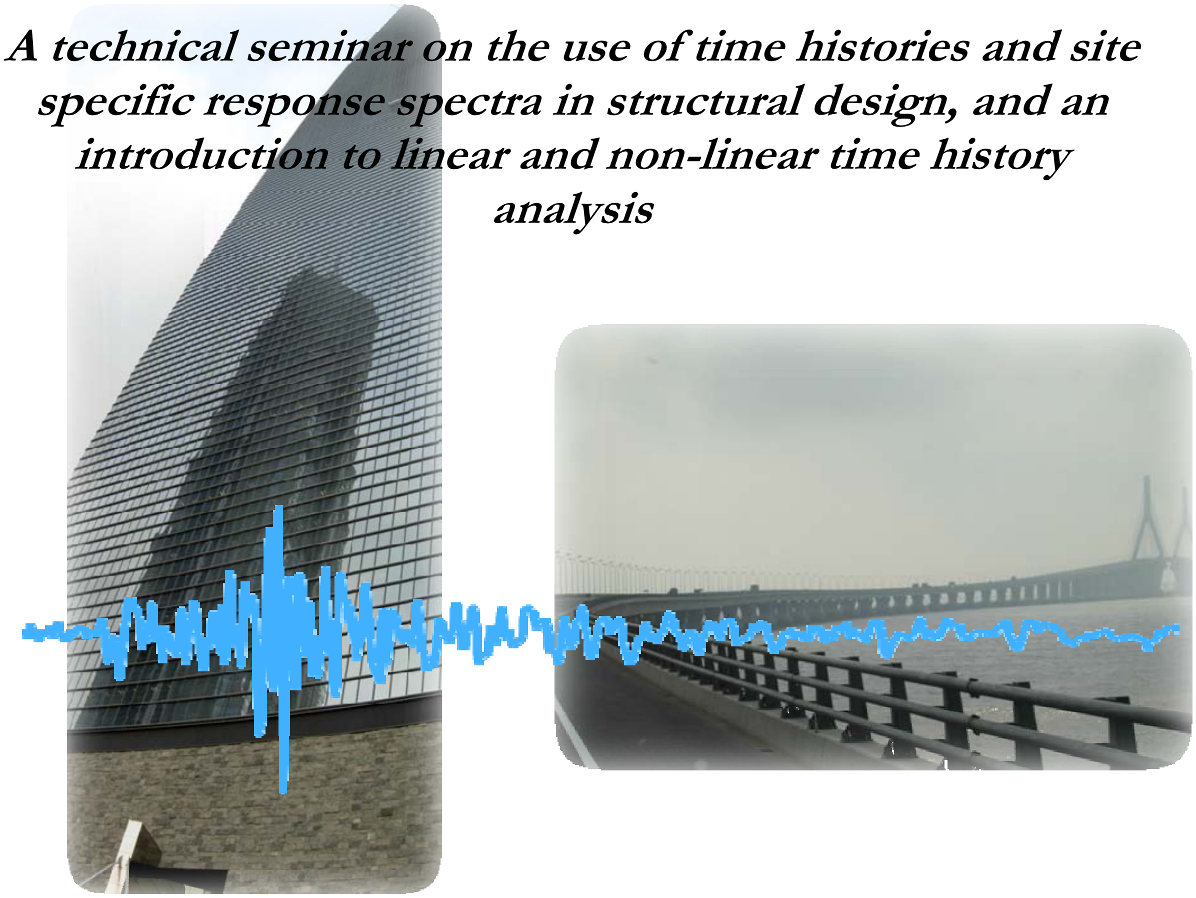
# TIME HISTORY ANALYSIS

***A technical seminar on the use of time histories and site specific response spectra in structural design, and an introduction to linear and non-linear time history analysis***



**PRESENTED BY:**
The Canadian Society for Civil Engineering
Vancouver Section

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UBC Department of Civil Engineering
Structural Engineers Association of BC, SEABC

November 14th and 15th, 2008
The University of British Columbia
Ponderosa Centre, Arbutus/Dogwood Rooms
Vancouver, BC

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## Technical seminar on the use of time histories and site specific response spectra in structural design, and an introduction to linear and non-linear time history analysis

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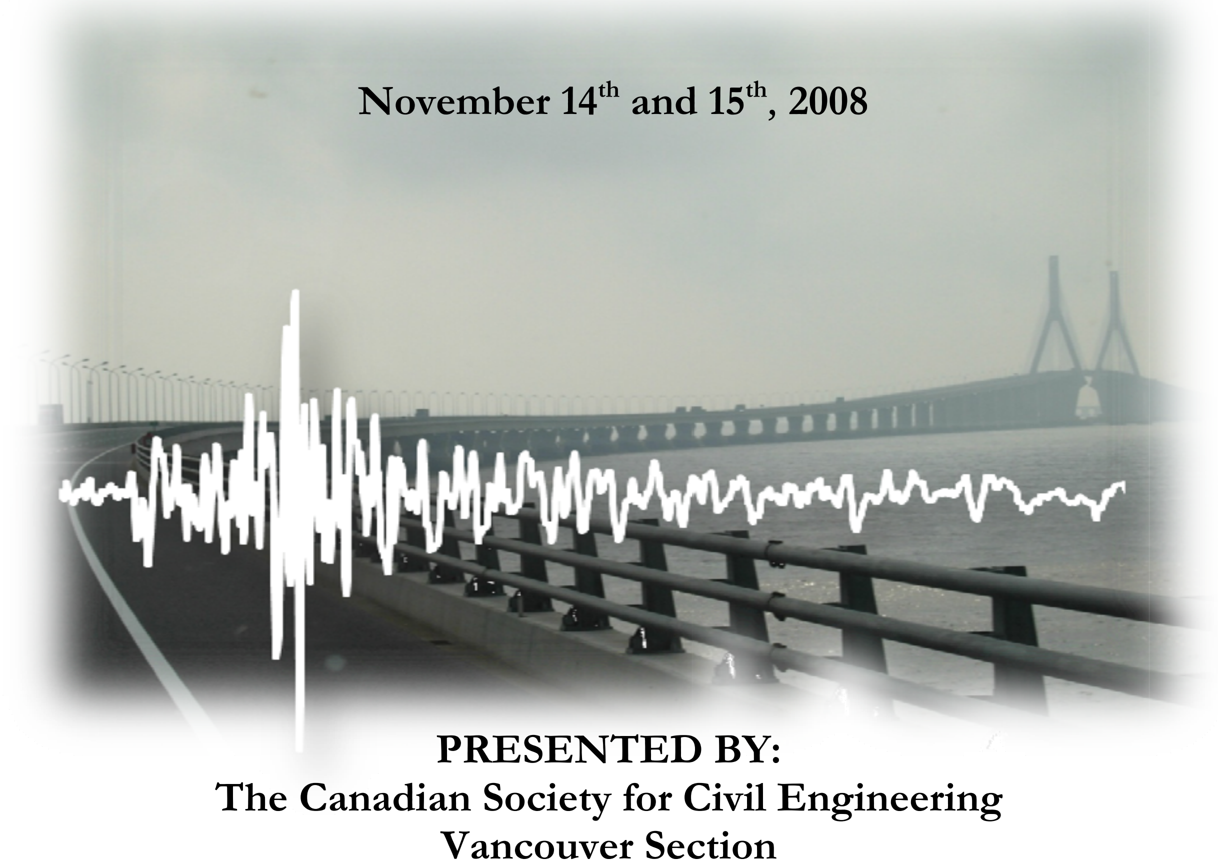

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**Vancouver, BC**

# Time History Analysis – Technical Seminar

**Final Program**

| Time | Topic | Speaker | Lecture |
|---|---|---|---|
| **Day One: Friday November 14, 2008** | | | |
| 07:30 to 08:00 | Registration | | |
| 08:00 to 08:10 | Welcome and Introduction | Carlos Ventura | |
| 08:10 to 08:55 | Time history versus response spectrum analysis | John Sherstobitoff | 1 |
| 08:55 to 09:25 | Origin and interpretation of ground motion time histories | Carlos Ventura | 2 |
| 09:25 to 09:55 | Selection and scaling of ground motion records | Tim Little | 3 |
| 09:55 to 10:25 | Latest approach to spectral matching of records | Adrian Wightman | 4 |
| 10:25 to 11:00 | Coffee Break | | |
| 11:00 to 11:45 | Site response analysis and soil-structure interaction | Liam Finn | 5 |
| 11:45 to 12:15 | Modelling the nonlinear response of structural concrete | Perry Adebar | 6 |
| 12:15 to 01:15 | **Lunch** | | |
| 01:15 to 02:15 | Impact of foundation modeling on the accuracy of seismic response history analysis | Farzad Naeim | 7 |
| 02:15 to 03:15 | Software options for structural time history analysis | Mahmoud Rezai | 8 |
| 03:15 to 03:45 | Coffee Break | | |
| 03:45 to 04:45 | Push-over analysis compared to time history analysis, a case study | Mark Sinclair | 9 |
| 04:45 to 05:30 | Questions Period, Closing Remarks and Information on Saturday Sessions | Carlos Ventura | |
| **Day Two: Saturday November 15, 2008** | | | |
| 08:30 to 09:30 | Time-history analysis for seismic design of bridges | Steve Zhu | 10 |
| 09:30 to 11:00 | Evolution of non-linear analysis for tall buildings: 1998-2008, two case studies | James Mutrie,<br>Clinton Hoffman<br>and Josif Golubovic | 11 |
| 11:00 to 11:30 | Coffee Break | | |
| 11:30 to 12:30 | Non-linear analysis of low rise buildings, braced frames, and rocking of foundations | Mahmoud Rezai | 12 |

# Foreword

The seminar covers both linear and non-linear time history analysis for buildings and bridges and examines the advantages of time history analysis compared with response spectrum methods for design of complex structures and structures with deep basements or deep pile foundations. The seminar will present a roadmap for guiding the user through the steps to effective use of time history analysis. The major steps are: obtaining appropriate input motions, modeling of the structure and soil, software options for analysis, and interpretation of results.

Considerable attention will be devoted to input motions, progressing from the field recording of time history records, their modification for engineering use, selection of appropriate time history records, current options and guidelines for scaling and spectrally matching of records, the propagation and modification of ground motions from the reference soil type to the base of the structure.

Recent developments in topics of particular relevance to design involving soil-structure interaction are presented in detail. These topics are the modification of free field motions by basement slabs and the effectiveness of various approximate models for the analysis of structures with deep pile foundations and multiple basements.

Case studies of a low-rise building, a high-rise building, and a bridge structure will be presented, focusing on interpretation and comparison of the results from response spectrum and time history analysis. These examples will also include discussions of issues of SSI.

The seminar will enable both structural and geotechnical engineers to reach a greater appreciation and understanding of their complementary roles in time history analysis of structures

The speakers in this seminar include well established professors from leading universities in North America and experienced senior engineers from engineering firms in Vancouver.

**The organizing committee of this seminar is comprised of:**

| | |
|---|---|
| Carlos E. Ventura, P.Eng. (Chairman) | UBC Civil Engineering Department |
| Max Bischof, P.Eng. | Bisco Engineering Inc. |
| Ron DeVall, P.Eng. | Read Jones Christoffersen Ltd. |
| Liam Finn, P.Eng | UBC Civil Engineering Department |
| Sharlie Huffman, P.Eng | Ministry of Transportation |
| Hugon Juarez Garcia | UBC Civil Engineering Department |
| Mahmoud Rezai, P.Eng | EQ-Tec Engineering Ltd. |
| John Sherstobitoff, P.Eng | Sandwell Engineering |
| Katherine Thibert | Sandwell Engineering |
| Shiva Tiwari | CH2M Hill |

**This seminar is presented by:**
The Canadian Society for Civil Engineering Vancouver Section (www.cscevancouver.ca)

**This seminar has received sponsorship and endorsement from the following organizations:**
UBC Department of Civil Engineering
Structural Engineering Association of BC, SEABC

**Additional contributing co-sponsors to the seminar are:**
Canadian Association for Earthquake Engineering, CAEE
Vancouver Geotechnical Society, VGS
ACI – BC Chapter
Consulting Engineers of British Columbia, CEBC
Canadian Institute of Steel Construction, CISC

The cooperation of these organizations is greatly appreciated by the organizing committee.

Vancouver, November 2008

## *DISCLAIMER*

While the authors have tried to be as accurate as possible, they cannot be held responsible for the designs of others that might be based on the material presented here. These notes are intended for the use of professional personnel competent to evaluate the significance and limitations of its contents and recommendations, and who will accept responsibility for the application of the material it contains. The authors and the sponsoring organizations disclaim any and all responsibility for the application of the stated principles and for the accuracy of any of the material contained herein.

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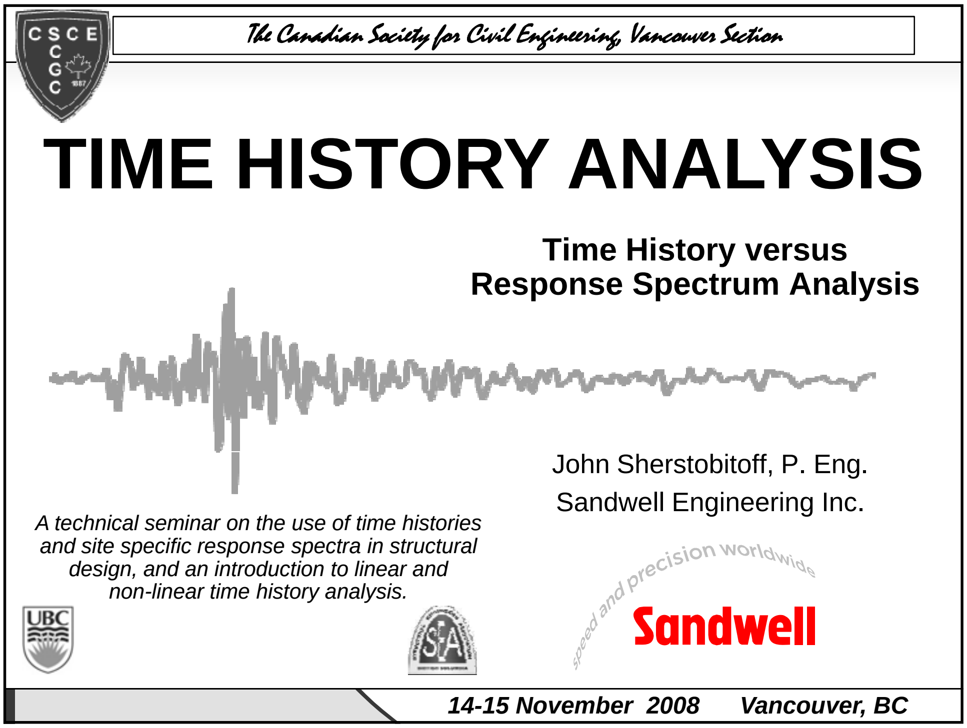
# TIME HISTORY ANALYSIS

## LECTURE # 1

# Time History versus Response Spectrum Analysis

**John Sherstobitoff, P. Eng.**
**Sandwell Engineering Inc.**

Currently Manager, Buildings and Infrastructure. Over 27 years at Sandwell after receiving a Master's Degree California Institute of Technology. In the past 17 years his work has focused on all aspects of seismic upgrading (buildings, dams, reservoirs, pipelines), including use of passive energy dissipation devices, fiber reinforced polymers (FRP). Currently part of Peer Review Group regarding Ministry of Education guidelines for seismic upgrade of schools, Seismic / Structural Working Group regarding Existing Buildings Code project.



The Canadian Society for Civil Engineering, Vancouver Section

# TIME HISTORY ANALYSIS

## Time History versus
## Response Spectrum Analysis

John Sherstobitoff, P. Eng.
Sandwell Engineering Inc.

*A technical seminar on the use of time histories and site specific response spectra in structural design, and an introduction to linear and non-linear time history analysis.*

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***14-15 November 2008 Vancouver, BC***

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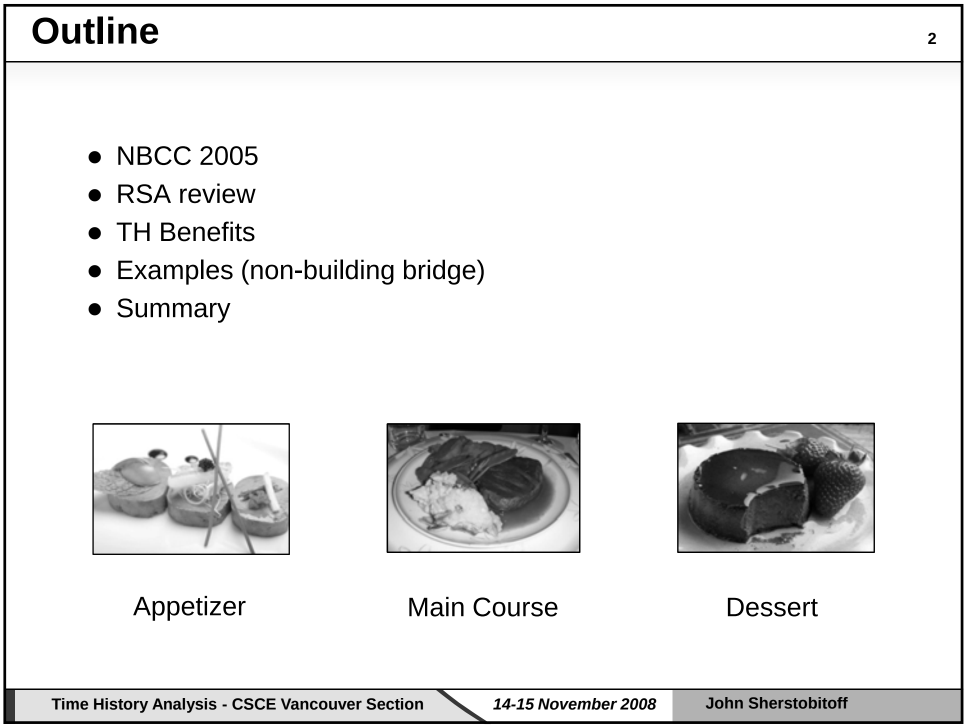


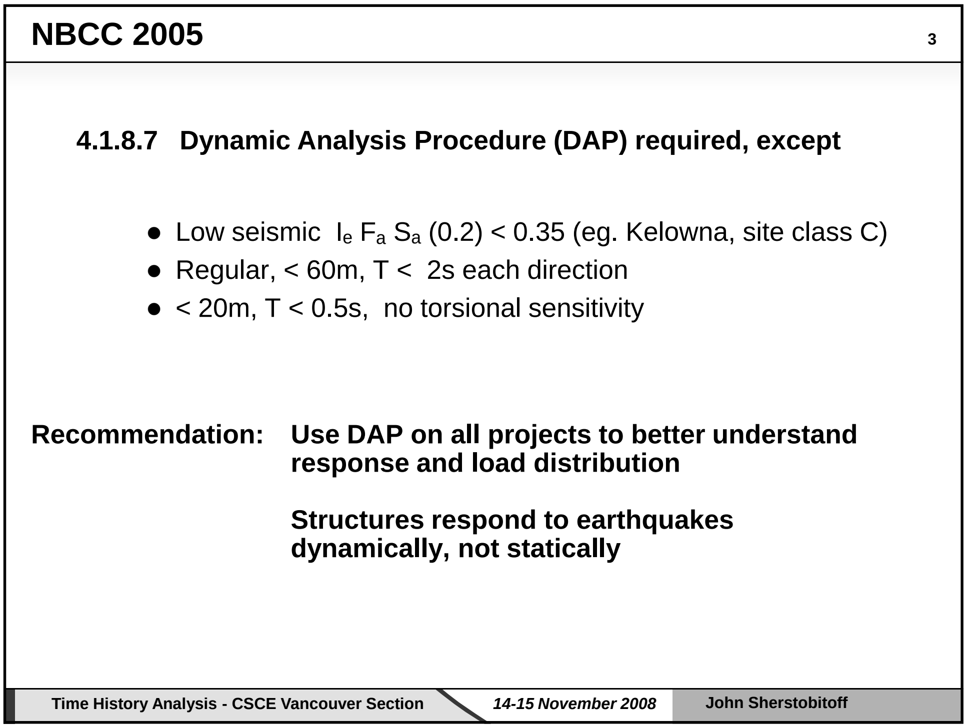
## Outline

- NBCC 2005
- RSA review
- TH Benefits
- Examples (non-building bridge)
- Summary

Appetizer

Main Course

Dessert



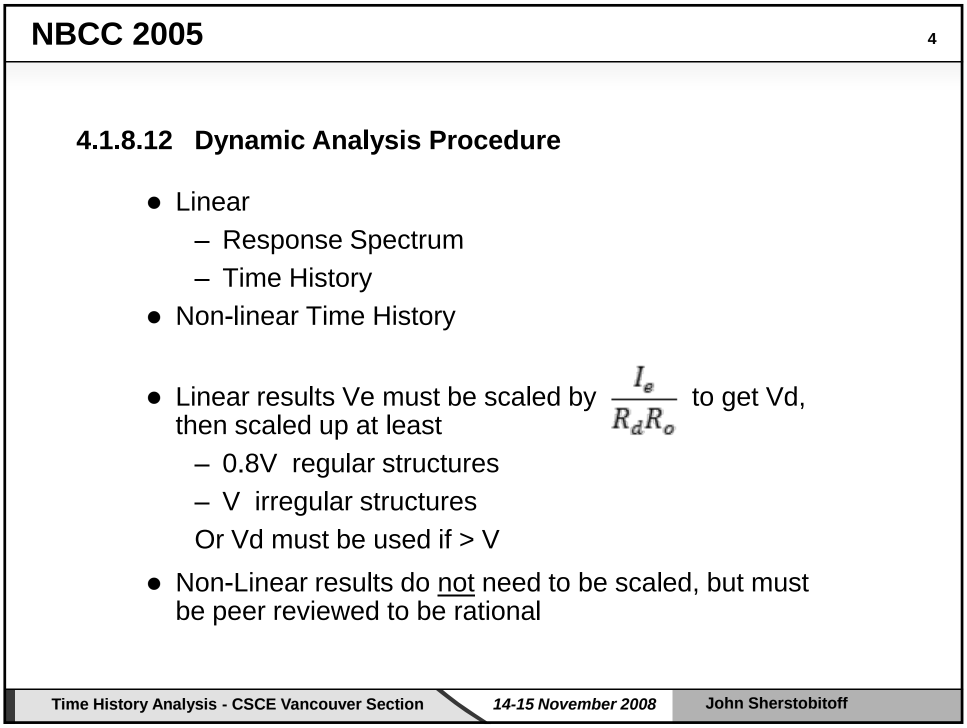
## NBCC 2005

**4.1.8.7 Dynamic Analysis Procedure (DAP) required, except**

- Low seismic $I_e F_a S_a (0.2) < 0.35$ (eg. Kelowna, site class C)
- Regular, < 60m, T < 2s each direction
- < 20m, T < 0.5s, no torsional sensitivity

**Recommendation:** **Use DAP on all projects to better understand response and load distribution**

**Structures respond to earthquakes dynamically, not statically**



## NBCC 2005

**4.1.8.12 Dynamic Analysis Procedure**

- Linear
  - Response Spectrum
  - Time History
- Non-linear Time History
- Linear results Ve must be scaled by $\frac{I_e}{R_d R_o}$ to get Vd, then scaled up at least
  - 0.8V regular structures
  - V irregular structures

  Or Vd must be used if > V
- Non-Linear results do <u>not</u> need to be scaled, but must be peer reviewed to be rational

## NBCC 2005

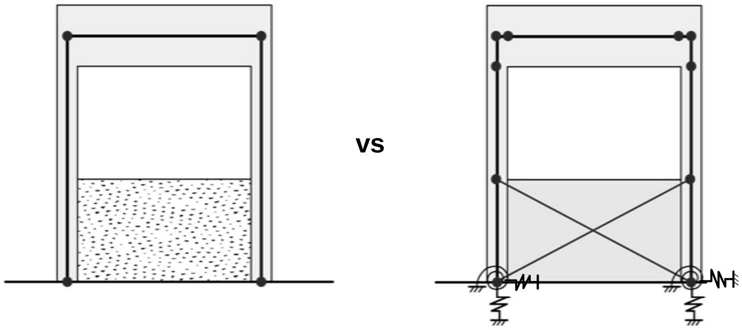
**Modeling**

**4.1.8.3 (6), (7), (8)**

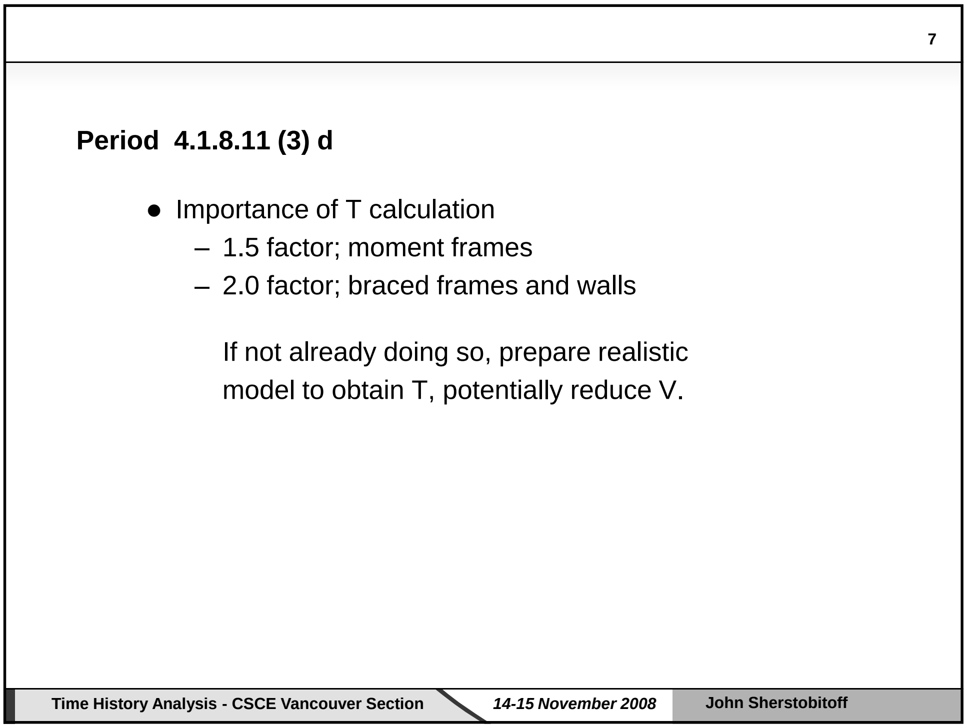
- Cracked sections concrete and masonry (… 0.35 Ig)
- Size of members and joints (offsets)
- P-delta
- Other effects that influence lateral stiffness and period

## NBCC 2005

**Modeling**



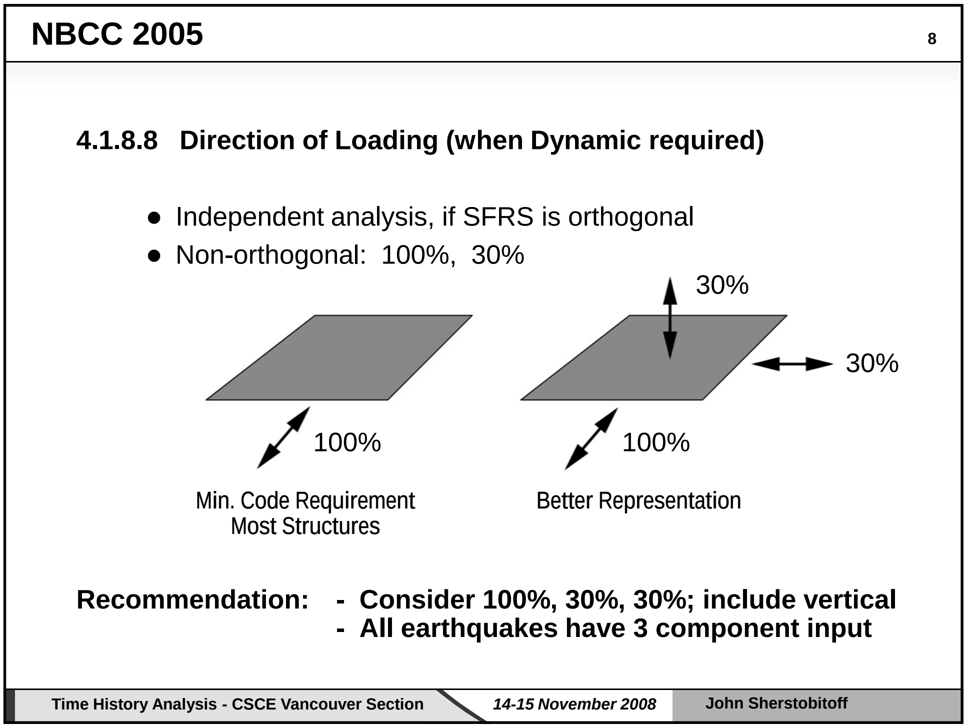
**Recommendation:** - **Sensitivity Analyses**
- **Consider soil structure interaction**



**Period 4.1.8.11 (3) d**

- Importance of T calculation
  - 1.5 factor; moment frames
  - 2.0 factor; braced frames and walls

If not already doing so, prepare realistic model to obtain T, potentially reduce V.



## NBCC 2005

**4.1.8.8 Direction of Loading (when Dynamic required)**

- Independent analysis, if SFRS is orthogonal
- Non-orthogonal: 100%, 30%

Min. Code Requirement
Most Structures

Better Representation

**Recommendation: - Consider 100%, 30%, 30%; include vertical**
**- All earthquakes have 3 component input**

## NBCC 2005

**Hazard**

- Probabilistic Approach
- 2% in 50 years
- 1/2500 annual probability of exceedance
- Median confidence level (50% chance ground motions higher)
- 84th percentile confidence level 1.5 – 3 times higher
- “designers should not place the same level of reliance on forces and deformations determined from a seismic analysis as they would for dead load and live load analysis”.

**Recommendation: - “structure should be designed to be able to resist ground motions in excess of DGM”**

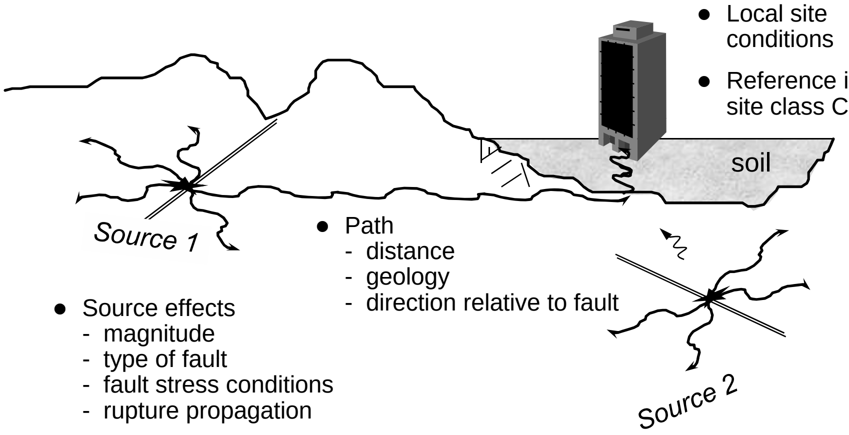
## NBCC 2005

**Hazard**

- Catalogues of earthquakes
- Geological structure of earth’s crust
- Magnitude recurrence relationships
- Aleatory uncertainty (physical variability)
- Epistemic uncertainty (modelling assumptions)
- Two source zone models (Historical, Regional)
- Attenuation

**Note: - Not based on a library of time history analysis records**
**- Cascadia not included (will be in future)**
**- site specific necessary for critical structures**

## Seismic Hazard



- Local site conditions
- Reference is site class C

- Path
  - distance
  - geology
  - direction relative to fault

- Source effects
  - magnitude
  - type of fault
  - fault stress conditions
  - rupture propagation

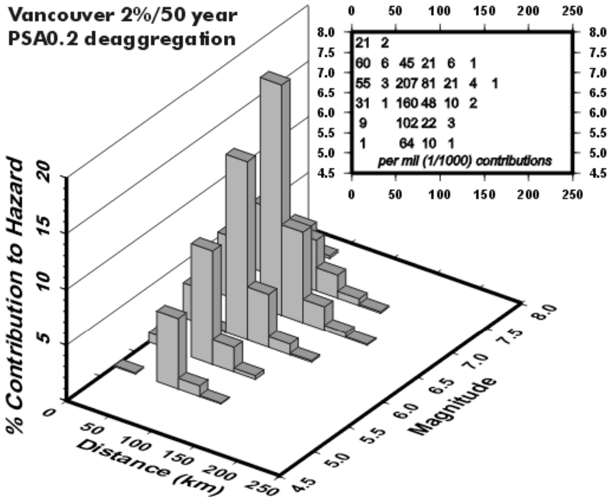
---

## NBCC 2005

**Uniform Hazard Spectrum**

- Composite of potential earthquakes
- Crustal, sub-crustal
- Large distant events (affecting long period)
- Moderate local events (affecting short period)
- Conservative to consider entire period range in single event
- No real EQ will match UHS, except synthetic EQ

Deaggregation of hazard contributions by magnitude and distance



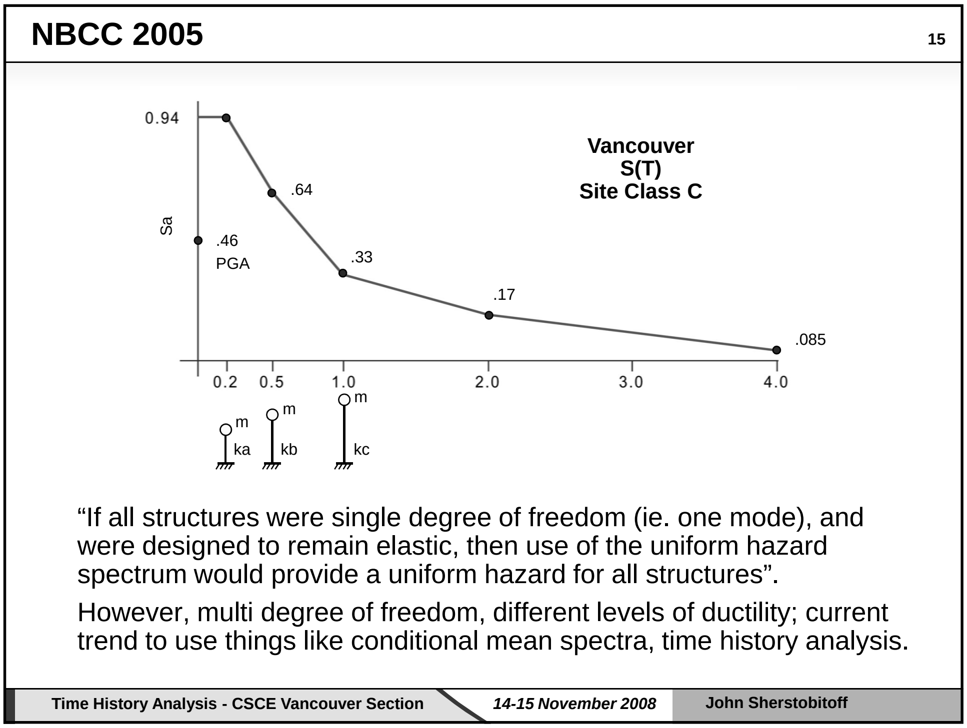
*J. Adams, GSC*

## Seismic Hazard

- Attenuation relationships typically:
  - Predict PGA and Sa (spectral accelerations) at various vibration periods

$$\ln(y) = f(M,D,F,\varepsilon)$$

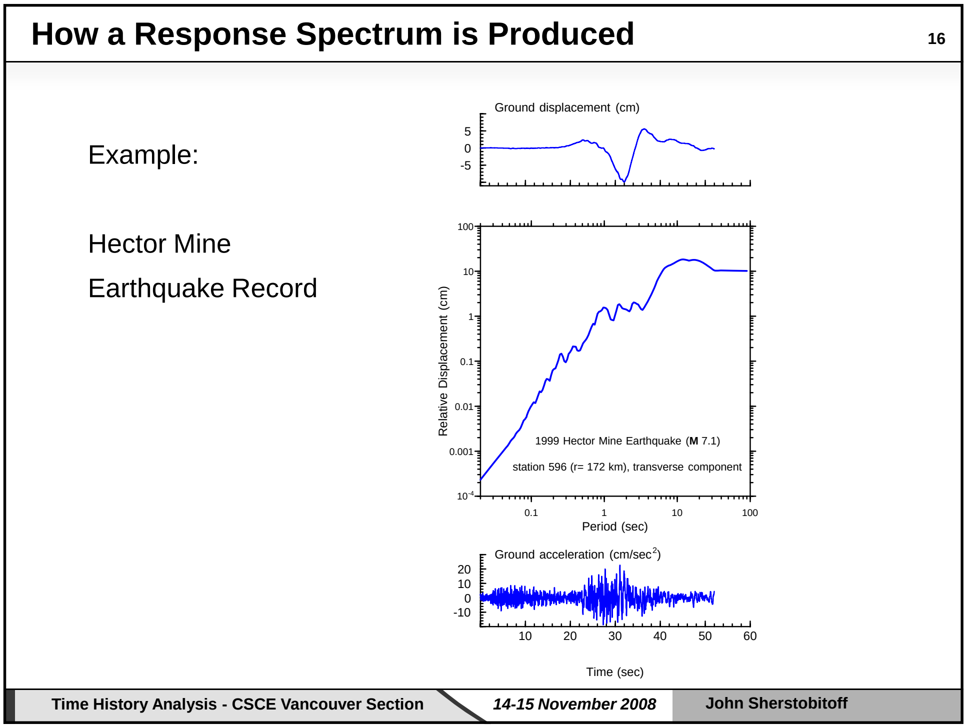
$y$ = PGA or Sa
$M$ = Magnitude
$D$ = Distance
$F$ = Fault Type Factor
$\varepsilon$ = Uncertainty Term



## NBCC 2005

"If all structures were single degree of freedom (ie. one mode), and were designed to remain elastic, then use of the uniform hazard spectrum would provide a uniform hazard for all structures".

However, multi degree of freedom, different levels of ductility; current trend to use things like conditional mean spectra, time history analysis.

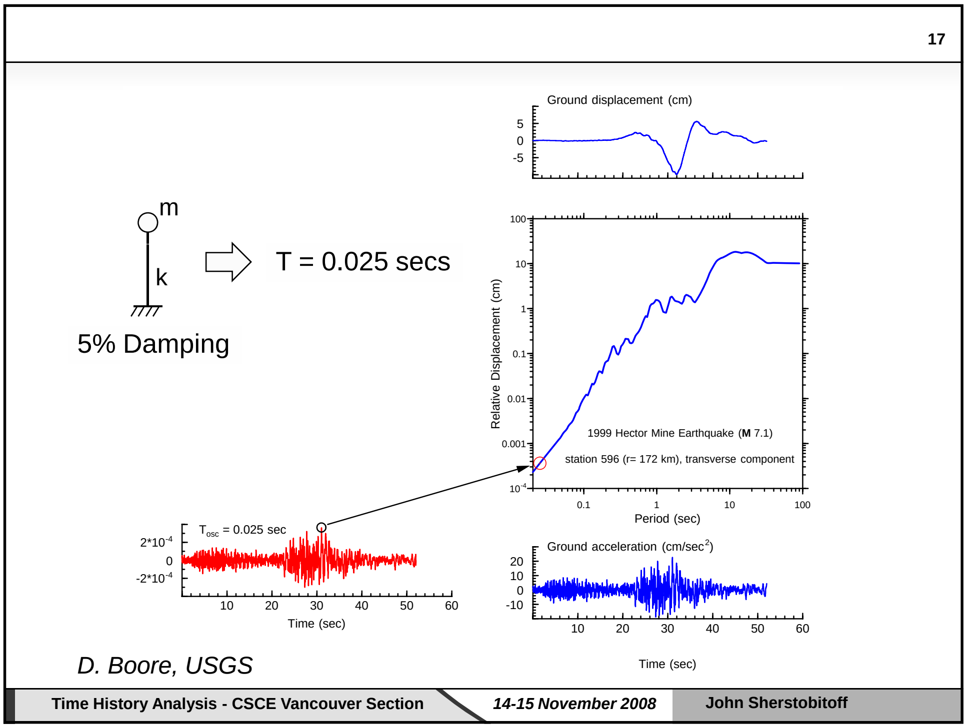


## How a Response Spectrum is Produced

Example:

Hector Mine

Earthquake Record



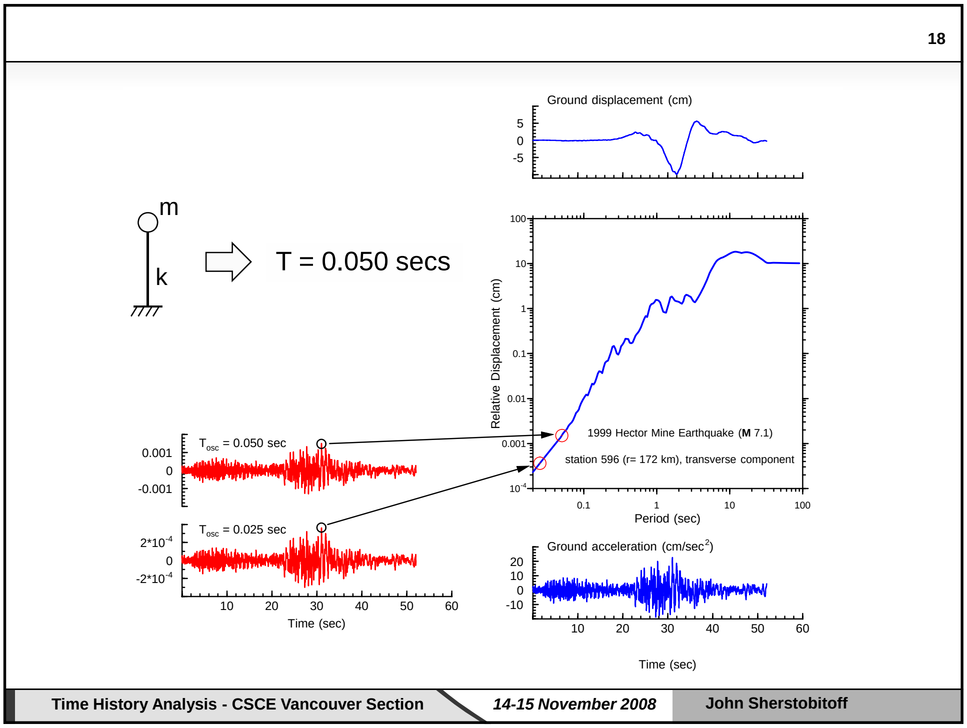
17

T = 0.025 secs

5% Damping

*D. Boore, USGS*

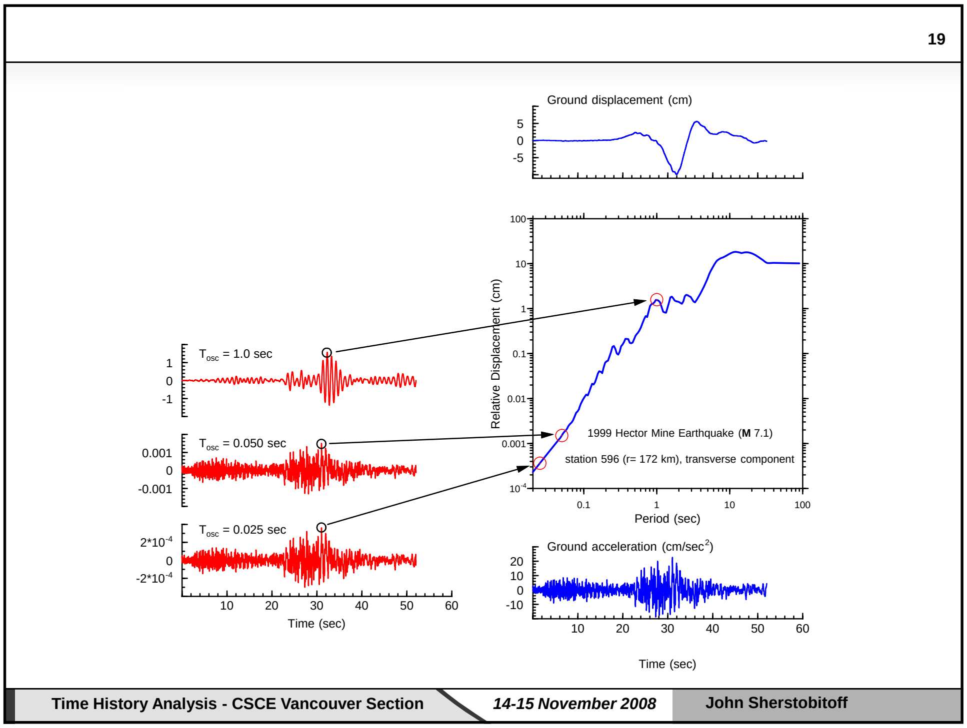
**Time History Analysis - CSCE Vancouver Section** ***14-15 November 2008*** **John Sherstobitoff**



18

T = 0.050 secs

**Time History Analysis - CSCE Vancouver Section** ***14-15 November 2008*** **John Sherstobitoff**


19
Ground displacement (cm)
5
0
-5
Relative Displacement (cm)
100
10
1
0.1
0.01
0.001
$10^{-4}$
0.1
1
10
100
Period (sec)
1999 Hector Mine Earthquake (**M** 7.1)
station 596 (r= 172 km), transverse component
Ground acceleration (cm/sec$^2$)
20
10
0
-10
10 20 30 40 50 60
Time (sec)
$T_{osc}$ = 1.0 sec
1
0
-1
$T_{osc}$ = 0.050 sec
0.001
0
-0.001
$T_{osc}$ = 0.025 sec
$2*10^{-4}$
0
$-2*10^{-4}$
10 20 30 40 50 60
Time (sec)
Time History Analysis - CSCE Vancouver Section
14-15 November 2008
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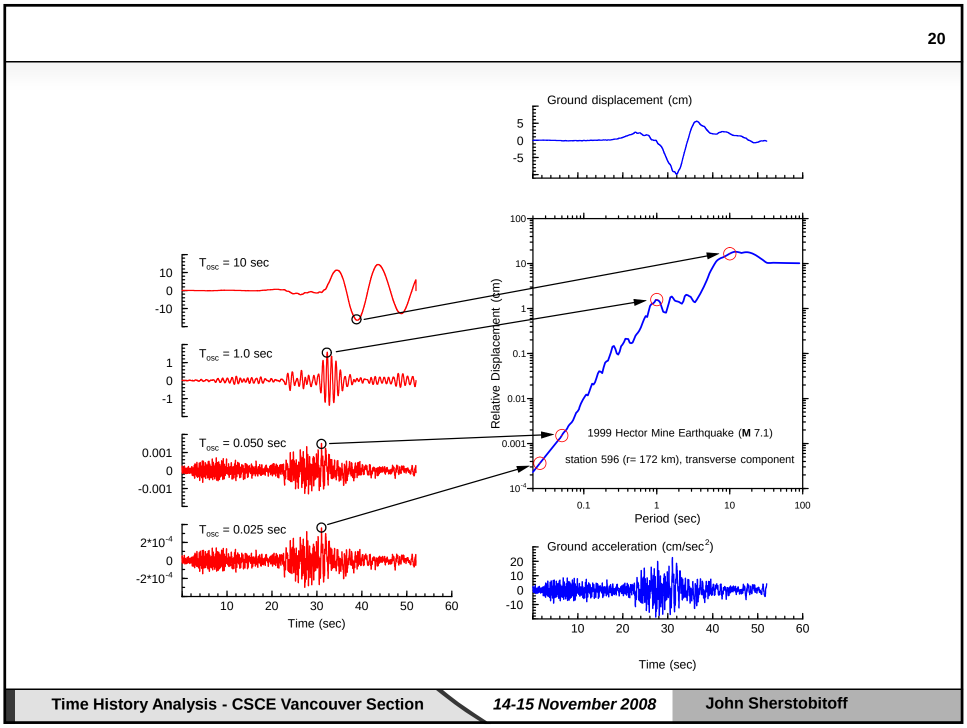
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Ground displacement (cm)
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100
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0.1
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Period (sec)
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$T_{osc}$ = 10 sec
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$T_{osc}$ = 0.050 sec
0.001
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-0.001
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$2*10^{-4}$
0
$-2*10^{-4}$
10 20 30 40 50 60
Time (sec)
Time History Analysis - CSCE Vancouver Section
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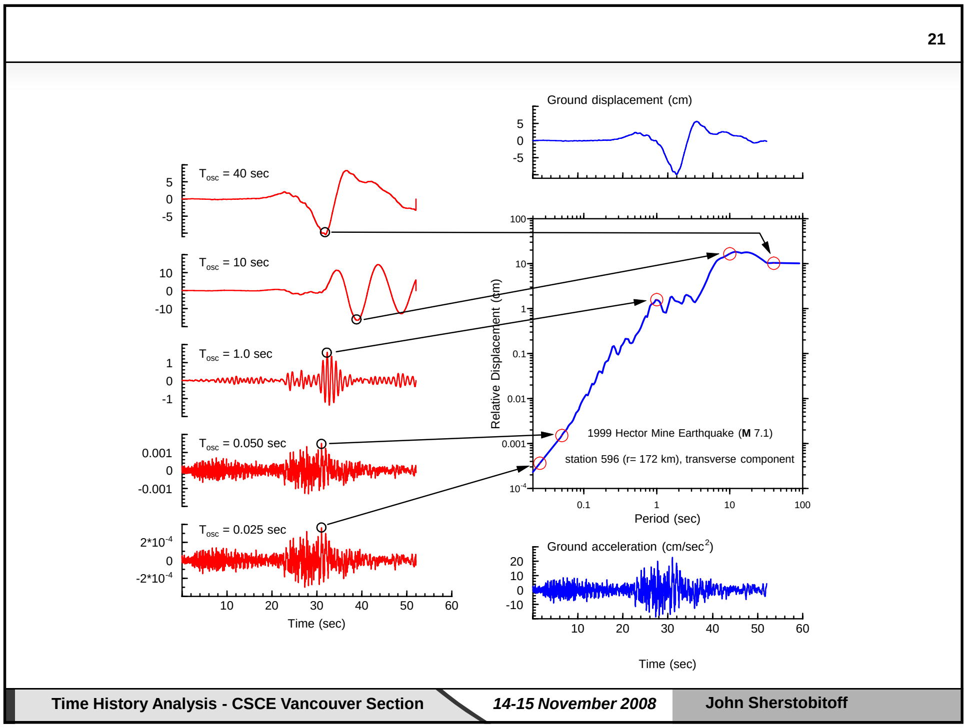
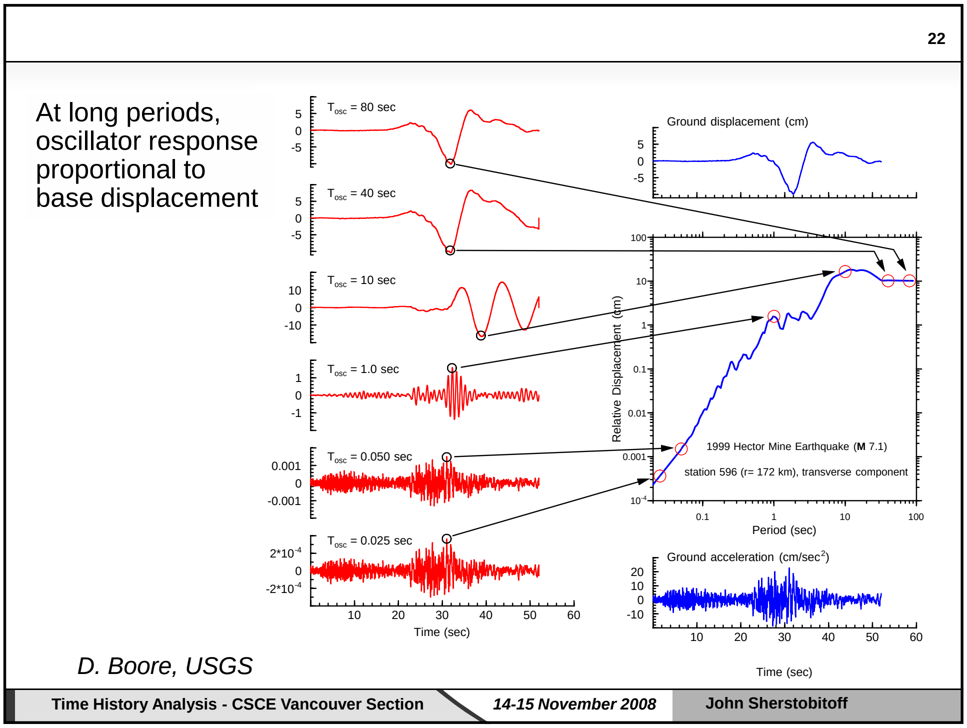
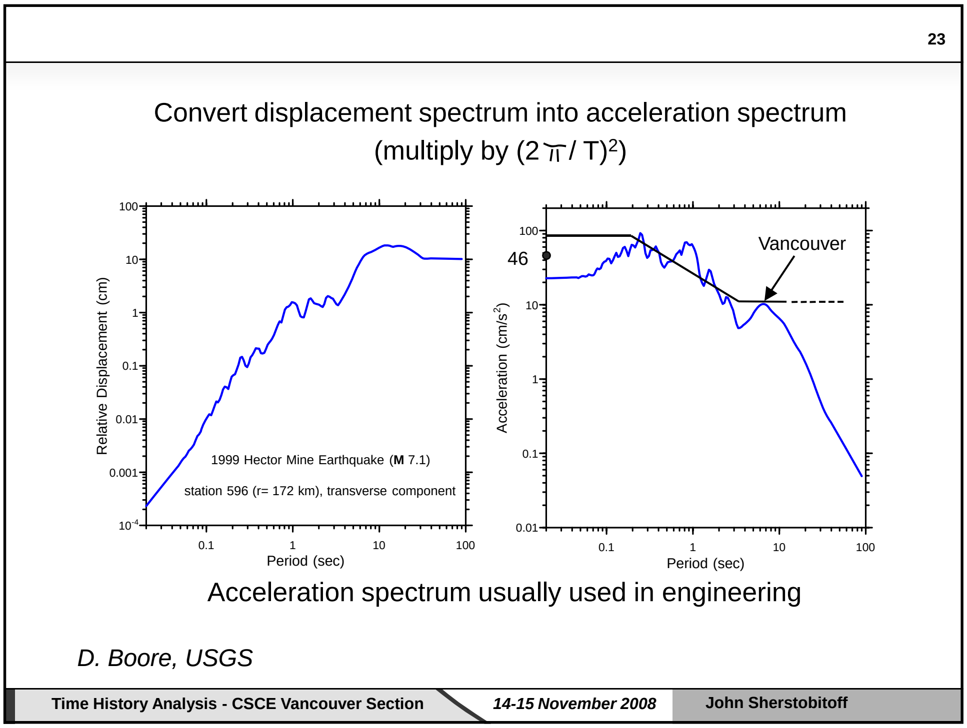




At long periods, oscillator response proportional to base displacement

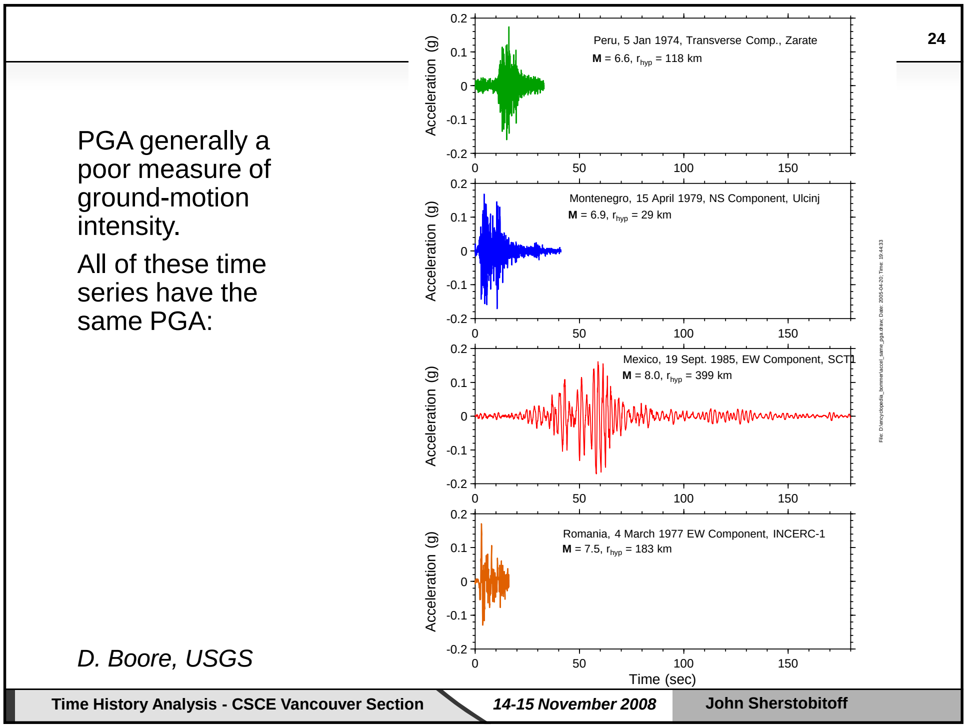
*D. Boore, USGS*



Convert displacement spectrum into acceleration spectrum
(multiply by $(2\pi / T)^2$)

Acceleration spectrum usually used in engineering

*D. Boore, USGS*

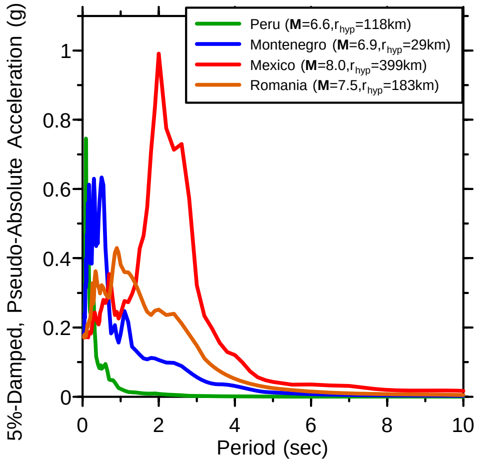


PGA generally a poor measure of ground-motion intensity.

All of these time series have the same PGA:

*D. Boore, USGS*

But the response spectra (and consequences for structures) are quite different



*D. Boore, USGS*

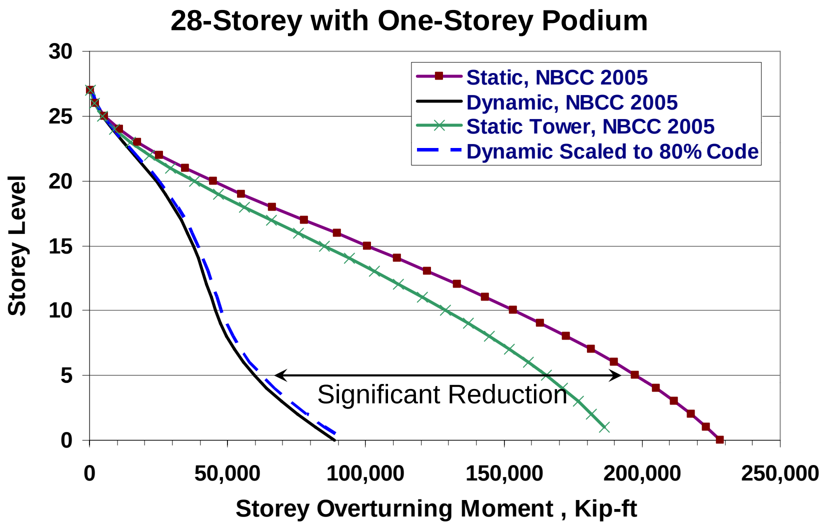
## Response Spectrum Analysis

Advantages over Equivalent Static Force Procedure

- More realistic load distribution.
- Changes in mass and stiffness are better modeled.
- Reductions in base shear in some torsionally eccentric buildings.
- Reductions in overturning moments and displacements for tall, long period buildings.
- Dynamic amplification of torque effects is captured.

. . . All while being relatively simple to do.

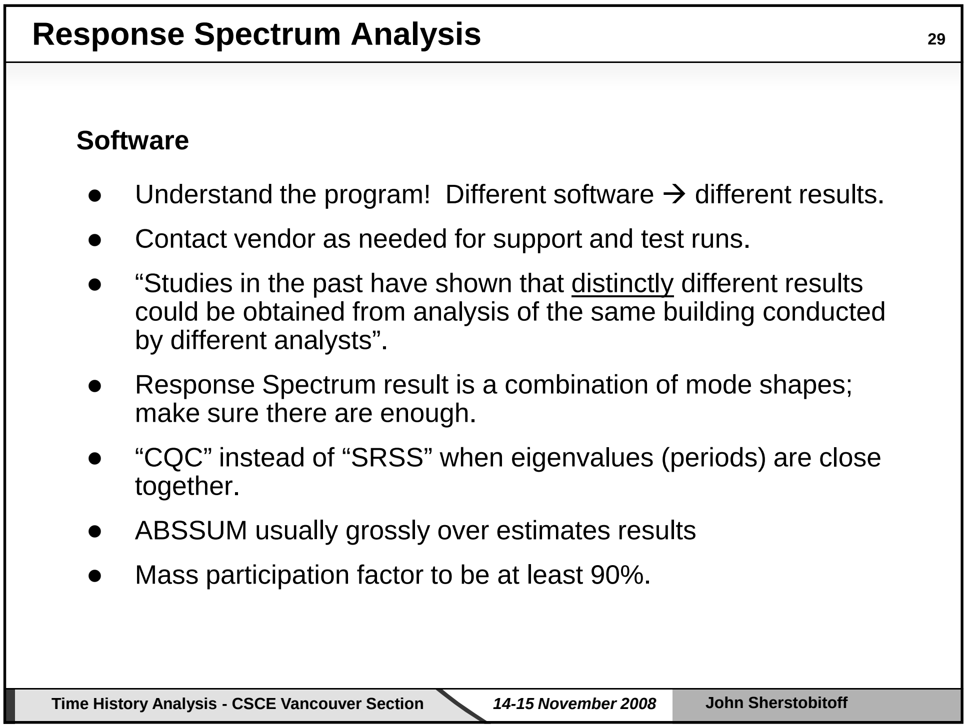
## Storey Moment (28-Storey Concrete Highrise)



## Response Spectrum Analysis

Some Computer Modeling items to consider in addition to 4.1.8.3

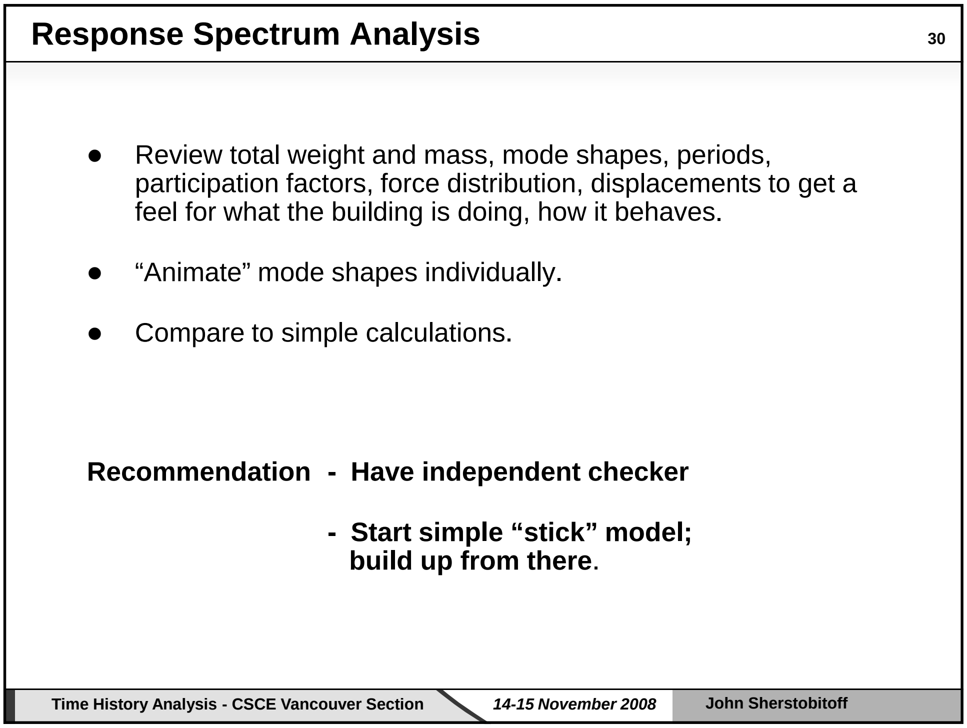
- shear displacements
- below grade structures
- diaphragm stiffness
- foundation flexibility at soil
- added mass (snow, large equipment)
- Beam vs shell vs plate elements



## Response Spectrum Analysis

**Software**

- Understand the program!  Different software → different results.
- Contact vendor as needed for support and test runs.
- "Studies in the past have shown that distinctly different results could be obtained from analysis of the same building conducted by different analysts".
- Response Spectrum result is a combination of mode shapes; make sure there are enough.
- "CQC" instead of "SRSS" when eigenvalues (periods) are close together.
- ABSSUM usually grossly over estimates results
- Mass participation factor to be at least 90%.

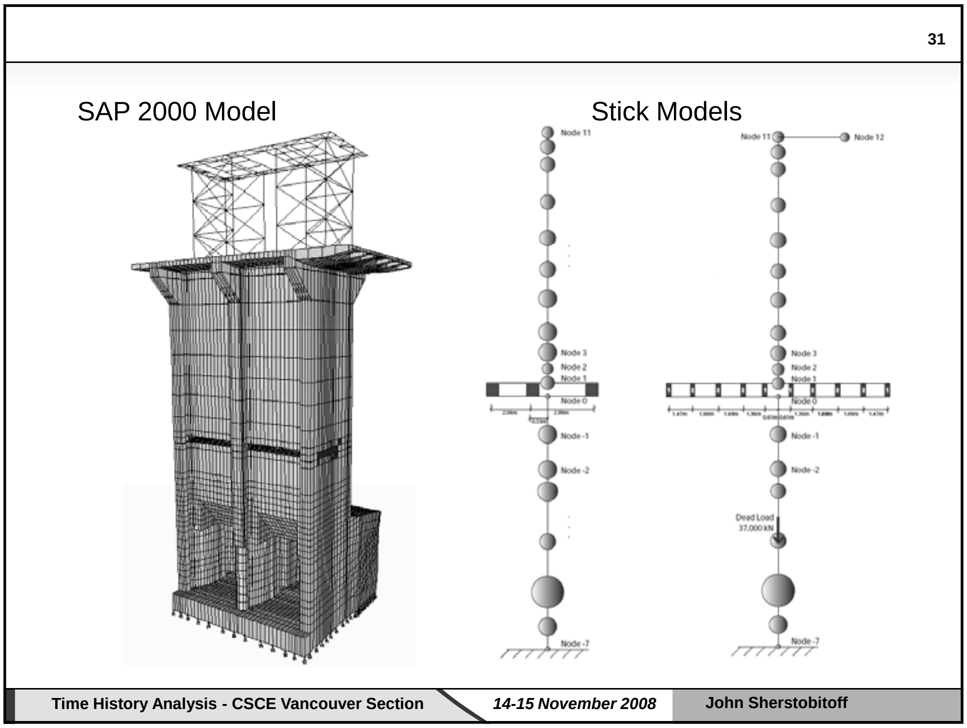


## Response Spectrum Analysis

- Review total weight and mass, mode shapes, periods, participation factors, force distribution, displacements to get a feel for what the building is doing, how it behaves.
- "Animate" mode shapes individually.
- Compare to simple calculations.

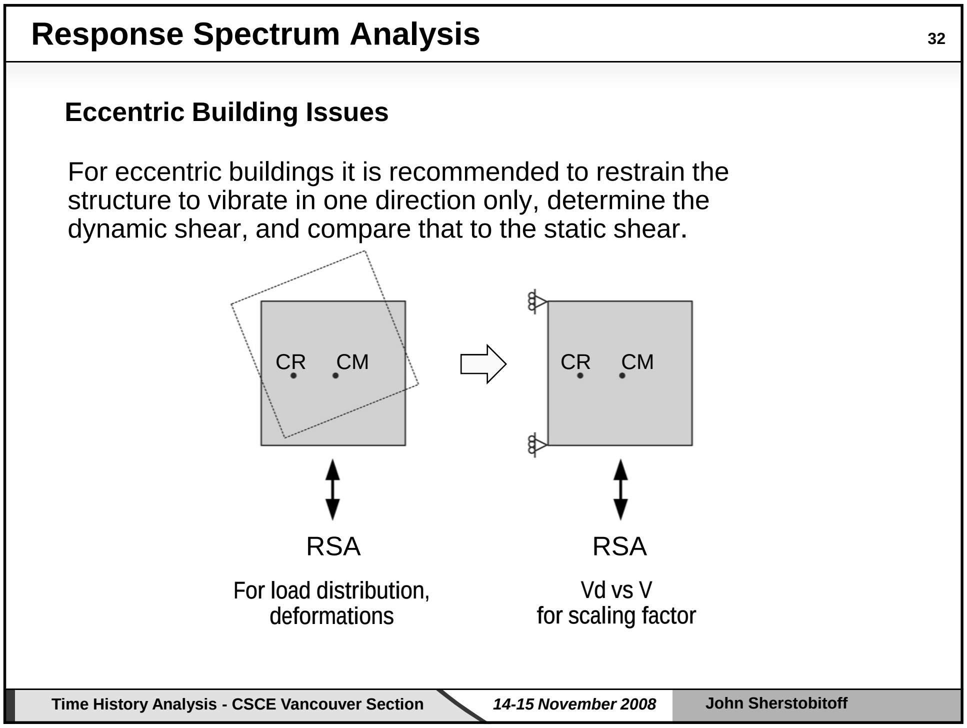
**Recommendation  -  Have independent checker**

**-  Start simple "stick" model;
build up from there**.



SAP 2000 Model

Stick Models



## Response Spectrum Analysis

### Eccentric Building Issues

For eccentric buildings it is recommended to restrain the structure to vibrate in one direction only, determine the dynamic shear, and compare that to the static shear.

RSA

For load distribution,
deformations

RSA

Vd vs V
for scaling factor

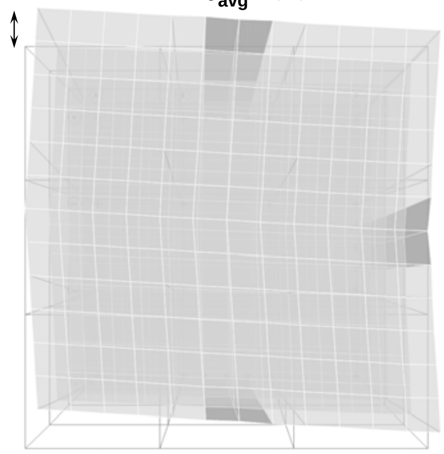
## Response Spectrum Storey Deflection

**Torsion**

- Torsional sensitivity ratio $B_x$ for each Level x is :

$$B_x = \delta_{max} / \delta_{avg}$$

- Start using center of mass offset at distances ±0.05D.
- If B< 1.7,  then regular, and OK to use this lesser offset



$\delta_{max}$ = 8.7"

$\delta_{avg}$= 6.0"

**$B_x$ = 1.42**

Therefore, the building is considered regular.

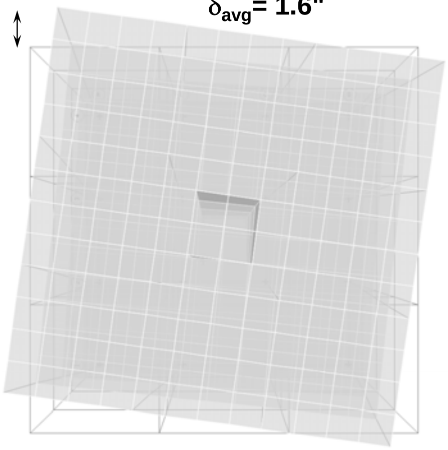
## Response Spectrum Storey Deflection

**Torsion**

- Torsional sensitivity ratio $B_x$ for each Level x is :

$$B_x = \delta_{max} / \delta_{avg}$$

- For irregular buildings induce accidental torsion by applying the equivalent static forces at distances ±0.10D from the centres of mass at each floor.

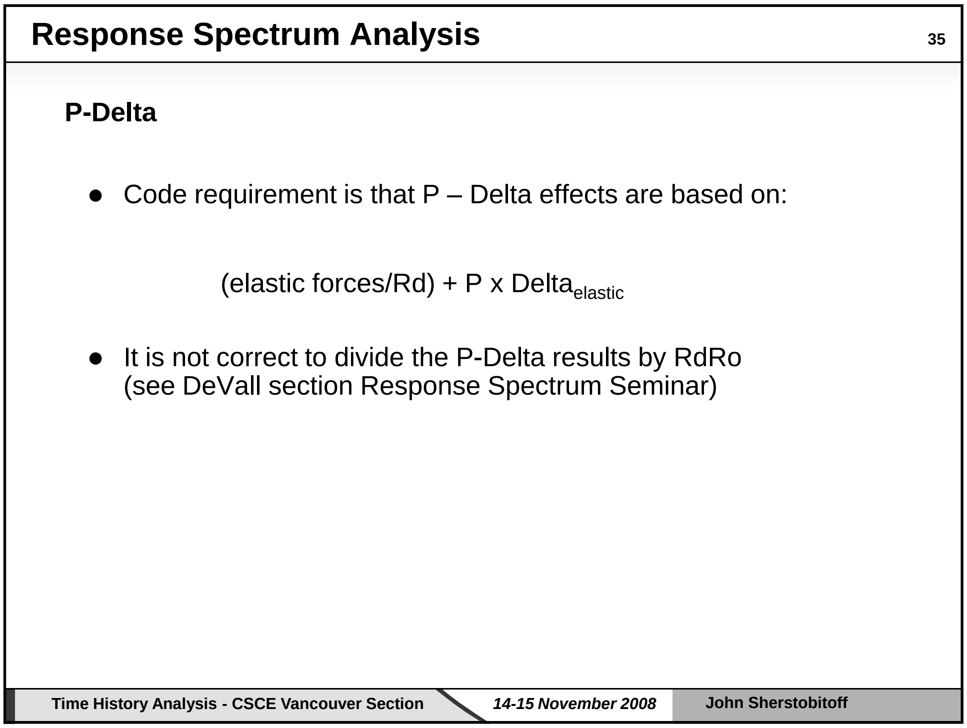


$\delta_{max}$ = 7.7"

$\delta_{avg}$= 1.6"

**$B_x$ = 4.8**

Therefore, confirms the building is irregular.



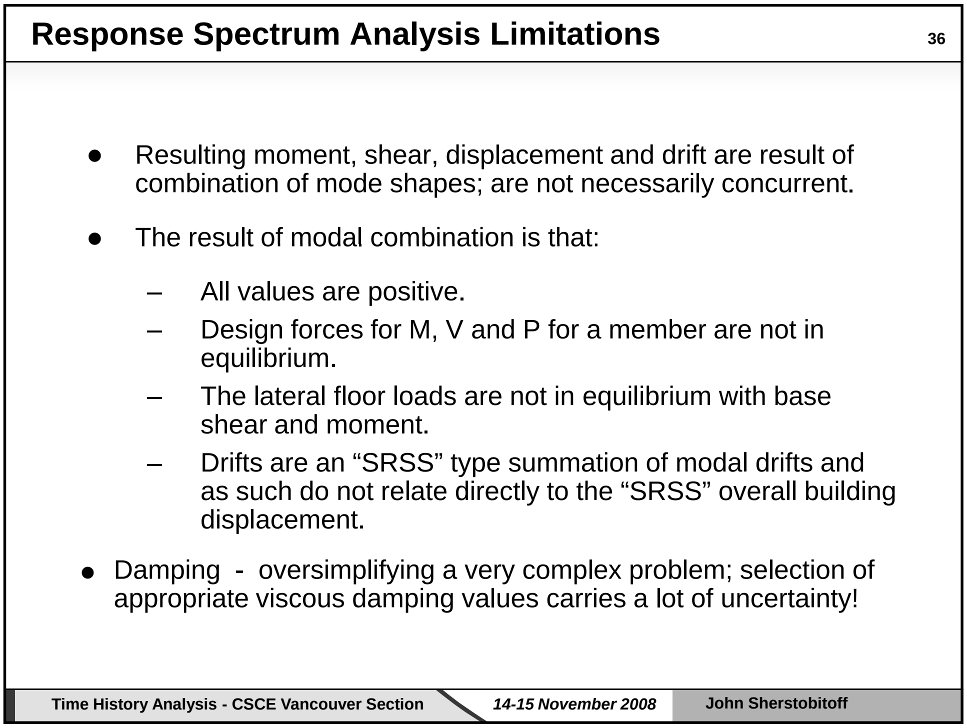
## Response Spectrum Analysis

**P-Delta**

- Code requirement is that P – Delta effects are based on:

(elastic forces/Rd) + P x Delta$_{elastic}$

- It is not correct to divide the P-Delta results by RdRo (see DeVall section Response Spectrum Seminar)



## Response Spectrum Analysis Limitations

- Resulting moment, shear, displacement and drift are result of combination of mode shapes; are not necessarily concurrent.
- The result of modal combination is that:
    - All values are positive.
    - Design forces for M, V and P for a member are not in equilibrium.
    - The lateral floor loads are not in equilibrium with base shear and moment.
    - Drifts are an “SRSS” type summation of modal drifts and as such do not relate directly to the “SRSS” overall building displacement.
- Damping - oversimplifying a very complex problem; selection of appropriate viscous damping values carries a lot of uncertainty!

## Response Spectrum Analysis Limitations

**Caution**

- Do **<u>not</u>** compute story shears from the story drifts derived from the SRSS of the story displacements.
- Calculate the shears in each mode (using modal drifts) and then SRSS the results.

## Response Spectrum Analysis

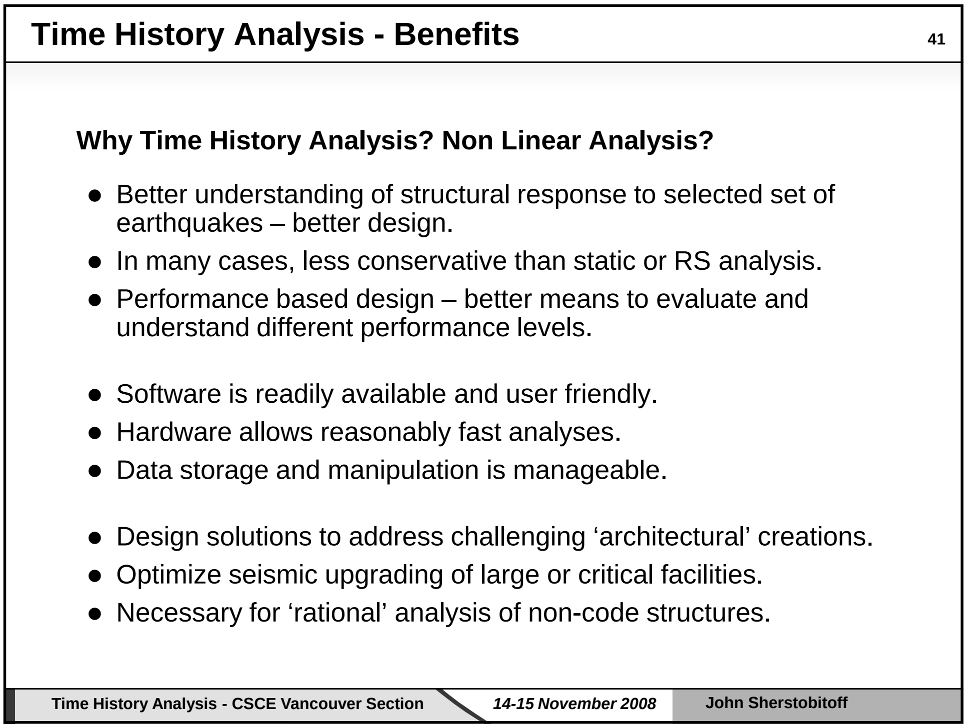
- Utilizes response spectrum to give structural designer a set of <u>possible</u> forces and deformations a real structure would experience under earthquake loads.
- For SDF systems, RSM gives quick and accurate peak response without the need for a time-history analysis.
- For low buildings (few modes) with $R_d$ = 1.5, quite reasonable
- For MDF systems, a true structural system, RSM gives a <u>reasonably</u> accurate peak response, without using a full time-history analysis.

## Response Spectrum Analysis

- Offers a standardized solution to evaluate structures.
- Method is simple, straightforward, yet powerful.
- Designer can assess design in a timely and efficient manner.
- With computer hardware and computer modeling software available today, RSM offers a way for designer to quickly verify and understand the sometimes non-intuitive results.
- A necessary initial step to understand behavior before embarking on TH analyses.

**Other Comments**

- Must read Commentary J in detail
- Excellent reference:
  - ‘Dynamic Analysis of Buildings for Earthquake-resistant Design’ by Saatcioglu and Humar, Canadian Journal of Civil Engineering, April 2003. (free download)



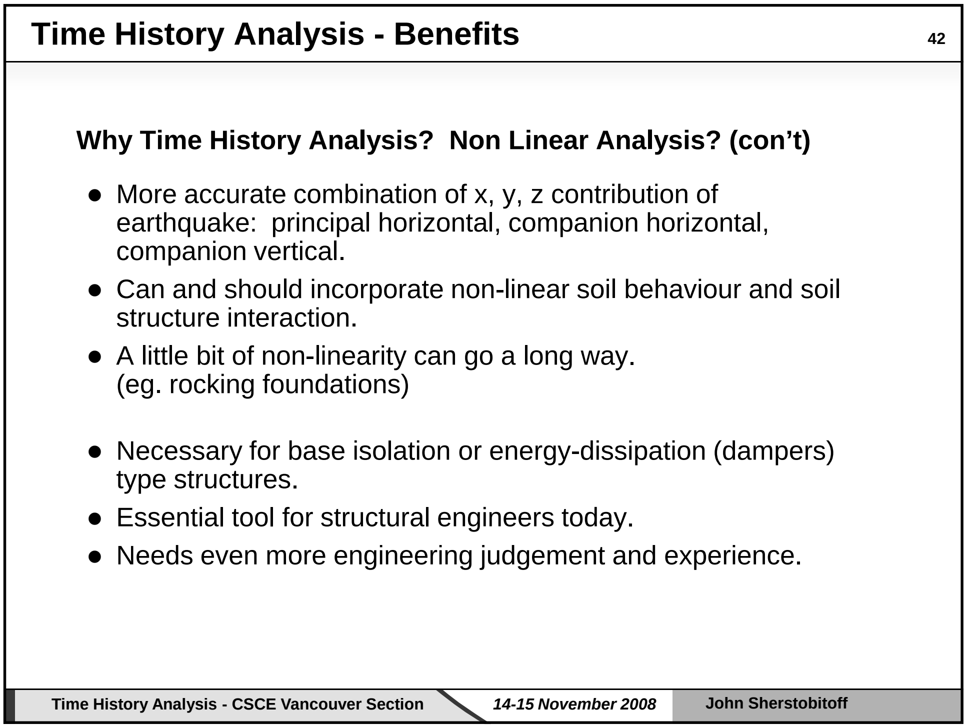
## Time History Analysis - Benefits

**Why Time History Analysis? Non Linear Analysis?**

- Better understanding of structural response to selected set of earthquakes – better design.
- In many cases, less conservative than static or RS analysis.
- Performance based design – better means to evaluate and understand different performance levels.

- Software is readily available and user friendly.
- Hardware allows reasonably fast analyses.
- Data storage and manipulation is manageable.

- Design solutions to address challenging ‘architectural’ creations.
- Optimize seismic upgrading of large or critical facilities.
- Necessary for ‘rational’ analysis of non-code structures.



## Time History Analysis - Benefits

**Why Time History Analysis?  Non Linear Analysis? (con’t)**

- More accurate combination of x, y, z contribution of earthquake:  principal horizontal, companion horizontal, companion vertical.
- Can and should incorporate non-linear soil behaviour and soil structure interaction.
- A little bit of non-linearity can go a long way.
  (eg. rocking foundations)

- Necessary for base isolation or energy-dissipation (dampers) type structures.
- Essential tool for structural engineers today.
- Needs even more engineering judgement and experience.

## Time History Analysis - Benefits

**Linear Time History**

- Information on time-wise fluctuations of structural parameters. (forces, deflections)
- Can indicate peak demands are only very infrequent, short duration spikes for which structure or soil cannot respond to.

*(example later)*

## Time History Analysis - Benefits

**Non-Linear Time History**

- Most representative of actual structure.
- No one period of structure.
- Inelastic hysteretic behaviour included.
  - Materials
  - Detailing
- Captures duration effects; changes in stiffness, strength.
- No $R_d R_o$ scaling.
- Capture $I_e$ effect by scaling up input, or reducing acceptable deflections, ductility.

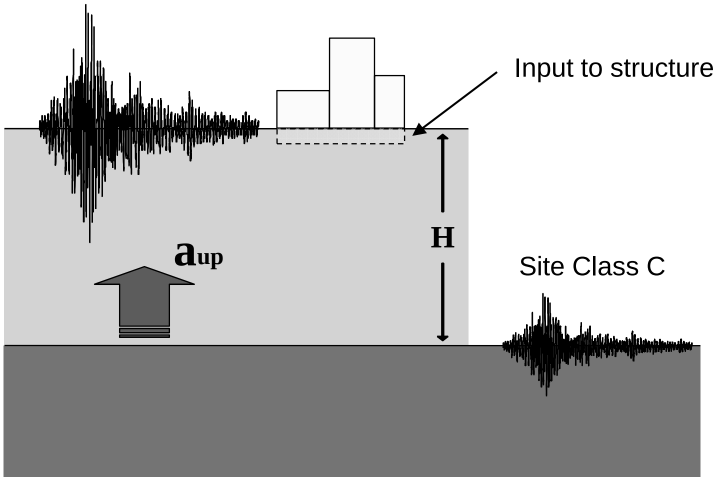
## Time History Analysis

**Considerations**

- Totally reliant on appropriate ground motions; how do we get these?
- How many to use? (min 3, preferred 7, > 20?)
- Include vertical component? Can have significant effect in certain structures.

- Where to apply motions?
  - At grade
  - Along height of basement walls
  - At bottom of basement

## Site Response



. . . or adjust S(T) using Fa, Fv values

## Near Fault Effects

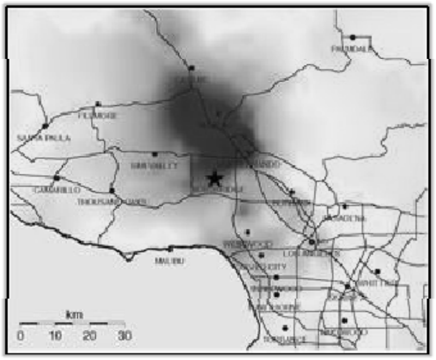
Directivity ground motion in direction of rupture propagation is more severe than in other directions.

At sites close to fault but away from epicenter.

Fling is related to permanent deformation at site.

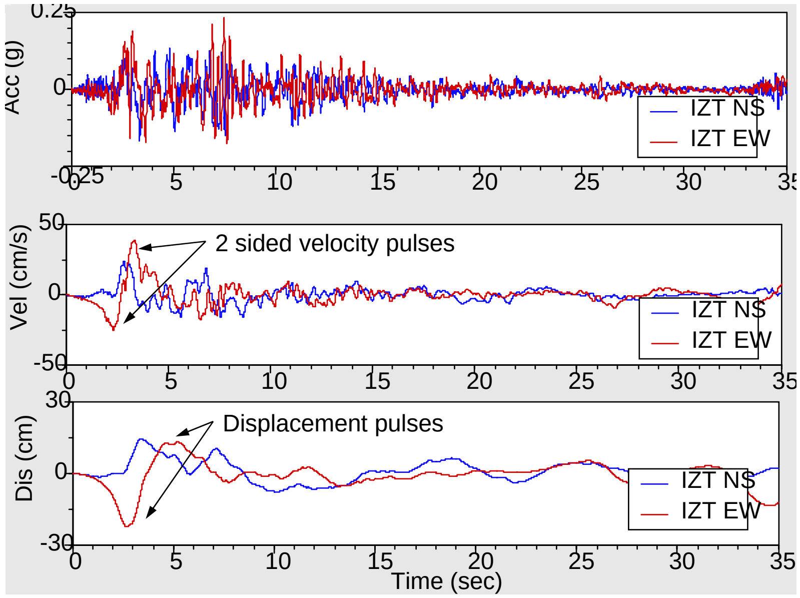
At sites near fault rupture independent of epicenter location.

*January 17, 1994 M6.7*
*Northridge Earthquake*

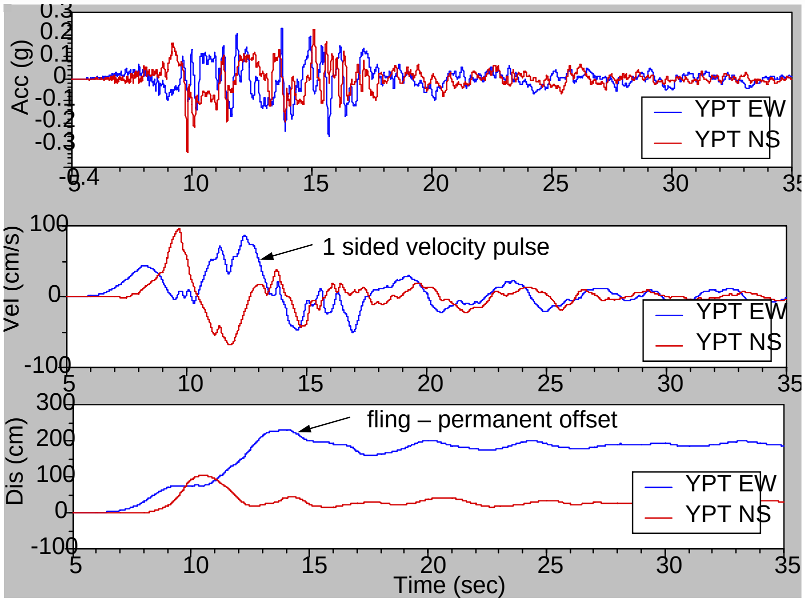


This ShakeMap of the shaking shows the result of rupture directivity toward the north.

## Kocaeli, Turkey - IZT station directivity



## Example of Fling



## Time History Analysis

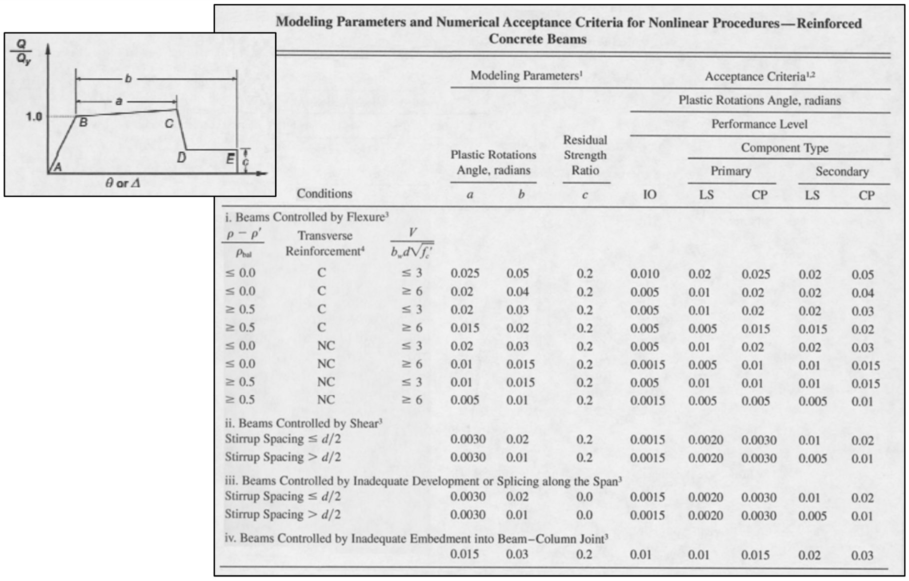
**Considerations (con't)**

- Which software, which solver?
- Step-by-step numerical integration, 'fast non-linear'?
- Convergence (may be converging to false result)

- Time step size (< .01 $T_1$ , even increment of TH data)
- Damping – modal, Rayleigh, hysteretic, added viscous damping
- Coping with data

- Member modeling (large variety in means to model non-linearity)
- Non-linear properties; backbone curves; how do we get these? (literature, software, testing)
- Strength degradation per cycle; difficult to model

## Backbone Curves from FEMA 356, ASCE/SEI 41-06

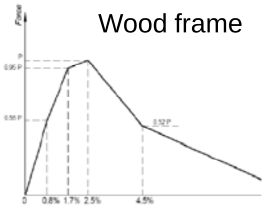


**Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams**

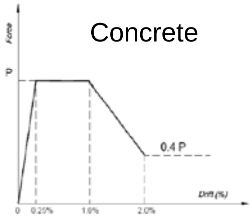
| Conditions | | | Modeling Parameters[1] | | | Acceptance Criteria[1,2] | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | Plastic Rotations Angle, radians | | Residual Strength Ratio | Plastic Rotations Angle, radians | | | | |
| | | | | | | Performance Level | | | | |
| | | | | | | | Component Type | | | |
| | | | | | | | Primary | | Secondary | |
| | | | *a* | *b* | *c* | IO | LS | CP | LS | CP |
| i. Beams Controlled by Flexure[3] | | | | | | | | | | |
| $\frac{\rho - \rho'}{\rho_{bal}}$ | Transverse Reinforcement[4] | $\frac{V}{b_w d\sqrt{f_c'}}$ | | | | | | | | |
| ≤ 0.0 | C | ≤ 3 | 0.025 | 0.05 | 0.2 | 0.010 | 0.02 | 0.025 | 0.02 | 0.05 |
| ≤ 0.0 | C | ≥ 6 | 0.02 | 0.04 | 0.2 | 0.005 | 0.01 | 0.02 | 0.02 | 0.04 |
| ≥ 0.5 | C | ≤ 3 | 0.02 | 0.03 | 0.2 | 0.005 | 0.01 | 0.02 | 0.02 | 0.03 |
| ≥ 0.5 | C | ≥ 6 | 0.015 | 0.02 | 0.2 | 0.005 | 0.005 | 0.015 | 0.015 | 0.02 |
| ≤ 0.0 | NC | ≤ 3 | 0.02 | 0.03 | 0.2 | 0.005 | 0.01 | 0.02 | 0.02 | 0.03 |
| ≤ 0.0 | NC | ≥ 6 | 0.01 | 0.015 | 0.2 | 0.0015 | 0.005 | 0.01 | 0.01 | 0.015 |
| ≥ 0.5 | NC | ≤ 3 | 0.01 | 0.015 | 0.2 | 0.005 | 0.01 | 0.01 | 0.01 | 0.015 |
| ≥ 0.5 | NC | ≥ 6 | 0.005 | 0.01 | 0.2 | 0.0015 | 0.005 | 0.005 | 0.005 | 0.01 |
| ii. Beams Controlled by Shear[3] | | | | | | | | | | |
| Stirrup Spacing ≤ *d*/2 | | | 0.0030 | 0.02 | 0.2 | 0.0015 | 0.0020 | 0.0030 | 0.01 | 0.02 |
| Stirrup Spacing > *d*/2 | | | 0.0030 | 0.01 | 0.2 | 0.0015 | 0.0020 | 0.0030 | 0.005 | 0.01 |
| iii. Beams Controlled by Inadequate Development or Splicing along the Span[3] | | | | | | | | | | |
| Stirrup Spacing ≤ *d*/2 | | | 0.0030 | 0.02 | 0.0 | 0.0015 | 0.0020 | 0.0030 | 0.01 | 0.02 |
| Stirrup Spacing > *d*/2 | | | 0.0030 | 0.01 | 0.0 | 0.0015 | 0.0020 | 0.0030 | 0.005 | 0.01 |
| iv. Beams Controlled by Inadequate Embedment into Beam–Column Joint[3] | | | | | | | | | | |
| | | | 0.015 | 0.03 | 0.2 | 0.01 | 0.01 | 0.015 | 0.02 | 0.03 |

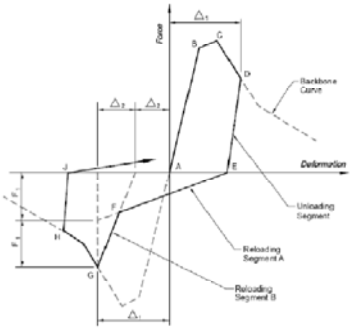
## Sample Backbone Curves – BC Schools Projects

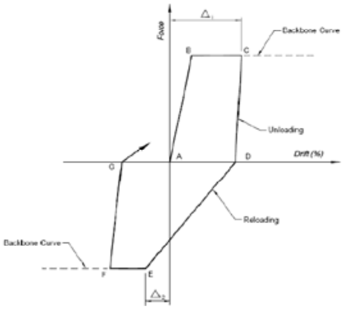
Wood frame



Concrete







## Time History Analysis - Benefits

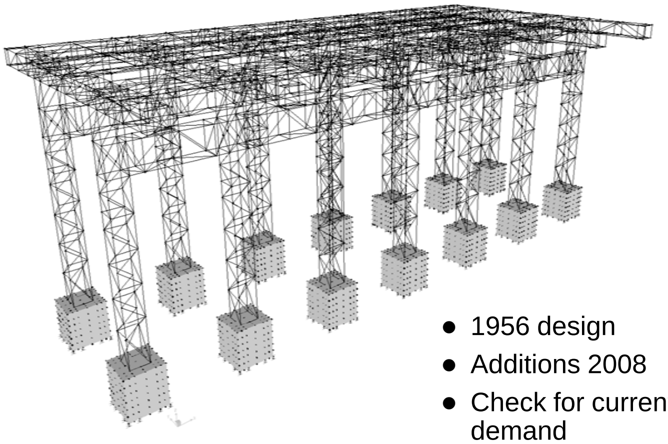
**Considerations (con't)**

- Soil structure interaction (compression-only linear springs, non-linear springs, soil damping)
- Still need to deal with accidental torsion similar to RSA
- Sensitivity ($f_y$, $f_c$, E, I)

- How to use results (peak, median, mean, mean + 1 std dev)
- More checking
- Requires complete independent review by qualified engineering team (from ground motion TH selection, through to design)

## Example #1

**Substation Structure**



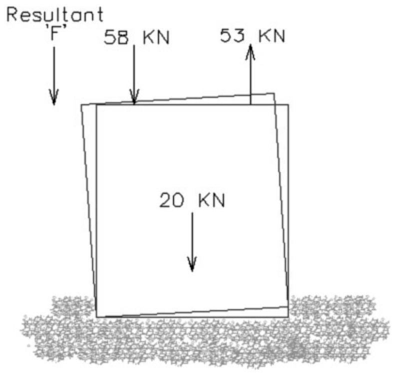
- 1956 design
- Additions 2008
- Check for current seismic demand
- Steel OK

## Example #1

**Static and RS analysis results for one footing**

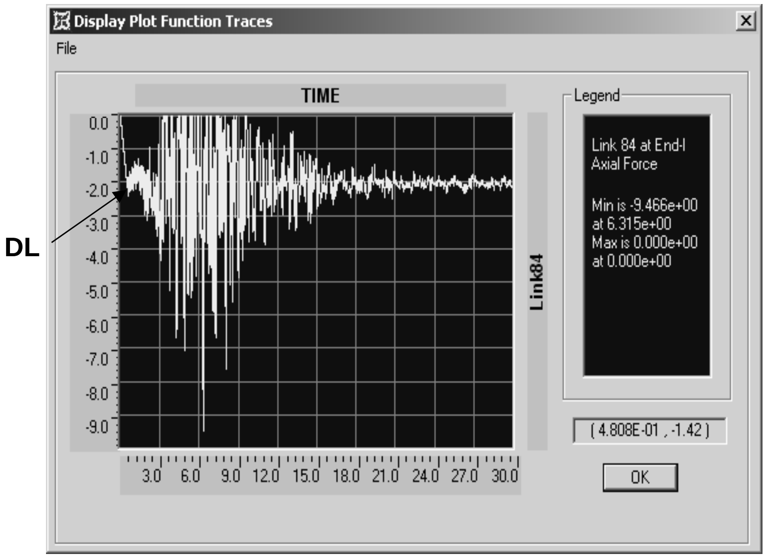
- Overturning problem “unstable”
- Suggests remediation required

- TH analysis confirmed OK with no remediation
- Using compression only springs as only non-linear components



## Example #1

**Compression-Only Soil Element Time History Axial Force (KN)**



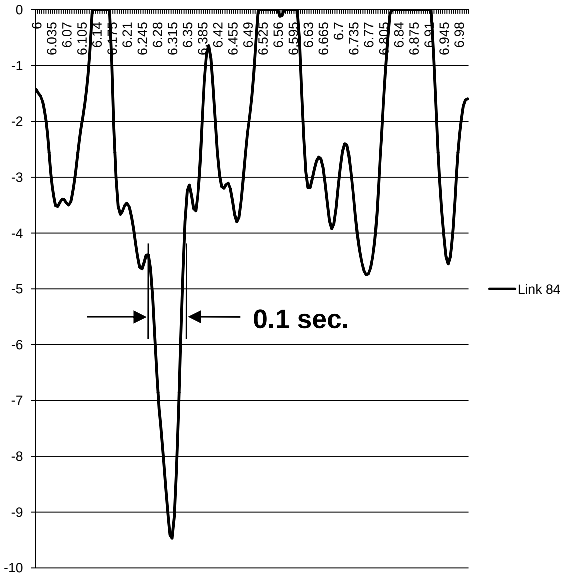
Link 84 at End-I
Axial Force

Min is -9.466e+00
at 6.315e+00
Max is 0.000e+00
at 0.000e+00

( 4.808E-01 , -1.42 )

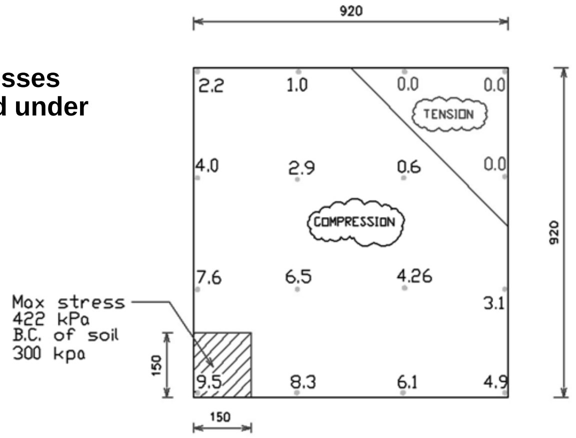
## Example #1

**Peak of 9.5 KN force for a period of 0.1 seconds**



## Example #1

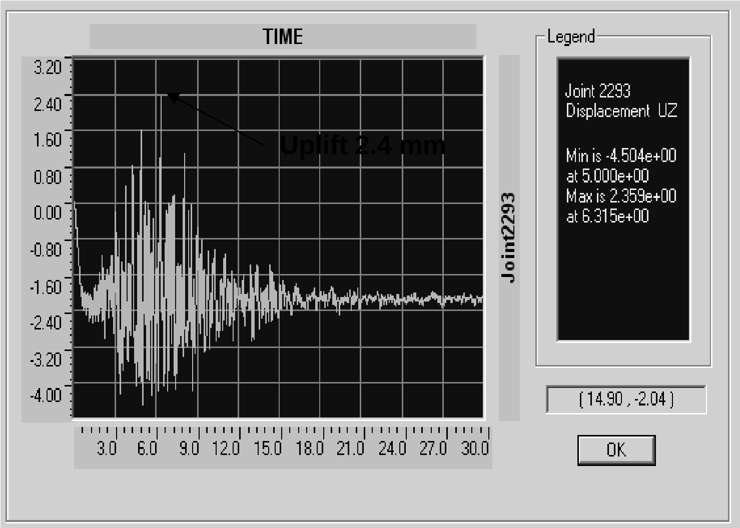
**Max. soil stresses approximated under one footing**



STRESS DISTRIBUTION
AT 6.315 SECOND (KN)

## Example #1

### Edge joint time history displacement (mm)

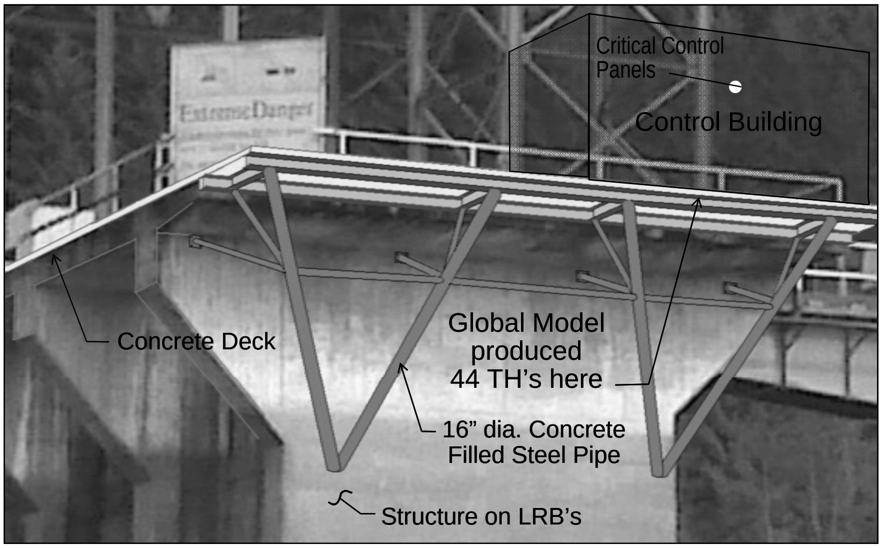


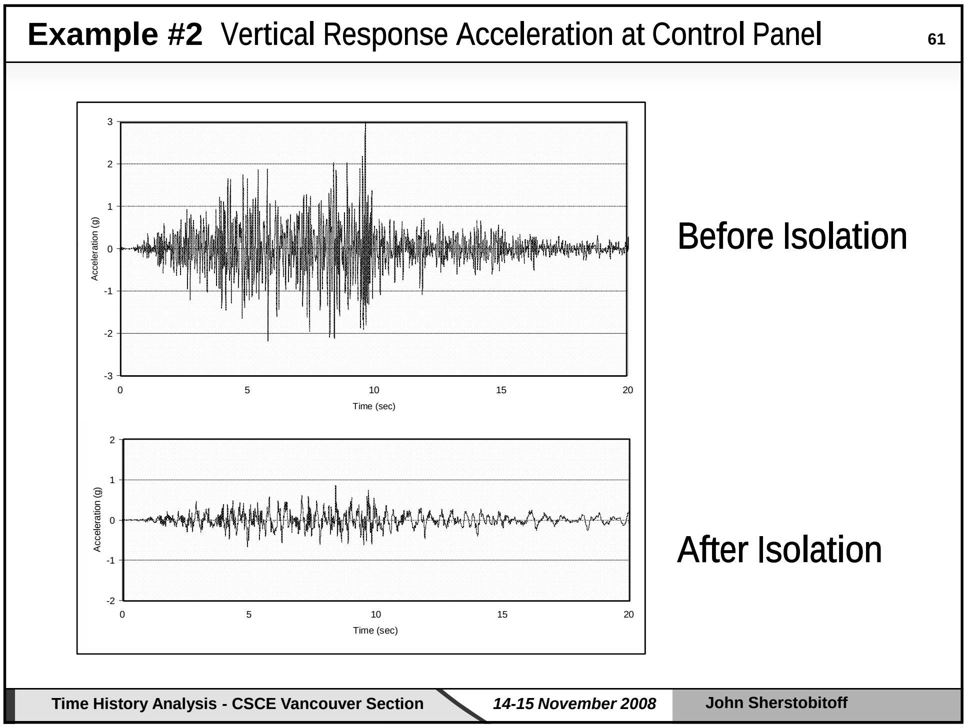
Joint 2293
Displacement UZ

Min is -4.504e+00
at 5.000e+00
Max is 2.359e+00
at 6.315e+00

( 14.90 , -2.04 )

## Example #2 Control Building on Intake Tower; 0.8g PGA Site

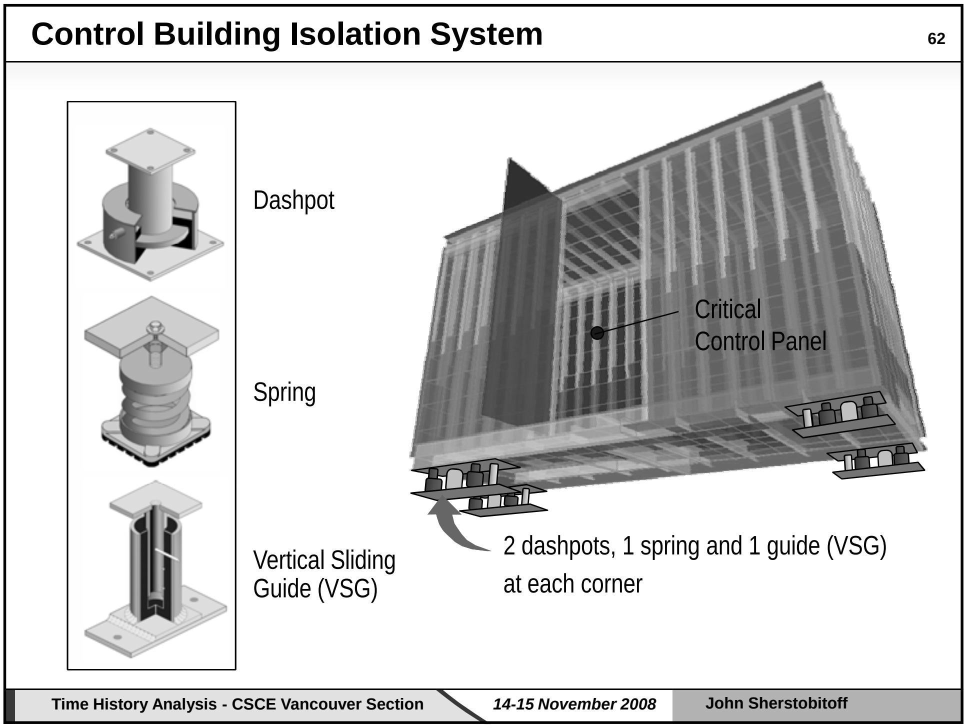




## **Example #2** Vertical Response Acceleration at Control Panel

Before Isolation

After Isolation



## Control Building Isolation System

Dashpot

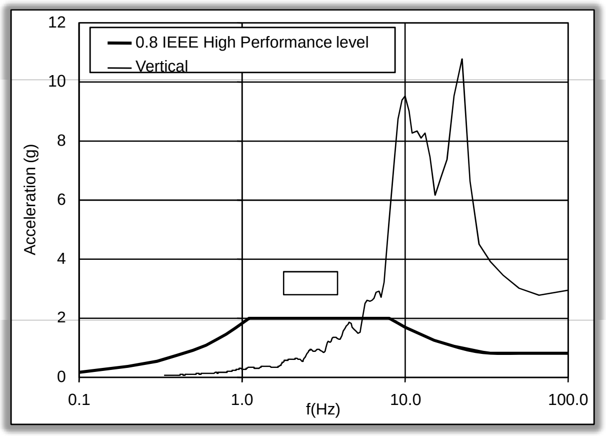
Spring

Vertical Sliding
Guide (VSG)

Critical
Control Panel

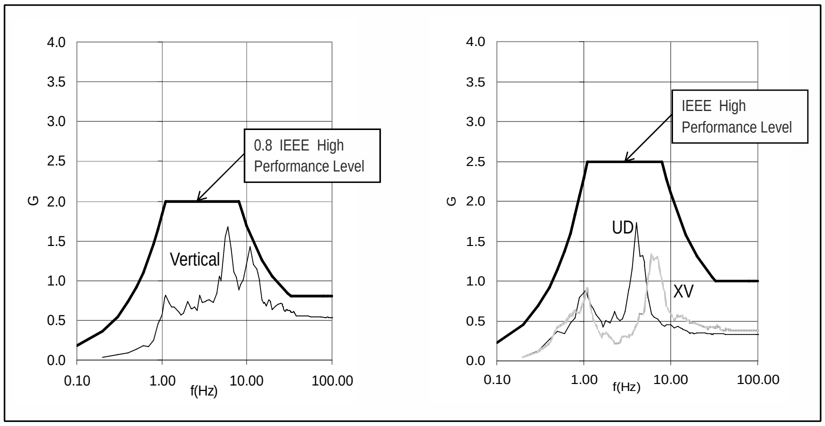
2 dashpots, 1 spring and 1 guide (VSG)
at each corner

## Example #2 No Isolation – Deck Response



UD_SPS_CHI_WNT

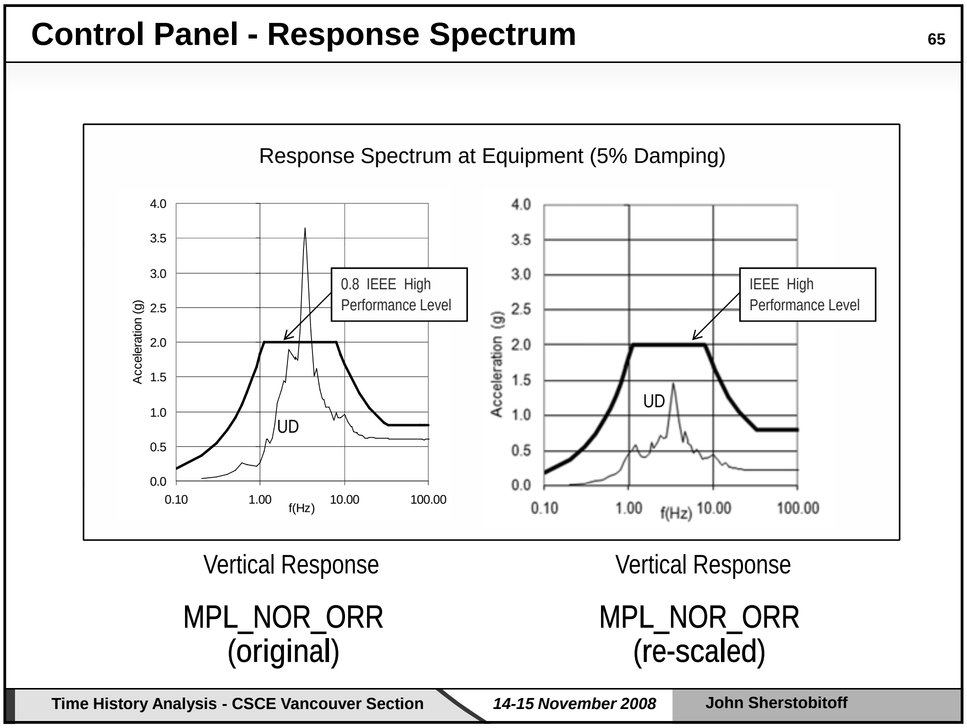
## Example #2 With Isolation



Vertical Response

Horizontal Response

UD_SPS_NOR_5108



## Control Panel - Response Spectrum

65

Response Spectrum at Equipment (5% Damping)

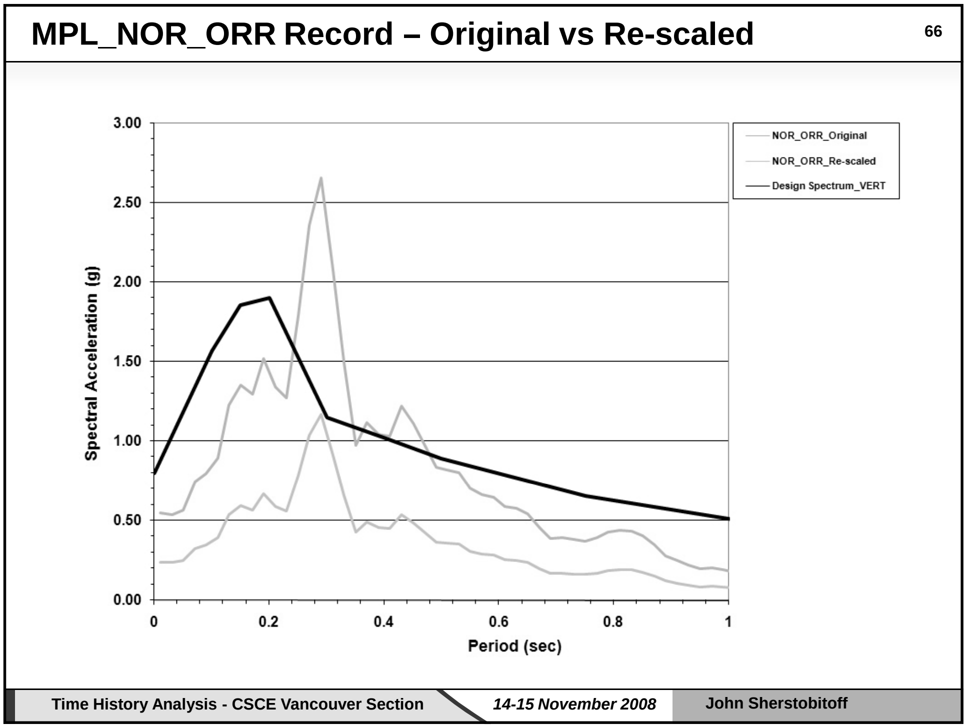
Vertical Response

MPL_NOR_ORR
(original)

Vertical Response

MPL_NOR_ORR
(re-scaled)

Time History Analysis - CSCE Vancouver Section 14-15 November 2008 John Sherstobitoff



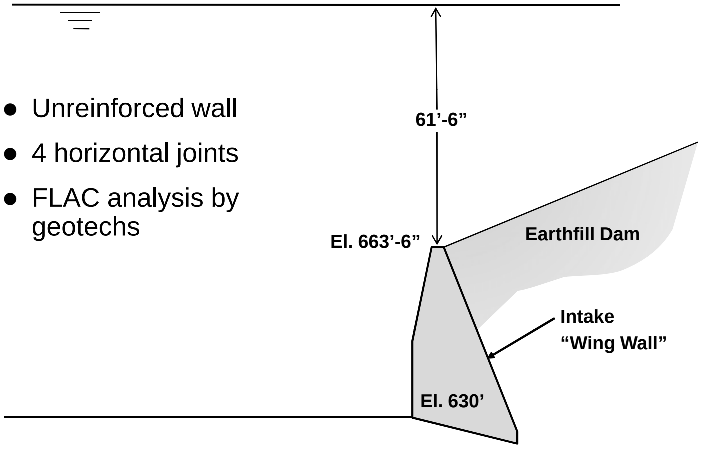
## MPL_NOR_ORR Record – Original vs Re-scaled

66

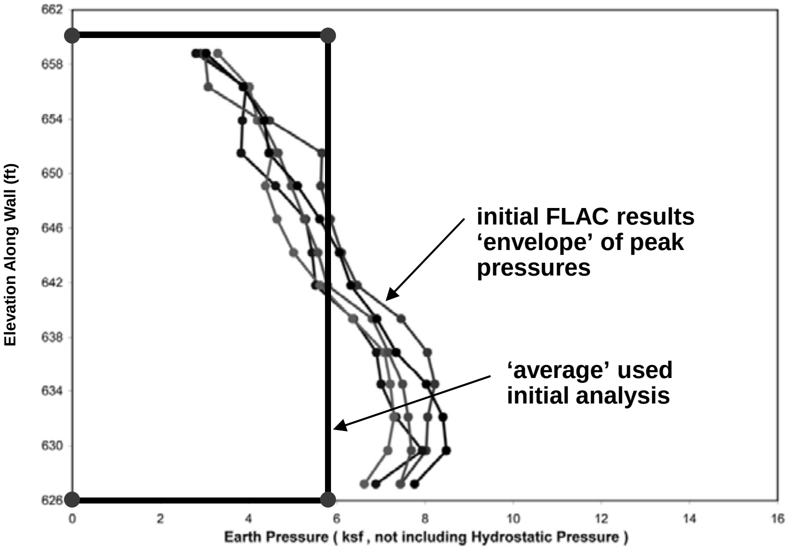
Time History Analysis - CSCE Vancouver Section 14-15 November 2008 John Sherstobitoff

## Example #3 Underwater Retaining Wall; 0.8g PGA Site

- Unreinforced wall
- 4 horizontal joints
- FLAC analysis by geotechs



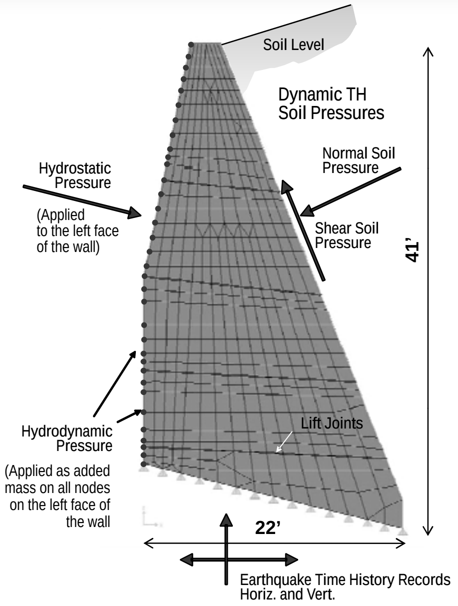
## Peak Horizontal Seismic Pressures on Intake Wing Wall



initial FLAC results 'envelope' of peak pressures

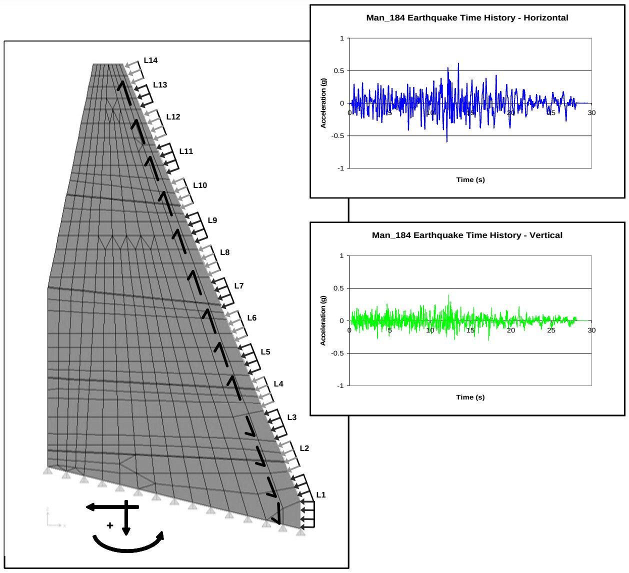
'average' used initial analysis

## Applied Loading



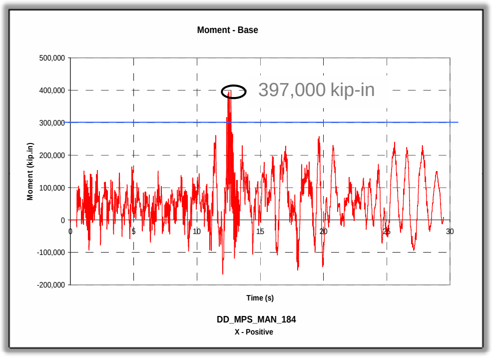
## Dynamic Soil Pressure Loading

- Normal and shear soil time history pressures applied at 14 elevations
- 12 records used, (SP and MP)
- Both positive and negative horizontal
- Total of 24 time histories



## Design

- Consider design moment of 300,000 kip-in
- 24% lower than peak of 397,000 kip-in
- 11 of 24 records contained one peak that exceeds 300,000
- Lower than average of 24 peak moments



## Summary

### Time History Analysis

- Careful structural modeling and sensitivity analysis.
- Appropriate selection of ground motion records.
- Thorough knowledge and familiarity with computer software employed.
- A very good tool to attain reasonably accurate assessment of inelastic seismic response.
- Special care should be exercised to make sure that the design and detailing can achieve the computed response.

## Schedule for Today

. . . the Main Course

- Ground motion records (Ventura)
- Selection of TH (Little)
- Matching UHRS (Wightman)
- Geotechnical aspects (Finn)
- Backbone curves (Adebar)
- Where to input TH (Naeim)
- Software and modeling (Rezai)
- Pushover vs TH (Sinclair)

## Schedule for Tomorrow

. . . the Dessert

- Bridges (Zhu)
- Tall buildings (Mutrie / Hoffman)
- Low rise buildings, misc. (Rezai)

# TIME HISTORY ANALYSIS

## LECTURE # 2

## Origin and Interpretation of Ground motion time histories

**Dr. Carlos E. Ventura, P.E., P.Eng.**
**Department of Civil Engineering**
**The University of British Columbia**

Dr. Carlos Ventura is a Civil Engineer with specializations in structural dynamics and earthquake engineering. He has been a faculty member of the UBC Department of Civil Engineering since 1992. He is currently the Director of the Earthquake Engineering Research Facility (EERF) at UBC, and is the author numerous technical and non technical papers and reports on earthquake engineering, structural dynamics and structural testing. He is a member of several national and international professional societies and advisory committees. Dr. Ventura has conducted research for more than twenty five years in the dynamic behaviour and analysis of structural systems subjected to extreme dynamic loads, including severe earthquakes. Dr. Ventura's research work includes experimental studies in the field and in the laboratory of structural systems and components.

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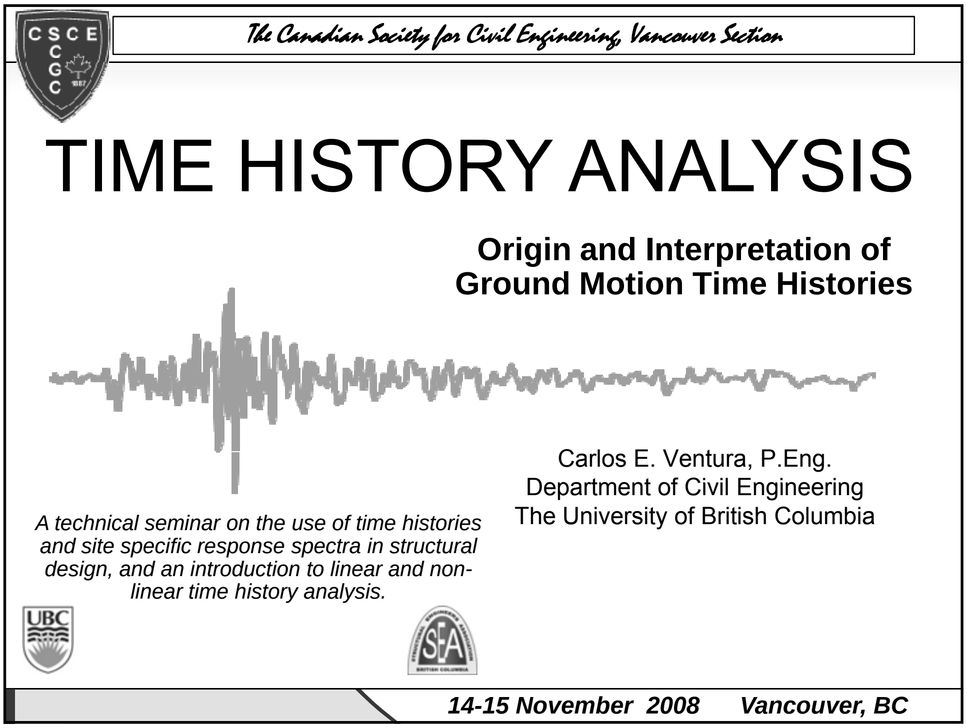


The Canadian Society for Civil Engineering, Vancouver Section

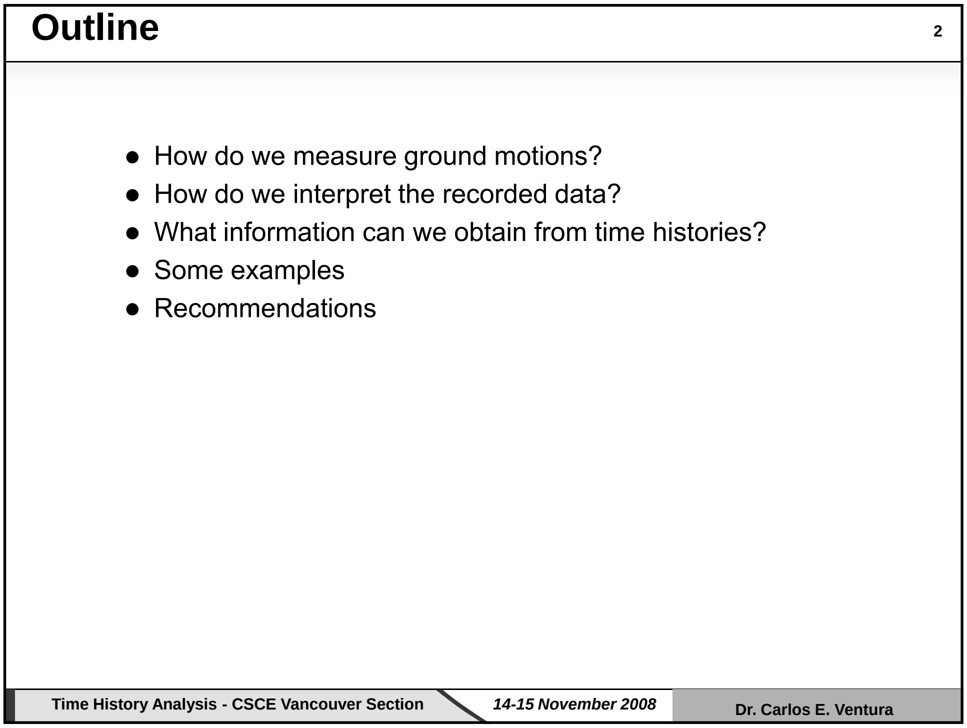
# TIME HISTORY ANALYSIS

## Origin and Interpretation of Ground Motion Time Histories

Carlos E. Ventura, P.Eng.
Department of Civil Engineering
The University of British Columbia

*A technical seminar on the use of time histories and site specific response spectra in structural design, and an introduction to linear and non-linear time history analysis.*

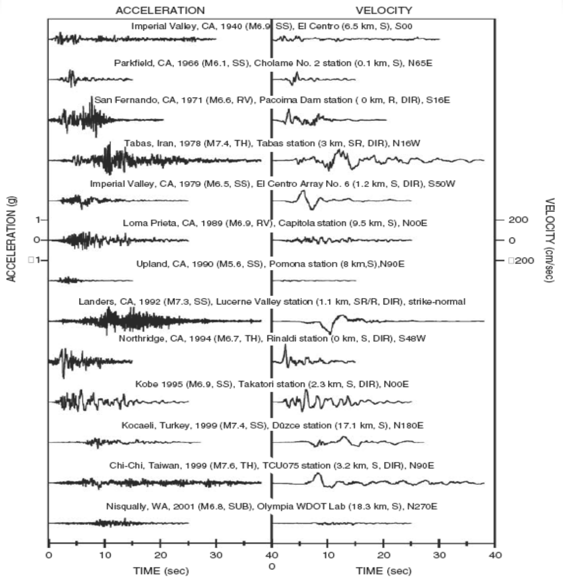
***14-15 November 2008 Vancouver, BC***



## Outline

- How do we measure ground motions?
- How do we interpret the recorded data?
- What information can we obtain from time histories?
- Some examples
- Recommendations

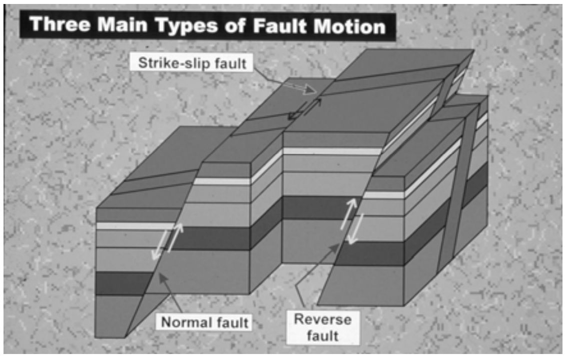
## Example of Recorded Ground Motions



## Tectonic Plates & Earthquakes

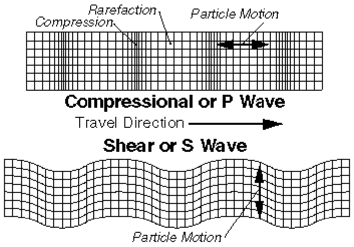
- ***The majority of the world's earthquakes occur near tectonic plate boundaries, but***
- ***Earthquakes also occur within the interior of tectonic plates***

***Different types of faults can exist within plates, depending on tectonic stress regime***



# Types of Seismic waves

- <u>P-waves:</u>
  - called compressional, or push-pull waves
  - Propagate parallel to the direction in which the wave is moving
  - Move through solids, liquids
- <u>S-waves:</u>
  - Called shear waves
  - Propagate the movement perpendicular to the direction in which the wave is moving
- <u>Surface waves</u> (Love and Rayleigh waves).
  - Complex motion
  - Up-and-down and side-to-side
  - Slowest
  - Most damage to structures, buildings



# Seismic Sensors

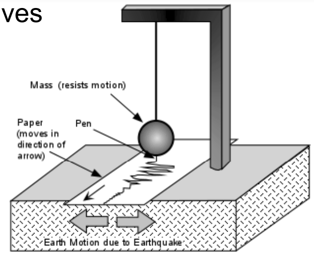
- Displacement Transducers /accelerometers
  - Some devices produce an output voltage proportional to the mass displacement relative to the case
  - Other devices measure acceleration of the case.
- Velocity Transducers – traditional type
  - In most cases a cylindrical coil, movable parallel to its axis within the field of a fixed permanent magnet.
  - Produce an induced voltage proportional to the rate of the magnetic flux change within the coil, hence proportional to the velocity of the coil in motion relative to the magnet.
- Seismometer Demo

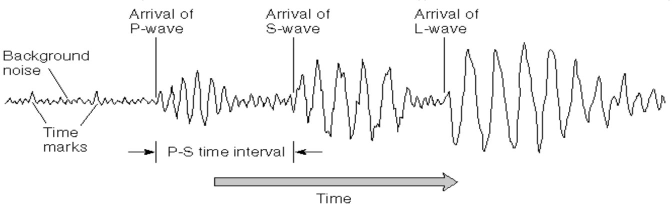
  http://www.ifg.tu-clausthal.de/java/seis/sdem_app-e.html

# Seismometer

**Seismometers:** instruments that detect seismic waves

- A basic seismometer consists of a freely suspending mass from a frame attached to the ground.
- The relative motion of the frame with respect to the heavy mass is printed as a seismogram.

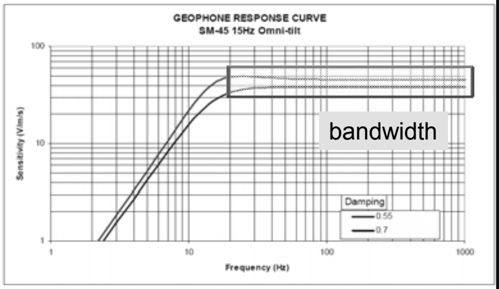




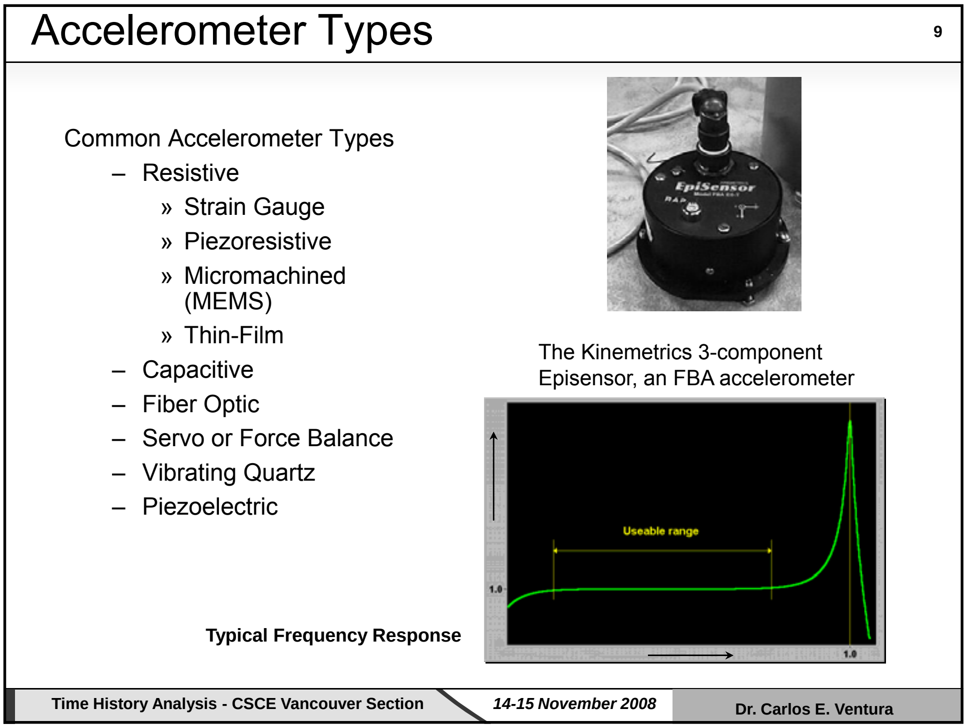
A Schematic Seismograph Showing the Arrival, Order, and Pattern Produced by P-, S-, and L- Waves

# Seismometers

- Modern digital broadband seismographs are capable of recording almost the whole seismological spectrum (50 Hz – 300 s).
- Their resolution of 24 bits (high dynamic range) allows for precise recording of small quakes, as well as unsaturated registration of the largest ones.



GEOPHONE RESPONSE CURVE
SM-45 15Hz Omni-tilt

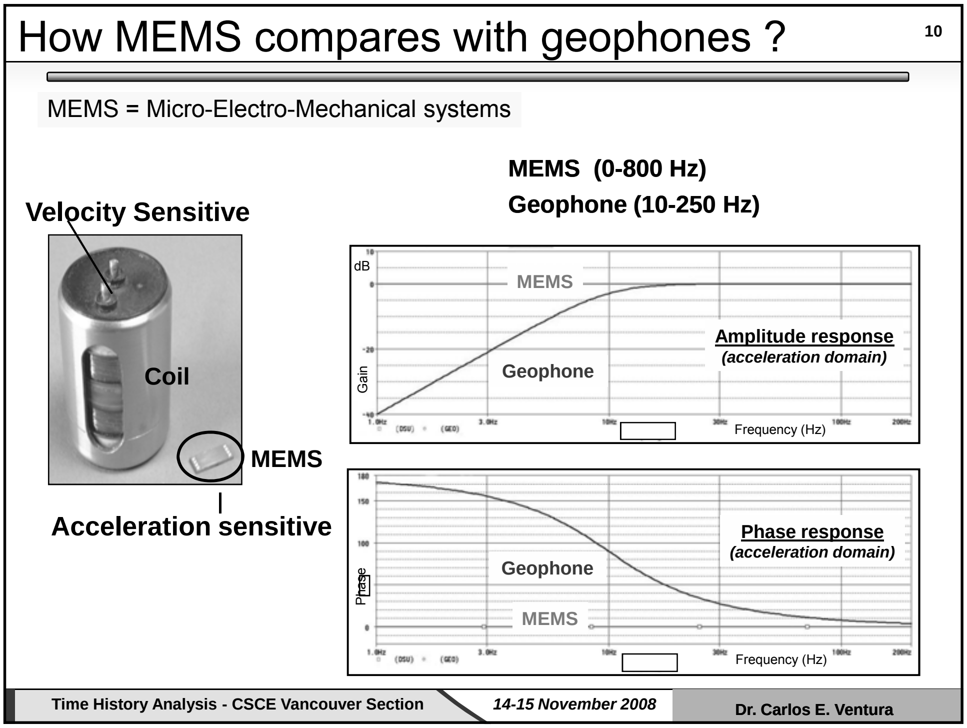


## Accelerometer Types

Common Accelerometer Types
- Resistive
  - » Strain Gauge
  - » Piezoresistive
  - » Micromachined (MEMS)
  - » Thin-Film
- Capacitive
- Fiber Optic
- Servo or Force Balance
- Vibrating Quartz
- Piezoelectric

The Kinematrics 3-component Episensor, an FBA accelerometer

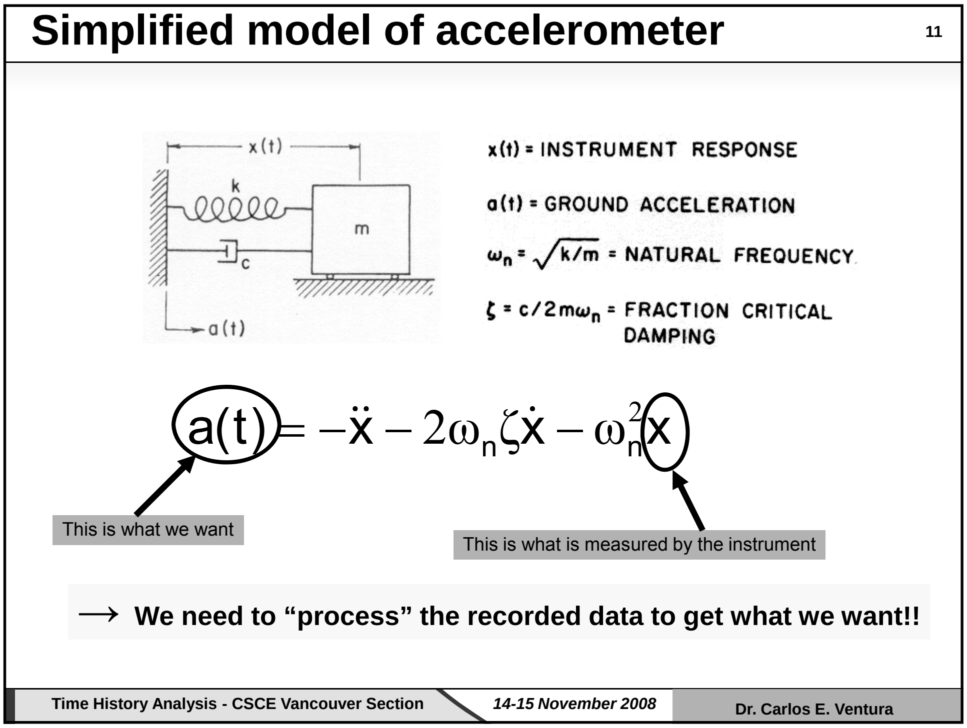
Typical Frequency Response



## How MEMS compares with geophones ?

MEMS = Micro-Electro-Mechanical systems

**MEMS (0-800 Hz)**
**Geophone (10-250 Hz)**



## Simplified model of accelerometer

x(t) = INSTRUMENT RESPONSE

a(t) = GROUND ACCELERATION

$\omega_n = \sqrt{k/m}$ = NATURAL FREQUENCY

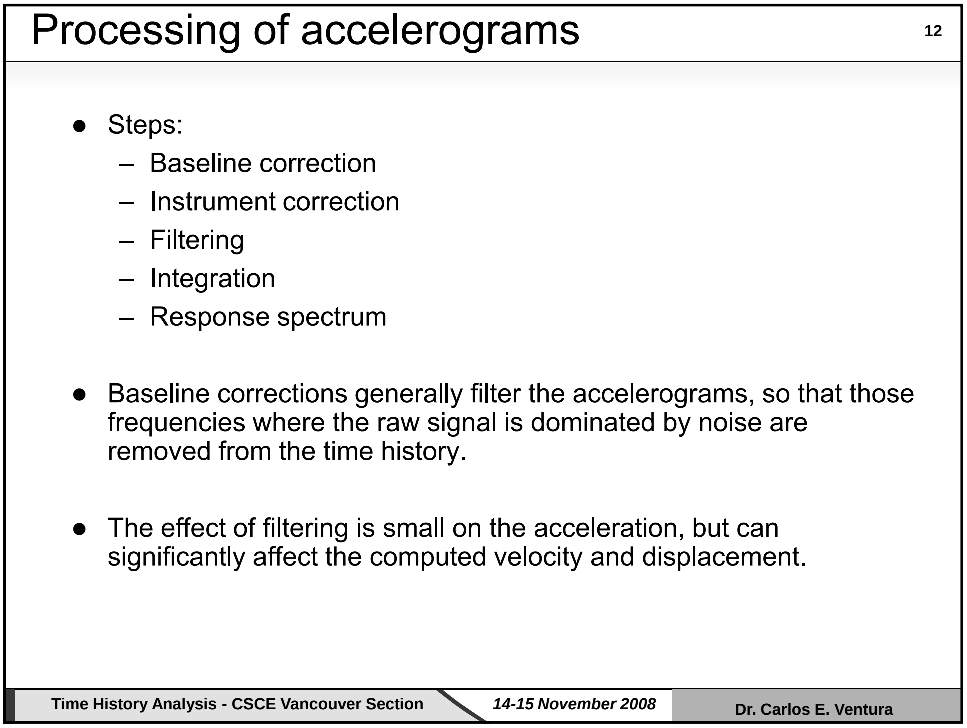
$\zeta = c/2m\omega_n$ = FRACTION CRITICAL DAMPING

$$\boxed{a(t)} = -\ddot{x} - 2\omega_n\zeta\dot{x} - \omega_n^2\boxed{x}$$

This is what we want

This is what is measured by the instrument

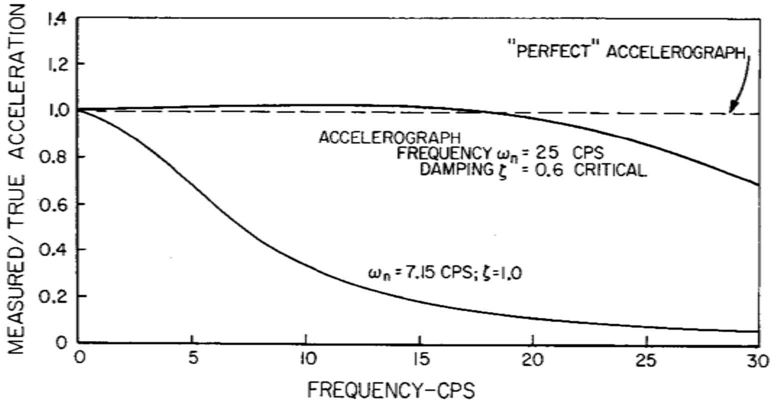
→ **We need to “process” the recorded data to get what we want!!**



## Processing of accelerograms

- Steps:
  - Baseline correction
  - Instrument correction
  - Filtering
  - Integration
  - Response spectrum
- Baseline corrections generally filter the accelerograms, so that those frequencies where the raw signal is dominated by noise are removed from the time history.
- The effect of filtering is small on the acceleration, but can significantly affect the computed velocity and displacement.

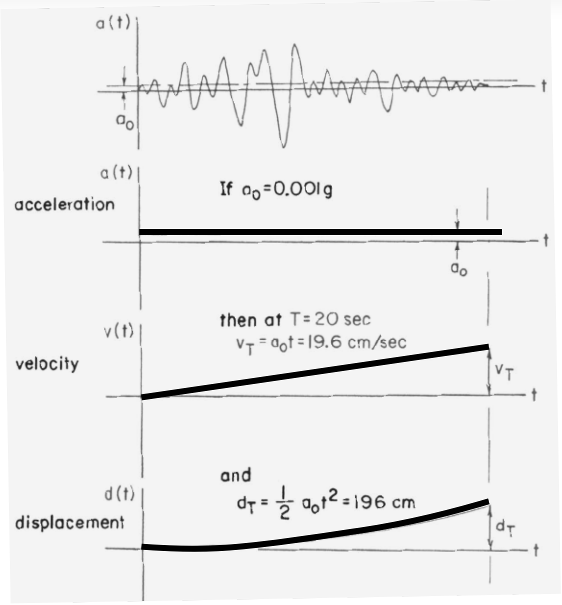
## Correction due to Instrument Response



**Corrections to the recorded motions are made primarily:**

- To remove instrument response
- To account for base line shift

## Base Line Correction



Recorded acceleration may not have a “zero” mean value

Mean value of recorded acceleration – a constant value

A constant acceleration value results in a linear velocity

.. and in a parabolic displacement

After Hudson

## Filtering

- A numerical process that is applied to a time series (in this case an accelerogram). The process removes the contributions of certain frequencies to the time series.
  - High pass filter: removes low frequencies (i.e. frequencies below the filter frequency $f_f$ ), but does not affect the high frequencies.
  - Low pass filter: removes high frequencies (above $f_f$ ), but does not affect the low frequencies.
  - Filter response is generally not “sharp”. In other words, there is a range of frequencies that are partly removed.
- Filter frequencies are often selected on the basis of noise models.
- When a record is filtered, signal is removed as well as noise.
  - If a particular frequency is important to a structure, then the accelerogram you use to test it should not have that frequency filtered out.
- Modern digital accelerograms require much less filtering than older analog accelerograms.

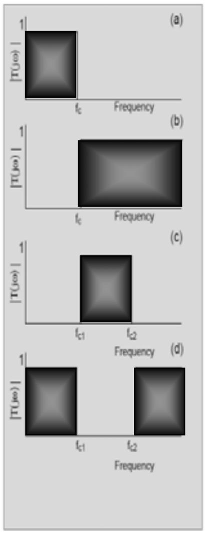
## Types of Filters

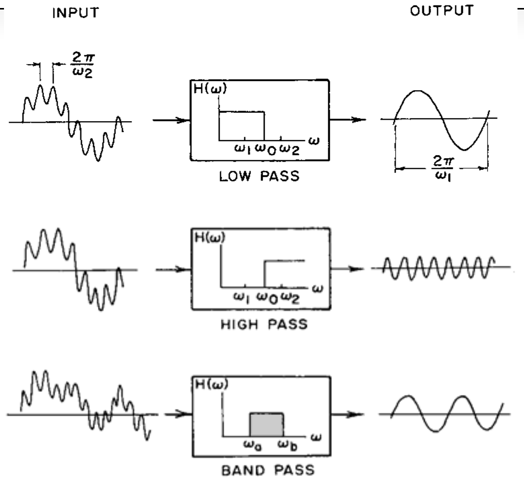
**Lowpass**

**Highpass**

**Band Pass**

**Band Reject**





High-pass filters generally have small effects on accelerations.

The effect is much greater on velocity and displacement.

## How do I find out how the data has been processed?

### Example 1: ground motion record from the 1999 Taiwan Chi-Chi Eq.

**Corrected accelerogram** 75049-b0073-01008. **Chan 2: 360 Deg** from Uncorrected Accelerogram Data
Processed: 08/30/01, CDMG TCU049
Chi-Chi, Taiwan Earthquake, 21 Sep 1999 (Avol1 v4.6 7/01 CSMIP)
Taiwan Central Weather Bureau CWB-TSMIP (Origin: 09/20/99 17:47:15.9 UTC; CWB)
75049-b0073-01008. Start time: 9/20/99, 17:47:04.0 UTC
Station No. 75049 24.179N, 120.690E A900 s/n 73 (3 Channels)
Taichung - Chiaoshiao School, TCU049 Chan 2: 360 Deg
Chi-Chi, Taiwan Earthquake, 21 Sep 1999 Mon Sep 20, 1999 10:47 PDT
Hypocenter(CWB): 23.853N,120.816E, H=8km ML=7.3; MS,MW=7.7; mb=6.5 (CWB)
**Instr Period = .0222 sec, Damping = .700, Sensitivity = 2.25 v/g**
Record length =150.000 sec.
Uncor Max = -.247 g, at 34.060 sec.
RMS accel of (uncor) record = .
**Accelerogram bandpass filtered with 3 dB pts at .04 and 40.00 cyc/sec**
15000 points of instrument- and baseline-corrected accel, veloc and displ data
At equally-spaced intervals of .010 sec.
Peak acceleration = -238.352 cm/sec/sec at 34.060 sec.
Peak velocity = 63.063 cm/sec at 35.450 sec.
Peak displacement = -43.496 cm at 52.010 sec.
Initial velocity = .039 cm/sec; Initial displacement = -.052 cm
Chi-Chi, Taiwan Earthquake, 21 Sep 1999 Mon Sep 20, 1999 10:47 PDT

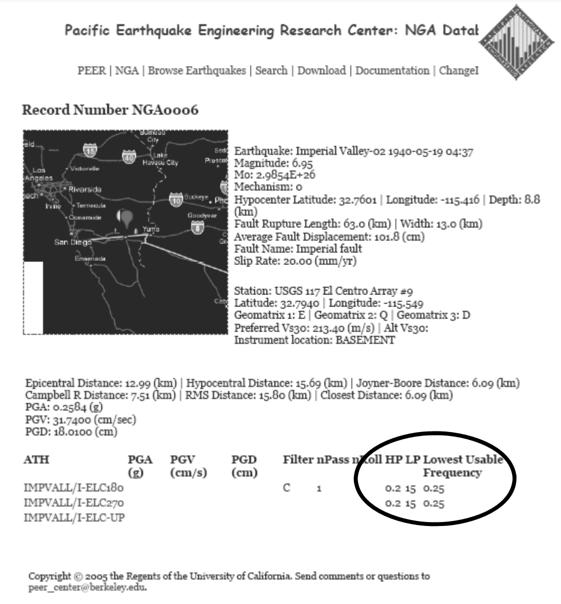
75049-b0073-01008. Taichung - Chiaoshiao School, TCU049 Chan 2: 360 Deg

## How do I find out how the data has been processed?

### Example 2:

ground motion record from the 1940 El Centro Eq.

*Obtained from the PEER NGA Database*



Pacific Earthquake Engineering Research Center: NGA Datal

PEER | NGA | Browse Earthquakes | Search | Download | Documentation | ChangeI

**Record Number NGA0006**

Earthquake: Imperial Valley-02 1940-05-19 04:37
Magnitude: 6.95
Mo: 2.9854E+26
Mechanism: 0
Hypocenter Latitude: 32.7601 | Longitude: -115.416 | Depth: 8.8 (km)
Fault Rupture Length: 63.0 (km) | Width: 13.0 (km)
Average Fault Displacement: 101.8 (cm)
Fault Name: Imperial fault
Slip Rate: 20.00 (mm/yr)

Station: USGS 117 El Centro Array #9
Latitude: 32.7940 | Longitude: -115.549
Geomatrix 1: E | Geomatrix 2: Q | Geomatrix 3: D
Preferred Vs30: 213.40 (m/s) | Alt Vs30:
Instrument location: BASEMENT

Epicentral Distance: 12.99 (km) | Hypocentral Distance: 15.69 (km) | Joyner-Boore Distance: 6.09 (km)
Campbell R Distance: 7.51 (km) | RMS Distance: 15.80 (km) | Closest Distance: 6.09 (km)
PGA: 0.2584 (g)
PGV: 31.7400 (cm/sec)
PGD: 18.0100 (cm)

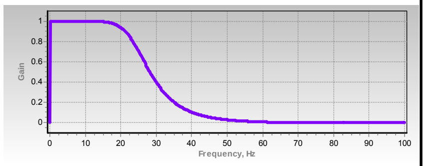
| ATH | PGA (g) | PGV (cm/s) | PGD (cm) | Filter | nPass | nRoll | HP | LP | Lowest Usable Frequency |
|---|---|---|---|---|---|---|---|---|---|
| IMPVALL/I-ELC180 | | | | C | 1 | | 0.2 | 15 | 0.25 |
| IMPVALL/I-ELC270 | | | | | | | 0.2 | 15 | 0.25 |
| IMPVALL/I-ELC-UP | | | | | | | | | |

# China, Wenchuan Earthquake (May 12, 2008)

## *STATION: 051WCW*

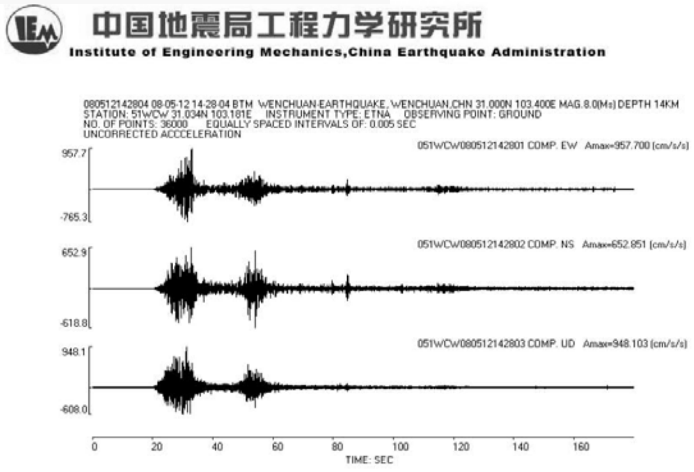
CORDINATES: 31.04N 103.18E
SITE TYPE: ALLUVIUM
DURATION OF RECORD: 180 SEC (but only 160 sec are displayed here)
PRE-EVENT TIME: 20 SEC
ACCELERATION UNITS: CM/SEC$^2$
NO. OF POINTS: 36000
EQUALLY SPACED INTERVALS OF: 0.005 SEC

Records processing parameters:
- Base line correction applied to the record (linear correction)
- Filter Type : Butterworth, Bandpass
  - Order: 4
  - Low Frequency: 0.1 Hz
  - High Frequency: 25 Hz



# Uncorrected Acceleration

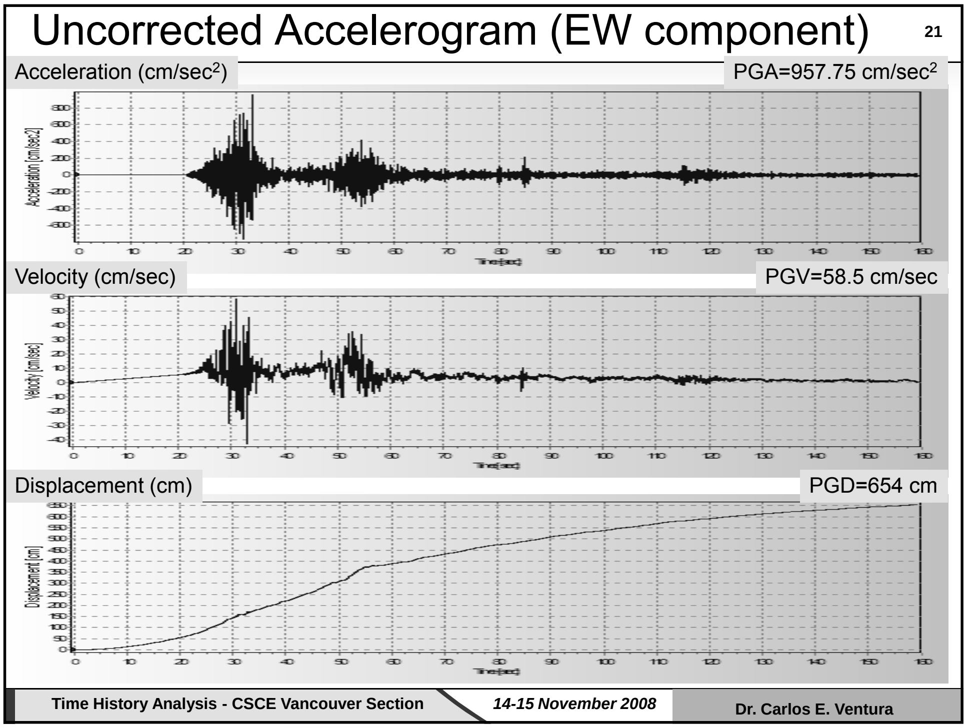
Instrument correction has been applied



中国地震局工程力学研究所
Institute of Engineering Mechanics,China Earthquake Administration

080512142804 08-05-12 14-28-04 BTM WENCHUAN-EARTHQUAKE, WENCHUAN,CHN 31.000N 103.400E MAG.8.0(Ms) DEPTH 14KM
STATION: 51WCW 31.034N 103.181E INSTRUMENT TYPE: ETNA OBSERVING POINT: GROUND
NO. OF POINTS: 36000 EQUALLY SPACED INTERVALS OF: 0.005 SEC
UNCORRECTED ACCCELERATION

Wolong Station (22.2km epicenter distance, 1.09 fault distance)

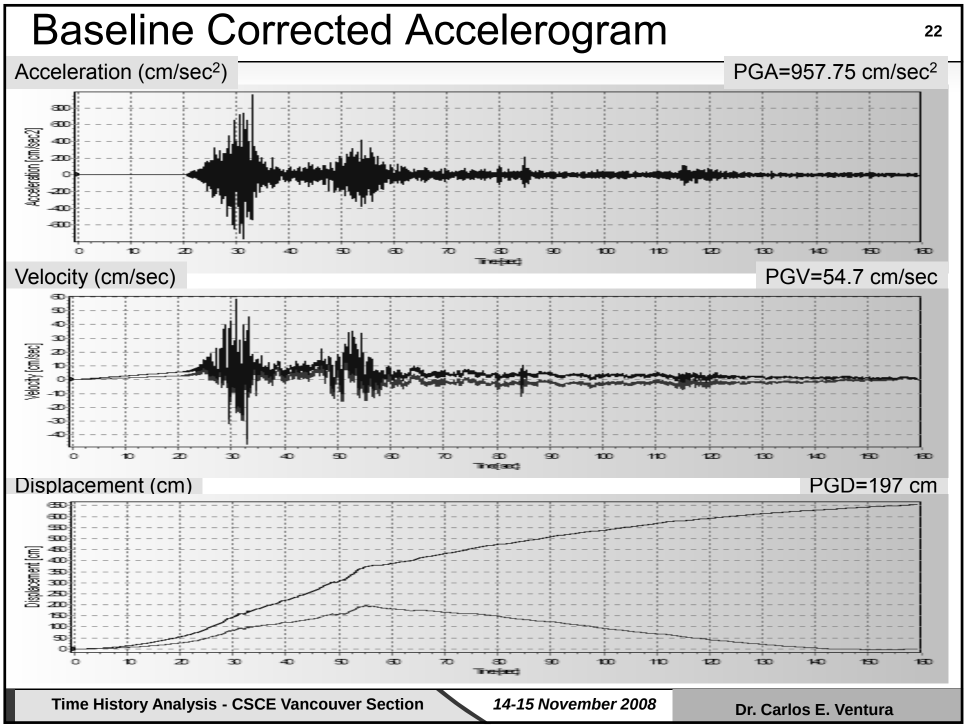


## Uncorrected Accelerogram (EW component)

Acceleration (cm/sec$^2$) PGA=957.75 cm/sec$^2$

Velocity (cm/sec) PGV=58.5 cm/sec

Displacement (cm) PGD=654 cm



## Baseline Corrected Accelerogram

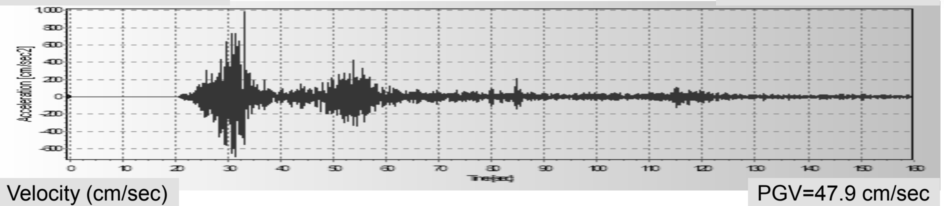
Acceleration (cm/sec$^2$) PGA=957.75 cm/sec$^2$

Velocity (cm/sec) PGV=54.7 cm/sec

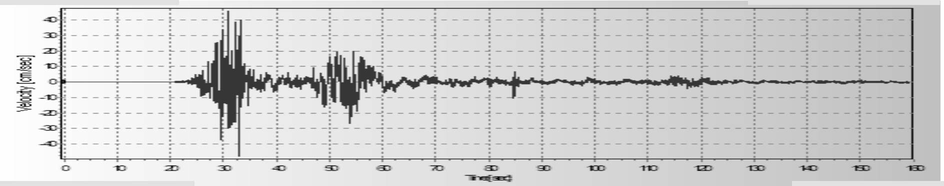
Displacement (cm) PGD=197 cm

## Baseline Corrected and Filtered Accelerogram

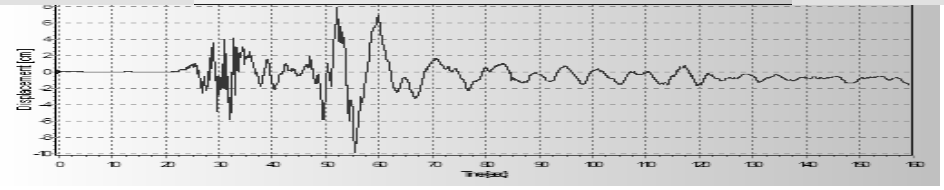
Acceleration (cm/sec$^2$) PGA=987.63 cm/sec$^2$



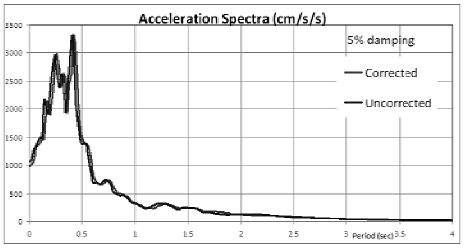
Velocity (cm/sec) PGV=47.9 cm/sec

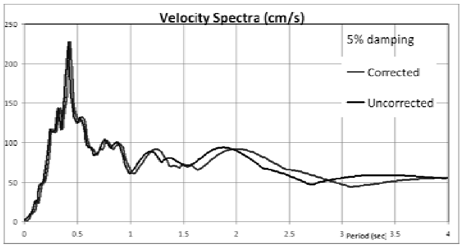


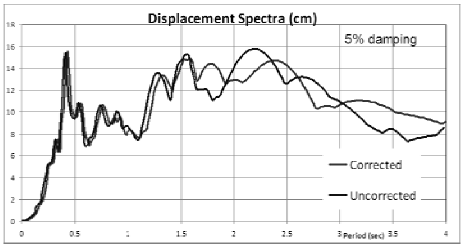
Displacement (cm) PGD=9.85 cm

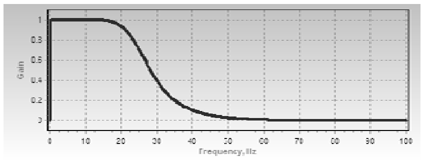


## Effect of Baseline Correction and Filtering

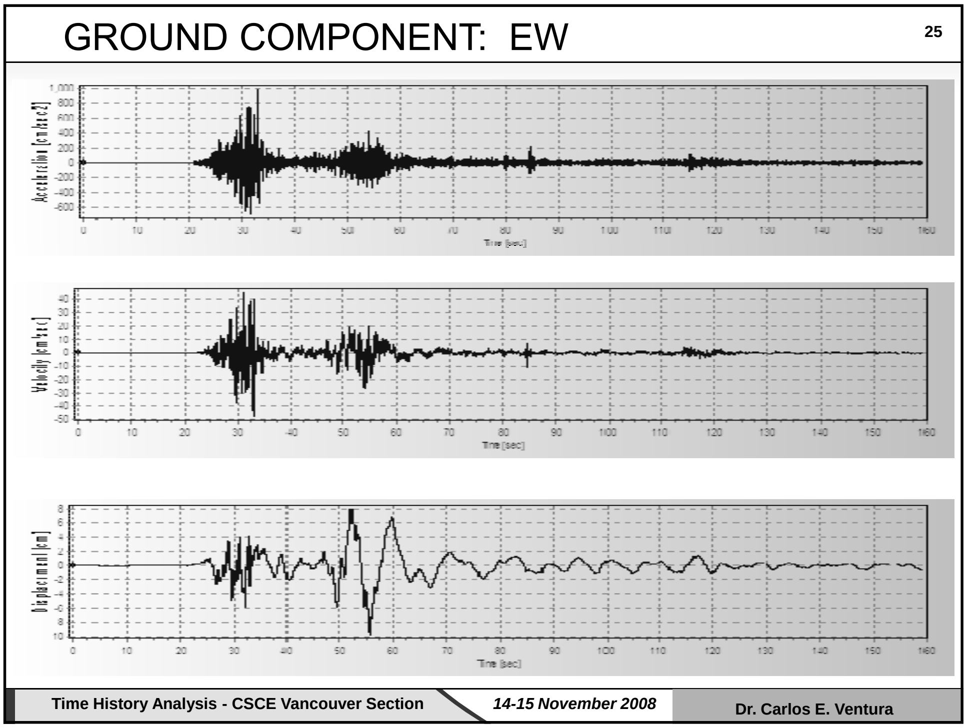




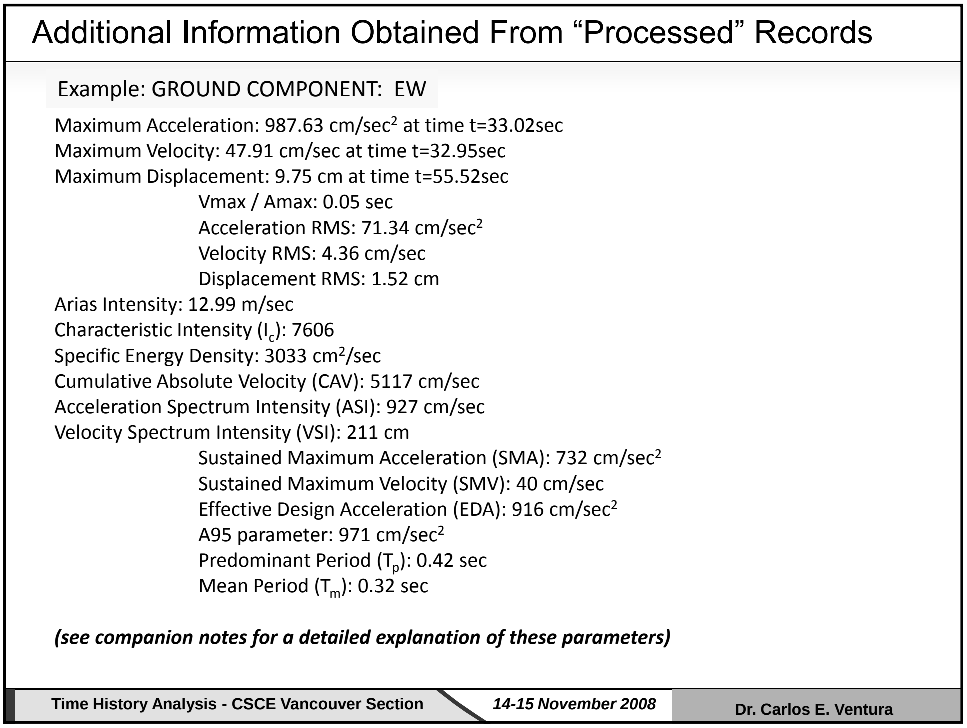




Bandpass Filter (0.1 to 25 Hz)



## GROUND COMPONENT: EW



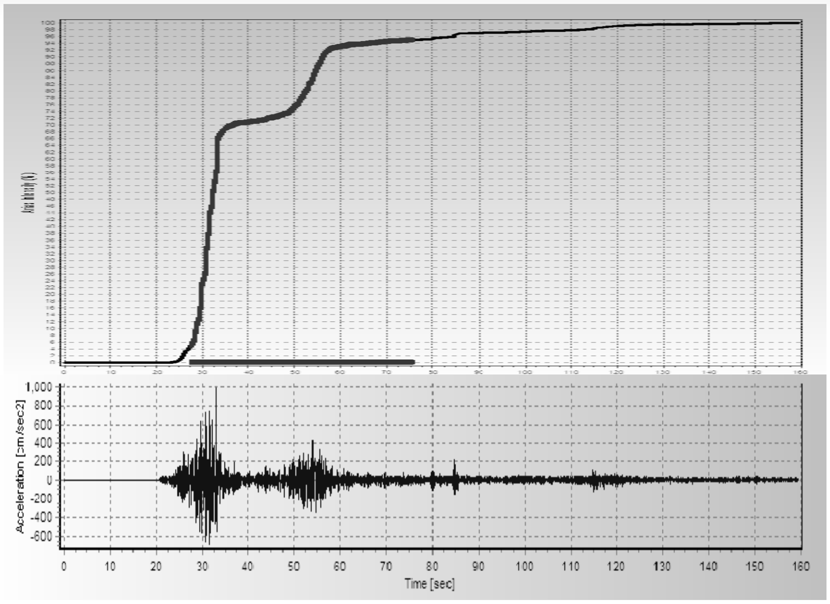
## Additional Information Obtained From “Processed” Records

Example: GROUND COMPONENT: EW

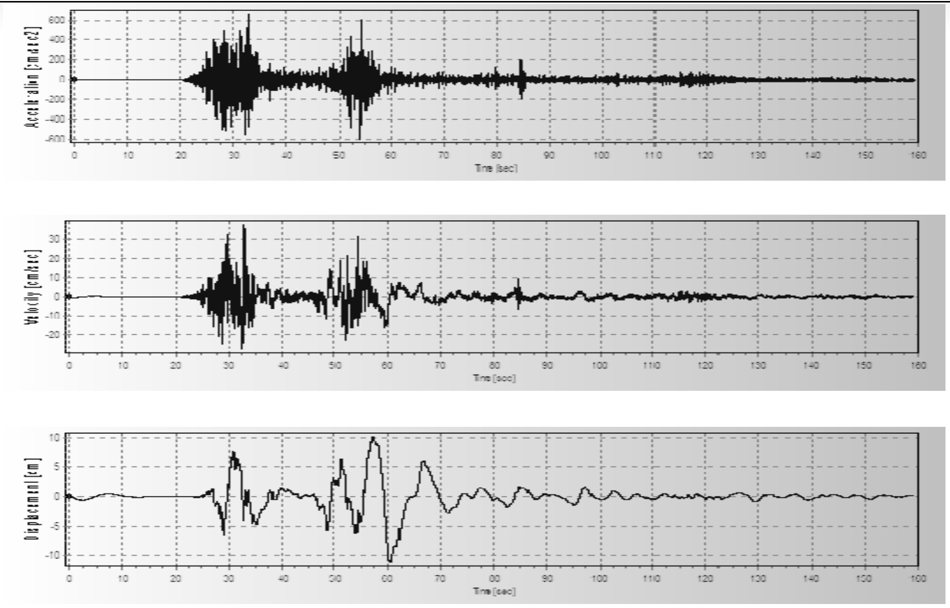
Maximum Acceleration: 987.63 cm/sec$^2$ at time t=33.02sec
Maximum Velocity: 47.91 cm/sec at time t=32.95sec
Maximum Displacement: 9.75 cm at time t=55.52sec
Vmax / Amax: 0.05 sec
Acceleration RMS: 71.34 cm/sec$^2$
Velocity RMS: 4.36 cm/sec
Displacement RMS: 1.52 cm
Arias Intensity: 12.99 m/sec
Characteristic Intensity ($I_c$): 7606
Specific Energy Density: 3033 cm$^2$/sec
Cumulative Absolute Velocity (CAV): 5117 cm/sec
Acceleration Spectrum Intensity (ASI): 927 cm/sec
Velocity Spectrum Intensity (VSI): 211 cm
Sustained Maximum Acceleration (SMA): 732 cm/sec$^2$
Sustained Maximum Velocity (SMV): 40 cm/sec
Effective Design Acceleration (EDA): 916 cm/sec$^2$
A95 parameter: 971 cm/sec$^2$
Predominant Period ($T_p$): 0.42 sec
Mean Period ($T_m$): 0.32 sec

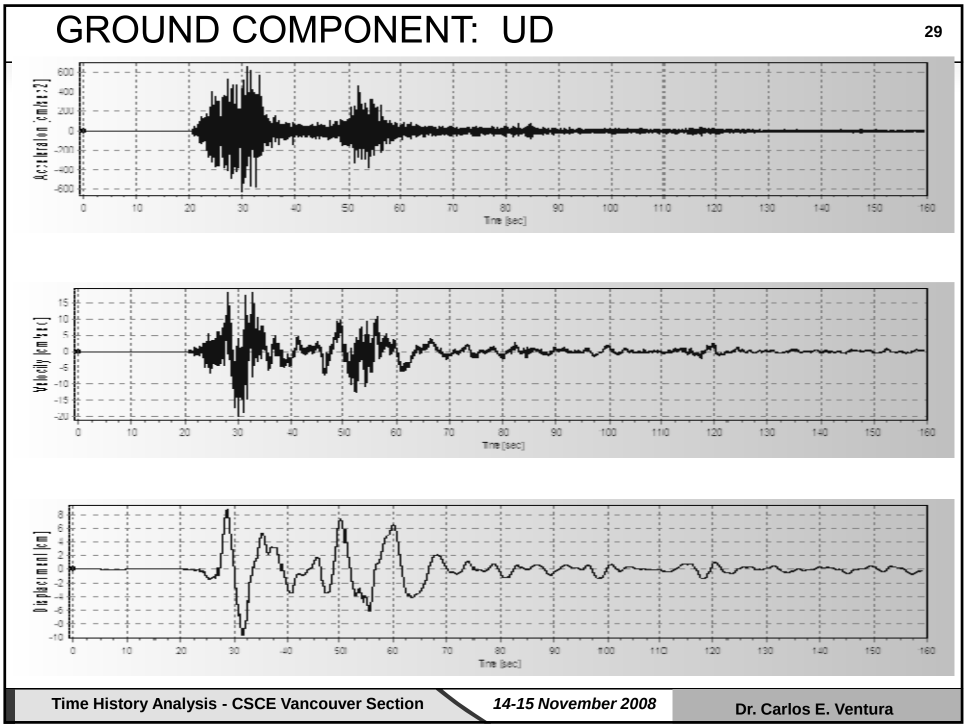
***(see companion notes for a detailed explanation of these parameters)***

## Significant Duration using Arias Intensity

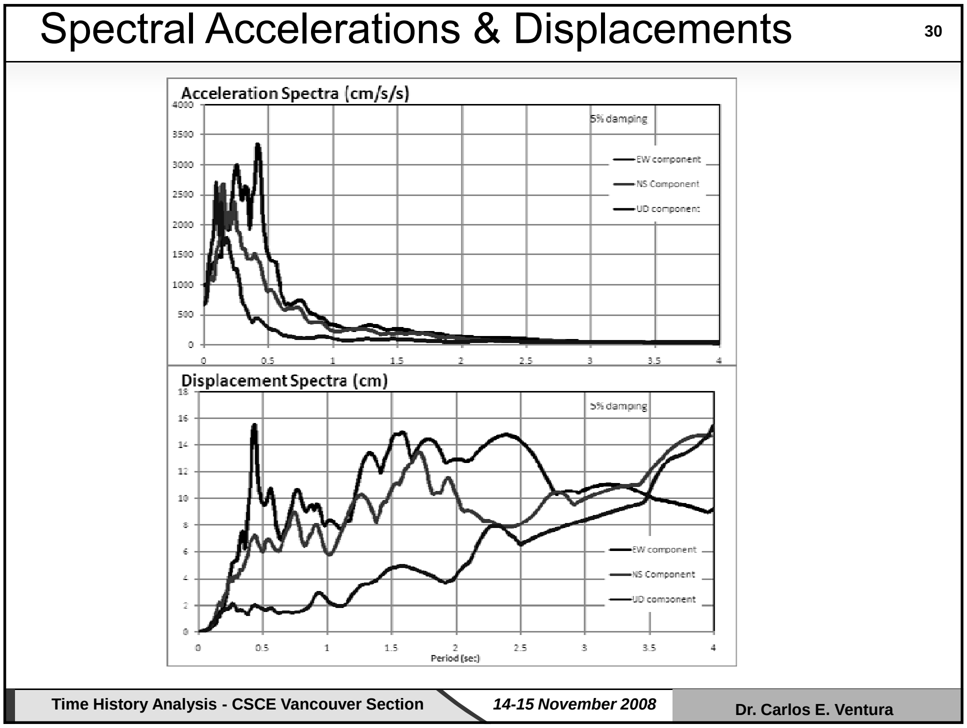


## GROUND COMPONENT: NS

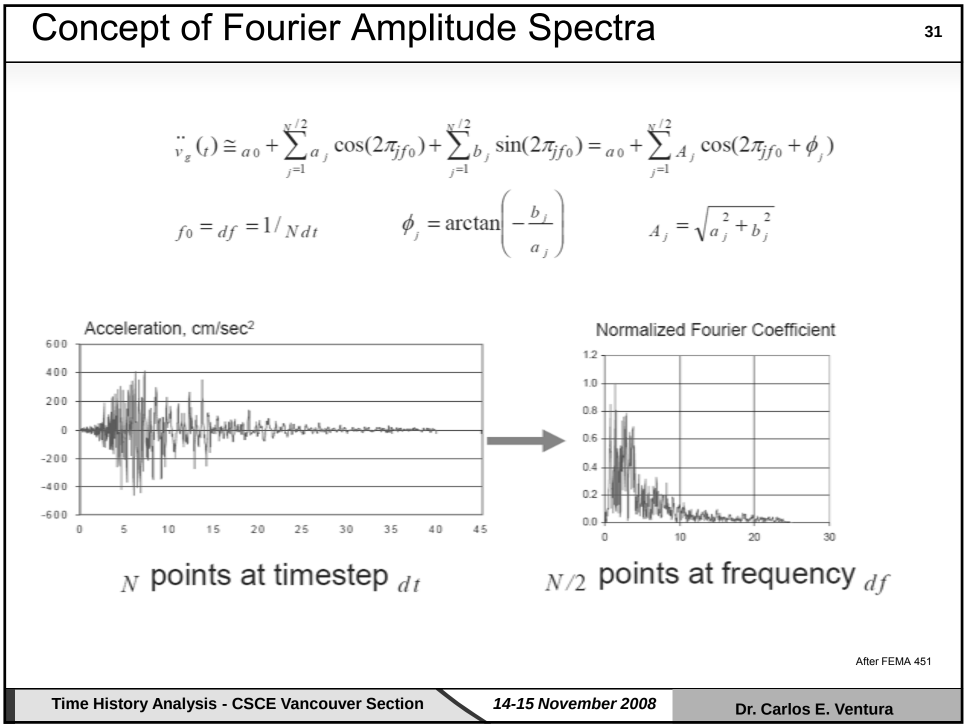




## GROUND COMPONENT: UD



## Spectral Accelerations & Displacements



## Concept of Fourier Amplitude Spectra

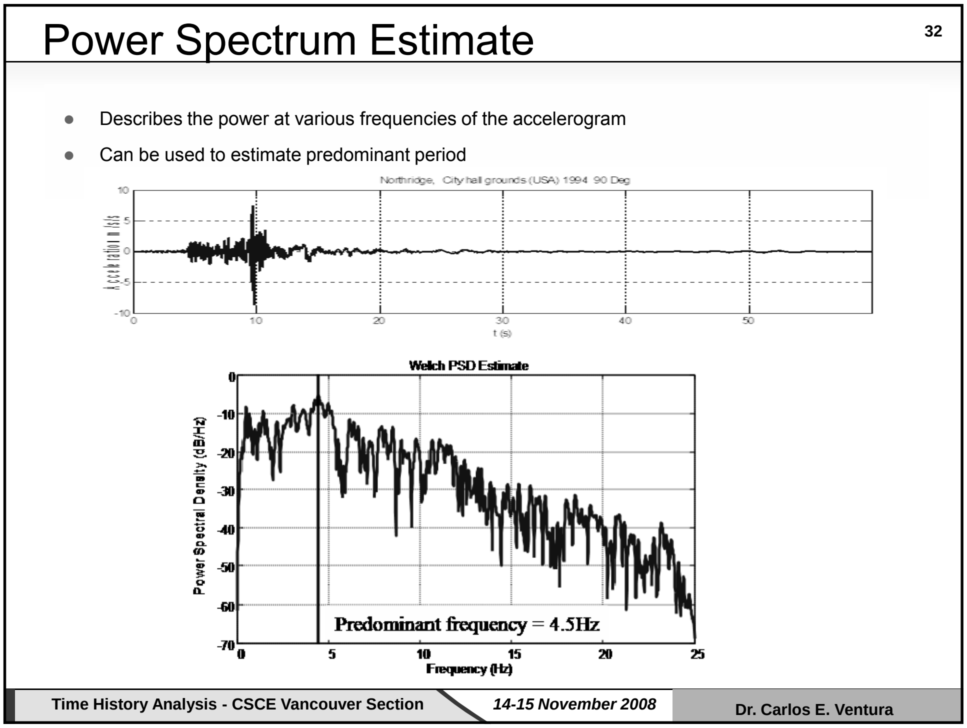
$$\ddot{v}_g(t) \cong a_0 + \sum_{j=1}^{N/2} a_j \cos(2\pi_{jf_0}) + \sum_{j=1}^{N/2} b_j \sin(2\pi_{jf_0}) = a_0 + \sum_{j=1}^{N/2} A_j \cos(2\pi_{jf_0} + \phi_j)$$

$$f_0 = df = 1/_{Ndt} \qquad \phi_j = \arctan\left(-\frac{b_j}{a_j}\right) \qquad A_j = \sqrt{a_j^2 + b_j^2}$$

$N$ points at timestep $dt$

$N/2$ points at frequency $df$

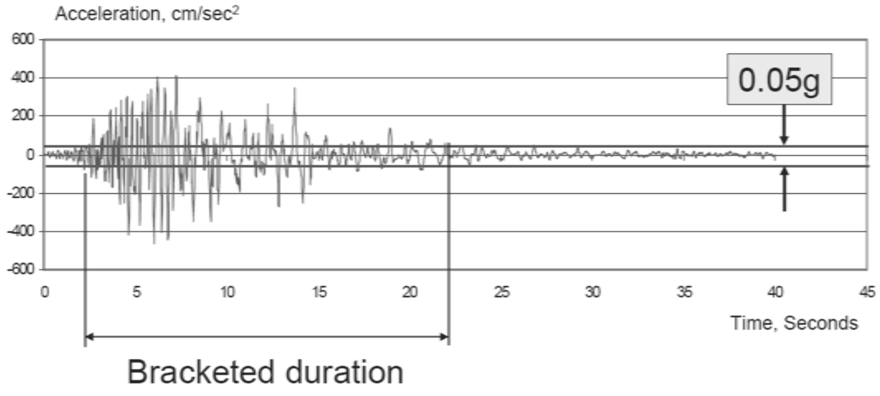
After FEMA 451



## Power Spectrum Estimate

- Describes the power at various frequencies of the accelerogram
- Can be used to estimate predominant period

## Bracketed Duration



After FEMA 451

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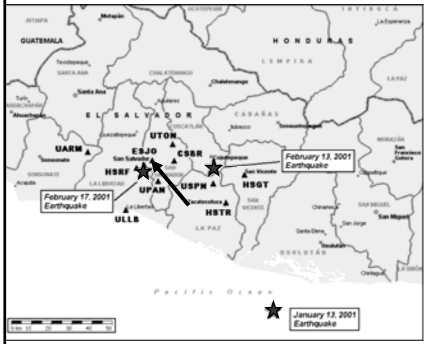
## Some Examples of Ground Motions and Important Observations

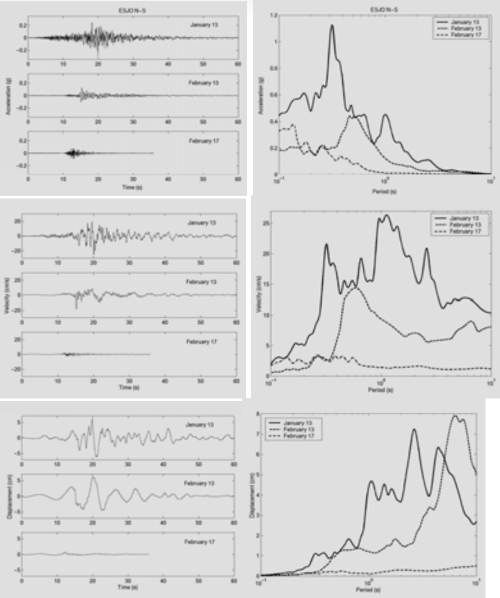
After FEMA 451

## Comparison of Crustal and Sub-crustal Ground Motions

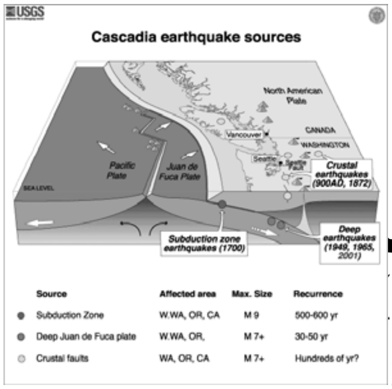
***El Salvador 2001 Earthquakes***

- January 13, Mw 7.7 subcrustal
- February 13 Mw 6.6 crustal
- February 17 Mw 5.1 crustal

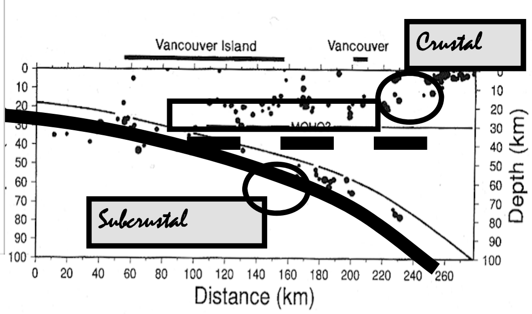




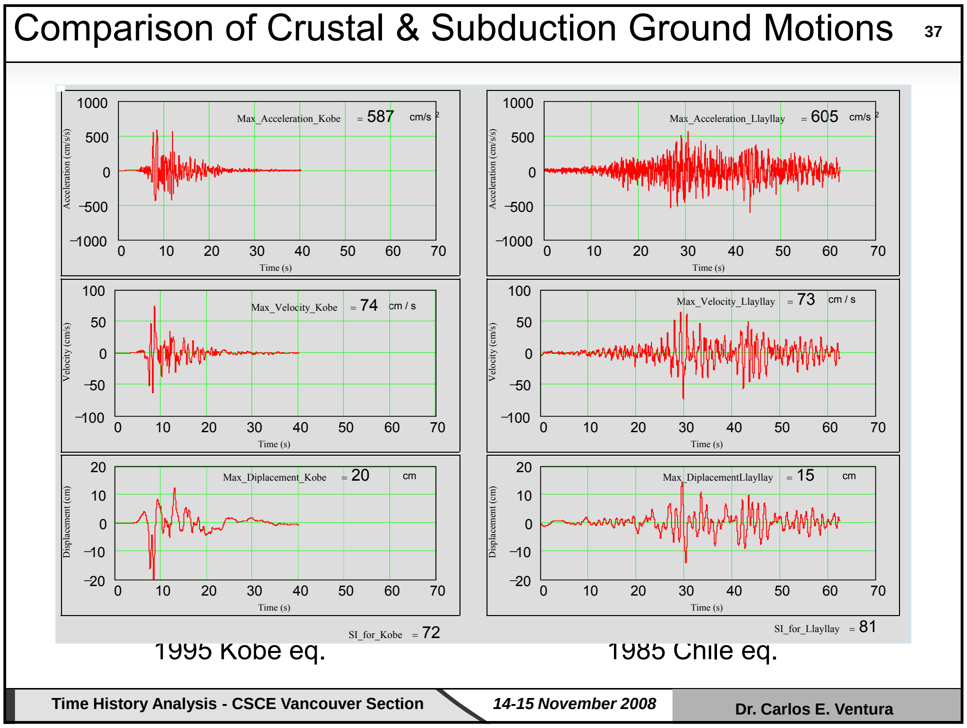
## Seismicity and Tectonics in British Columbia



| Source | Affected area | Max. Size | Recurrence |
|---|---|---|---|
| ● Subduction Zone | W.WA, OR, CA | M 9 | 500-600 yr |
| ● Deep Juan de Fuca plate | W.WA, OR, | M 7+ | 30-50 yr |
| ○ Crustal faults | WA, OR, CA | M 7+ | Hundreds of yr? |



- *Crustal earthquakes at the North American Plate*
- *Subcrustal earthquakes at the Juan de Fuca Plate*
- *Subduction events at the interface of the two plates*



## Comparison of Crustal & Subduction Ground Motions

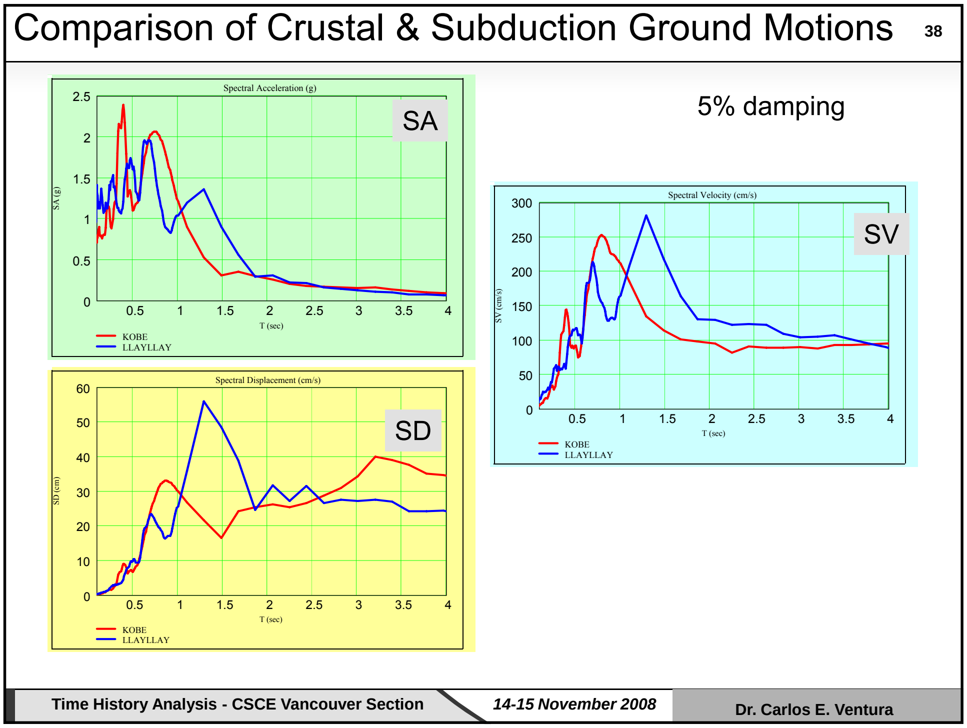
1995 Kobe eq.

1985 Chile eq.

Time History Analysis - CSCE Vancouver Section

*14-15 November 2008*

Dr. Carlos E. Ventura



## Comparison of Crustal & Subduction Ground Motions

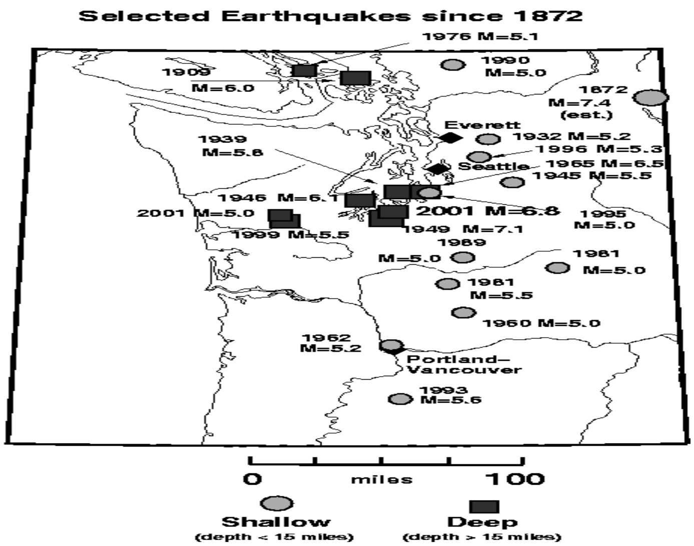
5% damping

Time History Analysis - CSCE Vancouver Section

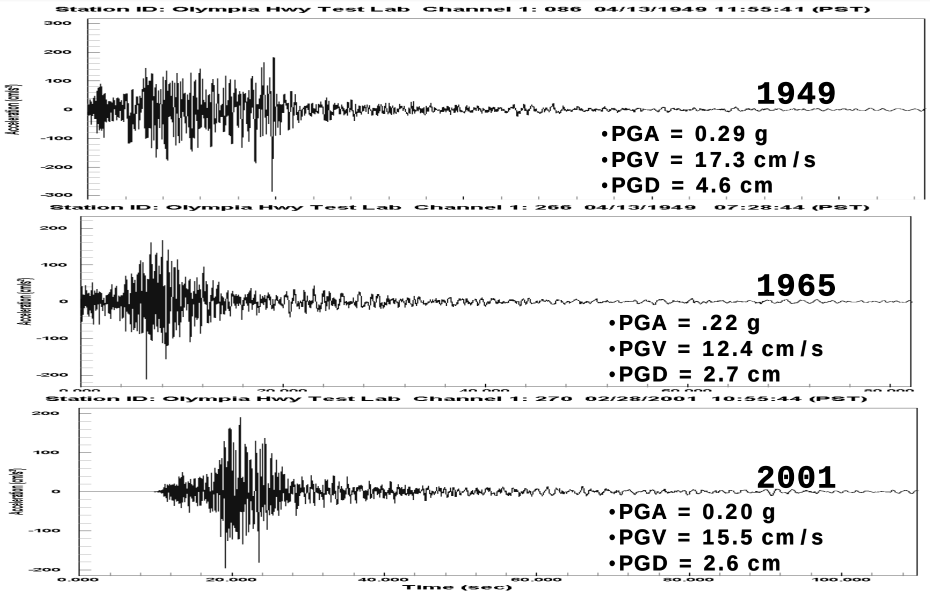
*14-15 November 2008*

Dr. Carlos E. Ventura

## Seattle Regional Earthquake History



## Acceleration Comparison



**1949**

- PGA = 0.29 g
- PGV = 17.3 cm / s
- PGD = 4.6 cm

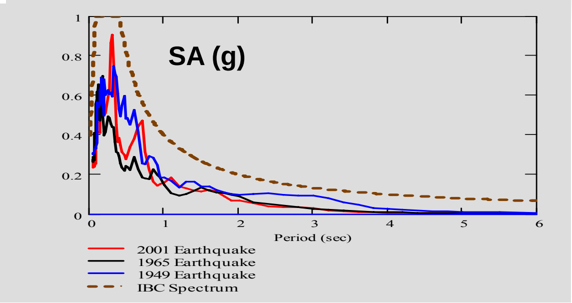
**1965**

- PGA = .22 g
- PGV = 12.4 cm / s
- PGD = 2.7 cm

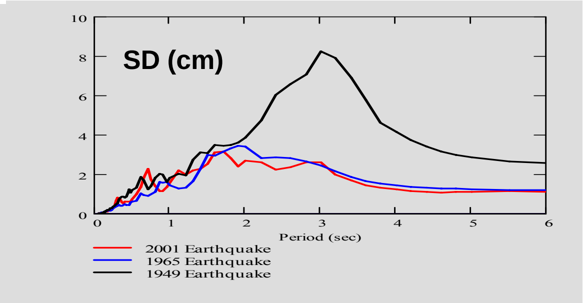
**2001**

- PGA = 0.20 g
- PGV = 15.5 cm / s
- PGD = 2.6 cm

## Comparison of Acceleration & Displacement Spectra



(5% damping)

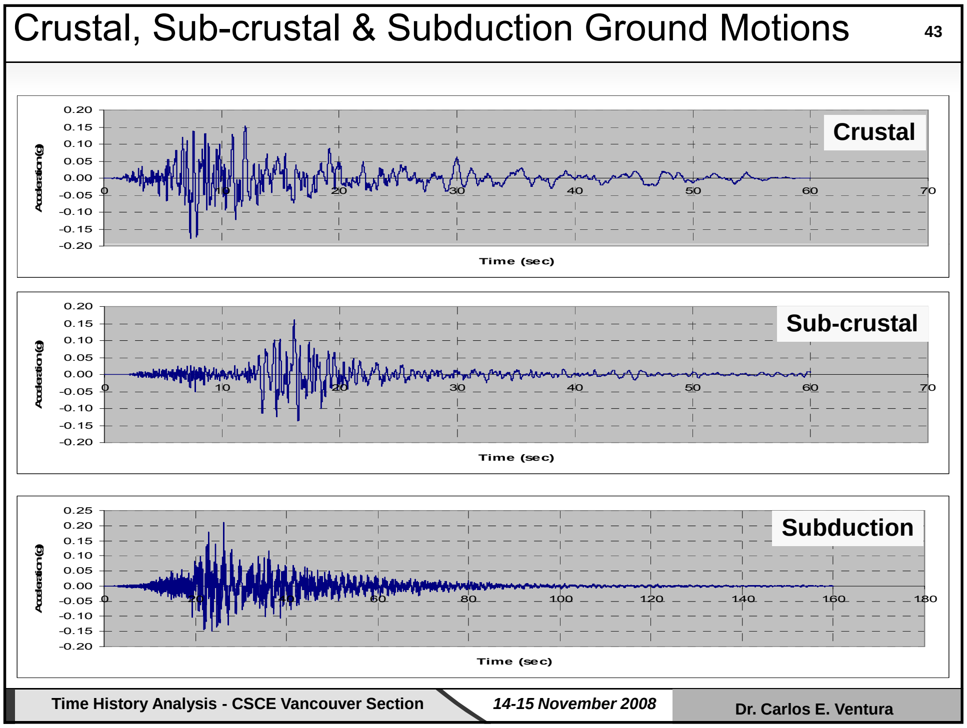


## Crustal, Subcrustal & Subduction Ground Motions

| Record | Date | Mag. | SF |
|---|---|---|---|
| Loma Prieta, CA | 18-Oct-1989 | 6.9 | 1.06 |
| Nisqually, WA | 28-Feb-2001 | 6.8 | 1.53 |
| Tokachi-oki, Japan | 25-Sep-2003 | 8.0 | 1.02 |

SF= scaling factor

These are part of the set of records used for the BC Schools Seismic Retrofitting Program



## Crustal, Sub-crustal & Subduction Ground Motions

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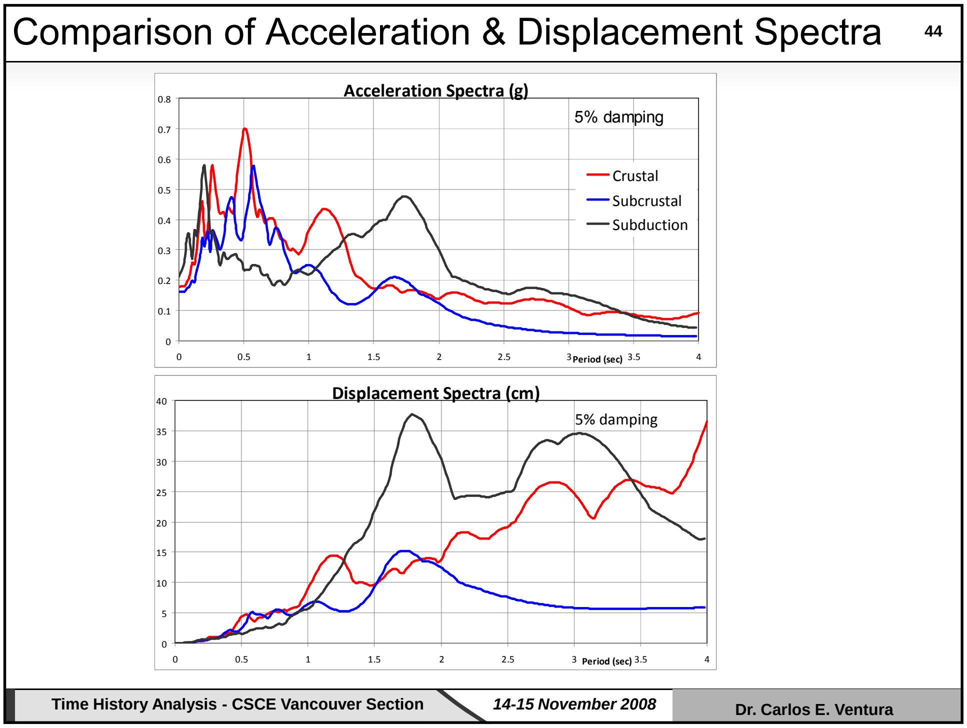
Time History Analysis - CSCE Vancouver Section

*14-15 November 2008*

Dr. Carlos E. Ventura



## Comparison of Acceleration & Displacement Spectra

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*14-15 November 2008*

Dr. Carlos E. Ventura

## Some source of Strong Motion Data

PEER Strong Motion Database: http://peer.berkeley.edu/smcat/index.html

COSMOS (Consortium of Organizations for Strong-Motion Observation Systems): http://www.cosmos-eq.org

The European Strong Motion Database (ESD) : http://www.isesd.cv.ic.ac.uk/

KiK-net (Japan's digital strong-motion seismograph network: http://www.kik.bosai.go.jp/

CESMD (Center for Engineering Strong Motion Data): http://www.strongmotioncenter.org/

Modern databases provide a very convenient way to search for strong motion data. Data can be searched in terms of Magnitude, epicentral distance, source mechanism, fault type, soil conditions, etc.

## "Good Practice" for data selection and usage

- Many different types of instruments are available and they often represent an excellent choice for ground motion measurements; however, accelerometers are not well-suited for all applications as no single sensor can meet every vibration requirement.
- It is easy to generate "bad data" without the proper *transducer.*
- Only get data from reliable sources and databases
- Be aware of methods used for data processing
- Use datasets that have been processed in the same manner
- Only use the records for the specified frequency band (i.e. do not use records that have been filtered at 2 seconds for the analysis of a structure with a natural period of 4 seconds)

Thank you!

# TIME HISTORY ANALYSIS

## LECTURE # 2

## Origin and Interpretation of Ground motion time histories

**Dr. Carlos E. Ventura, P.E., P.Eng.**
**Department of Civil Engineering**
**The University of British Columbia**

# NOTES

# EARTHQUAKE EXCITATION

***Earthquake Damage Mechanisms:*** Earthquakes can damage structures in various ways, such as:

- by inertial forces generated by severe ground shaking;
- by direct fault displacement at the site;
- by foundation failure due to consolidation, settlement and liquefaction of the supporting soil;
- by landslides, or other surficial movements;
- by water waves generated by seismic motions (tsunamis & seiches);
- by fires resulting from earthquake shaking;
- by large-scale tectonic changes in ground elevation.

Earthquake ground motion is usually measured by strong-motion accelerographs, which record the acceleration of the ground at particular locations. The recorded accelerograms are generally corrected for instrument errors and adjusted for baseline, and are integrated to obtain velocity and displacement time histories.

The peak values of ground acceleration, velocity and displacement are of most interest in seismic design. These parameters, in combination with other factors such as magnitude, epicentral distance, distance to the fault, duration of strong shaking, soil conditions of the site, and frequency content of the motion, affect the seismic behaviour of a structure.

***Characteristics of Earthquake Ground Motions:*** The characteristics of earthquake ground motion which are of most interest in earthquake engineering applications are:

1. Peak ground motions (acceleration, velocity and displacement) primarily influence the vibration amplitudes
2. Duration of strong motion has a pronounced effect on the severity of the shaking.
3. Frequency content spectral shapes relate to frequencies or periods of vibration of a structure (resonance conditions).

A ground motion with moderate peak acceleration and a long duration may be more damaging than a ground motion with a larger acceleration and a shorter duration. In a structure, ground motion is amplified the most when the frequencies that dominate the motion are close to the vibration frequencies of the structure.

***(Note: the following sections were obtained from http://www.isesd.cv.ic.ac.uk/ on November 11, 2008)***

***From recording to a usable digital form of record:*** Earthquake strong ground motions are recorded by instruments known as accelerographs because the records produced, called accelerograms, are proportional to, or approximately proportional to, the acceleration of the ground. Accelerograms are also known as "strong-motion records" and (acceleration) time-histories. Strong-motion instruments usually consist of three mutually perpendicular transducers (accelerometers), two measuring components of the horizontal motion and the third measuring the vertical component of motion.

**Analogue (optical-mechanical) instruments:** These were the first type of accelerograph developed and they record the ground motion in the form of either a photographic trace on film or paper, or a scratch trace on waxed paper. They do not record all the time but are triggered by a minimum level of ground acceleration, usually of the order of 0.005 to 0.01g in the vertical direction. Therefore they do not record the entire ground motion, which occurred during the earthquake. After recovering the paper or film from the instrument, the trace of the strong ground motion is digitized either by hand or by machine. This digitized record is then ready for use, after checking that there are no obvious digitization errors. The majority of records within this databank were recorded by analogue instruments such as the SMA-1 made by Kinemetrics Inc.

**Digital instruments:** In the past twenty or thirty years instruments have been developed which record the strong ground motion in a digital form and hence the separate digitisation step is no longer required. These instruments record on reusable media (magnetic or solid state) and so are able to record continuously. If the threshold trigger level is exceeded then the record is retained together with the ground motion which occurred in the seconds before the instrument triggered (pre-event time). Therefore they record the entire ground motion which occurred during the earthquake as long as the post-event time is sufficient. Recently digital instruments have become increasingly deployed but there still fewer digital records in the databank than those from analogue accelerographs.

**Errors in accelerograms in usable digital form:** In this databank, uncorrected records are those records which have not undergone any adjustment except for the removal of any obvious spurious peaks or backward time steps. These records however can be expected to be affected by errors, especially if they are from analogue instruments, which will be most prominent in the high frequency ($\gtrsim$ 20Hz) and low frequency ($\lesssim$ 0.5Hz) ranges. High frequency errors may affect estimates of the peak ground acceleration and short period spectral quantities. Low frequency errors will affect the velocity and displacement time-histories (obtained by integrating the acceleration time history), because both are long-period quantities, and also long period spectral values.

Records from analogue instruments are particularly affected by long period errors because of the digitisation stage which is not required for records from digital instruments. An excellent discussion of the errors in digitised analogue records is provided by Trifunac et al. (1973).

**Instrumental errors:** Sources of errors in the strong-motion records due to the instrument include:

1. Transducer distortions of amplitude and phase
2. Imperfections of the transducer design: most existing transducers are not true single-degree-of-freedom (SDOF) systems

3. Transverse play of the recording paper/film causing variations up to several millimetres
4. Non-uniform velocity of the record-driving mechanism
5. Non-uniform time marks
6. Misalignment of the transducers
7. Clipping: if sensitivity setting of instrument is too high, the largest peaks may go off scale
8. Variable trace thickness: influences accuracy of digitisation
9. Sensitivity calibration
10. Drift: over long time intervals, temperature and humidity effects can cause drift but for periods of minutes this is not important
11. Instrument slip

**Photographic processing errors:** Sources of errors in the strong-motion records due to the photographic processing include:

1. Warping of film negatives caused by chemical processing and ageing
2. Errors from optical enlargement during printing of film negatives resulting from lens imperfection and non-parallelism of the planes of original film and projected image
3. Poisson effect in film processing because during film copying, the original and copy are held together under longitudinal tension

**Digitisation errors:** Sources of errors in the strong-motion records due to the digitisation of the analogue record include:

1. Digitisation rate: the greater the number of digitised points, the better the accuracy with which the digital data approximates the continuous function of the accelerogram
2. Inadequate resolution of the digitising equipment
3. Low-pass filtering effects of optical-mechanical digitisation because digitisation approximates a continuous function by a sequence of discrete points
4. Systematic and random digitisation errors:
   - Imperfections in the mechanical traverse mechanism of the digitiser creates systematic long period errors
   - Human "imperfection" introduces random intermediate and high frequency errors
5. Baseline shifts (translations and/or rotations relative to the digitiser axes) during digitisation can be considered as random long period errors

**Instrument correction:** The output from accelerographs, which do not have instrument correction built in, is the relative displacement response as a function of time, *t* , *y(t)* . Most accelerographs are SDOF systems so this relative displacement obeys the second order differential equation:

$$\ddot{y}(t) + 2\beta\omega\dot{y}(t) + \omega^2 y(t) = -\ddot{U}$$

where $\beta$ is the undamped critical damping ratio (usually about 0.6 in most analogue instruments), $\omega$ is the transducer natural angular frequency (usually about 25 × 2$\pi$ in most analogue instruments), $\ddot{U}$ ; is the ground acceleration (the dots signify differentiation with respect to time).

The transducer undamped natural angular frequency, $\omega$, is usually high enough so that *y(t)* is proportional to the ground acceleration, $\ddot{U}$ ;, for frequencies less than about 25Hz. However for higher frequencies it is important that an instrument correction is performed to find the "true" ground acceleration, $\ddot{U}$ ;. A number of different methods have been used to achieve such a correction, for example a finite difference method (Trifunac, 1972), high-frequency oscillator approach (Trifunac, 1972), discrete Fourier transform filter and digital differentiation (Sunder & Connor, 1982).

**Baseline correction:** The major problem with the recovery of true ground velocity and displacement is that the zero acceleration level (baseline or centreline) is not indicated on the accelerogram (Schiff & Bogdanoff, 1967; Trifunac, 1971). The main difficulties in determining the baseline position are: a) initial part of shock is not recorded, b) final acceleration or velocity cannot be assumed to be zero, due to the presence of background noise, c) the final displacement is not known and d) sometimes the final part of the shock is not recorded.

One of the main polynomial correction methods was developed at the Earthquake Engineering Research Laboratory (California Institute of Technology). A parabolic acceleration baseline (cubic baseline on the velocity) is assumed which is fixed by minimizing the mean square ground velocity (Hudson et al., 1969). Graizer (1979) develops a technique based on this idea and uses this method to correct the 65° component of the Parkfield-Cholame Shandon Array 2W record from the Parkfield earthquake (28/6/1966) and achieves a good match with theoretical results. Graizer (1979) minimizes the mean square ground velocity in the 'quiet' periods before and after the main portion of shaking and also uses polynomials of up to degree 10, thereby achieving a more stable correction.

Iwan et al., 1985 introduce a simple baseline correction method, specifically for the Kinemetrics PDR-1 digital accelerograph, which allows three parts of the acceleration baseline (that before the strong motion, that during the strong motion and that after the strong motion) to have different zero levels. This procedure was used because tests revealed an instrument anomaly, thought to be due to mechanical or electrical hysteresis within the transducer, which prevented the true ground displacement being recovered simply through integrating twice the acceleration time-history. Results obtained by Iwan et al., 1985 and by other investigators show that realistic ground displacements can be obtained by this method.

**Filtering:** In order to remove the short and long period errors from accelerograms the time-histories are often filtered. Many different types of filter have been used to filter strong-motion records, for example Ormsby filters (Trifunac et al., 1973), frequency-domain filters, elliptical filters (Sunder & Connor, 1982; Sunder & Schumacker, 1982) and Butterworth filters (Converse, 1992). This filtering will remove the errors in the stop bands however it will also remove any ground motions within these period ranges and hence outside the pass band the corrected accelerogram can no longer be expected to adequately represent the true ground motion. Usually however the stop bands adopted are outside the range of engineering interest. The choice of the low-frequency cut-off often has a large effect on long-period time-domain parameters such as peak ground velocity (PGV) and peak ground displacement (PGD) and hence such parameters are associated with much uncertainty unless these cut-off frequencies were chosen with care.

***(Note: the following sections were obtained from http://www.seismosoft.com/en/HomePage.aspx on November 11, 2008)***

# Ground Motion Parameters

Commonly computed ground motion parameters (Kramer, 1996) are:

**Peak ground values of acceleration (PGA), velocity (PGV) and displacement (PGD)**

$$\mathrm{PGA} = \max|a(t)| \quad ; \quad \mathrm{PGV} = \max|v(t)| \quad ; \quad \mathrm{PGD} = \max|d(t)|$$

**Peak velocity and acceleration ratio (Vmax/Amax)**

$$v_{\max}/a_{\max} = \frac{\max|v(t)|}{\max|a(t)|}$$

**Root-mean-square (RMS) of acceleration, velocity and displacement**

$$a_{rms} = \sqrt{\frac{1}{t_r}\int_0^{t_r}[a(t)]^2\,dt} \quad ; \quad v_{rms} = \sqrt{\frac{1}{t_r}\int_0^{t_r}[v(t)]^2\,dt} \quad ; \quad d_{rms} = \sqrt{\frac{1}{t_r}\int_0^{t_r}[d(t)]^2\,dt}$$

**Arias Intensity (Ia)**

$$I_a = \frac{\pi}{2g}\int_0^{\infty}[a(t)]^2\,dt$$

**Characteristic Intensity (Ic)**

$$I_c = \left(a_{rms}\right)^{\frac{3}{2}} \cdot \sqrt{t_r}$$

**Specific Energy Density (SED)**

$$\mathrm{SED} = \int_0^{t'} \left[v(t)\right]^2 dt$$

**Cumulative Absolute Velocity (CAV)**

$$\mathrm{CAV} = \int_0^{t'} \left|a(t)\right| dt$$

**Acceleration (ASI) and Velocity (VSI) Spectrum Intensity**

$$\mathrm{ASI} = \int_{0.1}^{0.5} S_a(\xi = 0.05, T)\,dT \;;\; \mathrm{VSI} = \int_{0.1}^{2.5} S_v(\xi = 0.05, T)\,dT$$

**Sustained maximum acceleration (SMA) and velocity (SMV):** This parameter gives the sustained maximum acceleration/velocity during three cycles, and is defined as the third highest absolute value of acceleration in the time history.

**Effective Design Acceleration (EDA):** This parameter corresponds to the peak acceleration value found after lowpass filtering the input time history with a cut-off frequency of 9 Hz.

**A95 parameter:** The acceleration level below which 95% of the total Arias intensity is contained. In other words, if the entire accelerogram yields a value of Ia equal to 100, the A95 parameter is the threshold of acceleration such that integrating all the values of the accelerogram below it, one gets an Ia=95.

**Predominant Period (Tp):** The predominant period *Tp* is the period at which the maximum spectral acceleration occurs in an acceleration response spectrum calculated at 5% damping.

**Mean Period (Tm):** The mean period *Tm* is the best simplified frequency content characterisation parameter, being estimated with the following equation, where *Ci* are the Fourier amplitudes, and *fi* represent the discrete Fourier transform frequencies between 0.25 and 20 Hz.

$$T_m = \frac{\sum C_i^2 / f_i}{\sum C_i^2}$$

**Husid plot:** The Husid plot represents the build-up of the Arias Intensity.

**Energy Flux plot:** The Energy flux plot represents the build-up of Specific Energy Density.

## *Record durations:*

**Bracketed duration:** The total time elapsed between the first and the last excursions of a specified level of acceleration (default is 5% of PGA).

**Uniform duration:** The total time during which the acceleration is larger than a given threshold value (default is 5% of PGA).

**Significant duration:** The interval of time over which a proportion (percentage) of the total Arias Intensity is accumulated (default is the interval between the 5% and 95% thresholds).

**Effective duration**: It is based on the significant duration concept but both the start and end of the strong shaking phase are identified by absolute criteria.

## References

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3. Hudson D.E., N.C. Nigam, and M.D. Trifunac (1969): Analysis of strong-motion accelerograph records. Pages A2(1)-A2(17) of: Proceedings of Fourth World Conference of Earthquake Engineering, vol. I.
4. Iwan W.D., M.A. Moser, and C.-Y. Peng (1985): Some observations on strong-motion earthquake measurements using a digital accelerograph. Bull. Seism. Soc. Am., vol. 75, no. 5, pp. 1225-1246.
5. Kramer S.L. [1996] *Geotechnical Earthquake Engineering*, Prentice-Hall
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11. Trifunac M., F. Udwadia, and A. Brady (1973): Analysis of errors in digitized strong-motion accelerograms. Bull. Seism. Soc. Am., vol. 63, no. 1, pp. 157-187.

# TIME HISTORY ANALYSIS

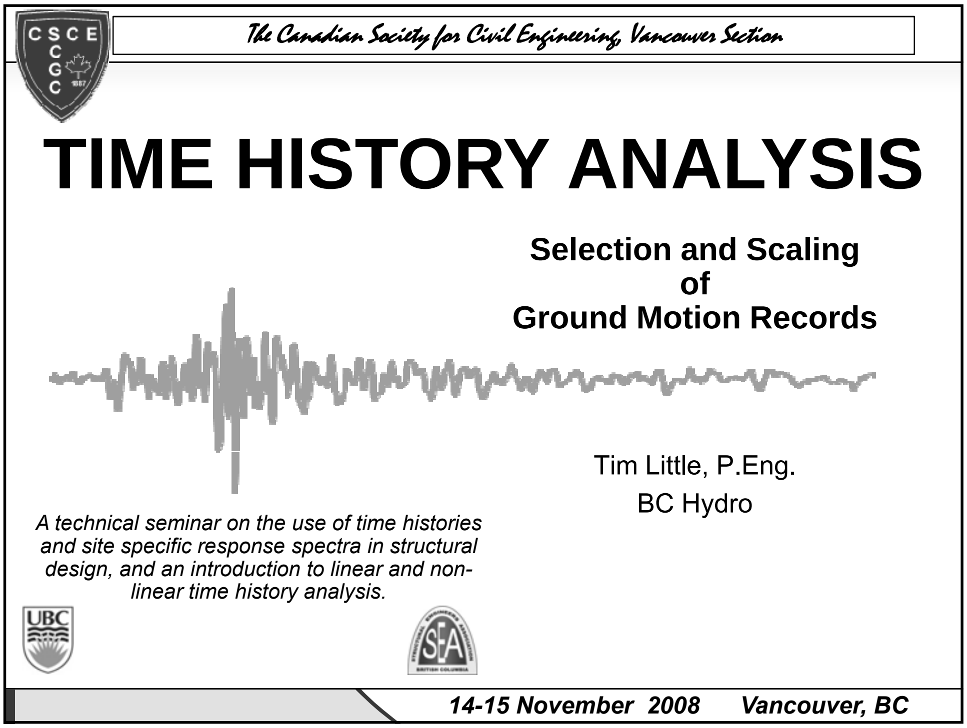
## LECTURE # 3

## Selection and Scaling of Ground Motions

**Timothy Little, P.Eng.**
**BC Hydro Engineering**

Tim Little is Chief Engineer of BC Hydro. He is a Geological Engineer with 32 years of experience in the hydroelectric and dam safety industry. Seismic hazard assessment and earthquake engineering have been a major focus of his work for more than 20 years. During that time, BC Hydro has been very proactive at assessing seismic hazards, has carried out many challenging and innovative seismic upgrade projects, and has installed and operated one of the largest strong motion instrument networks in Canada.

Since 1995, Tim has been a member of CANCEE, the committee responsible for recommending seismic provisions for the National Building Code of Canada. He is also the Canadian representative on the ICOLD (International Commission on Large Dams) Seismic Committee, which prepares guidelines on seismic design aspects of dams.



The Canadian Society for Civil Engineering, Vancouver Section

# TIME HISTORY ANALYSIS

## Selection and Scaling of Ground Motion Records

Tim Little, P.Eng.
BC Hydro

*A technical seminar on the use of time histories and site specific response spectra in structural design, and an introduction to linear and non-linear time history analysis.*

***14-15 November 2008 Vancouver, BC***

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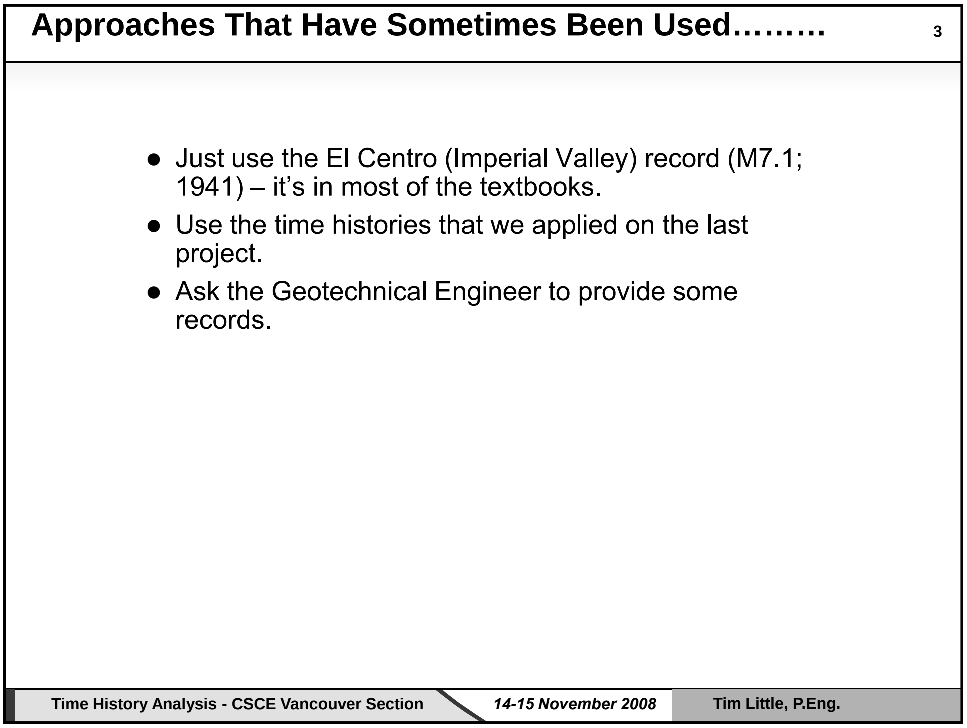
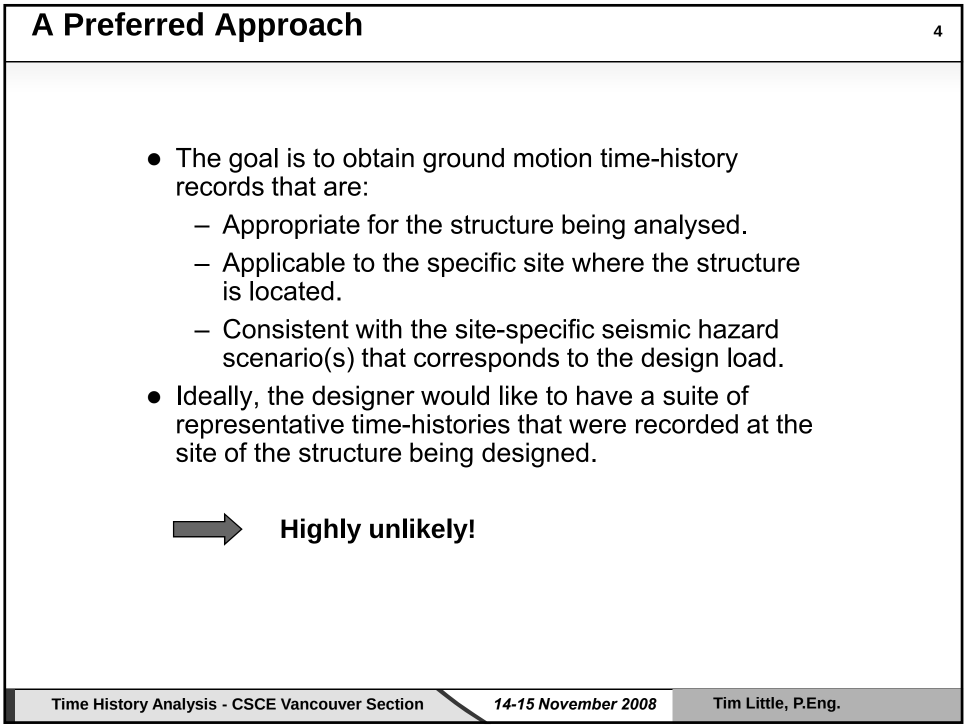
## Outline

- Introduction & general approach to selecting time histories
- Tectonic & geological conditions
- Site conditions
- Design earthquake scenarios
- Spectral matching
- Example



## Approaches That Have Sometimes Been Used………

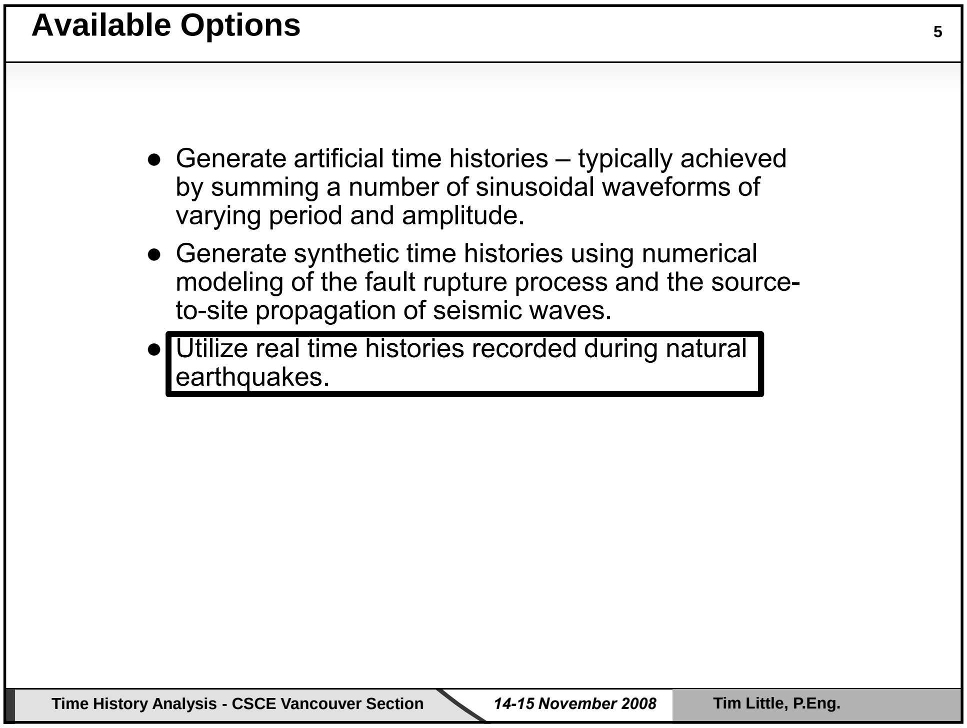
- Just use the El Centro (Imperial Valley) record (M7.1; 1941) – it's in most of the textbooks.
- Use the time histories that we applied on the last project.
- Ask the Geotechnical Engineer to provide some records.



## A Preferred Approach

- The goal is to obtain ground motion time-history records that are:
  - Appropriate for the structure being analysed.
  - Applicable to the specific site where the structure is located.
  - Consistent with the site-specific seismic hazard scenario(s) that corresponds to the design load.
- Ideally, the designer would like to have a suite of representative time-histories that were recorded at the site of the structure being designed.

➡ **Highly unlikely!**

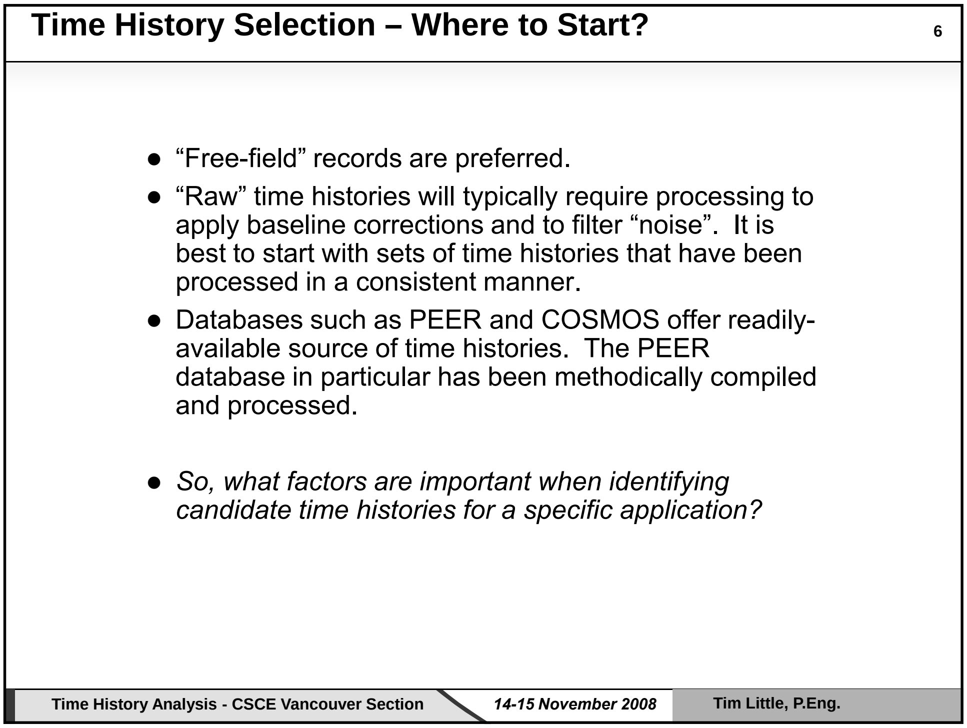


## Available Options

5

- Generate artificial time histories – typically achieved by summing a number of sinusoidal waveforms of varying period and amplitude.
- Generate synthetic time histories using numerical modeling of the fault rupture process and the source-to-site propagation of seismic waves.
- Utilize real time histories recorded during natural earthquakes.

**Time History Analysis - CSCE Vancouver Section** ***14-15 November 2008*** **Tim Little, P.Eng.**



## Time History Selection – Where to Start?

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- “Free-field” records are preferred.
- “Raw” time histories will typically require processing to apply baseline corrections and to filter “noise”. It is best to start with sets of time histories that have been processed in a consistent manner.
- Databases such as PEER and COSMOS offer readily-available source of time histories. The PEER database in particular has been methodically compiled and processed.

- *So, what factors are important when identifying candidate time histories for a specific application?*

**Time History Analysis - CSCE Vancouver Section** ***14-15 November 2008*** **Tim Little, P.Eng.**

## Typical Ground Motion Attenuation Relationship

$\ln Y = f_1(M) + f_2(R) + f_3(F) + f_4(HW) + f_5(S) + f_6(D) + \varepsilon$, where:

- *Y = Peak ground motion (PGA or Sa)*
- *M = Magnitude*
- *R = Source-to-site distance*
- *F = Style of faulting*
- *HW = Hanging-wall effect*
- *S = Shallow site condition factor*
- *D = Sediment depth factor*
- *ε = Random error term*

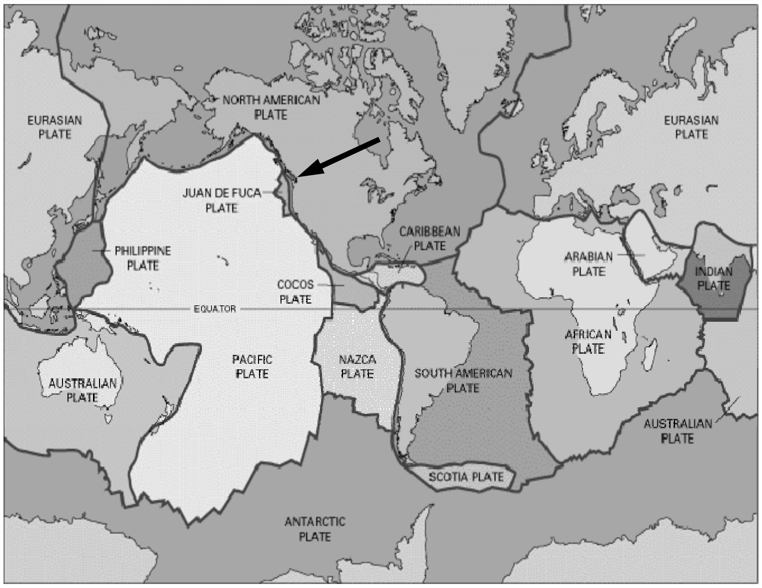
*Ref: Campbell and Bozorgnia (2008)*

## A General Approach to Screen Time Histories

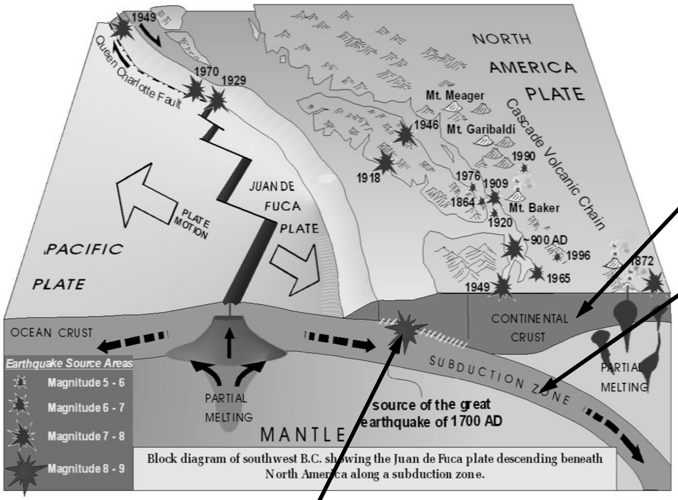
- Identify time history records that :
  - Are from similar tectonic and geologic settings.
  - Were recorded on similar site conditions.
  - Match the design earthquake scenario (M, R, ε).
  - Appropriately match the design response spectrum.

## Tectonic & Geological Conditions

## Earth’s Major Tectonic Plates



## Southwestern B.C. Seismic Sources



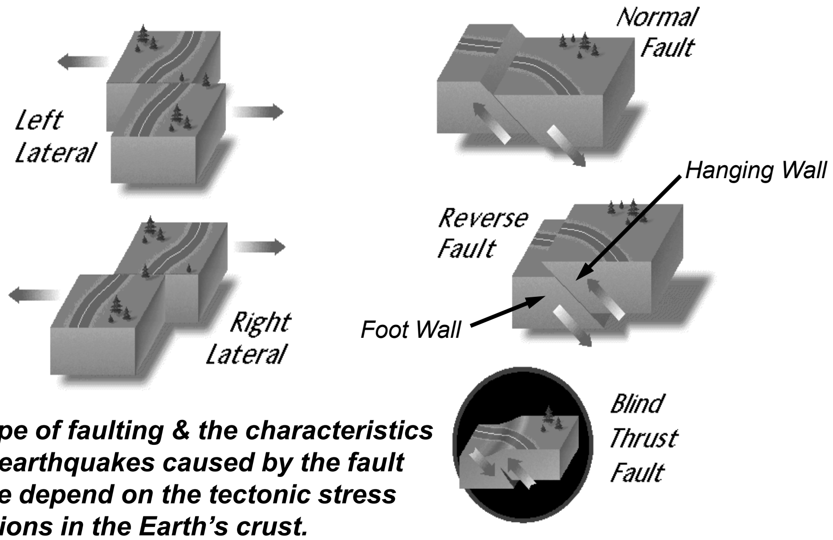
Block diagram of southwest B.C. showing the Juan de Fuca plate descending beneath North America along a subduction zone.

***Crustal EQs***

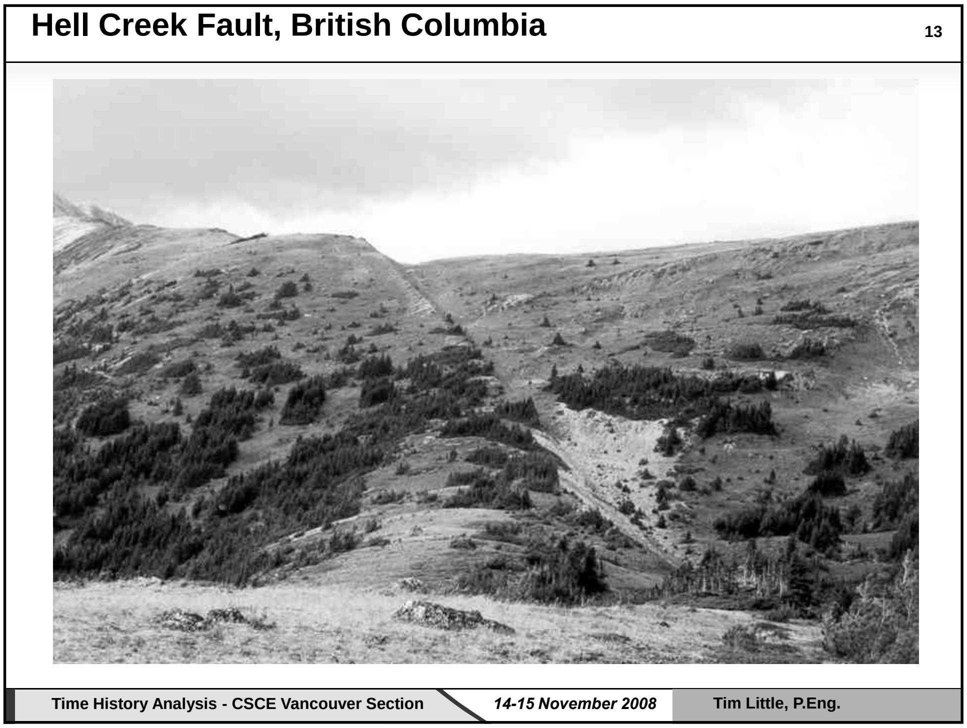
***Intraplate EQs***

***Interplate EQs***

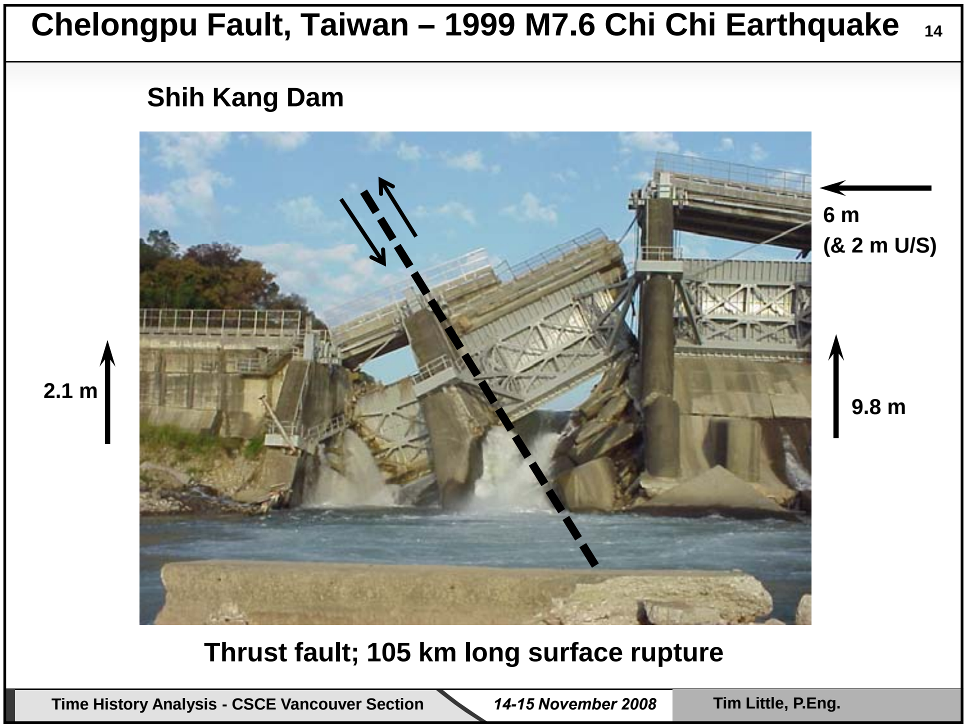
## Types of Faults



***The type of faulting & the characteristics of the earthquakes caused by the fault rupture depend on the tectonic stress conditions in the Earth's crust.***



## Hell Creek Fault, British Columbia



## Chelongpu Fault, Taiwan – 1999 M7.6 Chi Chi Earthquake

**Shih Kang Dam**

2.1 m

6 m
(& 2 m U/S)

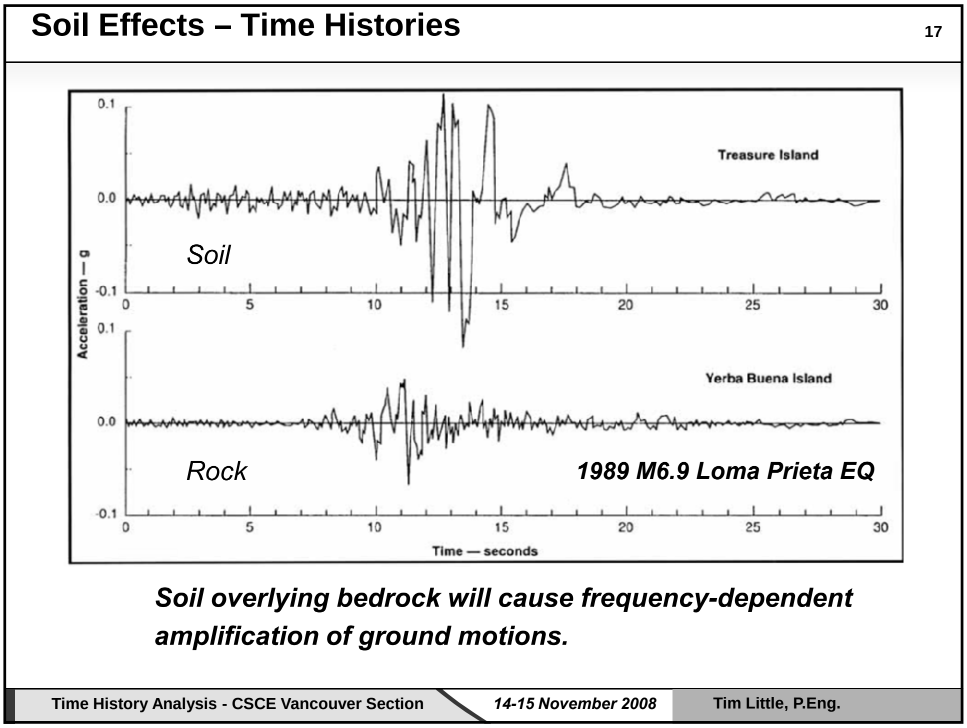
9.8 m

Thrust fault; 105 km long surface rupture

## Tectonic/Geological Considerations - Summary

- Candidate time history records should be from a similar tectonic setting, e.g. plate boundary region, continental interior, subduction zone.
- Records should be from earthquakes caused by similar styles of faulting, e.g. strike-slip, thrust or normal.
- For near-fault conditions (< 10km), records that show directivity effects (e.g. fault fling or directivity) should be considered.

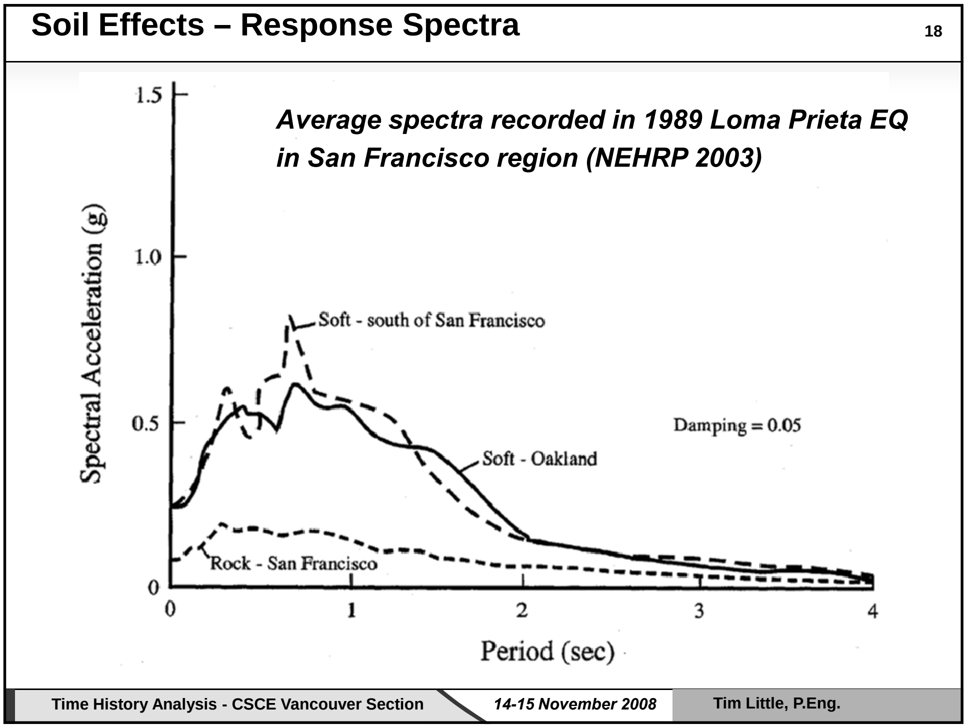
## Site Conditions



## Soil Effects – Time Histories

***1989 M6.9 Loma Prieta EQ***

***Soil overlying bedrock will cause frequency-dependent amplification of ground motions.***

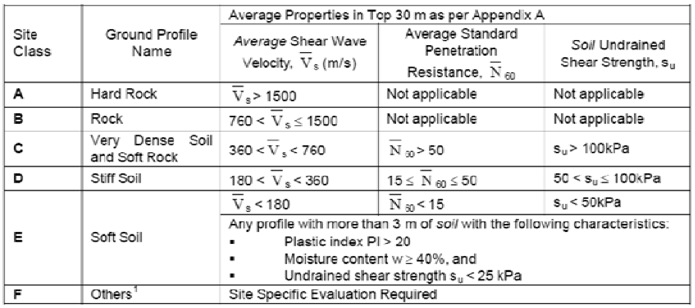


## Soil Effects – Response Spectra

***Average spectra recorded in 1989 Loma Prieta EQ in San Francisco region (NEHRP 2003)***

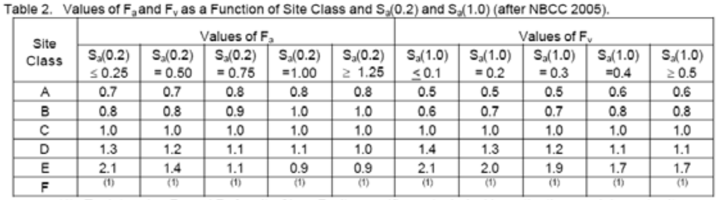
## NBCC 2005 Soil Classifications & Amplification Factors

Table 1. NBCC2005 Site Classification for Seismic Site Response (after NBCC 2005).



| Site Class | Ground Profile Name | Average Properties in Top 30 m as per Appendix A | | |
|---|---|---|---|---|
| | | *Average* Shear Wave Velocity, $\overline{V}_s$ (m/s) | Average Standard Penetration Resistance, $\overline{N}_{60}$ | *Soil* Undrained Shear Strength, $s_u$ |
| **A** | Hard Rock | $\overline{V}_s > 1500$ | Not applicable | Not applicable |
| **B** | Rock | $760 < \overline{V}_s \leq 1500$ | Not applicable | Not applicable |
| **C** | Very Dense Soil and Soft Rock | $360 < \overline{V}_s < 760$ | $\overline{N}_{60} > 50$ | $s_u > 100$kPa |
| **D** | Stiff Soil | $180 < \overline{V}_s < 360$ | $15 \leq \overline{N}_{60} \leq 50$ | $50 < s_u \leq 100$kPa |
| **E** | Soft Soil | $\overline{V}_s < 180$ | $\overline{N}_{60} < 15$ | $s_u < 50$kPa |
| | | Any profile with more than 3 m of *soil* with the following characteristics:<br>▪ Plastic index PI > 20<br>▪ Moisture content w ≥ 40%, and<br>▪ Undrained shear strength $s_u < 25$ kPa | | |
| **F** | Others[1] | Site Specific Evaluation Required | | |

Table 2. Values of $F_a$ and $F_v$ as a Function of Site Class and $S_a(0.2)$ and $S_a(1.0)$ (after NBCC 2005).



| Site Class | Values of $F_a$ | | | | | Values of $F_v$ | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | $S_a(0.2)$ $\leq 0.25$ | $S_a(0.2)$ $= 0.50$ | $S_a(0.2)$ $= 0.75$ | $S_a(0.2)$ $=1.00$ | $S_a(0.2)$ $\geq 1.25$ | $S_a(1.0)$ $\leq 0.1$ | $S_a(1.0)$ $= 0.2$ | $S_a(1.0)$ $= 0.3$ | $S_a(1.0)$ $=0.4$ | $S_a(1.0)$ $\geq 0.5$ |
| A | 0.7 | 0.7 | 0.8 | 0.8 | 0.8 | 0.5 | 0.5 | 0.5 | 0.6 | 0.6 |
| B | 0.8 | 0.8 | 0.9 | 1.0 | 1.0 | 0.6 | 0.7 | 0.7 | 0.8 | 0.8 |
| C | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| D | 1.3 | 1.2 | 1.1 | 1.1 | 1.0 | 1.4 | 1.3 | 1.2 | 1.1 | 1.1 |
| E | 2.1 | 1.4 | 1.1 | 0.9 | 0.9 | 2.1 | 2.0 | 1.9 | 1.7 | 1.7 |
| F | (1) | (1) | (1) | (1) | (1) | (1) | (1) | (1) | (1) | (1) |

(1) To determine Fa and Fv for site Class F, site specific geotechnical investigations and dynamic site response analyses shall be performed.

## Site Condition Considerations - Summary

- Candidate time history records should be from rock sites, or sites with soil conditions comparable to those at the site being analysed.
- It is often difficult to find time histories from sites with comparable soil conditions. An alternate approach is to select records from rock sites, then incorporate the site-specific soil conditions & properties of the structure site into the design analysis.

## Design Earthquake Scenarios

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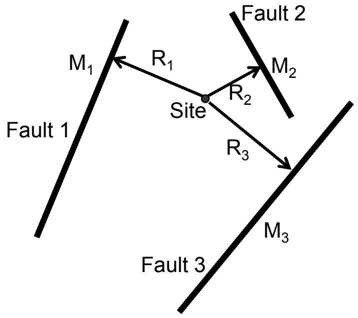
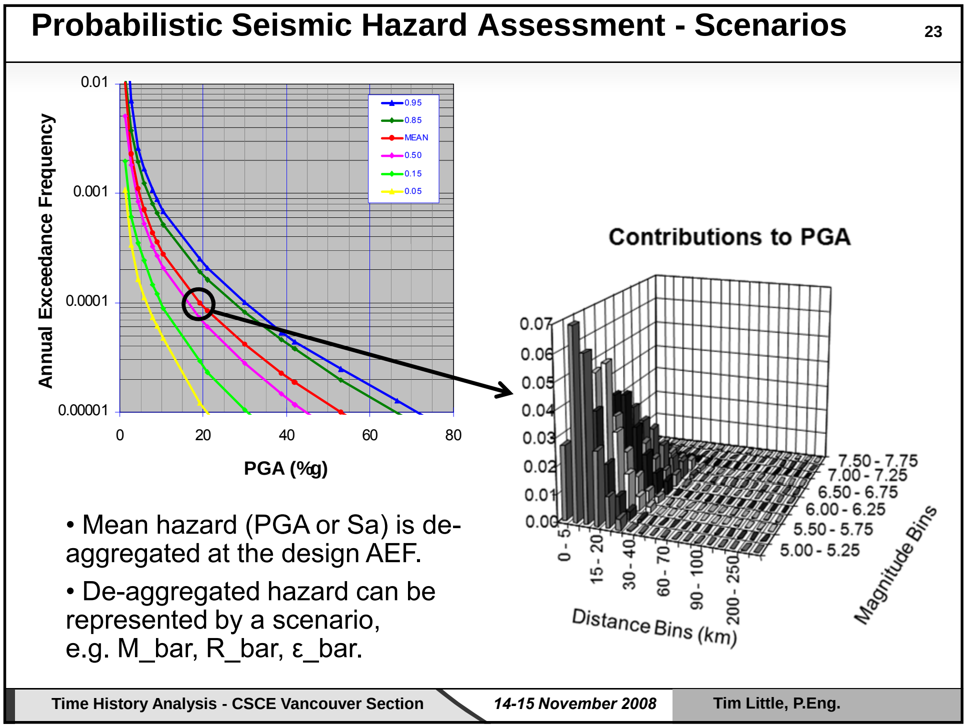
## Deterministic Seismic Hazard Assessment - Scenarios



| Source | Magnitude | Distance |
|---|---|---|
| Fault 1 | $M_1$ | $R_1$ |
| Fault 2 | $M_2$ | $R_2$ |
| Fault 3 | $M_3$ | $R_3$ |

Identification of active faults is difficult in most of Canada at this time & DSHA cannot be reliably applied.

- M based on empirical relations that correlate M with fault length, area, slip rate, etc.
- R = closest source-to-site distance
- $\varepsilon$ = 0 or 1 typically (i.e. 50th or 84th %ile)



## Probabilistic Seismic Hazard Assessment - Scenarios

- Mean hazard (PGA or Sa) is de-aggregated at the design AEF.
- De-aggregated hazard can be represented by a scenario, e.g. M_bar, R_bar, ε_bar.

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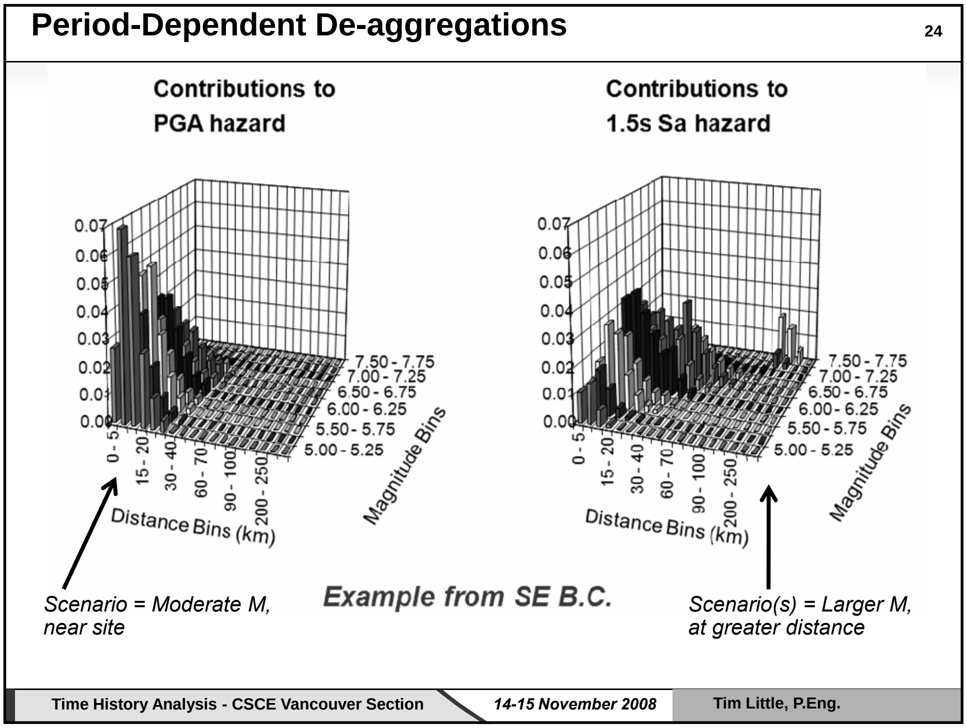

## Period-Dependent De-aggregations

*Scenario = Moderate M, near site*

***Example from SE B.C.***

*Scenario(s) = Larger M, at greater distance*

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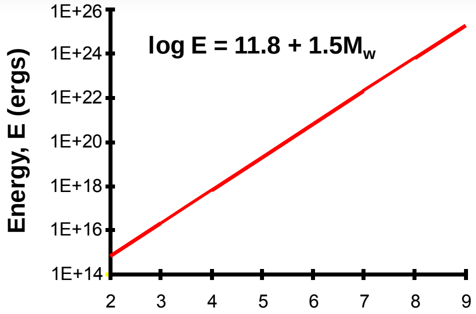
## Why Magnitude is Important



$\log E = 11.8 + 1.5M_w$

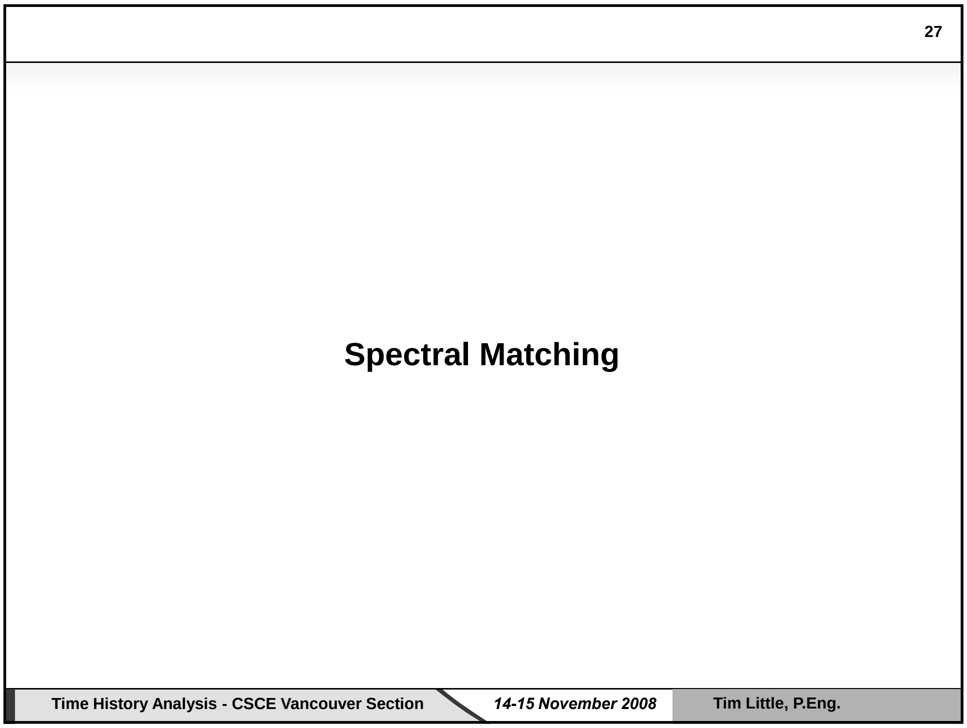
An increase of one unit of magnitude is equivalent to:

- A 10X increase in ground motion
- A 32X increase in released energy

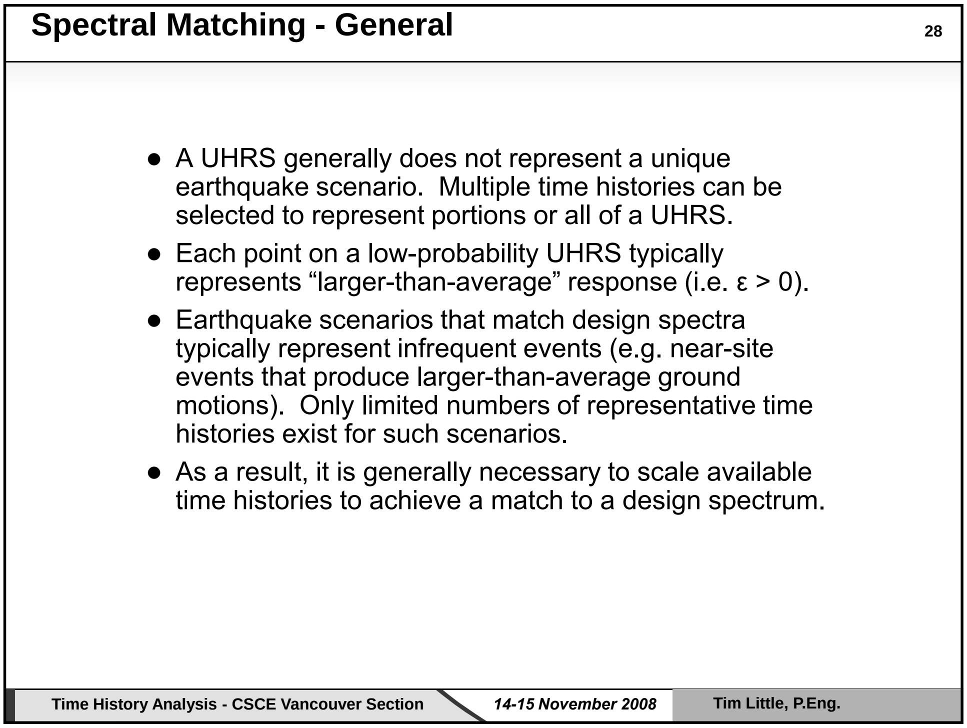
| Magnitude | Approx. Duration of Strong Shaking (sec) |
|---|---|
| 4.0 to 4.9 | <5 |
| 5.0 to 5.9 | 2 to 15 |
| 6.0 to 6.9 | 10 to 30 |
| 7.0 to 7.9 | 20 to 50 |
| 8.0 to 8.9 | 30 to 90 |

## Design EQ Scenario Considerations - Summary

- De-aggregation of hazard provides a method for selecting appropriate magnitude/distance scenarios.
- De-aggregation is typically done for PGA or Sa hazard corresponding to primary vibration mode of the structure, but don't forget about other modes.
- Magnitudes of candidate time histories should be similar to that of design scenario(s) (e.g. M_bar), typically within about ± 0.2M to 0.5M.
- Distances of candidate time histories should be similar to that of design scenario(s) (e.g. R_bar), typically within about ± 50%.
- Duration should be similar to that typically expected for the scenario magnitude.



## Spectral Matching



## Spectral Matching - General

- A UHRS generally does not represent a unique earthquake scenario. Multiple time histories can be selected to represent portions or all of a UHRS.
- Each point on a low-probability UHRS typically represents “larger-than-average” response (i.e. $\varepsilon > 0$).
- Earthquake scenarios that match design spectra typically represent infrequent events (e.g. near-site events that produce larger-than-average ground motions). Only limited numbers of representative time histories exist for such scenarios.
- As a result, it is generally necessary to scale available time histories to achieve a match to a design spectrum.

## Available Scaling Methods

- Linear scaling
  - The entire acceleration time history is scaled by a constant factor to achieve a match to target PGA or Sa at the fundamental period of the structure.
  - Frequency content and original phasing of the record are preserved.
- Frequency domain scaling
  - Involves adjusting Fourier amplitudes while maintaining Fourier phases, similar to addition or subtraction of sinusoidal waves of different periods to the full length of the original time history.
  - May produce modified time histories that significantly differ in appearance from the original time histories.
- Time domain scaling
  - “Wavelets” of finite duration are added to or subtracted from the time history to provide a match to the target spectrum at specific periods.

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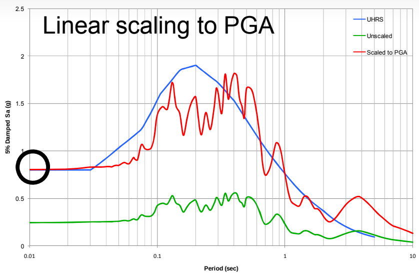
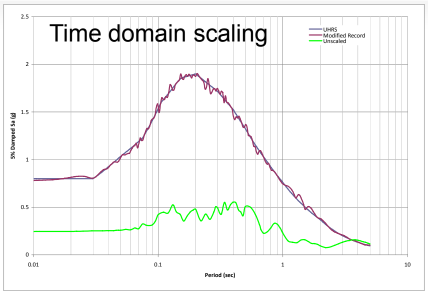
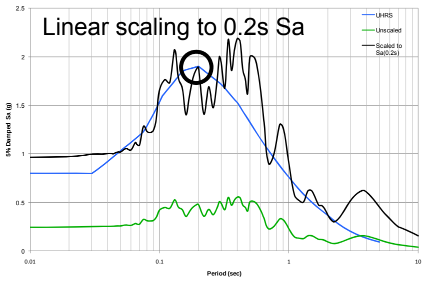
## Comparison of Some Scaling Methods







- Linear scaling factor is typically selected to achieve a match at the fundamental period of the structure.
- Other modes may be significant contributors to structural response.
- It may be necessary to aim for a general match over a range of periods.

## Spectral Matching Considerations - Summary

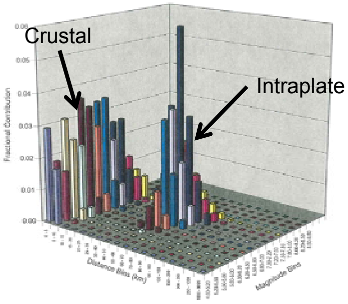
- General response spectrum shape for the candidate time history should be similar to that of the target response spectrum (e.g. no major peaks or troughs).
- It is preferable to avoid large scaling factors. For linear scaling to PGA or Sa, a rule-of-thumb is to try to avoid scaling factors larger than about 2 to 3.
- It is recommended that multiple records be selected. After scaling, the spectral shape corresponding to the mean response for the selected records should be equal to or slightly greater than the target spectrum.

## An Example

## Seismic Hazard

- Site located near Mission, B.C.
- Dynamic analyses being carried out for dam, powerhouse and soil abutment, each with different fundamental vibration periods.
- Design AEF = 1/10,000; design PGA = 0.7g.
- Contributions to hazard from both crustal and intraplate earthquakes.

| Period | Crustal earthquakes | | Deep earthquakes | |
|---|---|---|---|---|
| | M_bar | R_bar (km) | M_bar | R_bar (km) |
| PGA | 6.3 | 6 | 7.0 | 57 |
| T=0.15 sec | 6.3 | 6 | 7.1 | 56 |
| T=0.5 sec | 6.7 | 8 | 7.1 | 60 |
| T=1.0 sec | 6.9 | 9 | 7.2 | 59 |
| T=1.5 sec | 7.0 | 10 | 7.2 | 61 |



## Time History Selection Criteria for Soil Abutment

- Soil abutment has fundamental period in the range of about 0.4 to 1.0 sec.
- Initial search criteria for candidate crustal earthquakes:
  - M = 6.5 to 7.2
  - R = 0 to 12 km
  - Fault source mechanism:
    - strike slip, or
    - reverse normal, or
    - reverse-oblique,
    - but not including normal or normal-oblique due to local tectonic setting.
  - Time histories recorded on bedrock, or on shallow stiff soil profile < 20 m thick overlying bedrock .
  - Candidate records should be from a variety of earthquakes and recording stations.

## Results of Search

- Screening of PEER and COSMOS databases identified 130 candidate records generally matching the search criteria. It was necessary to relax some of the search criteria, in particular the target distance.
- Response spectra for each of the 130 records were plotted and compared to the target spectrum.
- Each record was scaled linearly to achieve a match with the target spectrum in the 0.4 to 1.0 sec period range.
- Eight records with the closest match were selected for the design analyses.

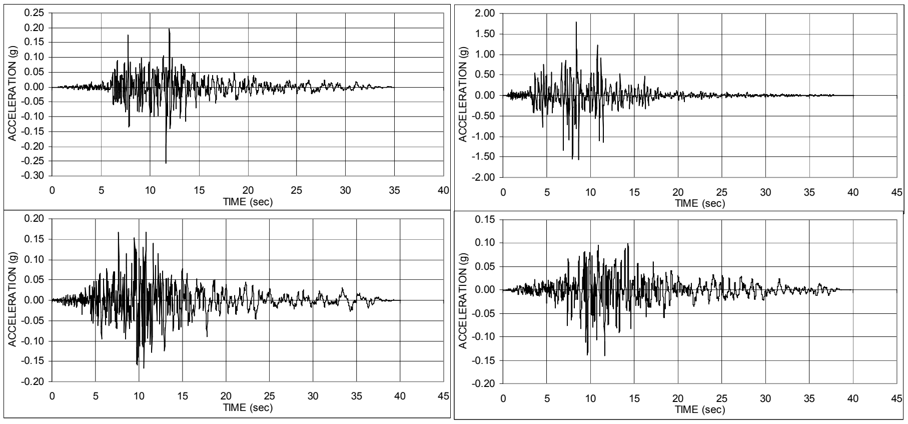
## Crustal Earthquake Time Histories Selected for Design

| Earthquake | M | Duration (sec) | Station | R (km) | $V_{s30}$ (m/s) | Scaling Factor |
|---|---|---|---|---|---|---|
| Gazli, USSR (1976) | 6.8 | 16 | Karaky | 3 | 660 | 1.0 |
| Tabas, Iran (1978) | 7.4 | 24 | Dayhook | 17 | 587 | 1.7 |
| Loma Prieta (1989) | 6.9 | 40 | Fremont | 43 | 285 | 3.9 |
| Cape Mendocino (1992) | 7.0 | 30 | Cape Mendocino | 9 | 539 | 0.7 |
| Northridge (1994) | 6.7 | 40 | San Gabriel | 42 | 694 | 3.4 |
| Northridge (1994) | 6.7 | 40 | Baldwin Hills | 26 | 297 | 3.0 |
| Northridge (1994) | 6.7 | 40 | Tarzana | 17 | 257 | 0.48 |
| Kocaeli, Turkey (1999) | 7.4 | 30 | Izmit | 5 | 811 | 2.2 |

## Unscaled Time Histories (1 of 2)

(a). -grn 270

(c) -tar 090



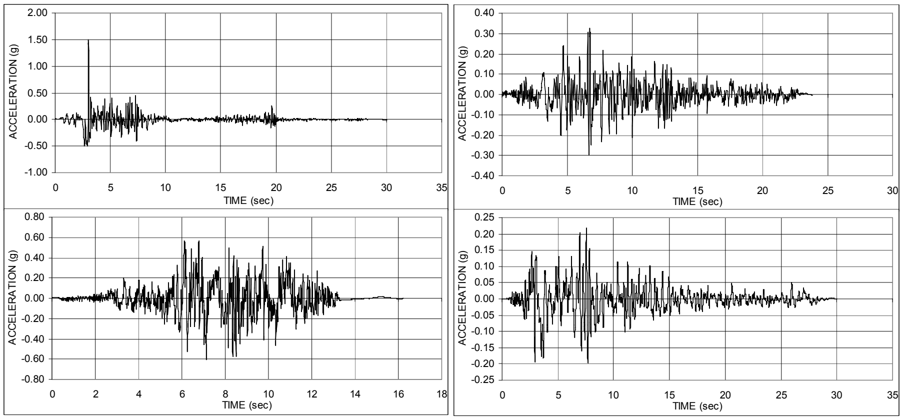
(b). -bld 360

(d) –fms 180

## Unscaled Time Histories (2 of 2)

(e) –cpm000

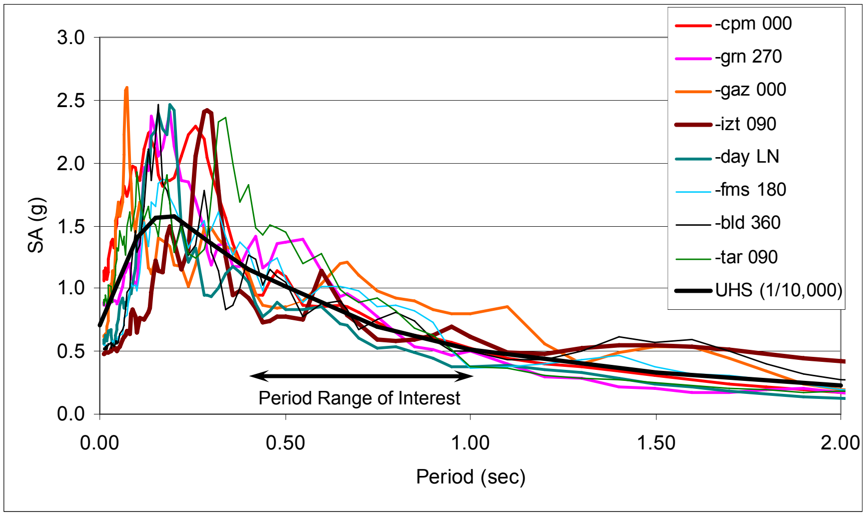
(g) –day LN



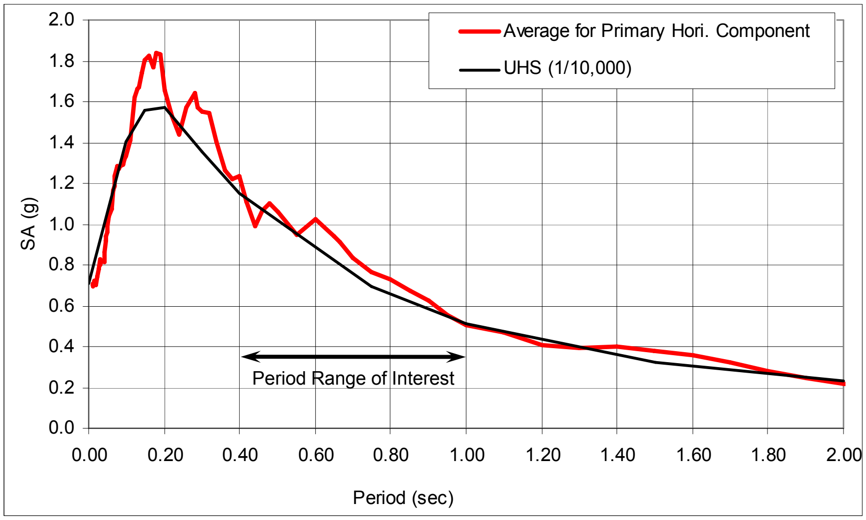
(f). –gaz000

(h) –izt 090

## Scaled Response Spectra



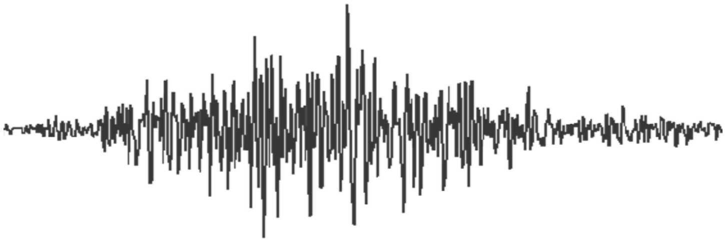
## Mean of Scaled Response Spectra

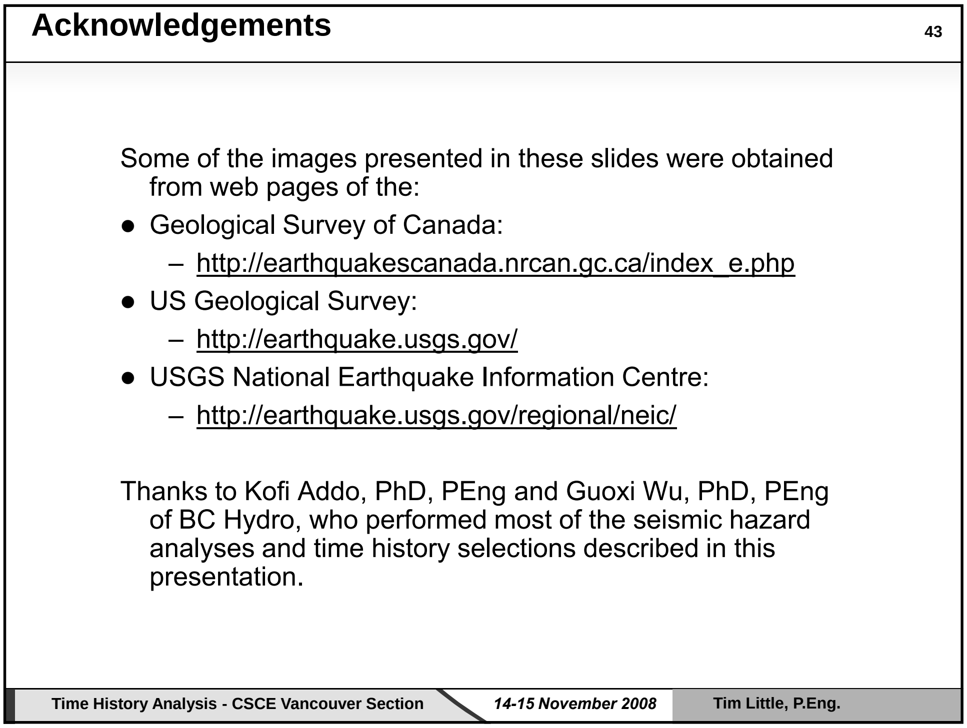


## Additional Comments

- Currently preferred methods of scaling time histories to match a target spectrum are:
  - Linear scaling
  - Time domain spectral matching
  - Conditional mean spectrum (CMS - ε) approach
- Generation of artificial or synthetic time histories is generally carried out only if no or very few appropriate natural time histories are available.
- If a 3D dynamic analysis is being carried out, the time history selection process must consider simultaneous scaling of all 3 components (2 horizontal, 1 vertical).

## *Thanks for your attention*





## Acknowledgements

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Some of the images presented in these slides were obtained from web pages of the:

- Geological Survey of Canada:
  - http://earthquakescanada.nrcan.gc.ca/index_e.php
- US Geological Survey:
  - http://earthquake.usgs.gov/
- USGS National Earthquake Information Centre:
  - http://earthquake.usgs.gov/regional/neic/

Thanks to Kofi Addo, PhD, PEng and Guoxi Wu, PhD, PEng of BC Hydro, who performed most of the seismic hazard analyses and time history selections described in this presentation.

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# TIME HISTORY ANALYSIS

## LECTURE # 3

## Selection and Scaling of Ground Motion Records

**Timothy Little, P.Eng.**
**BC Hydro Enginnering**

# NOTES

# SELECTION & SCALING OF GROUND MOTION RECORDS

## NOTES

With the increasing capability of personal computers and the availability of commercial seismic engineering software, dynamic analysis using acceleration time histories can now be readily carried out for many types of structures. The engineers performing the dynamic analyses often rely on other specialists to provide the necessary earthquake records. This seminar presentation is intended to provide those engineers with an understanding of the importance of carrying out an appropriate seismic hazard assessment and of selecting earthquake time histories in a structured manner that is consistent with the computed seismic hazard.

For more details of approaches to seismic hazard assessment, refer to Abrahamson (2007) and McGuire (2004).

For more details of approaches to selecting and scaling time history records, refer to Bommer and Acevedo (2004) and USACE (2003), Section 5 and Appendices B, C, D.

## References

Abrahamson, N.A., 2007, "Probabilistic Seismic Hazard Analysis", in Appendix B of "Evaluation of Earthquake Ground Motions" by I.M. Idriss and R.J. Archuleta, Federal Regulatory Energy Commission, Engineering Guidelines for the Evaluation of Hydropower Projects, Chapter 13 (Draft). Available for download on the internet at http://www.ferc.gov/industries/hydropower/safety/guidelines/eng-guide/chap13-draft.pdf

Bommer, J.J. and A.B. Acevedo (2004), "The use of real earthquake accelerograms as input to dynamic analysis", Journal of Earthquake Engineering, Vol. 8, Special Issue 1, pp. 43-91.

McGuire, R.K., 2004, "Seismic Hazard and Risk Analysis", Earthquake Engineering Research Institute Monograph MNO-10, 221 pp.

US Army Corps of Engineers (2003), Engineering and Design - Time-History Dynamic Analysis of Concrete Hydraulic Structures. Available for download on the internet at http://www.usace.army.mil/publications/eng-manuals/em1110-2-6051/ .

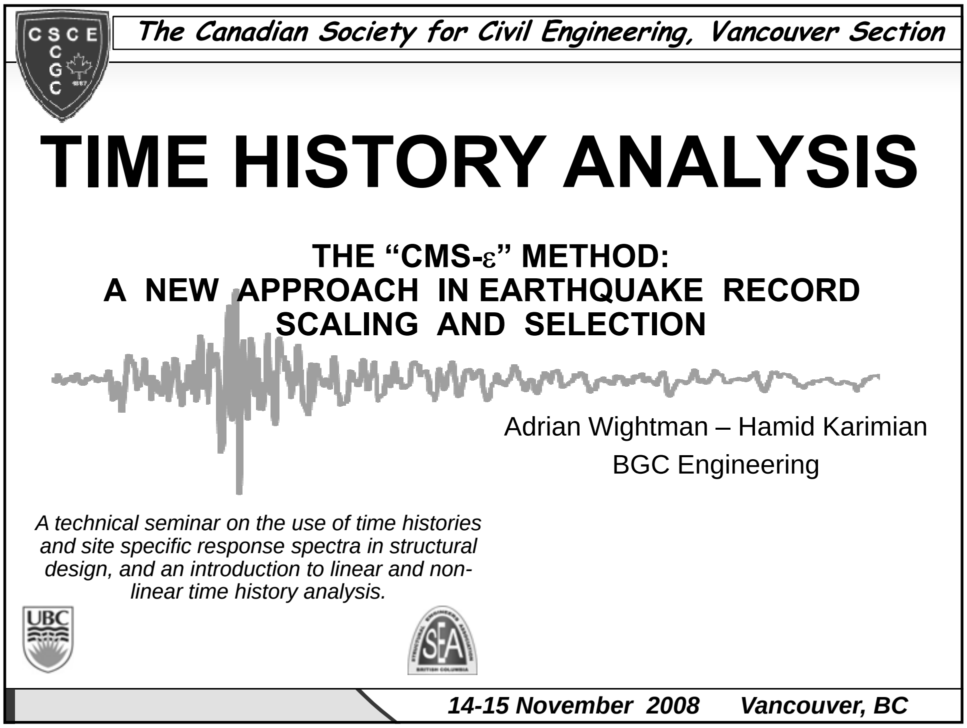
# TIME HISTORY ANALYSIS

## LECTURE # 4

## The "CMS-ε" Method:
## A New Approach in Earthquake Record Scaling and Selection

**Adrian Wightman – Hamid Karimian**
**BGC Engineering**

Adrian Wightman is a geotechnical consulting engineer specializing in earthquake geotechnical engineering, dam engineering, and dam safety. He has been working in the consulting field in Vancouver for the past 39 years.



*The Canadian Society for Civil Engineering, Vancouver Section*

# TIME HISTORY ANALYSIS

## THE “CMS-ε” METHOD: A NEW APPROACH IN EARTHQUAKE RECORD SCALING AND SELECTION

Adrian Wightman – Hamid Karimian
BGC Engineering

*A technical seminar on the use of time histories and site specific response spectra in structural design, and an introduction to linear and non-linear time history analysis.*

***14-15 November 2008 Vancouver, BC***

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## INTRODUCTION

REFERENCES:

- Abrahamson, N. A., 1992, “Non-Stationary Spectral Matching”, Seismological Research Letters Vol. 63, No. 2, 1992, pages 30.

- Abrahamson, N. A., 2006, “Seismic Hazard Assessment: Problems with Current Practice and Future Developments”, First European Conference on Earthquake Engineering and Seismology, Geneva, Switzerland, 3-8 September 2006

- Baker, J. W., Cornell, C. A., 2005, “A Vector-Valued Ground Motion Intensity Measure Consisting of Spectral Acceleration and Epsilon”, Earthquake Engineering and Structural Dynamics, April 2005, Vol. 34, No. 10, pp. 1193-1217.

- Baker, J. W., Cornell, C. A., 2006a, “Correlation of Response Spectral Values for Multicomponent Ground Motions”, Bulletin of the Seismological Society of America, February 2006, Vol. 96, No. 1, pp. 215-227.

- Baker, J. W., Cornell, C. A., 2006b, “Spectral Shape, Epsilon and Record Selection”, Earthquake Engineering and Structural Dynamics, April 2006, Vol. 35, No. 9, pp. 1077-1095

- Somerville, P. G., Smith, N. F., Graves, R. W., Abrahamson, N. A., 1997, “Modification of Empirical Strong Ground Motion Attenuation Relations to Include the Amplitude and Duration Effects on Rupture Directivity”, Seismological Research Letters, Vol. 68, No. 1, January/February 1997, pp. 199-222.

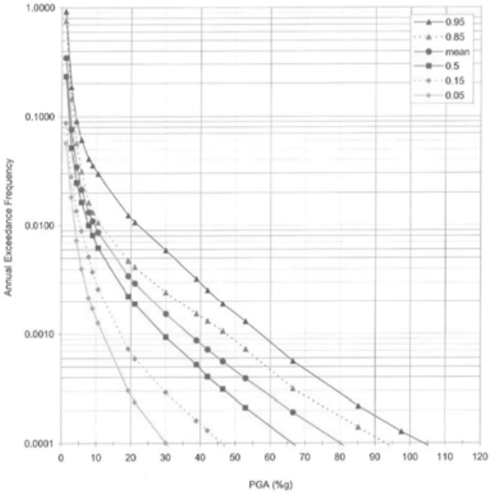
- USACE, 2003, “Engineering and Design – Time History Dynamic Analysis of Concrete Hydraulic Structures”, US Army Corps of Engineers Manual EM 1110-2-6051, December 2003

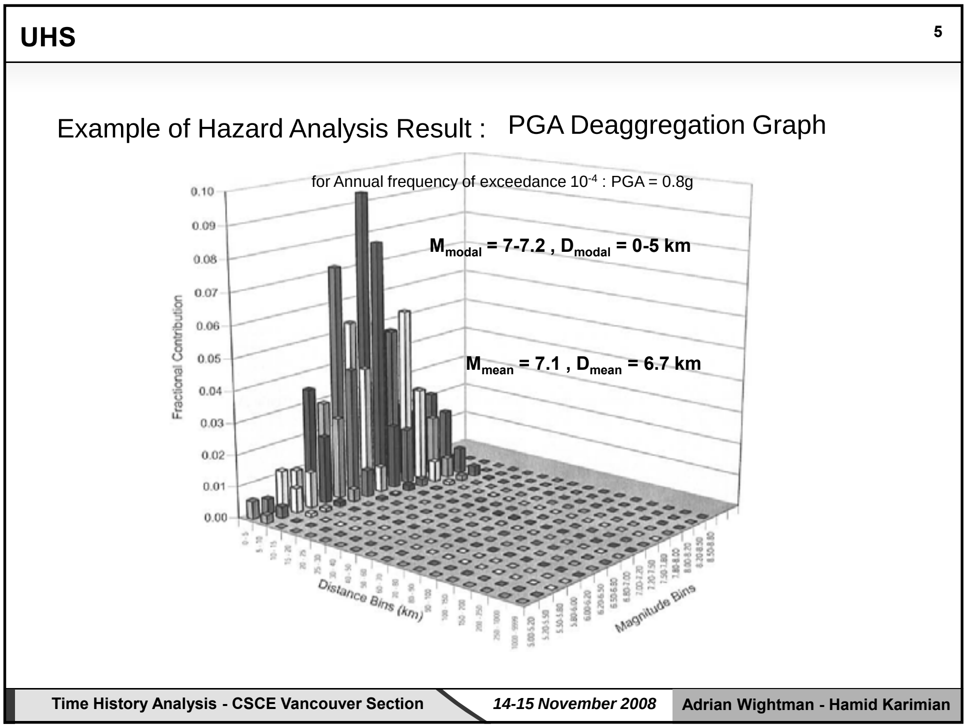
## OUTLINE

- Introduction
- UHS
- Epsilon
- Conditional mean spectrum
- Worked Example – Dam Safety
- Potential Application – NBCC
- Benefits of CMS-ε

## UHS

- Example of Hazard Analysis Result : PGA Hazard Curve





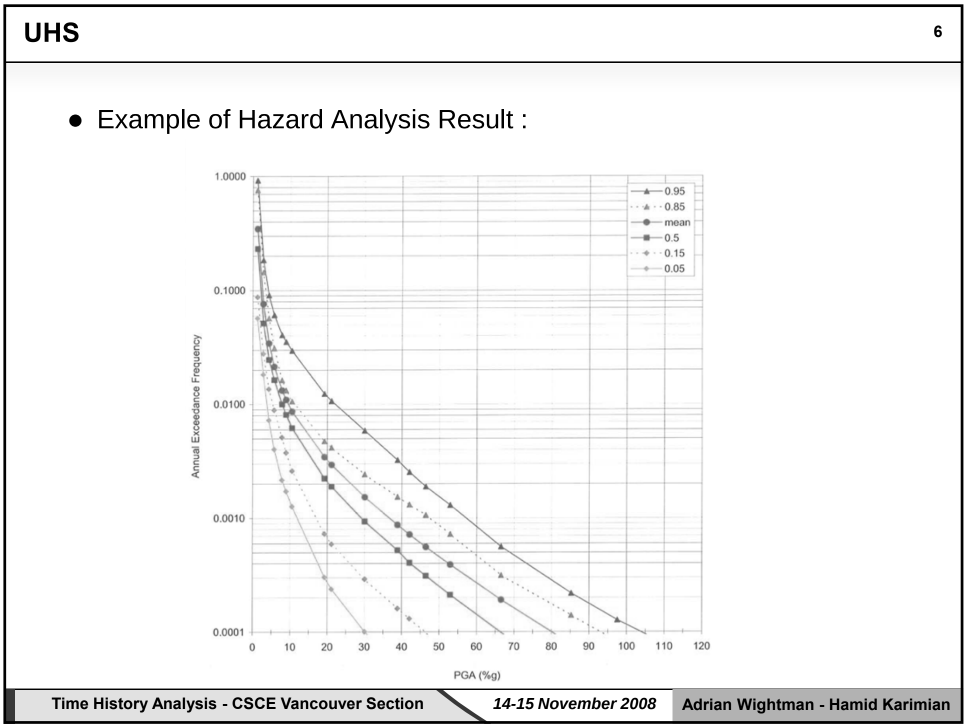
## UHS

Example of Hazard Analysis Result : PGA Deaggregation Graph

for Annual frequency of exceedance $10^{-4}$ : PGA = 0.8g

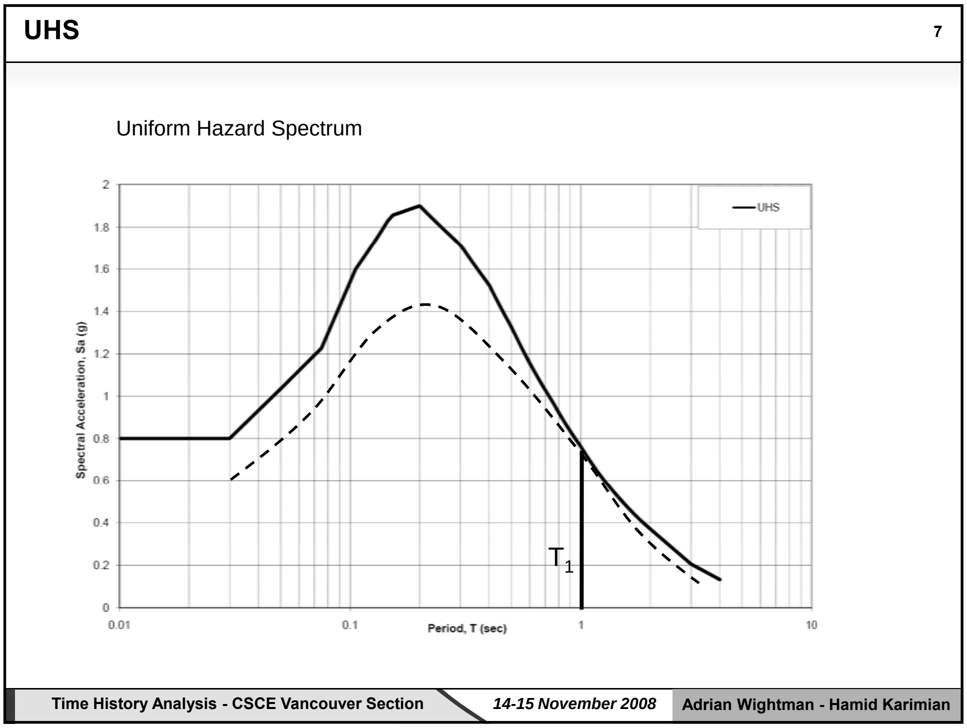
$M_{modal}$ = 7-7.2 , $D_{modal}$ = 0-5 km

$M_{mean}$ = 7.1 , $D_{mean}$ = 6.7 km



## UHS

- Example of Hazard Analysis Result :



## UHS

Uniform Hazard Spectrum

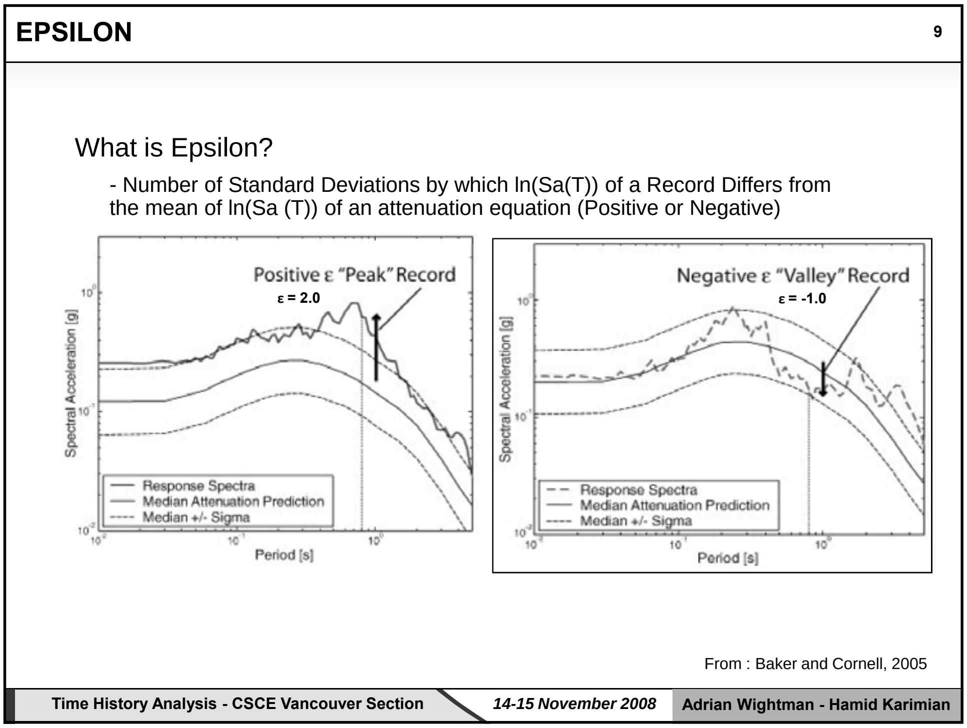
## EPSILON

### What is Epsilon?

- Number of standard deviations by which ln(Sa(T)) of a record differs from the mean of ln(Sa (T)) of an attenuation equation (Positive or Negative)

$$\ln Y = b_1 + b_2\left(M - 6\right) + b_3\left(M - 6\right)^2 + b_5 \ln r + b_V \ln \frac{V_S}{V_A} \pm \varepsilon.\sigma_{\ln Y}$$

From: Boore, Joyner, Fumal (1997)

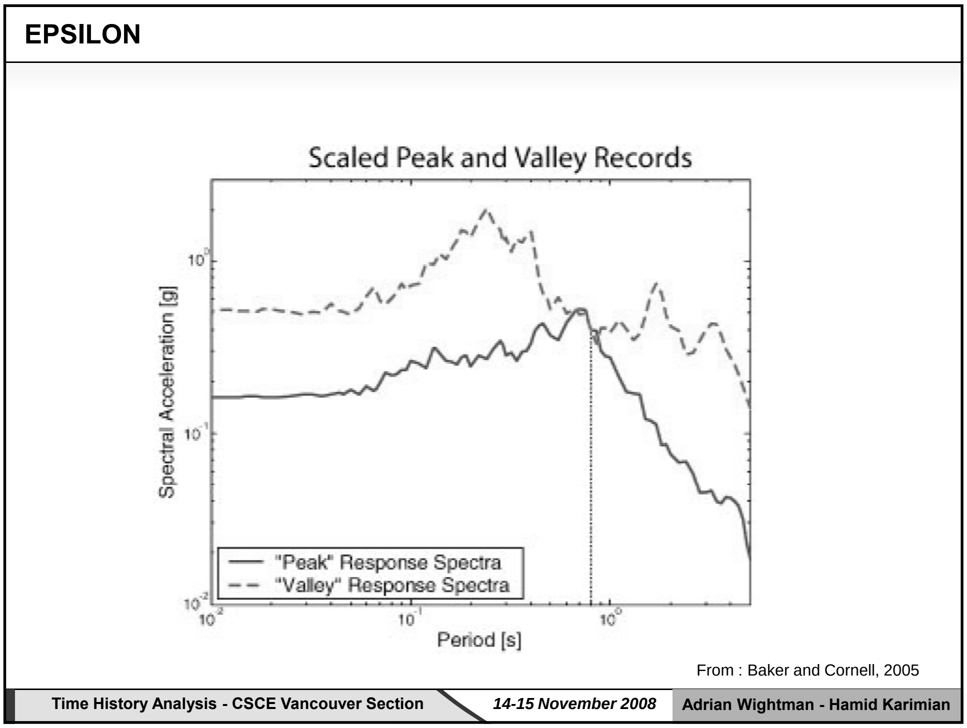


## EPSILON

What is Epsilon?

- Number of Standard Deviations by which ln(Sa(T)) of a Record Differs from the mean of ln(Sa (T)) of an attenuation equation (Positive or Negative)

From : Baker and Cornell, 2005



## EPSILON

From : Baker and Cornell, 2005

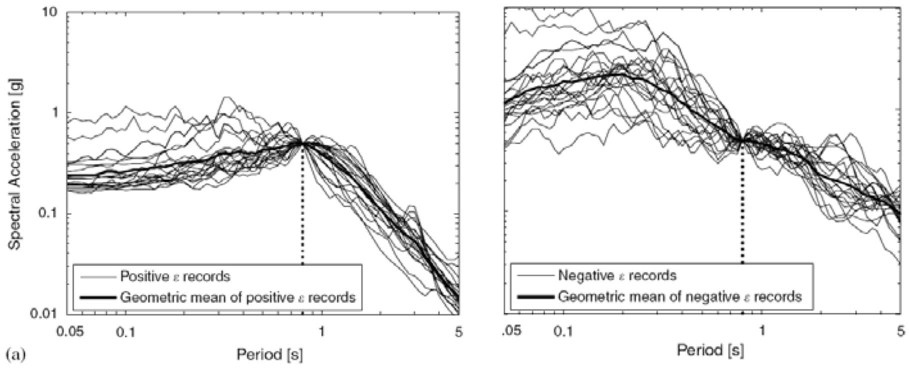
## EPSILON

- Records selected from PEER database
  - Stiff soil sites: USGS B-C
  - Free field records or 1st storey
  - M > 5.5
  - R < 100Km
  - All 3 components available
  - High-pass corner fy < 0.2Hz; low pass > 18Hz
  - No Chi-chi bias
  - A total of 191 records selected

## EPSILON

Effect of Epsilon on the Shape of Spectral Acceleration

- From PEER Database, All records with M > 5.5 and D < 100 km



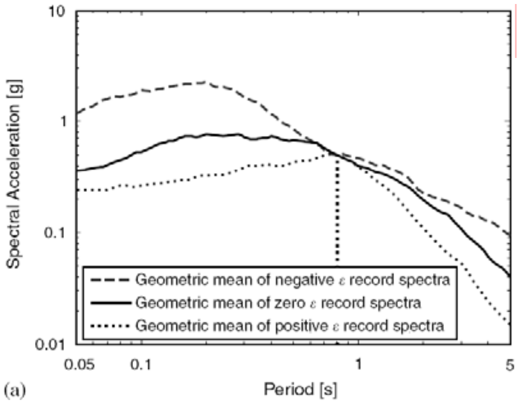
a: Response Spectra of records with top 20 epsilons – matched to Sa(0.8) = 0.5g

b: Response Spectra of records with 20 lowest epsilons – matched to Sa(0.8) = 0.5g

From : Baker and Cornell, 2006

## EPSILON

Effect of Epsilon on the Shape of Spectral Acceleration



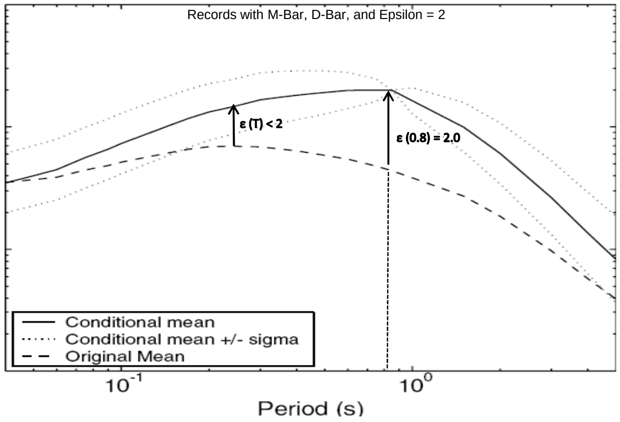
- General shape of negative epsilon records show a LOCAL sagging

- General shape of positive epsilon records show a LOCAL peak

The results of this analysis can be restated in words as follows. A record with a positive ε has a higher than expected $S_a$ value at the specified period. But $S_a$ values are not perfectly correlated, so a higher-than-average value at one period does not imply correspondingly higher-than-average values at all periods—in fact, the conditional expected values of $S_a$ at other periods tend back towards the marginal expected value. Thus, records with positive ε values tend to have peaks in the response spectrum at the specified period, and records with negative ε values tend to have valleys. Therefore, ε is an indicator of spectral shape, and this is why it is effective in predicting the response of non-linear MDOF models.

## EPSILON

For a Scenario EQ with known M-Bar and D-Bar:

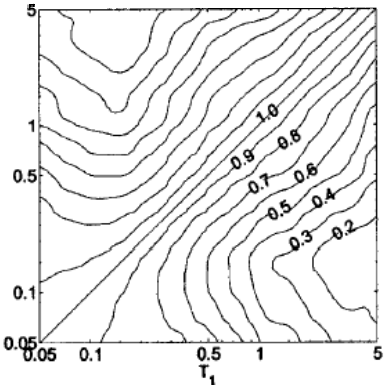


## EPSILON CORRELATIONS

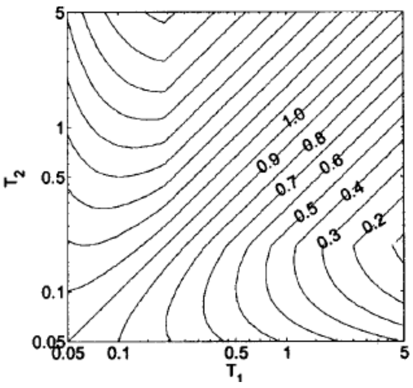
Statistic Analysis using Records from PEER Data Base (Baker & Cornell,2006):

- 1st Horizontal Component of Ground Motion: $\varepsilon_h(T)$ = ? , $\varepsilon_h(T1)$ = known

$$\varepsilon_h(T) = \rho_{Sa_h(T),Sa_h(T1)} \times \varepsilon_h(T1) \quad \textbf{Where: } \rho_{Sa_h(T),Sa_h(T1)} = f(T,T1)$$



(a): $\rho_{Sa_h(T),Sa_h(T1)}$ contours from statistical analysis



(b): $\rho_{Sa_h(T),Sa_h(T1)}$ contours from prediction model

## EPSILON CORRELATIONS

Correlation equation for same component, different periods

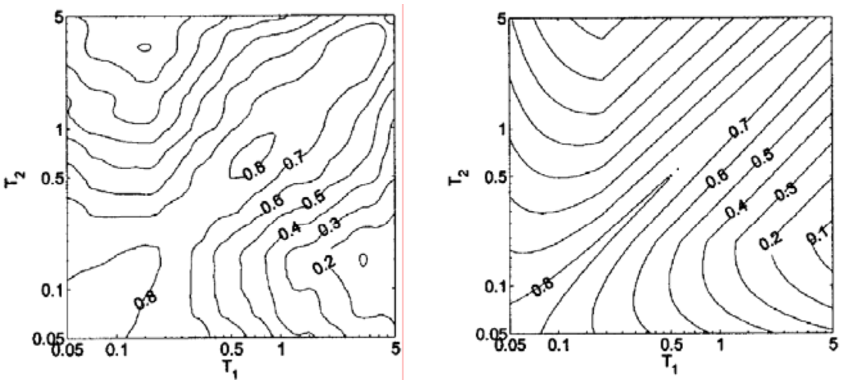
$$\rho_{\varepsilon_x,\varepsilon_y} = \left(\mathbf{0.79 - 0.023 * \ln\sqrt{T_{\min}T_{\max}}}\right)\times\left(\mathbf{1-\cos}\left(\frac{\pi}{2}-\left(\mathbf{0.359+0.163}\,I_{(T_{\min<0.189})}\,\mathbf{\ln}\frac{T_{\min}}{\mathbf{0.189}}\right)\mathbf{\ln}\frac{T_{\max}}{T_{\min}}\right)\right)$$

## EPSILON CORRELATIONS

Statistic Analysis using Records from PEER Data Base (Baker & Cornell, 2006):

- 1st and 2nd Horizontal Component of Ground Motion: $\varepsilon_{h2}(T)$ = ? , $\varepsilon_{h1}(T1)$ = known

$$\varepsilon_{h2}(T) = \rho_{Sa_{h2}(T),Sa_{h1}(T1)} \times \varepsilon_{h1}(T1) \quad \textbf{Where: } \rho_{Sa_{h2}(T),Sa_{h1}(T1)} = f(T, T1)$$



(a): $\rho_{Sa_{h2}(T),Sa_{h1}(T1)}$ contours from statistical analysis

(b): $\rho_{Sa_{h2}(T),Sa_{h1}(T1)}$ contours from prediction model

## EPSILON CORRELATIONS

- Correlation equation for perpendicular horizontal components, different periods

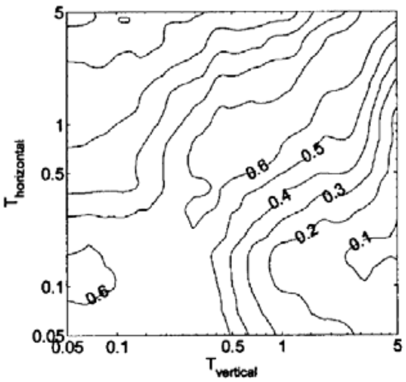
$$\rho_{\varepsilon_x,\varepsilon_z} = \left(\mathbf{0.64} + \mathbf{0.021} * \ln\sqrt{T_{\min}T_{\max}}\right) \times \left(1 - \cos\left(\frac{\pi}{2} - \left(\ln\frac{T_{\max}}{T_{\min}}\right)\left(0.29 + 0.094 I_{(T_{\min<0.189})} \ln\frac{T_{\min}}{0.189}\right)\right)\right)$$

## EPSILON CORRELATIONS

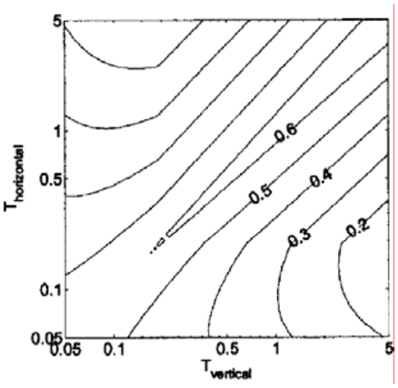
Statistic Analysis using Records from PEER Data Base (Baker & Cornell 2006):

- Horizontal & Vertical Component of Ground Motion: $\varepsilon_v(T)$ = ? , $\varepsilon_h(T1)$ = known

$$\varepsilon_v(T) = \rho_{Sa_v(T),Sa_h(T1)} \times \varepsilon_h(T1) \quad \textbf{Where: } \rho_{Sa_v(T),Sa_h(T1)} = f(T,T1)$$



(a): $\rho_{Sa_v(T),Sa_h(T1)}$ contours from statistical analysis



(b): $\rho_{Sa_v(T),Sa_h(T1)}$ contours from prediction model

## EPSILON CORRELATIONS

- Correlation equation for horizontal and vertical components, different periods

$$\rho_{\varepsilon_x,\varepsilon_z} = \left(\mathbf{0.64 + 0.021 * \ln \sqrt{T_{min} T_{max}}}\right) \times \left(\mathbf{1 - \cos}\left(\frac{\pi}{\mathbf{2}} - \left(\mathbf{\ln \frac{T_{max}}{T_{min}}}\right)\left(\mathbf{0.29 + 0.094} I_{(T_{min<0.189})} \mathbf{\ln \frac{T_{min}}{0.189}}\right)\right)\right)$$

## CMS-ε TARGET SPECTRA

Required Input Data to Develop CMS-ε Target Spectra:

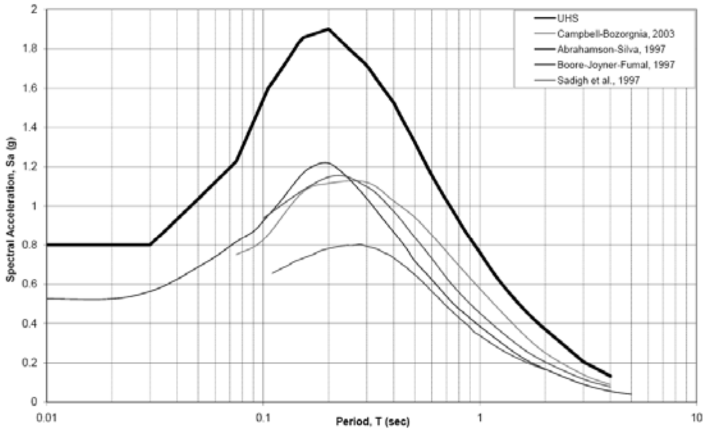
- Period of Significance for the Structure
- Spectral Acceleration at Period of Significance and for Design Return Period, Sa(T1)
  (Can be obtained from the UHS)
- Attenuation Equation(s) (same as used in hazard analysis)
- Parameters for Scenario Earthquake (e.g. M-Bar, D-Bar, etc)
  (Can be obtained from the Deaggregation Data)

Procedure to Develop CMS-ε Target Spectra:

- Procedure explained through a worked example

## CMS-ε TARGET SPECTRA WORKED EXAMPLE

- Period of Significance for the Structure : 0.6 sec
- Sa(0.6) for probability of exceedance of 10e-4 = 1.15g
- Scenario EQ Parameters: M-Bar = 7.3, D-bar = 8.6 km
- Attenuation Equations: CB, AS, BJF, Sadigh

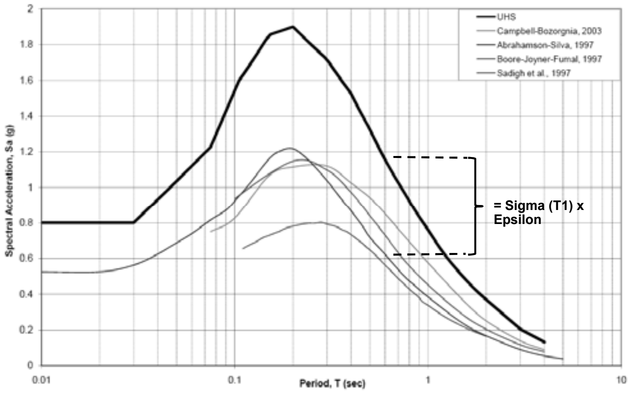


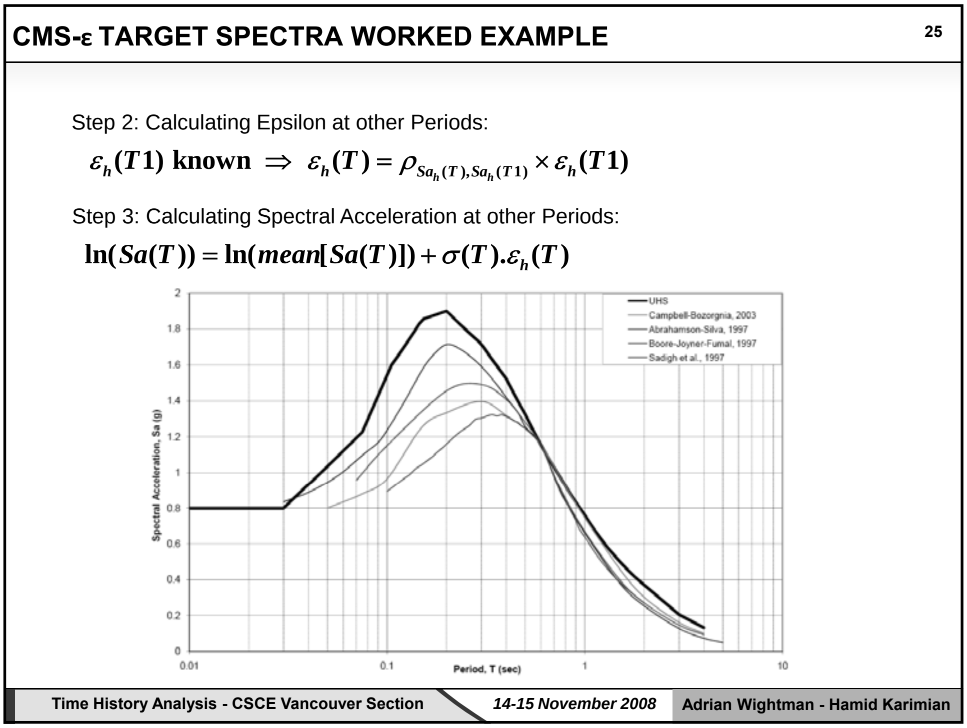
## CMS-ε TARGET SPECTRA WORKED EXAMPLE

Step 1: Finding Epsilon at T1 = 0.6s for each attenuation equation

$$\varepsilon_h(T1) = \frac{\ln(Sa^*(T1)) - \ln(mean[Sa(T1)])}{\sigma(T1)} \xrightarrow{\text{e.g.}} \varepsilon_h(0.6\,\mathrm{sec}) = \frac{\ln(1.15) - \ln(0.63)}{\sigma(0.56)} = 1.11$$

**$mean[Sa(T1)]$ & $\sigma(T1)$ are mean and standard deviation of the attenuation equation**





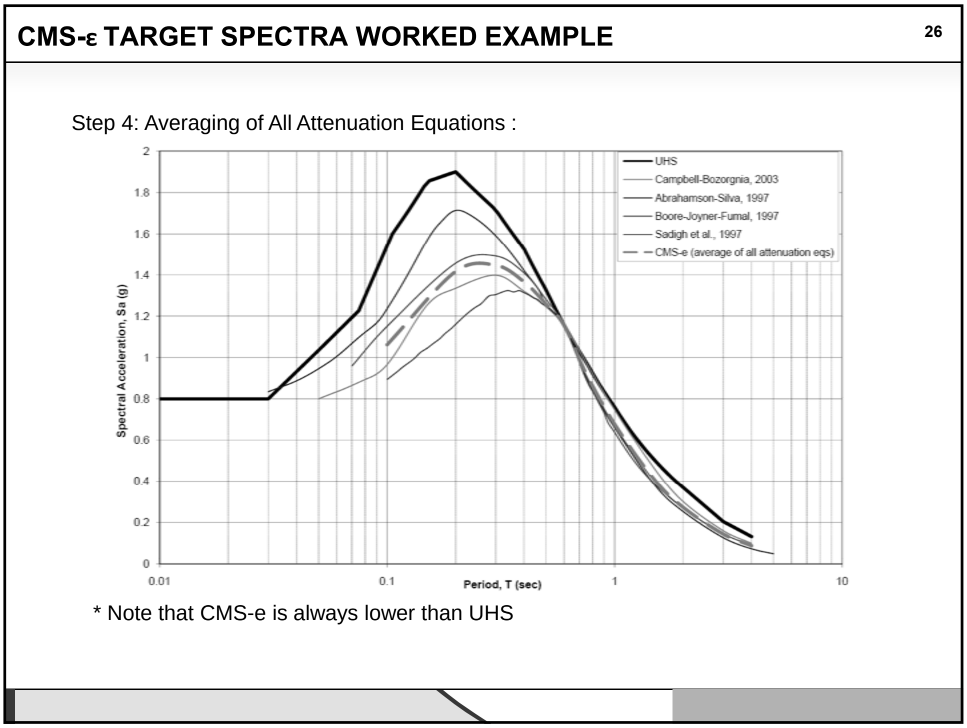
## CMS-ε TARGET SPECTRA WORKED EXAMPLE

Step 2: Calculating Epsilon at other Periods:

$$\varepsilon_h(T1)\ \textbf{known} \Rightarrow \varepsilon_h(T) = \rho_{Sa_h(T),Sa_h(T1)} \times \varepsilon_h(T1)$$

Step 3: Calculating Spectral Acceleration at other Periods:

$$\ln(Sa(T)) = \ln(mean[Sa(T)]) + \sigma(T).\varepsilon_h(T)$$



## CMS-ε TARGET SPECTRA WORKED EXAMPLE

Step 4: Averaging of All Attenuation Equations :

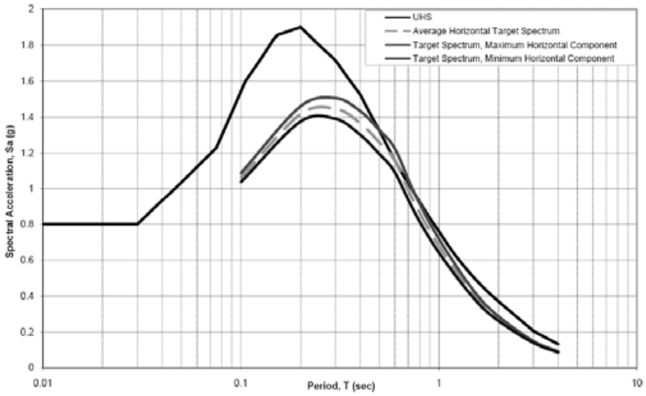
* Note that CMS-e is always lower than UHS

## CMS-ε TARGET SPECTRA WORKED EXAMPLE

Step 5: Maximum and Minimum Horizontal Ground Motion Components (if required)

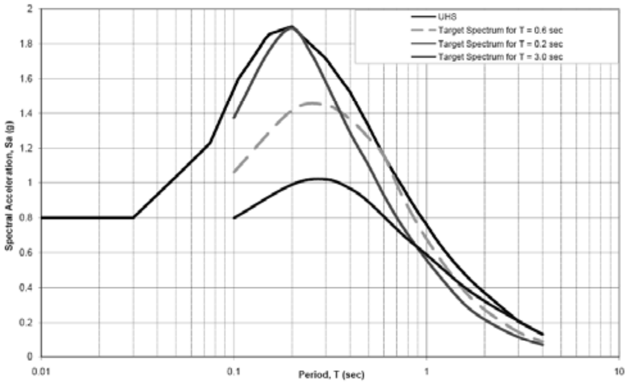
- UHS (and CMS-ε at Step 4) usually is for average horizontal component

$$\left.\begin{array}{l} \ln(Sa(T)) = \dfrac{\ln_{\max}(Sa(T)) + \ln_{\min}(Sa(T))}{2} \\ \varepsilon_{\min}(T) = \rho_{Sa_{\max}(T),Sa_{\min}(T)} \times \varepsilon_{\max}(T) \end{array}\right\} \varepsilon_{\min}(T) \text{ and } \varepsilon_{\max}(T) \text{ for each period}$$



## CMS-ε TARGET SPECTRA WORKED EXAMPLE

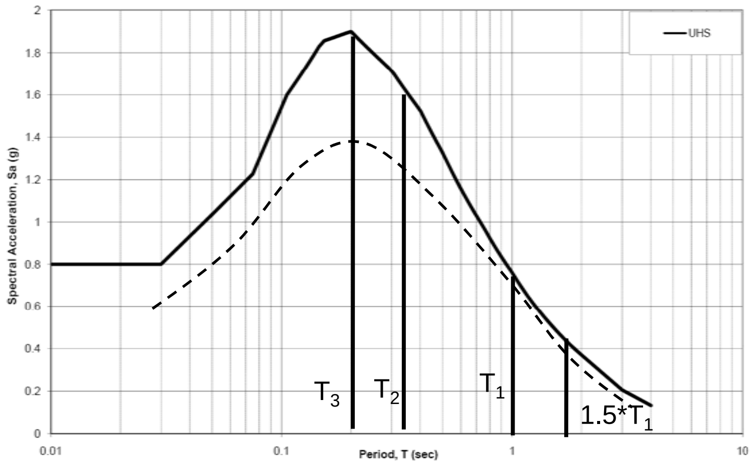
Comparison of Scenario Target Spectra: CMS-ε for Various Periods of Significance



- Sa(T) for each CMS-e is equal to the UHS value at period of significance
- Sa(T) for each CMS-e is less than the UHS value at any other period

## UHS

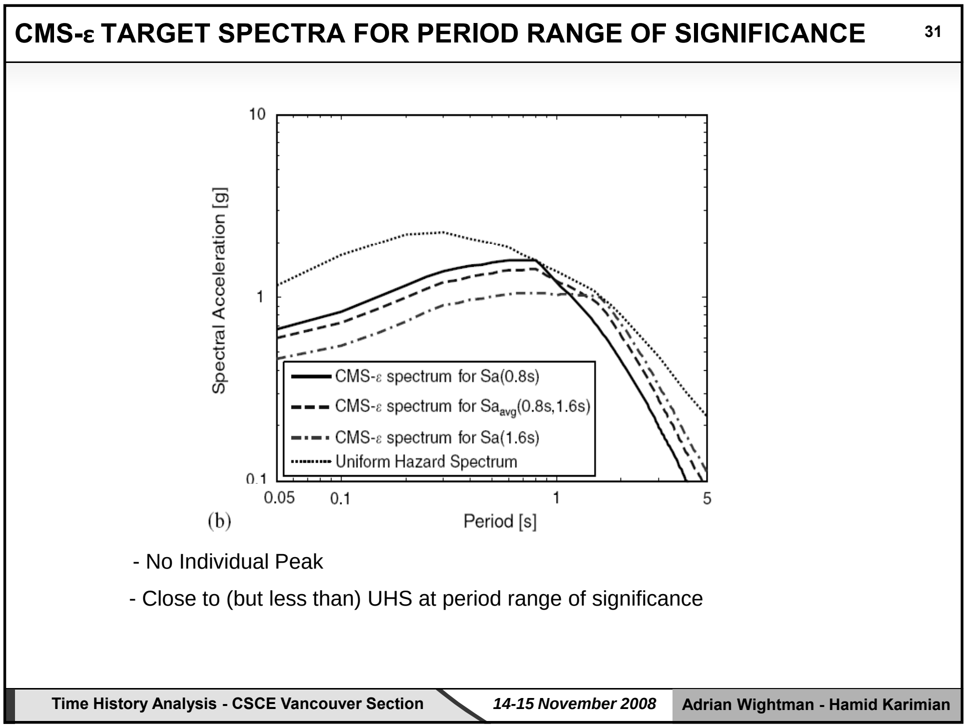
Uniform Hazard Spectrum



## CMS-ε TARGET SPECTRA WORKED EXAMPLE

….loosely speaking, rather than worrying about a spectrum that is 'very' strong at a single period, one might worry more about *an equally rare spectrum* that is 'somewhat' strong at several periods.

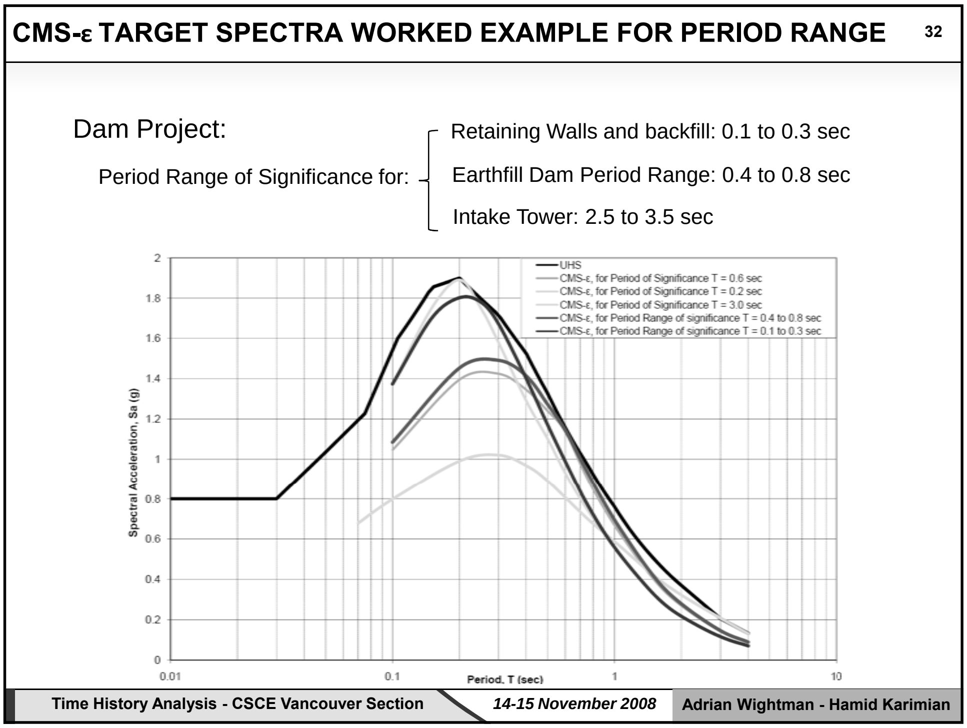
From : Baker and Cornell, 2006



## CMS-ε TARGET SPECTRA FOR PERIOD RANGE OF SIGNIFICANCE

(b)

- No Individual Peak
- Close to (but less than) UHS at period range of significance



## CMS-ε TARGET SPECTRA WORKED EXAMPLE FOR PERIOD RANGE

Dam Project:

Period Range of Significance for:
- Retaining Walls and backfill: 0.1 to 0.3 sec
- Earthfill Dam Period Range: 0.4 to 0.8 sec
- Intake Tower: 2.5 to 3.5 sec

## *EARTHQUAKE RECORD SELECTION*

---

## EARTHQUAKE RECORD SELECTION DIFFERENT APPROACHES

Potential Record Selection Strategies

1. Select records randomly: AR Method

2. Select records based on M-Bar and R-Bar from deaggregation results, no direct attempt to match Epsilon: MR-BR Method (Common Method)

3. Select records based on Epsilon value representing site hazard, no direct attempt to match M and R: ε –BR Method

4. Select records that their response spectra match the shape of CMS- ε target spectrum no direct attempt to match M, R, or ε value: CMS–ε Method*

* In the CMS- ε method, M, R, and ε value are already attributed to construct the CMS–ε target spectrum

## EARTHQUAKE RECORD SELECTION OTHER SELECTION CRITERIA

Criteria for Earthquake record Selection:

- Primary Criterion: 1. Spectral Shape of the Record
  (or Epsilon in the absence of Target spectrum)

- Secondary Criteria: 2. Pulse Characteristics and Directivity Effects
  3. Earthquake “Energy” * → Equivalent Number of Cycles
    → Earthquake Magnitude
    → Significant Duration
  4. Sub-surface Conditions (Monitor for liquefaction, etc)

## EARTHQUAKE RECORD SELECTION WORKED EXAMPLE

Dam Project: Selecting Record for Period Range 0.4 to 0.8 sec

1. Source PEER Database: Number of Records ~ 3500

2. M-Bar = 7.3 → Ignore Records with M < 6.5 : Number of records ~ 1100

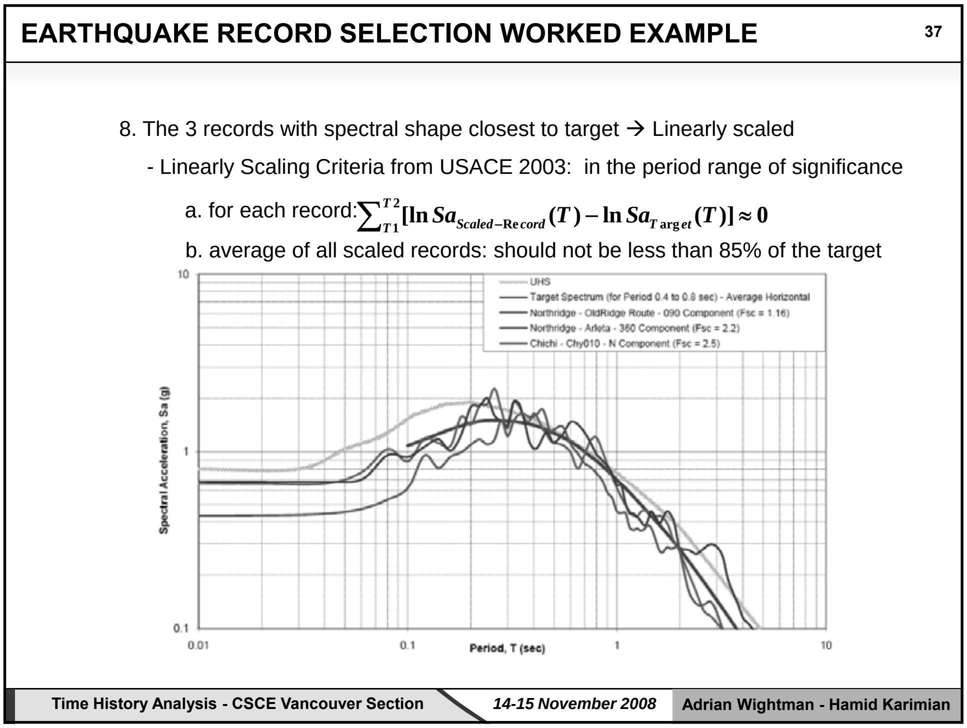
3. Spectral Shape of Average Horizontal vs. CMS-ε Target
   - Matching the overall shape
   - Linear Scaling Factor <~ 2 : Number of selected records ~ 15

4. Spectral shape of two horizontal components match with Target spectra:
   : Number of selected records ~10

5. Check for duration and equivalent number of cycles (geotechnical criteria)

6. Check diversity earthquake sources (no more than 2 records from 1 earthquake)

7. Check directivity for long P records (high Sa in long P, fling in V time history, & V/A)



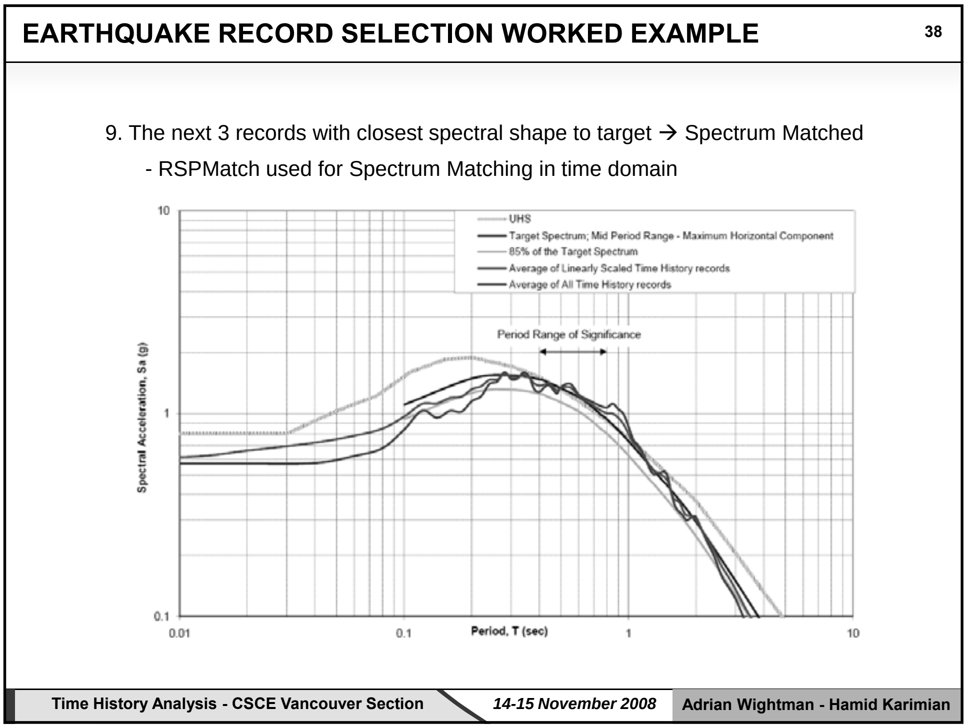
## EARTHQUAKE RECORD SELECTION WORKED EXAMPLE

8. The 3 records with spectral shape closest to target → Linearly scaled

- Linearly Scaling Criteria from USACE 2003: in the period range of significance

a. for each record: $\sum_{T1}^{T2}[\ln Sa_{Scaled-\mathrm{Re}cord}(T)-\ln Sa_{T\arg et}(T)]\approx 0$

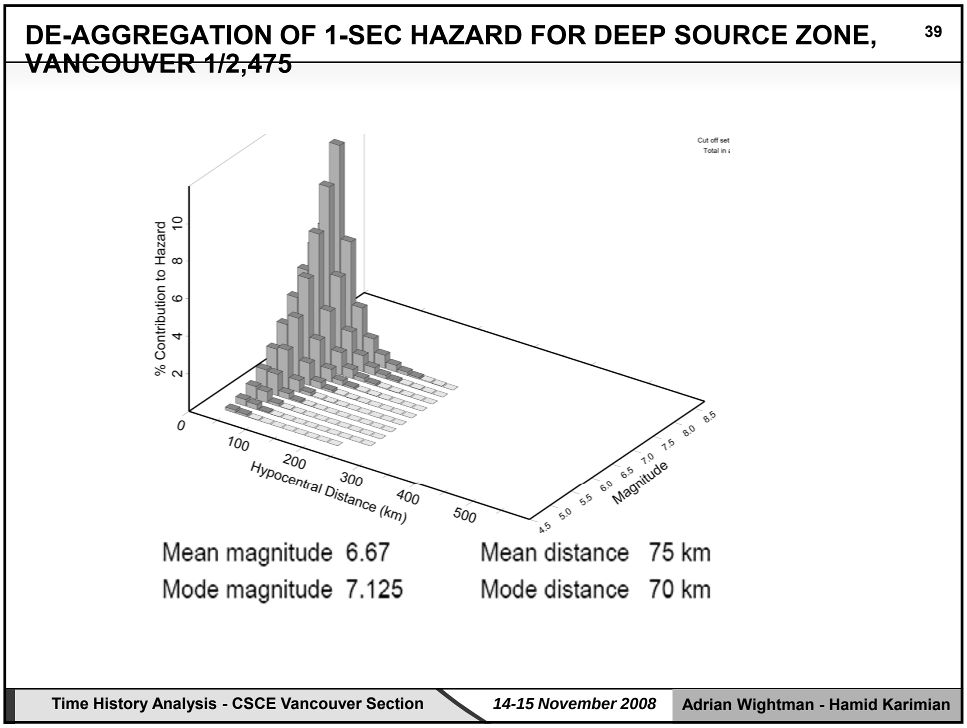
b. average of all scaled records: should not be less than 85% of the target



## EARTHQUAKE RECORD SELECTION WORKED EXAMPLE

9. The next 3 records with closest spectral shape to target → Spectrum Matched

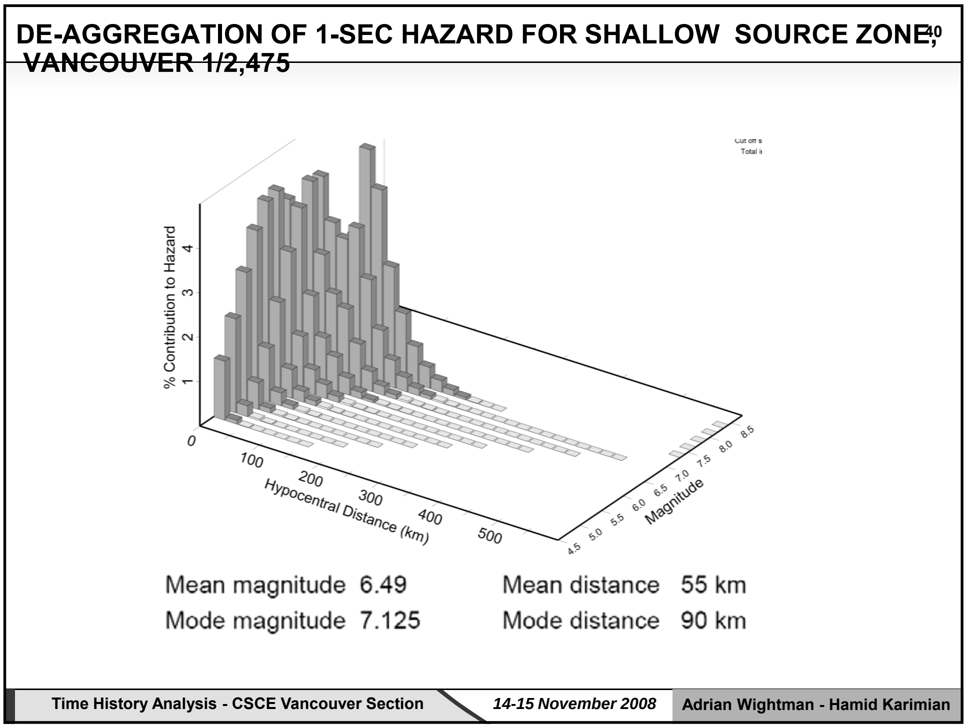
- RSPMatch used for Spectrum Matching in time domain



## DE-AGGREGATION OF 1-SEC HAZARD FOR DEEP SOURCE ZONE, VANCOUVER 1/2,475

Mean magnitude 6.67
Mode magnitude 7.125

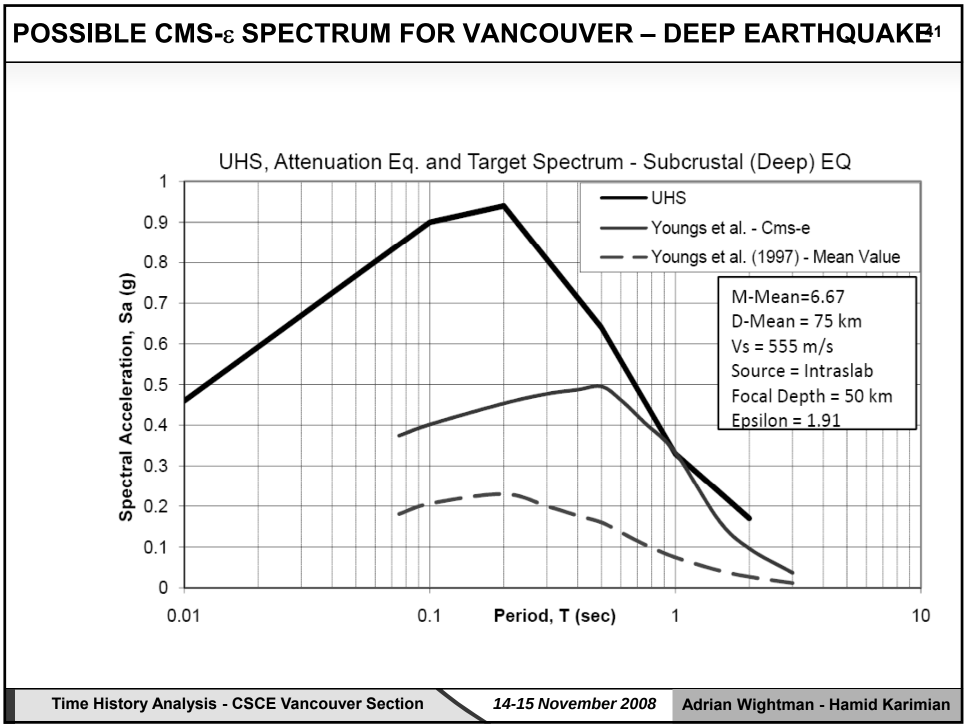
Mean distance 75 km
Mode distance 70 km



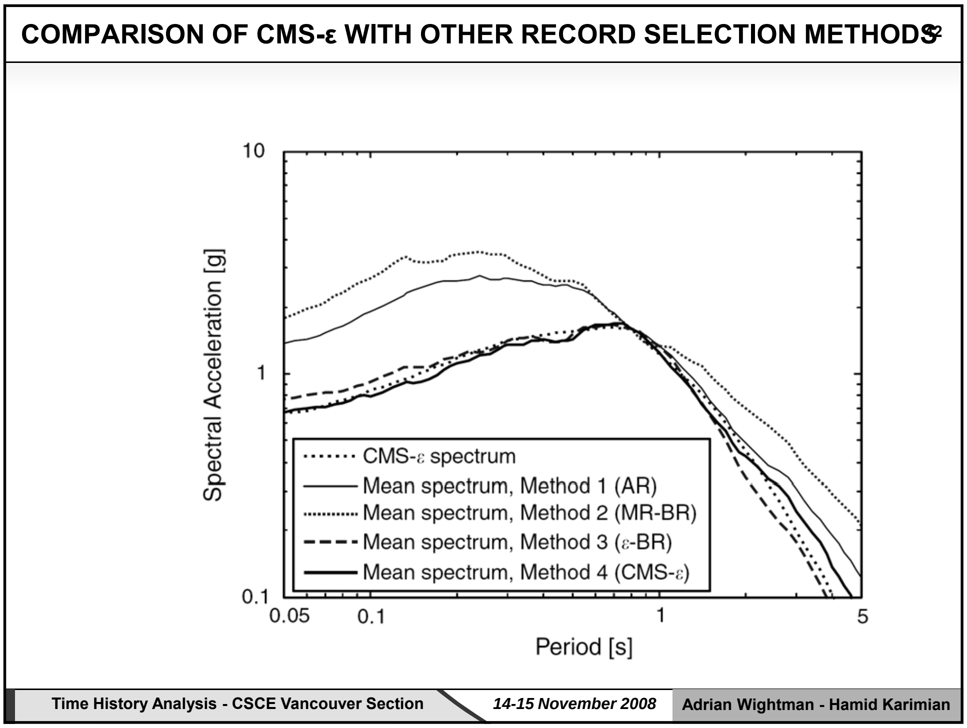
## DE-AGGREGATION OF 1-SEC HAZARD FOR SHALLOW SOURCE ZONE, VANCOUVER 1/2,475

Mean magnitude 6.49
Mode magnitude 7.125

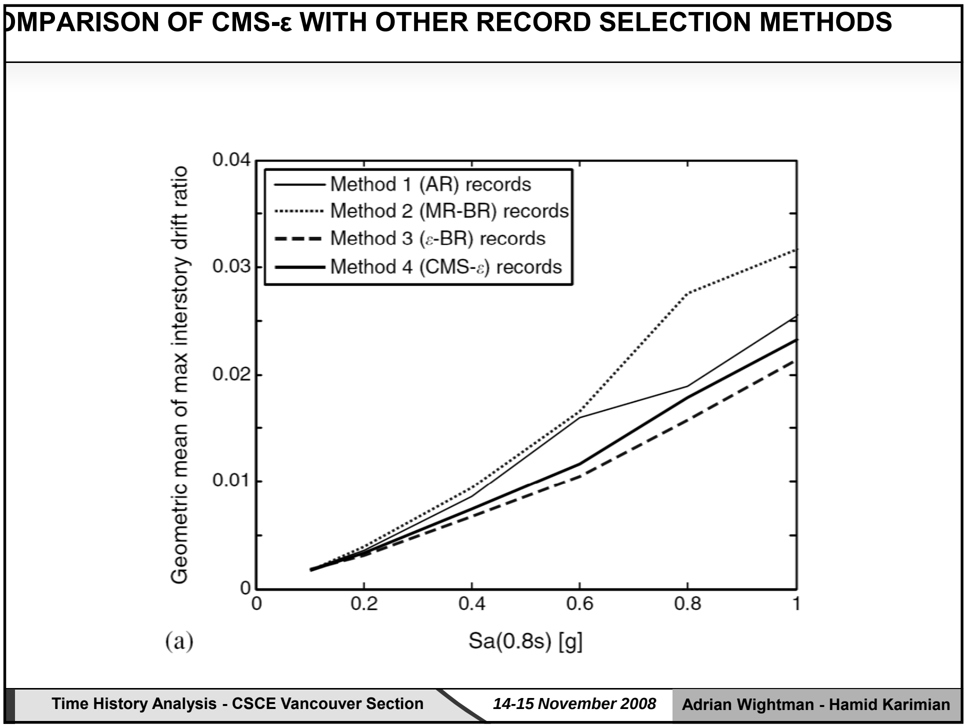
Mean distance 55 km
Mode distance 90 km



## POSSIBLE CMS-ε SPECTRUM FOR VANCOUVER – DEEP EARTHQUAKE

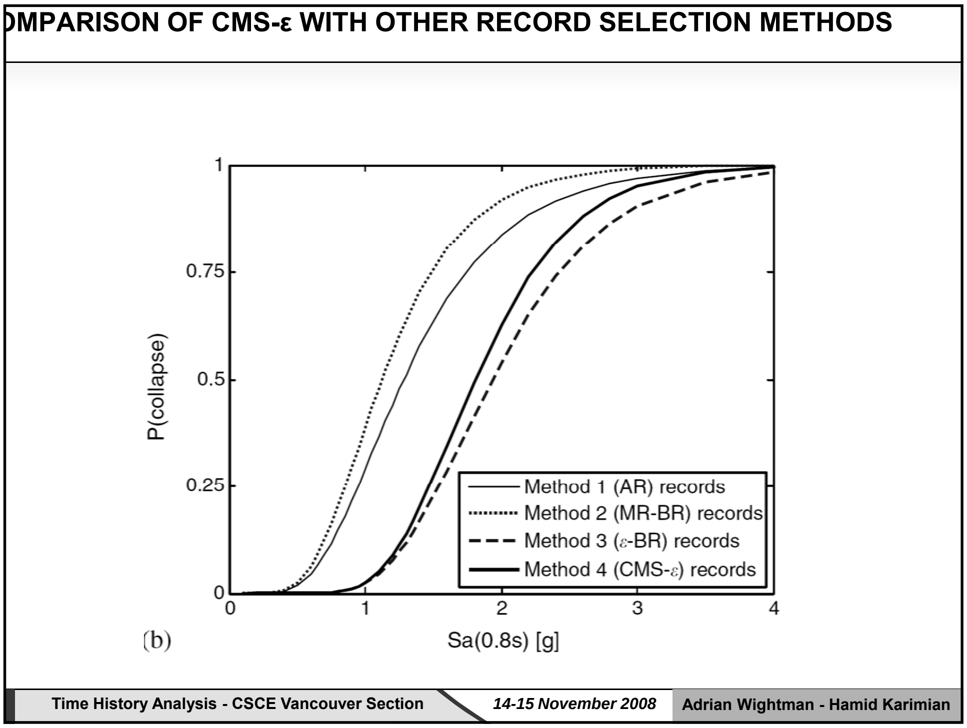


## COMPARISON OF CMS-ε WITH OTHER RECORD SELECTION METHODS



MPARISON OF CMS-ε WITH OTHER RECORD SELECTION METHODS

Time History Analysis - CSCE Vancouver Section | 14-15 November 2008 | Adrian Wightman - Hamid Karimian



MPARISON OF CMS-ε WITH OTHER RECORD SELECTION METHODS

Time History Analysis - CSCE Vancouver Section | 14-15 November 2008 | Adrian Wightman - Hamid Karimian

*END*

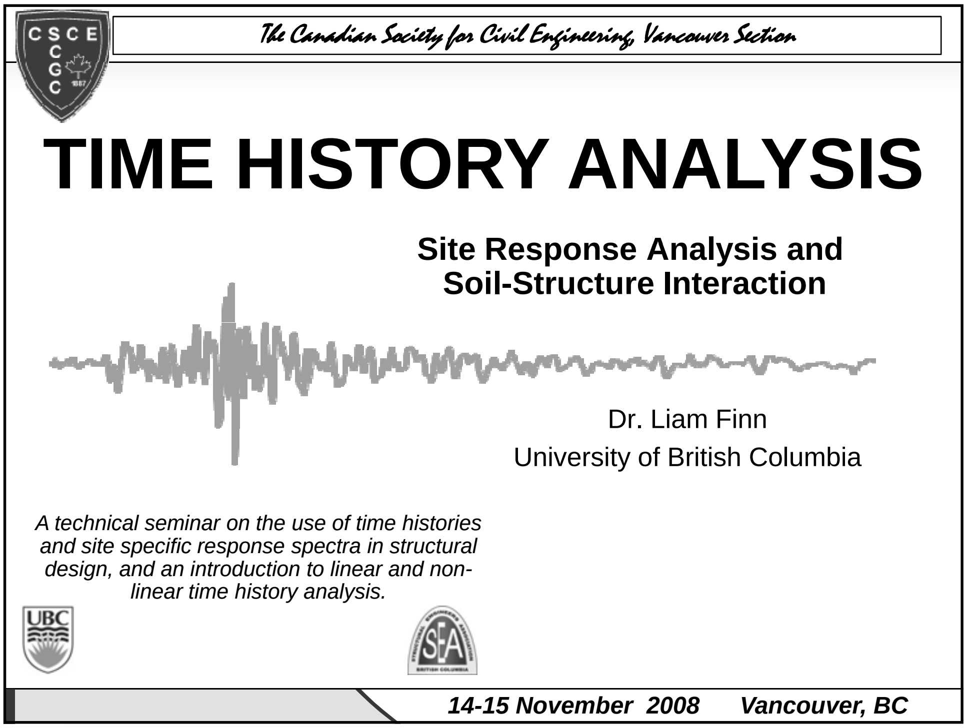
# TIME HISTORY ANALYSIS

## LECTURE # 5

## Site Response Analysis and Soil-Structure Interaction

**Liam Finn**
**University of British Columbia**

Liam Finn was Professor of Civil Engineering at UBC from 1961-1999 and Anabuki Research Professor of Foundation Geodynamics, Kagawa University, Japan 2000-2005. He is Editor of the Journal of Soil Dynamics and Earthquake Engineering and an international consultant in geotechnical earthquake engineering. He is an Honorary International Member of the Japanese Geotechnical Society and the Chinese Soil Dynamics Society and is Honorary Professor of the Institute of Building Construction in Beijing. He is a Fellow of Churchill College, Cambridge and a Fellow of the Engineering Institute of Canada. In 2005 he presented the 10th Mallet- Milne Lecture, “A Study of Piles during Earthquakes: Issues of Design and Analysis” presented to ICE, London, UK. He currently sits on the Technical Review Board for the Seismic Retrofit of BC Schools.



*The Canadian Society for Civil Engineering, Vancouver Section*

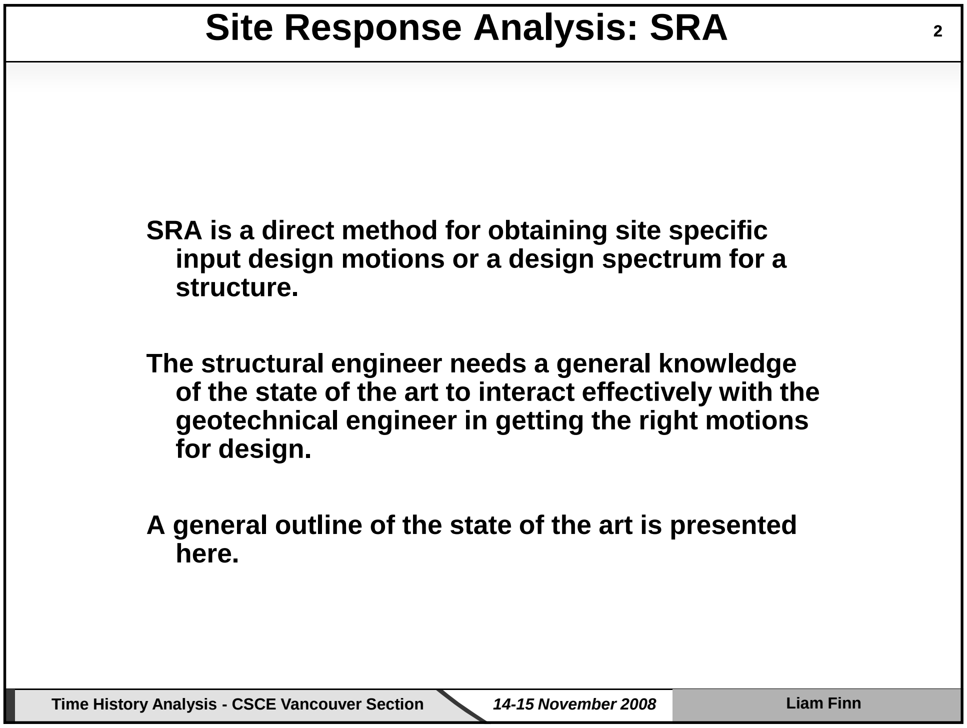
# TIME HISTORY ANALYSIS

## Site Response Analysis and Soil-Structure Interaction

Dr. Liam Finn
University of British Columbia

*A technical seminar on the use of time histories and site specific response spectra in structural design, and an introduction to linear and non-linear time history analysis.*

***14-15 November 2008 Vancouver, BC***



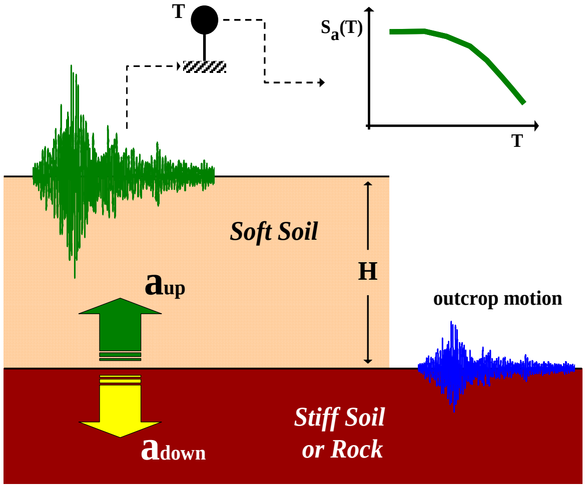
## Site Response Analysis: SRA

**SRA is a direct method for obtaining site specific input design motions or a design spectrum for a structure.**

**The structural engineer needs a general knowledge of the state of the art to interact effectively with the geotechnical engineer in getting the right motions for design.**

**A general outline of the state of the art is presented here.**

# Elements of Site Response Analysis: SRA



# Key Steps in Site Response Analysis

- **Selection of type of analysis:**
  - ***Equivalent linear or nonlinear analysis***
  - ***Sensitivity to best estimates of soil properties***
- **Selection of input motions for analysis**
  - ***Basis for selection of candidate motions***
    - ***Who picks the motions?***
    - ***How many motions?***
    - ***Scaling motions to required intensity***
    - ***Interpretation of results – dispersion etc***

## Appropriate Type of Analysis

Equivalent linear analysis is the simplest and is most widely used.

Recommended procedures for implementation of DMG Special Publication 117 Guidelines for analyzing and mitigating liquefaction hazards in California, SCEC, USC, Los Angeles, 1999, states,

**"In general, equivalent linear analyses are considered to have reduced reliability as ground shaking levels increase to values greater than 0.4g in the case of softer sols or where the maximum shear strain amplitudes exceed 1%-2%. For these cases, true nonlinear site response programs may be used."**

**In current practice the limiting criterion for reliable use of equivalent linear analysis is often 1% shear strain.**

## Non-Linear Programs

***The computer program DESRA-2, developed by Lee and Finn (1978), was the first widely used non-linear,effective stress program.***

***Other nonlinear programs which are based on modifications of DESRA include MARDES (Chang et al,1991), D-MOD (Matasovich, 1993) and SUMDES (Li et al., 1992).***

***PLAXIS and FLAC are becoming standard of practice programs for all kinds of analyses in geotechnical engineering including site response analyses.***

***These programs are computational platforms which contain different models of soil behavior. Which model?***

***How to calibrate it?***

***Their effective use requires a higher level of competence and theoretical understanding.***

## Selecting Candidate Motions-1

**This topic has been covered in detail in Lecture #3.**

**Here I would like to emphasize again some important issues and highlight some new developments.**

**Selection should be made if possible from a data base of uniformly processed records.**

**The large PEER NGA Data Base is uniformly processed.**

**If a conditional mean spectrum (CMS- ) is used for design, it is not necessary to try to match Magnitude and Distance when selecting records. Motions should only match the spectrum over the range of interest. (See Lecture #4).**

**Such motions exhibit minimum dispersion in the results of structural analyses despite significant differences in M & R.**

## Selecting Candidate Motions-2

**Selection using Magnitude M and Distance to site, R.**

***In Canada mostly aereal seismic sources.***

***M and R are selected as the values contributing most to the hazard at the site.***

**Use Mode Magnitude $M_m$ and Mode Distance $R_m$ to guide record selection, if specific faults are not being considered. Can be obtained from GSC.**

**Candidate motions should come from same seismic environment as the target site.**

- **Shallow crustal earthquakes**
- **Deep crustal earthquakes**
- **Subduction earthquakes**

**Records should preferably have been recorded on a site with similar velocity distributions with depth, either increasing or decreasing.**

## Example: BC School Retrofit Project

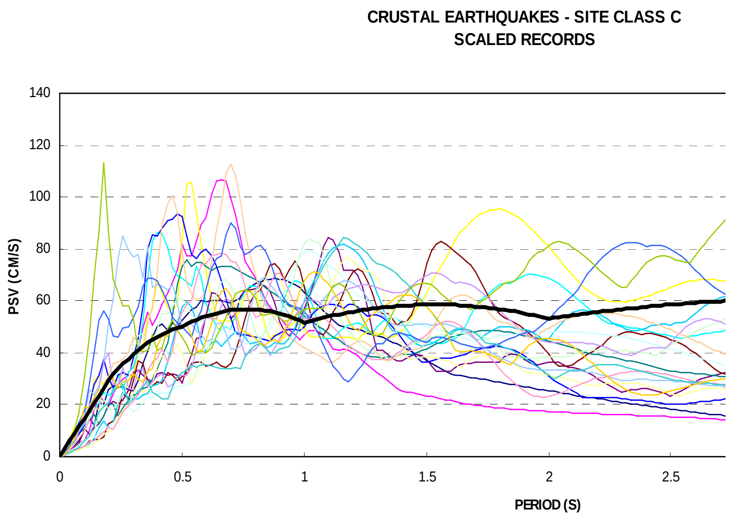
**For the Schools Retrofit Project in British Columbia only ground motion records from crustal sources have used but now motions from three distinct sources are under consideration:**

- ***Subduction Zone***
- ***Sub-crustal sources ( PUG)***
- ***Crustal sources***

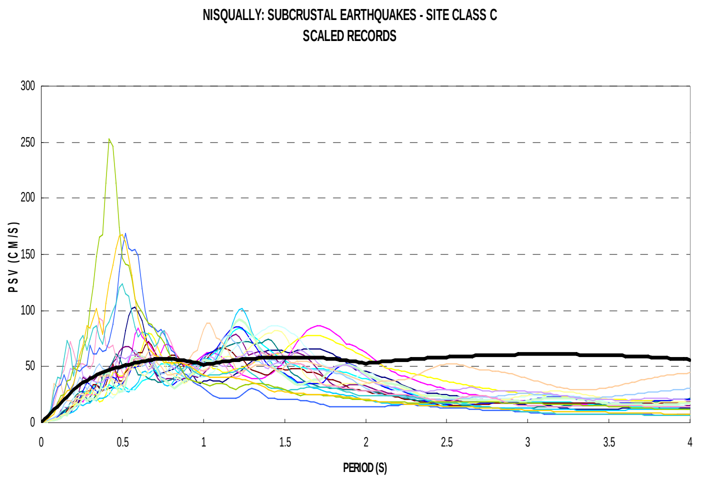
***Examples of crustal and sub-crustal motions are shown in Slides #10 - #12 . Note the narrow band spectra of the sub-crustal motions – also typical of large magnitude rare earthquakes.***

**The Puget Sound Source, PUG, with quake events in the subducting plate contributes most to risk in the Lower Mainland but in general practice shallow crustal records from California are usually used for all designs.**

## Crustal Records - PSV

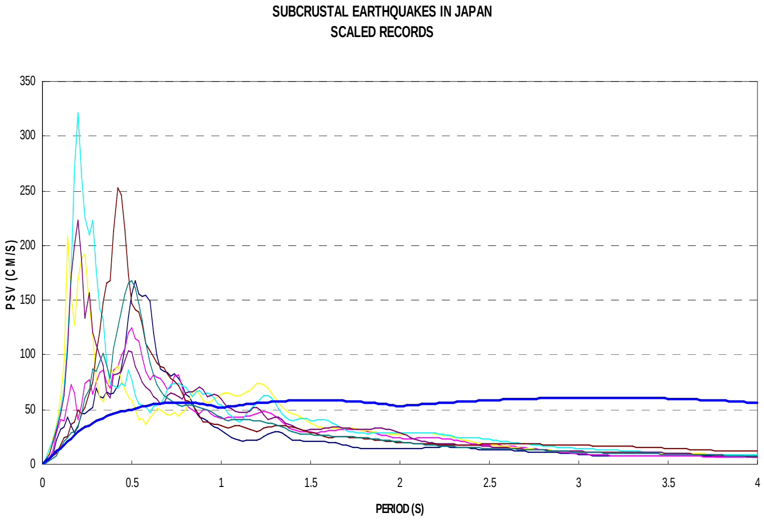


## Sub-Crustal Records – PSV: All from same Quake



NISQUALLY: SUBCRUSTAL EARTHQUAKES - SITE CLASS C
SCALED RECORDS

## Sub-Crustal Records from Japan - PSV



SUBCRUSTAL EARTHQUAKES IN JAPAN
SCALED RECORDS

## How many Input Motions are required ?

One reason for using multiple input motions is to protect against the great variety in the characteristics of motions that are appropriate candidates for input motions based on criteria such as magnitude, distance and site class.

Multiple motions also provide the data to obtain relatively reliable statistics on the input motions such as median, mean and standard deviation of spectral accelerations.

In IDA analysis they also provide the statistical distribution of response data to evaluate the probability of exceeding design criteria such as limiting drift ratio or collapse.

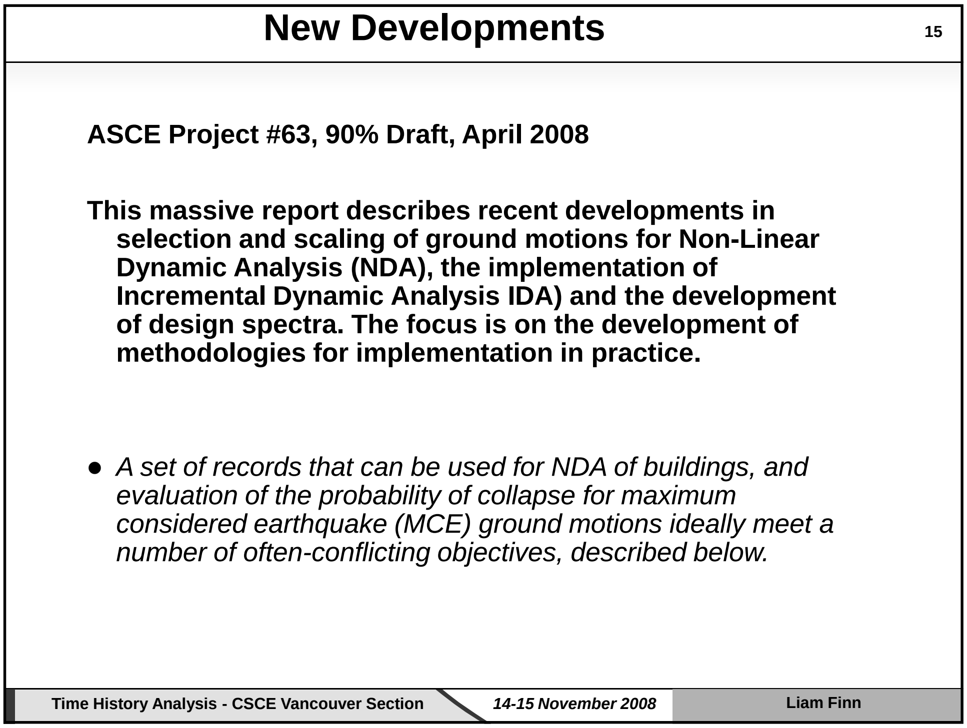
## Scaling Candidate Motions

Linear scaling of a selected record does not alter the frequency content of the record and is preferred, if a good enough match can be obtained in the spectral range of interest. Now with large data bsaes (PEER has 3350 motions) chances of a reasonable match have improved.

Matching in frequency domain has been shown to yield increased displacements in nonlinear response analysis.

For the BC Schools Seismic Retrofit Program input motions (10-20) were linearly scaled to match the average spectral velocity for Site Class C over the range 0.5s-1.5s.

This matching criterion is now under review.

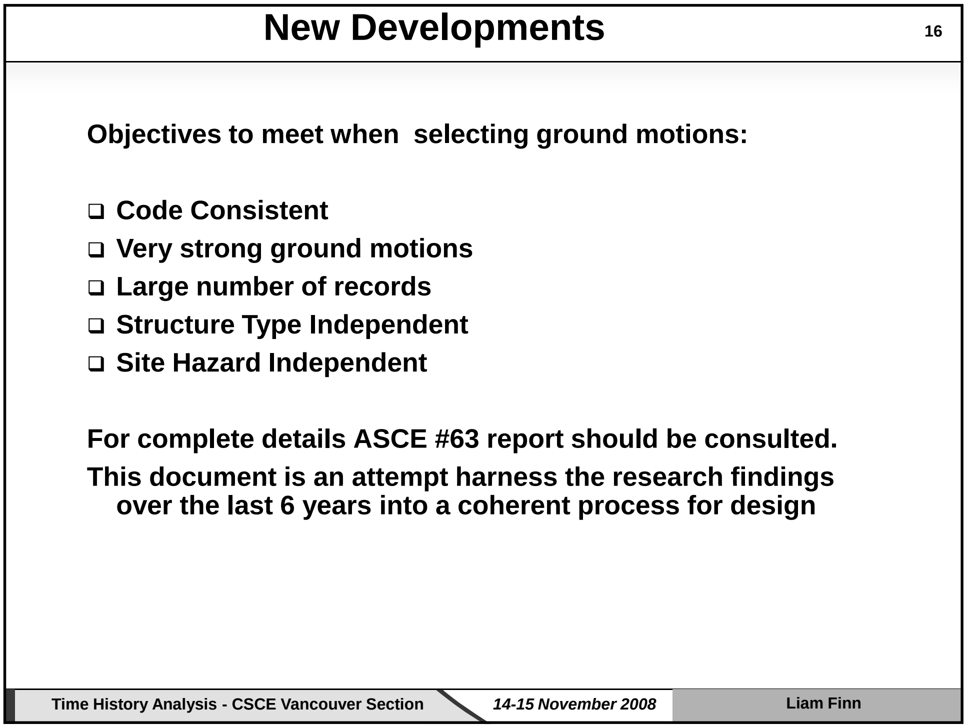


# New Developments

**ASCE Project #63, 90% Draft, April 2008**

**This massive report describes recent developments in selection and scaling of ground motions for Non-Linear Dynamic Analysis (NDA), the implementation of Incremental Dynamic Analysis IDA) and the development of design spectra. The focus is on the development of methodologies for implementation in practice.**

- *A set of records that can be used for NDA of buildings, and evaluation of the probability of collapse for maximum considered earthquake (MCE) ground motions ideally meet a number of often-conflicting objectives, described below.*



# New Developments

**Objectives to meet when selecting ground motions:**

- ☐ **Code Consistent**
- ☐ **Very strong ground motions**
- ☐ **Large number of records**
- ☐ **Structure Type Independent**
- ☐ **Site Hazard Independent**

**For complete details ASCE #63 report should be consulted.**

**This document is an attempt harness the research findings over the last 6 years into a coherent process for design**

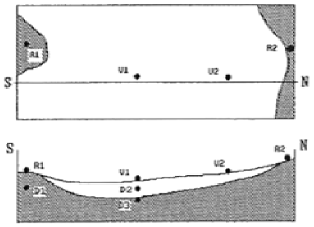
## Field Validation Exercise

A major blind prediction exercise was conducted at the Turkey Flat Site in California to evaluate capability to predict site response.

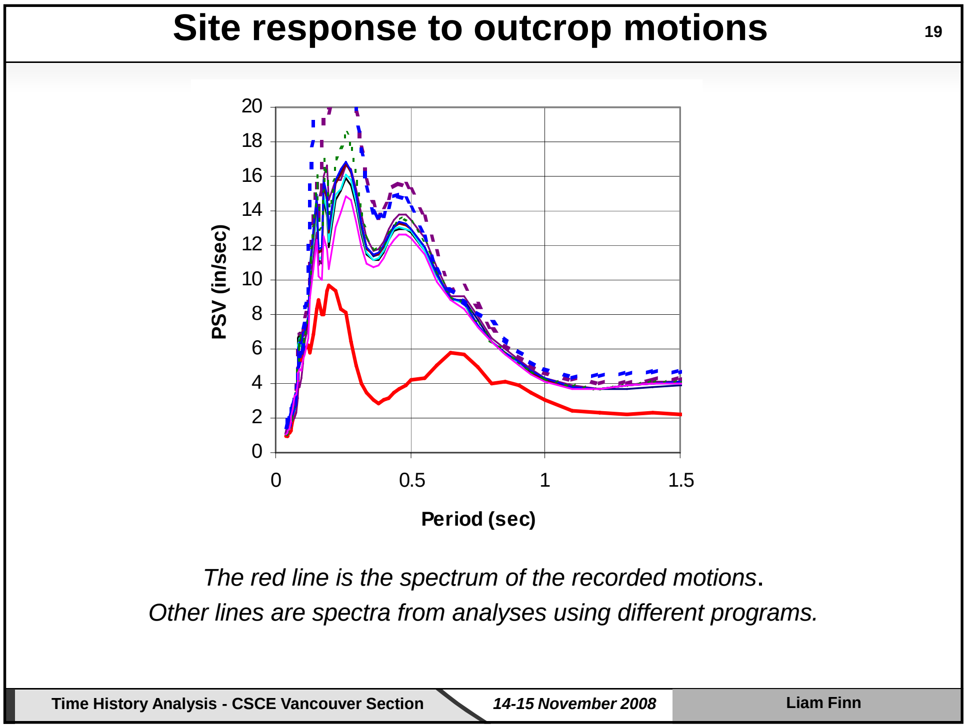
Many analysis programs, linear, equivalent linear and non-linear were evaluated.

## Turkey Flat Prediction Experiment

**Check on the reliability of the process for site response analysis under ideal conditions**

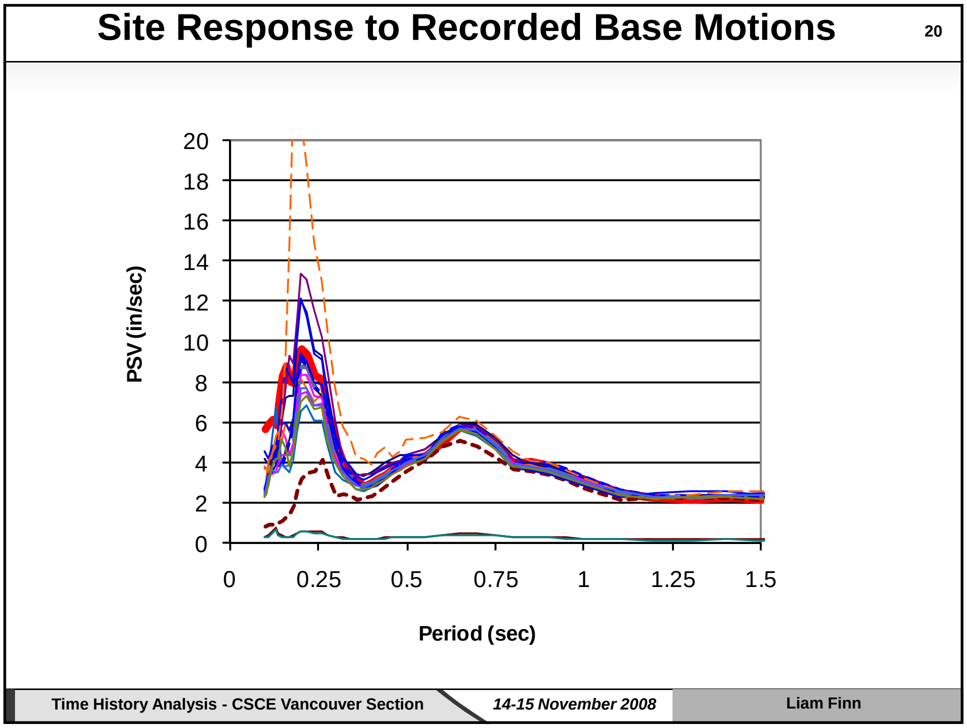


Turkey Flat site effects test area and the strong-motion array stations (after Tucker and Real, 1986)



## Site response to outcrop motions

*The red line is the spectrum of the recorded motions.*
*Other lines are spectra from analyses using different programs.*



## Site Response to Recorded Base Motions

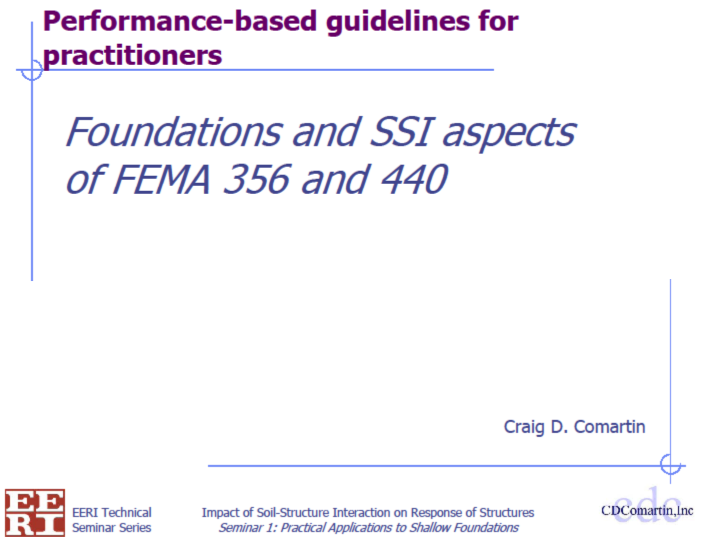
## Results of Turkey Flat Validation Study

***Despite a wealth of data on geology and soil properties and that recorded outcrop motions available, predictions of site responses were disappointing.***

***Pay attention to where outcrop motions were recorded. Avoid risks of topographical effects***

***Site response analysis is not a routine*** **process.** ***Go through a checklist of the essential requirements cited above before proceeding with an analysis.***

## Reference on SSI for Shallow Foundations



**Performance-based guidelines for practitioners**

*Foundations and SSI aspects of FEMA 356 and 440*

Craig D. Comartin

EERI Technical Seminar Series

Impact of Soil-Structure Interaction on Response of Structures
*Seminar 1: Practical Applications to Shallow Foundations*

CDComartin,Inc

## Soil-Structure Interaction

Inertial effects

- ☐Foundation stiffness and strength
- ☐Radiation damping

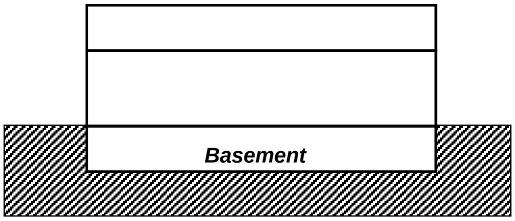
*Inertial effects are have been represented by equations for estimating in period lengthening and system damping since 1976 but have been ignored in design until recently. They will not be dealt with here.*

Kinematic effects ( considered here)

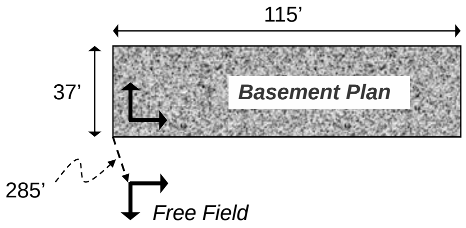
- ☐Base slab averaging (x,y)
- ☐Embedment (z)

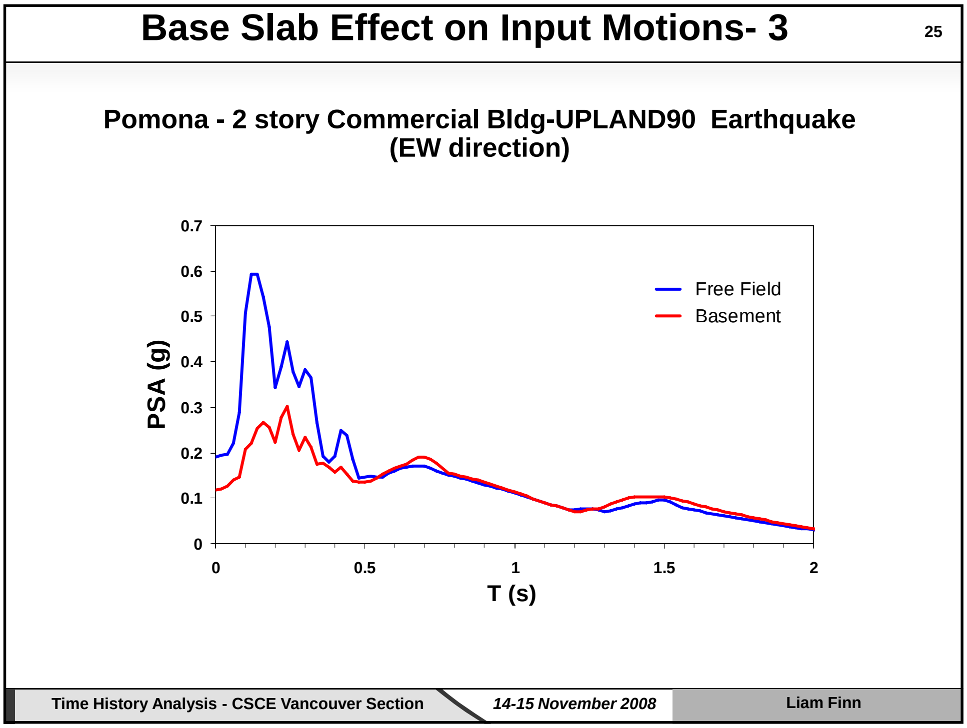
☐Courtesy C. D. Cromartin & EERI, 2007

## POMONA BUILDING



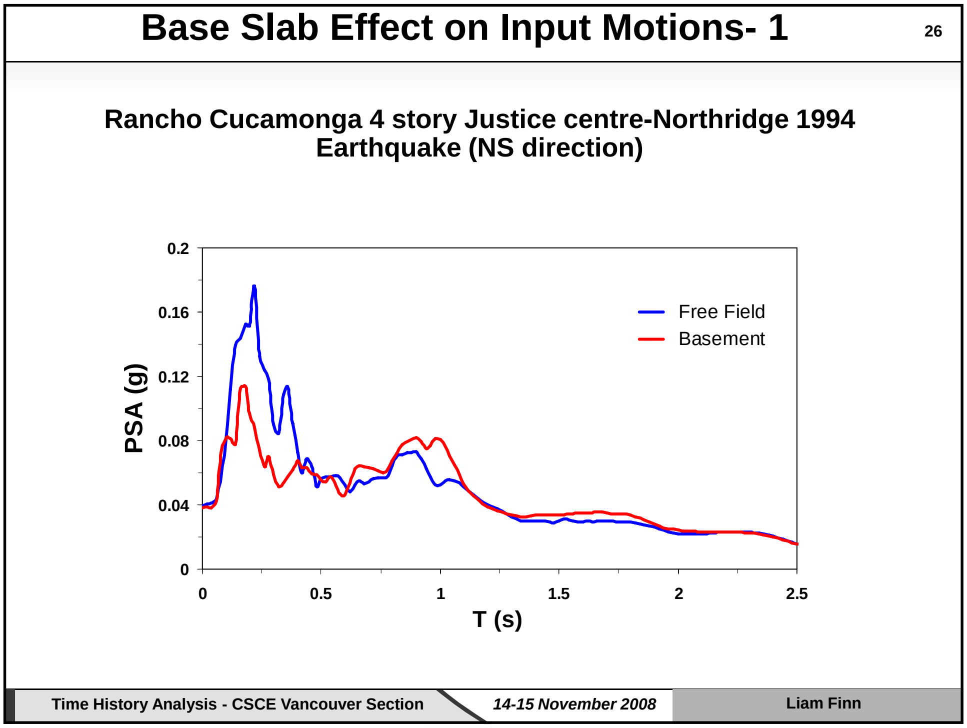
**Pomona - 2 story Commercial Bldg**





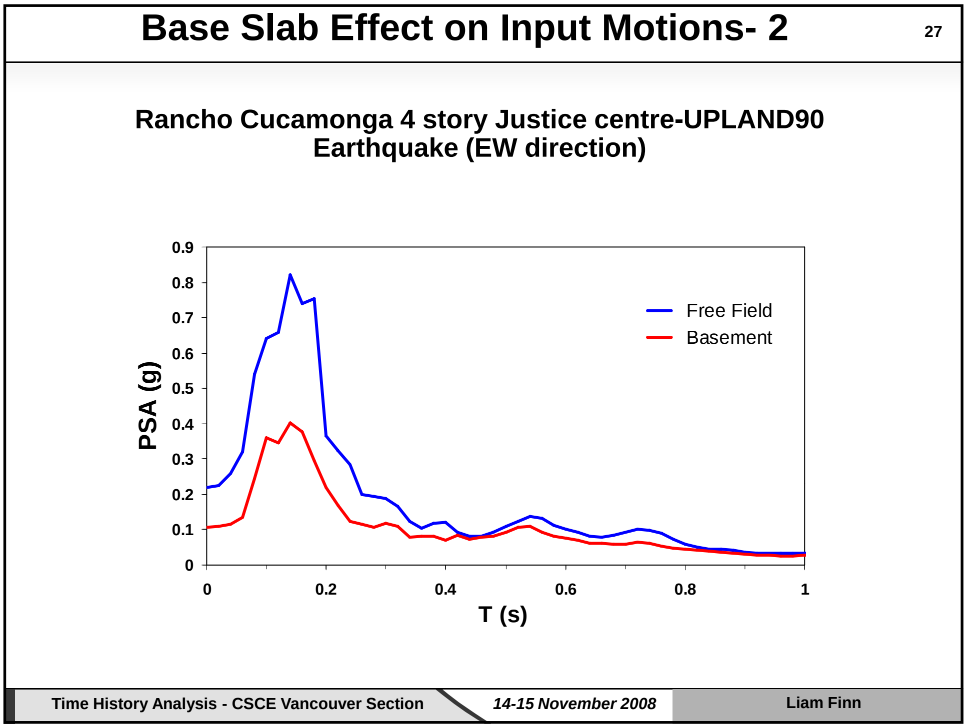
# Base Slab Effect on Input Motions- 3

**Pomona - 2 story Commercial Bldg-UPLAND90 Earthquake (EW direction)**



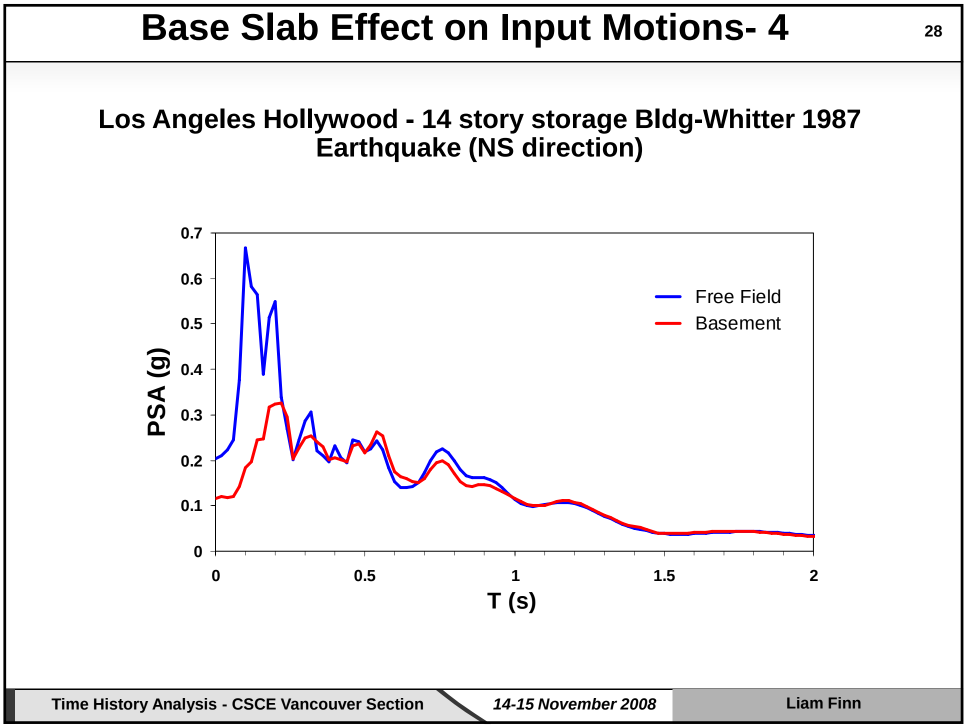
# Base Slab Effect on Input Motions- 1

**Rancho Cucamonga 4 story Justice centre-Northridge 1994 Earthquake (NS direction)**



## Base Slab Effect on Input Motions- 2

**Rancho Cucamonga 4 story Justice centre-UPLAND90 Earthquake (EW direction)**



## Base Slab Effect on Input Motions- 4

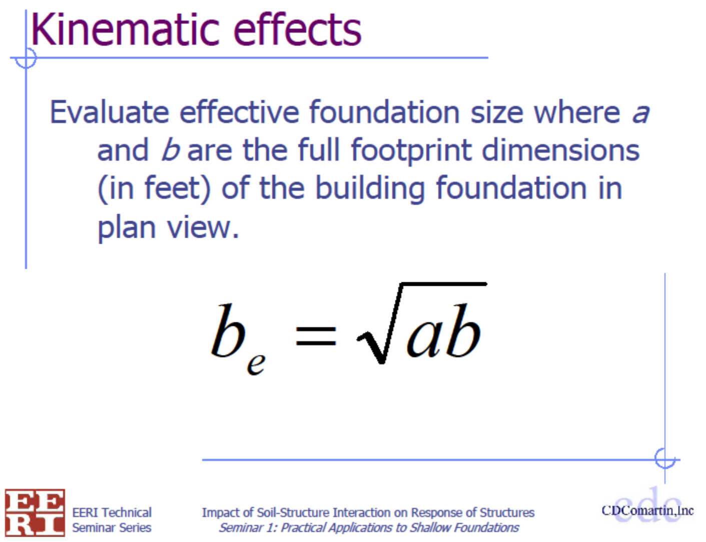
**Los Angeles Hollywood - 14 story storage Bldg-Whitter 1987 Earthquake (NS direction)**

## Findings on Kinematic Interaction

The base slab significantly reduces the spectral values of the free field motions for periods below 0.5s and if used for design could lead to reduced seismic demand on some structures depending on period.

Procedures for making appropriate reductions in the free field spectra follow in Slides #30 – 33.

## Effective Foundation Size



### Kinematic effects

Evaluate effective foundation size where *a* and *b* are the full footprint dimensions (in feet) of the building foundation in plan view.

$$b_e = \sqrt{ab}$$

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*Seminar 1: Practical Applications to Shallow Foundations*

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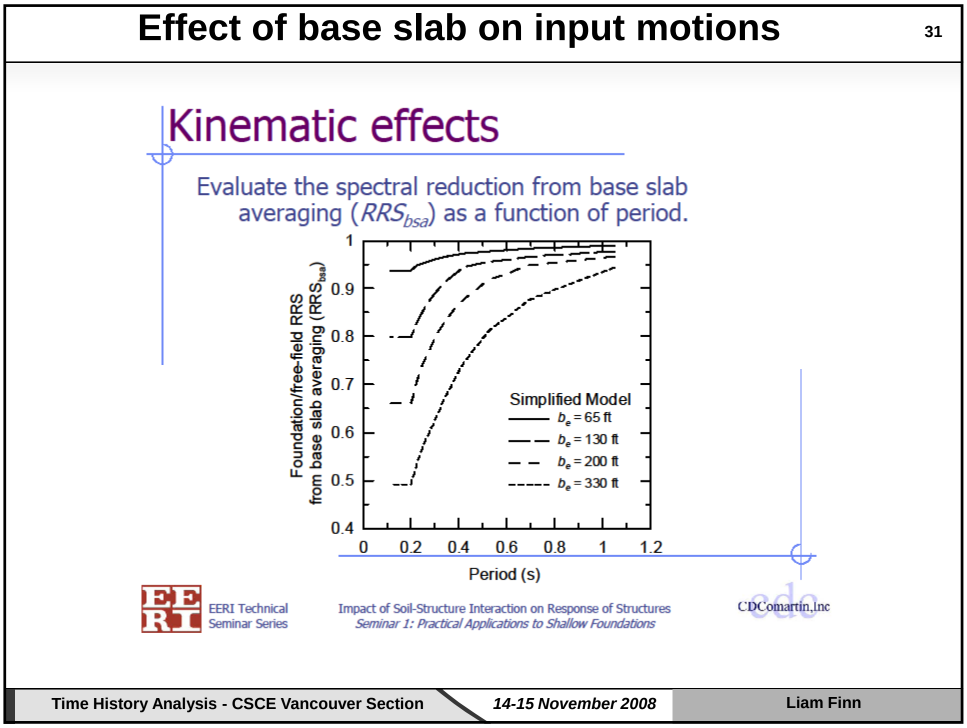

## Effect of base slab on input motions

### Kinematic effects

Evaluate the spectral reduction from base slab averaging (*RRS*$_{bsa}$) as a function of period.

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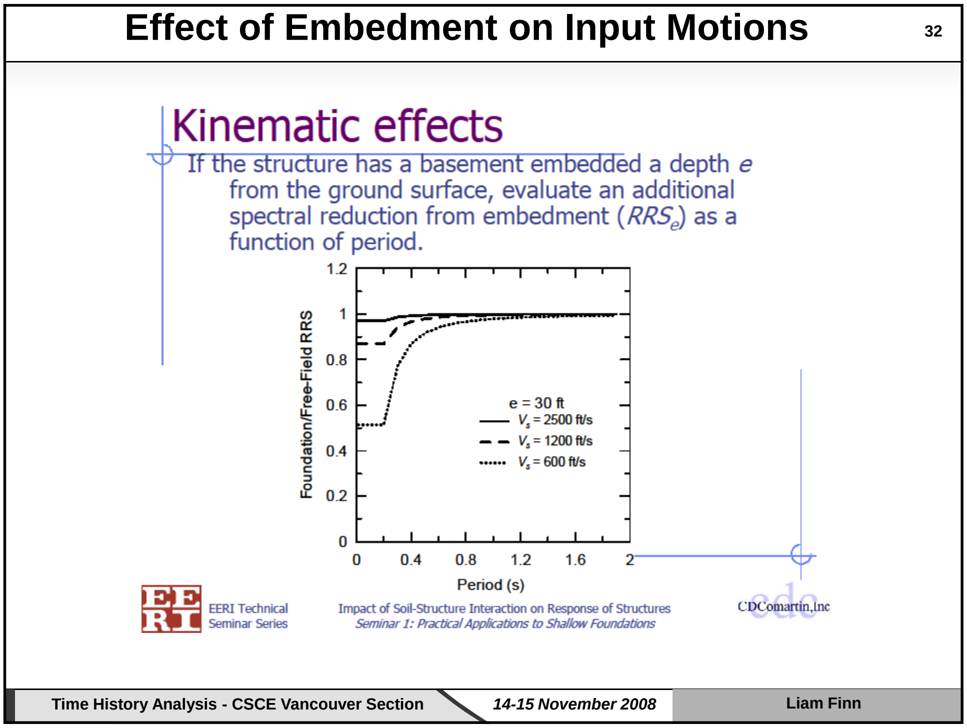

## Effect of Embedment on Input Motions

### Kinematic effects

If the structure has a basement embedded a depth *e* from the ground surface, evaluate an additional spectral reduction from embedment (*RRS*$_e$) as a function of period.

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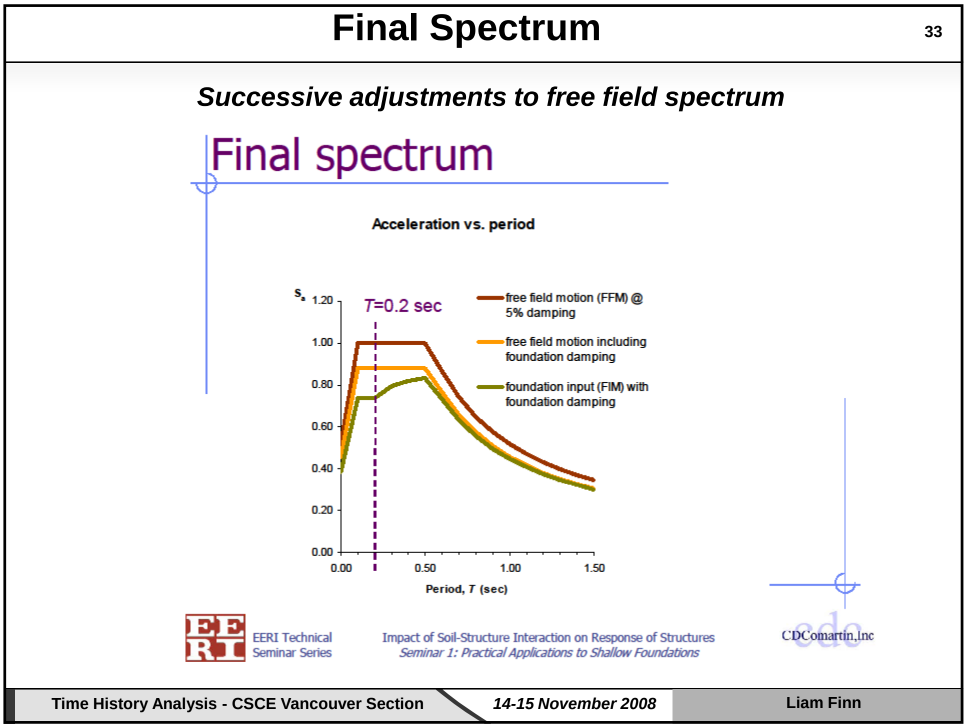

## Final Spectrum

*Successive adjustments to free field spectrum*

Final spectrum

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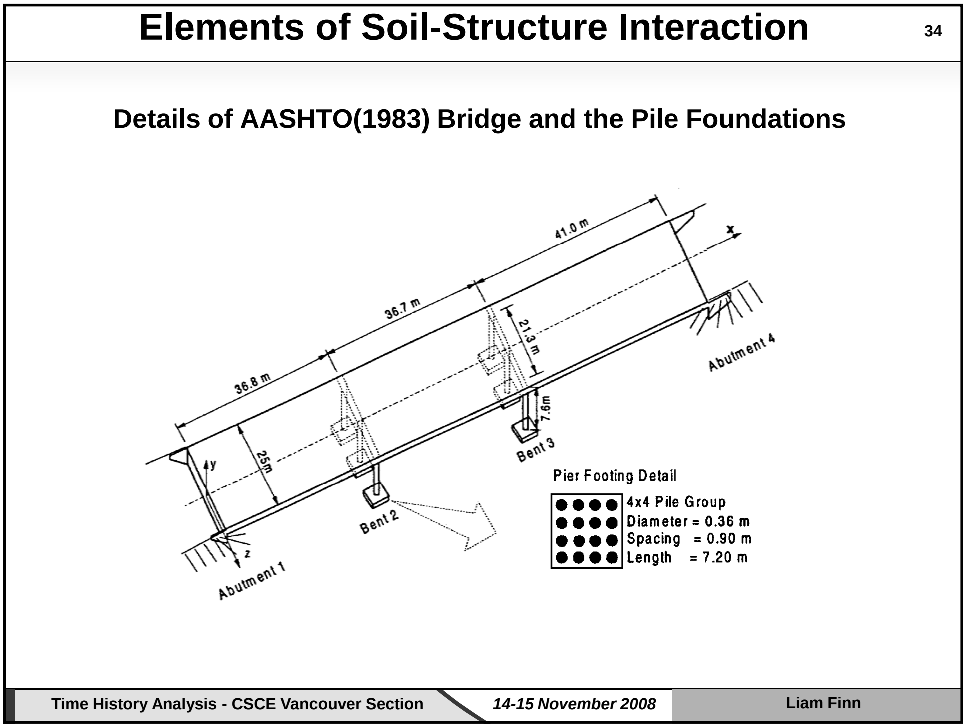


## Elements of Soil-Structure Interaction

**Details of AASHTO(1983) Bridge and the Pile Foundations**

Pier Footing Detail

4x4 Pile Group
Diameter = 0.36 m
Spacing = 0.90 m
Length = 7.20 m

## Soil-Structure Interaction

**Impact of modelling concepts on structural response using continuum analysis**

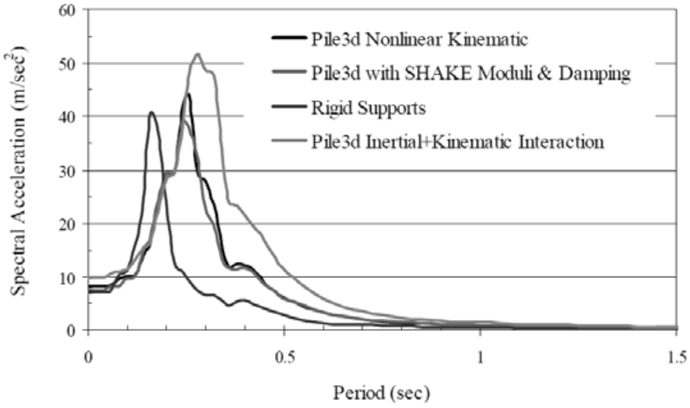
***Complete modelling of Structure –Foundation-Soil***

***Simplified Complete Modelling***

***Kinematic Modelling of Foundation (to get foundation springs)***

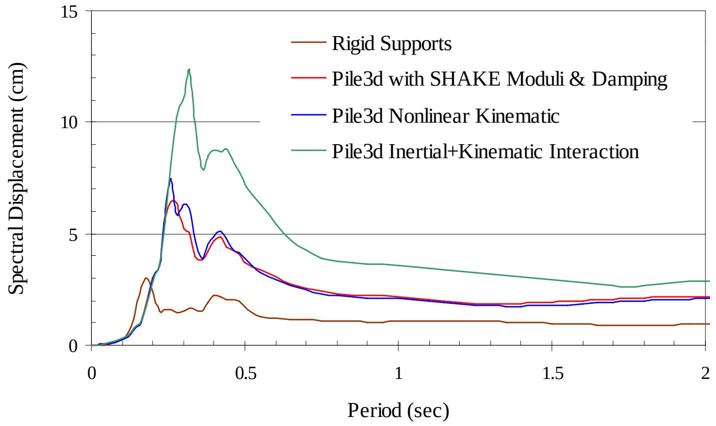
## Modelling Impact on Spectral Accelerations

**Note changes in acceleration and period of peak response**



## Modelling Impact on Spectral Displacements

**Displacements increase with increasing flexibility of models**



## Findings from Analyses

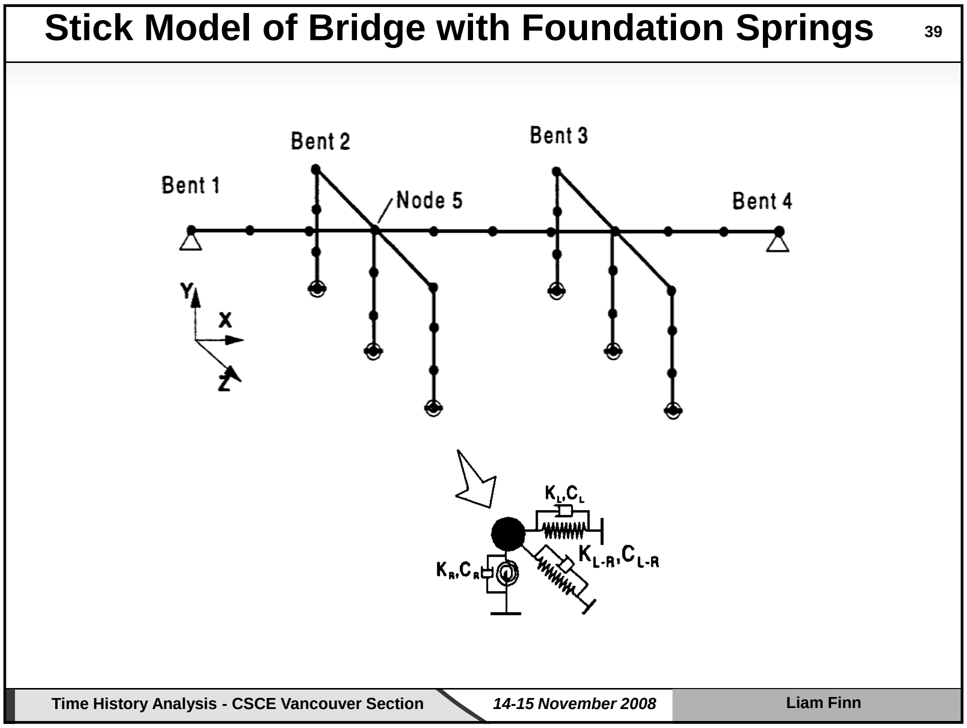
The analyses show that how adequately the kinematic analysisthe kinematic analysis represents the true response of the bridge depends on the impact of the inertial interaction which is neglected in the kinematic analysis.

In engineering practice there is no such thing as a standard kinematic analysis – several versions of increasing approximation are used often with no evaluation of reliability.

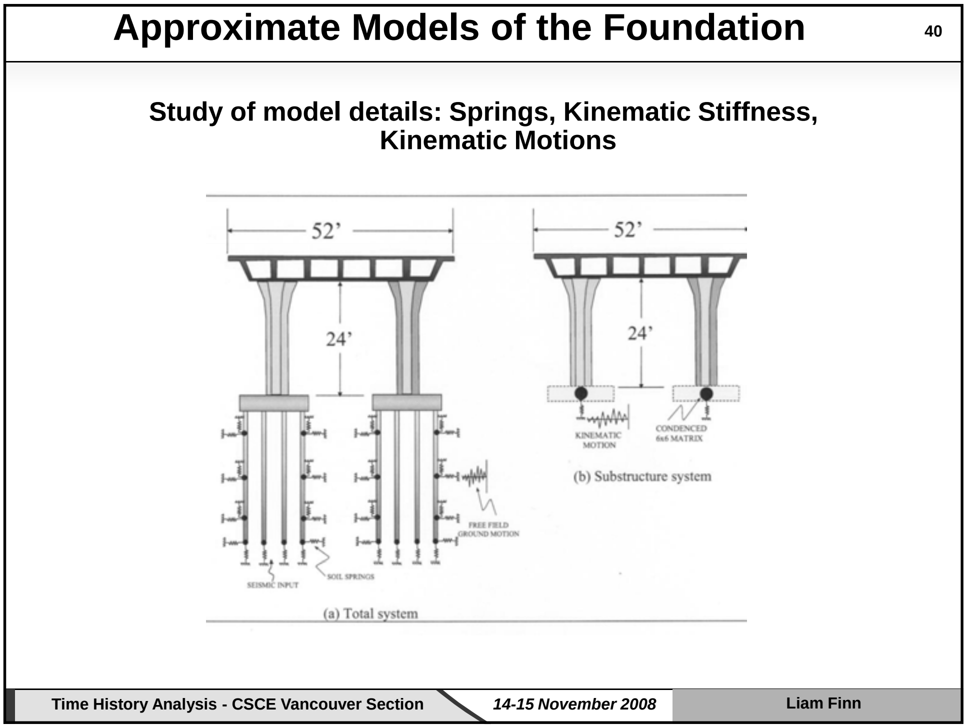
For a full discussion of the issues see the following references; 1-for for a better understanding of Slides #36 and #37 and 2- for general theory of nonlinear analysis of pile foundations.

1.W. D. Liam Finn, *CHARACTERIZING PILE FOUNDATIONS FOR EVALUATION OF PERFORMANCE BASED SEISMIC DESIGN OF CRITICAL LIFELINE STRUCTURES,* Invited keynote lecture, 13th WCEE, Vancouver, BC, Canada, August, 2004

2.WU, G. and FINN, W.D. Liam, "*DYNAMIC NONLINEAR ANALYSIS OF PILE FOUNDATIONS USING FINITE ELEMENT METHOD IN THE TIME DOMAIN*", Canadian Geotechnical Journal, Vol. 34, 1997, pp. 44-52.



## Stick Model of Bridge with Foundation Springs



## Approximate Models of the Foundation

**Study of model details: Springs, Kinematic Stiffness, Kinematic Motions**

## Determining kinematic motions and stiffnesses

**Note that inertial effects on stiffness are neglected and the pile cap is replaced by weightless rigid links. The free field motions are applied to the ends of the springs.**

**Pile cap 6x6 stiffness matrix and kinematic motions are applied to the master node**

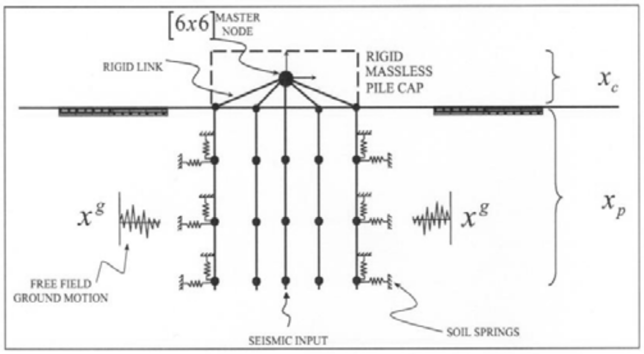


Figure 5. Substructure system.

## Determining kinematic motions and stiffnesses

**In the analysis of pile foundations in practice, the soil is replaced by linear or nonlinear Winkler springs.**

**The nonlinear springs recommended by the American Petroleum Institute (API) are often used.**

**These springs are for a single pile. For use in pile groups the springs need adjustment- softened. There are many suggestions for how this can be done.**

**There are commercial programs available for implementing the API springs such as L-PILE and L-GROUP**

**The springs are also often linearized for convenience in dynamic analysis.**

**The literature on the springs be very confusing.**

**For a detailed review from the point of selection and application of these springs and a discussion of their reliability see**

**Finn, W. D. Liam (2005). A Study of Piles during Earthquakes: Issues of Design and Practice, Bulletin of Earthquake Engineering, 3:131-234, Springer.**

## Selecting Approximate Input Motion

**Candidate Input Motion to Match Acceleration Spectrum**
**(Free field acceleration at depth of 16 m?)**

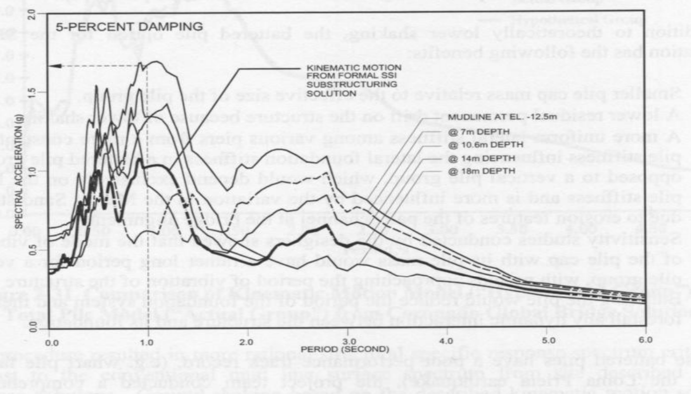
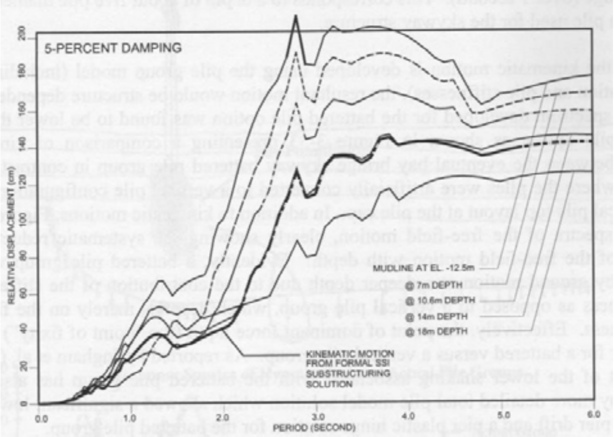
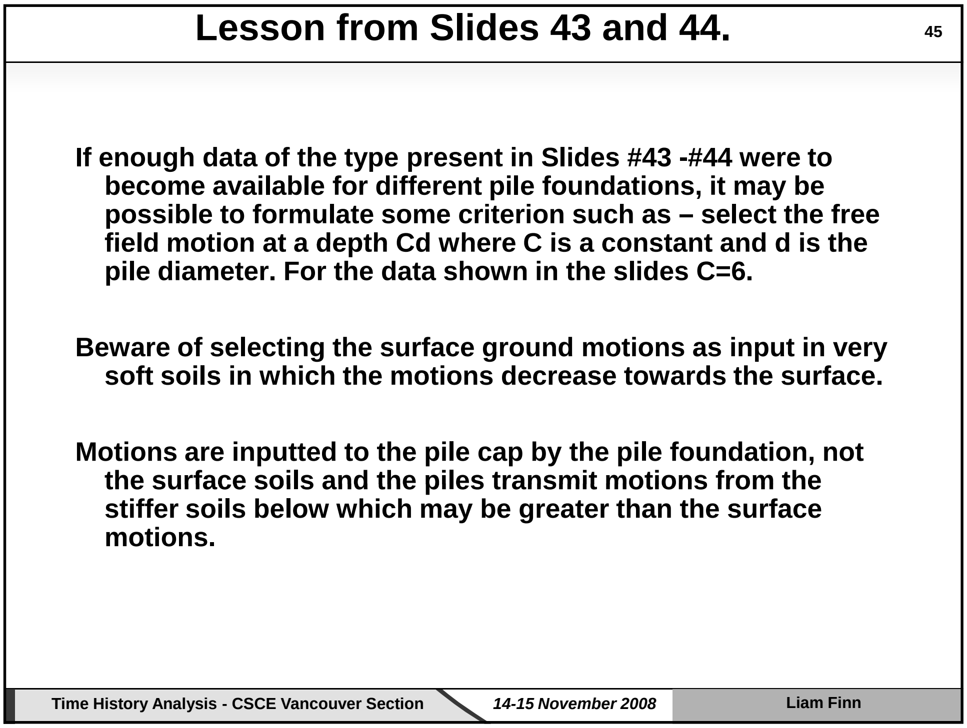


**Figure 3-32 Kinematic Motion vs. Depth-Varying Free-Field Motion**

## Selecting Approximate Input Motion

**Candidate Input Motion to Match Displacement Spectrum**
**(Free field acceleration at depth of 14 m)**





## Lesson from Slides 43 and 44.

If enough data of the type present in Slides #43 -#44 were to become available for different pile foundations, it may be possible to formulate some criterion such as – select the free field motion at a depth Cd where C is a constant and d is the pile diameter. For the data shown in the slides C=6.

Beware of selecting the surface ground motions as input in very soft soils in which the motions decrease towards the surface.

Motions are inputted to the pile cap by the pile foundation, not the surface soils and the piles transmit motions from the stiffer soils below which may be greater than the surface motions.

## Vale atque vale

**Thank you**

**Merci beaucoup**

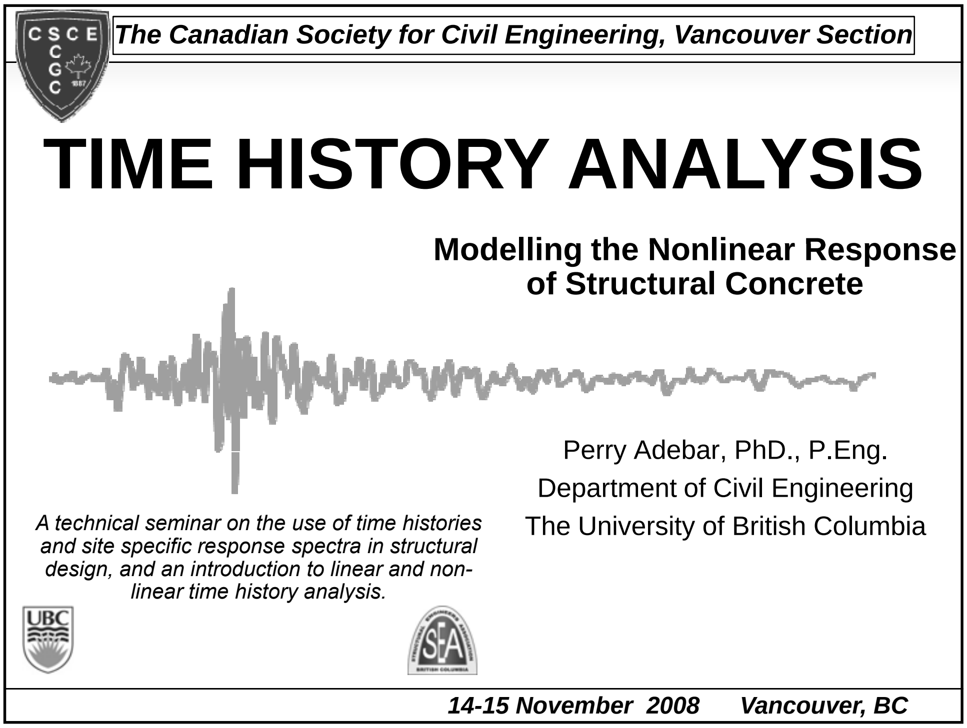
# TIME HISTORY ANALYSIS

## LECTURE # 6

## Modelling the Nonlinear Response of Structural Concrete

**Perry Adebar, PhD., P.Eng.**
**Department of Civil Engineering**
**The University of British Columbia**

Dr. Perry Adebar is Professor of Civil Engineering, University of British Columbia, Vancouver, Canada. The research he has been involved in over the past 20 years has had a direct impact on Canadian practice for seismic design of concrete wall buildings, pile cap design, and shear design of structural concrete. He became a Professor of Civil Engineering at UBC in 1990. Dr. Adebar is a member of several ACI Committees. His research and teaching interests include the field of concrete structures. He received his PhD from the University of Toronto in 1990.



The Canadian Society for Civil Engineering, Vancouver Section

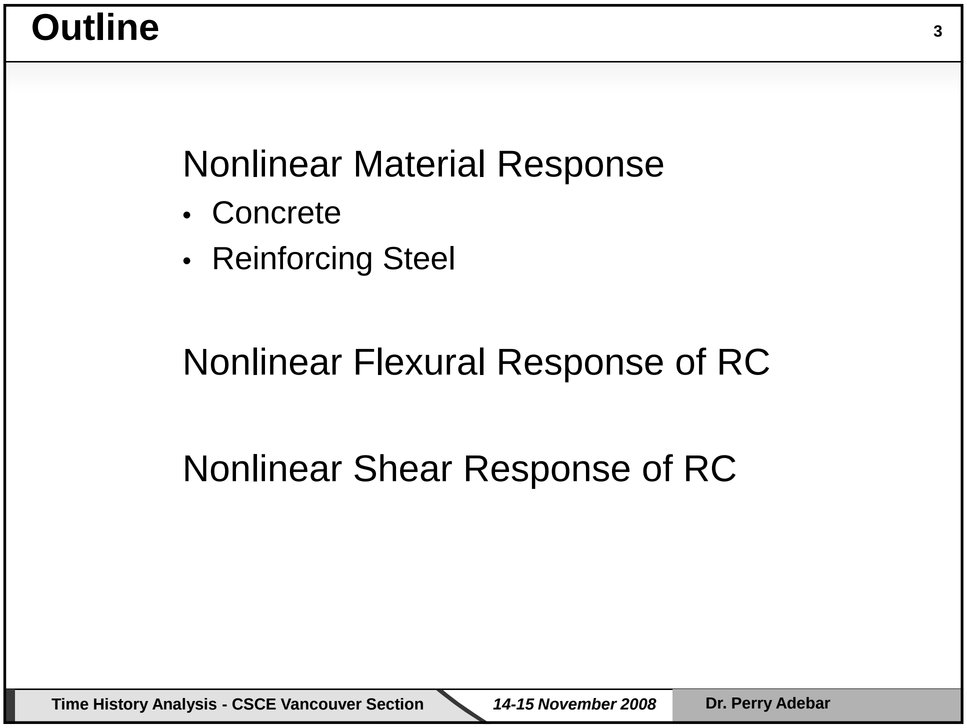
# TIME HISTORY ANALYSIS

## Modelling the Nonlinear Response of Structural Concrete

Perry Adebar, PhD., P.Eng.
Department of Civil Engineering
The University of British Columbia

*A technical seminar on the use of time histories and site specific response spectra in structural design, and an introduction to linear and non-linear time history analysis.*

***14-15 November 2008 Vancouver, BC***



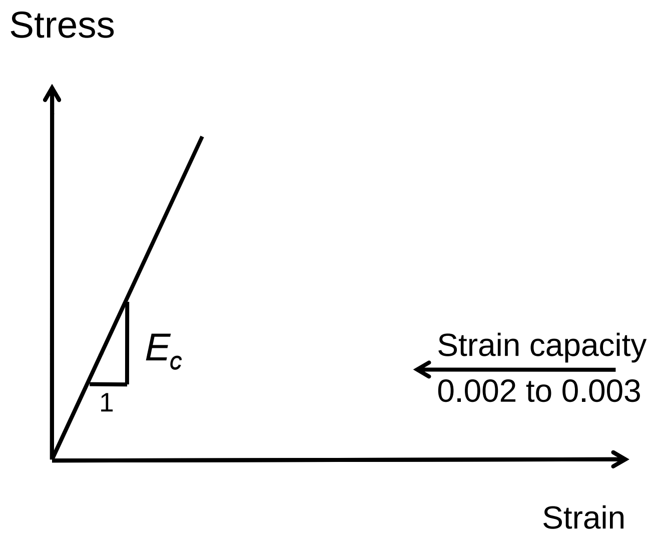
## Outline

Nonlinear Material Response
- Concrete
- Reinforcing Steel

Nonlinear Flexural Response of RC

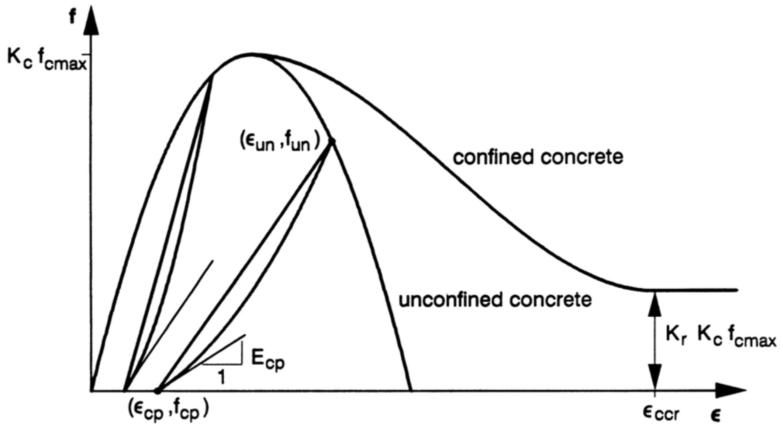
Nonlinear Shear Response of RC

## Concrete – monotonic compression



Strain capacity
0.002 to 0.003

## Concrete – cyclic compression response

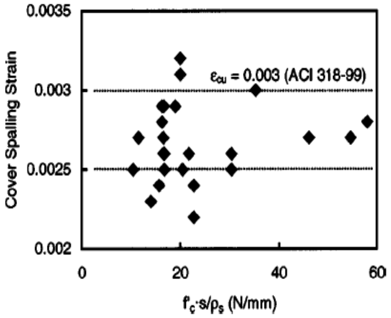


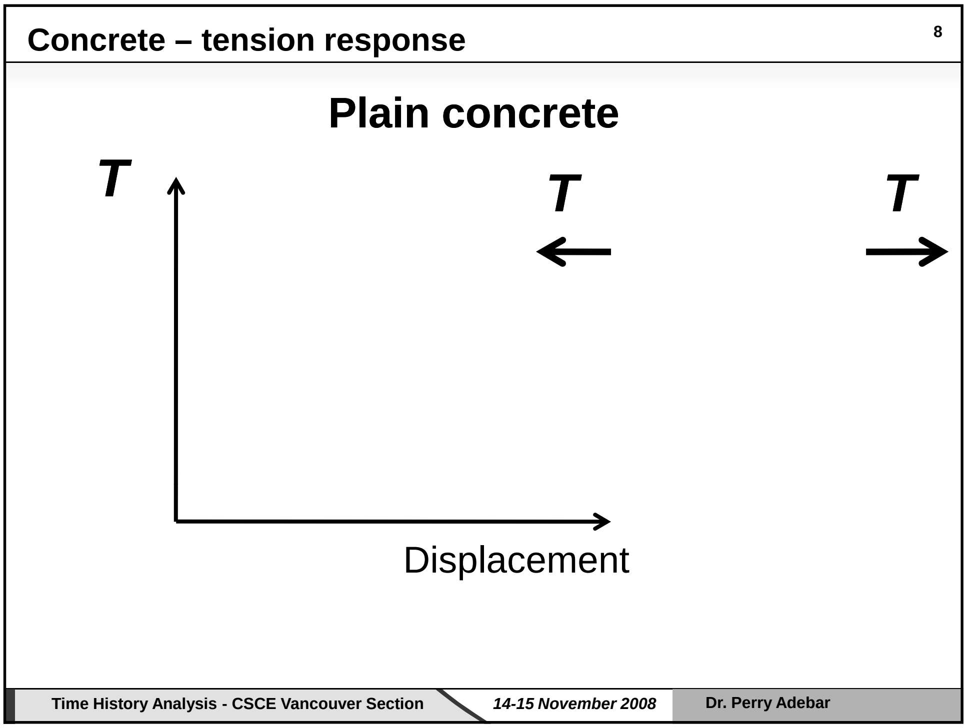
## Concrete – influence of strain gradient

Strain capacity
0.003 to 0.005

## Concrete – cover spalling

Cover spalling at
strain = 0.0025





## Concrete – tension response

### Plain concrete

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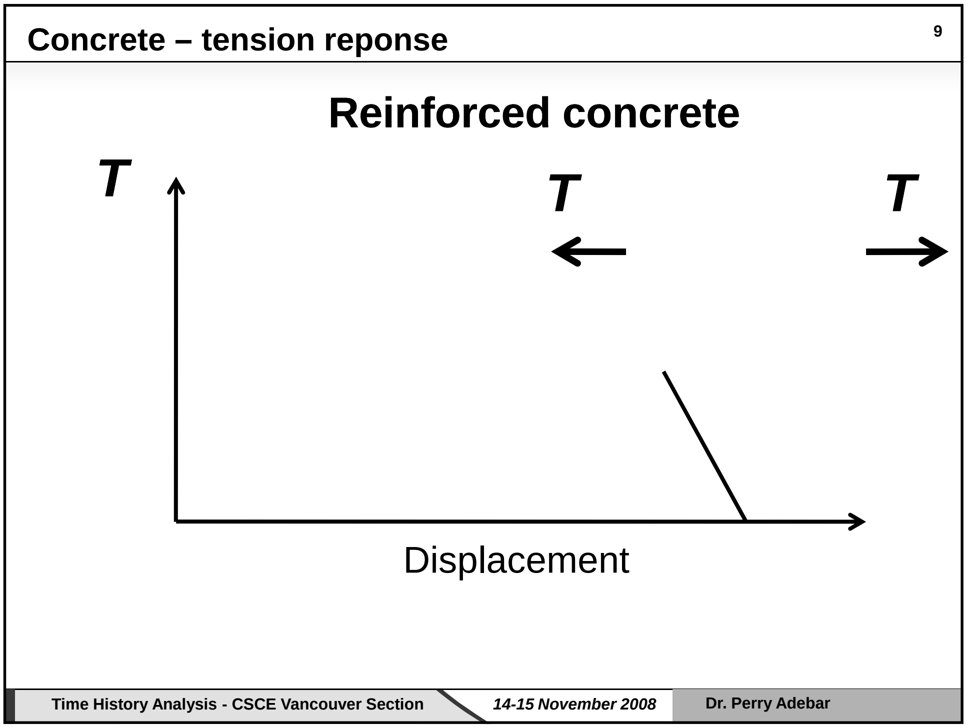
*14-15 November 2008*

Dr. Perry Adebar



## Concrete – tension reponse

### Reinforced concrete

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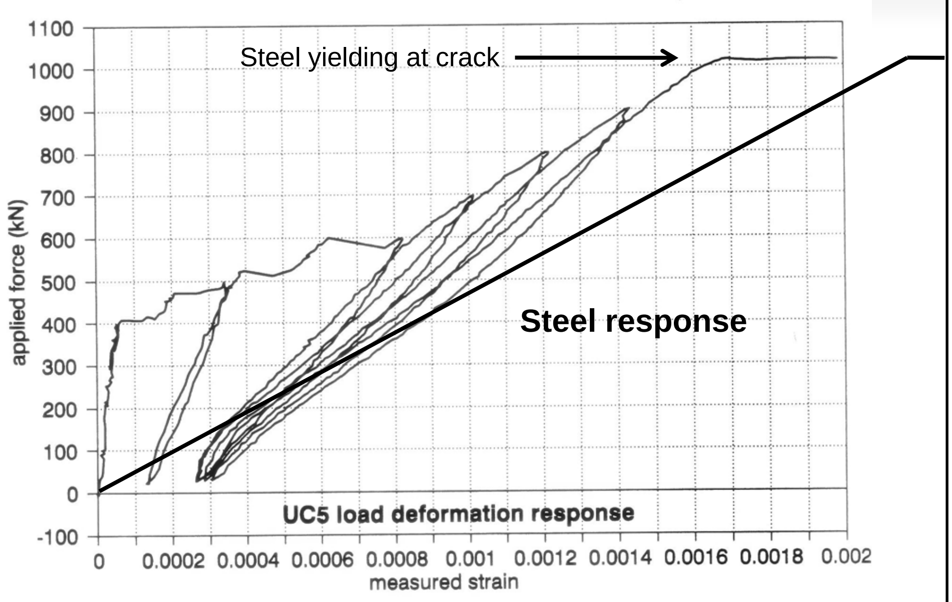
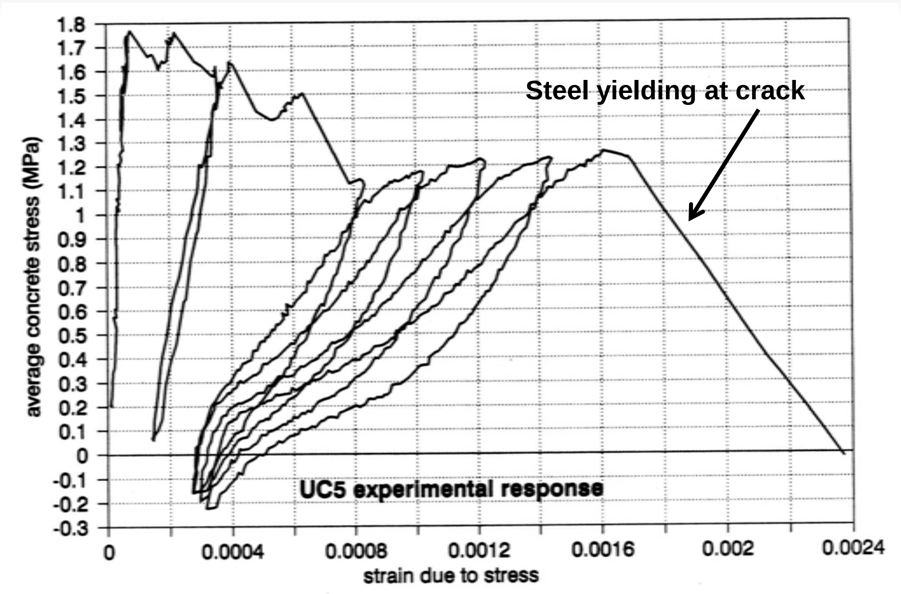
## Measured concrete tension response



Steel yielding at crack

**Steel response**

## Measured concrete tension response



**Steel yielding at crack**

## Concrete tension model in PERFORM

Half of concrete tension strength = ½ x 0.3 $\sqrt{f_c'}$

Steel yield strain = 0.002

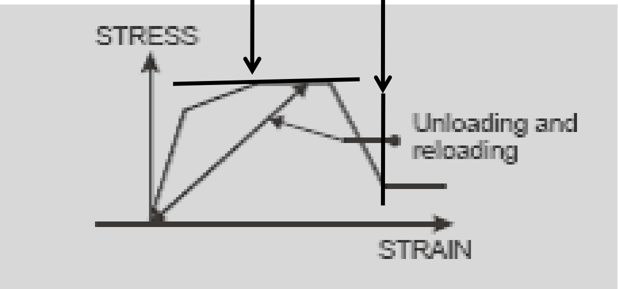
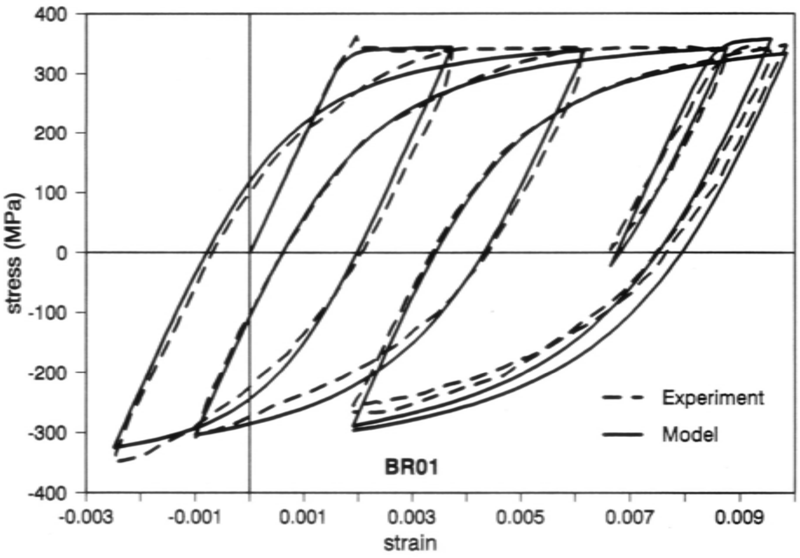


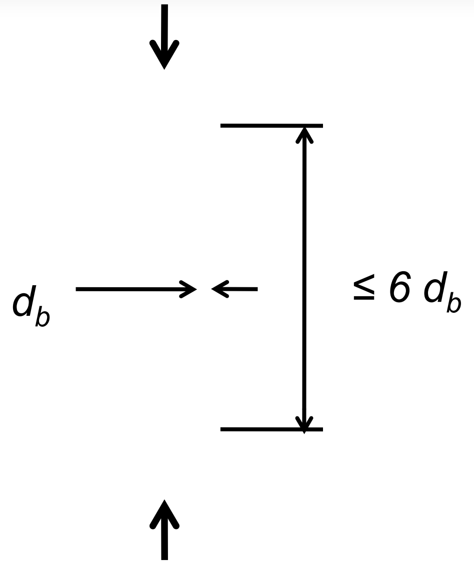
Figure 1.9 Concrete Material in Tension

Perform Model

## Reinforcing bar – Cyclic response



## Reinforcing bar – Buckling due to Bauschinger Effect



## Reinforcing bar – Tension strain capacity

Tension strain capacity of
reinforcing bar in concrete = 0.05 (5%)

# Nonlinear Flexural Response of Reinforced Concrete

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## PERFORM

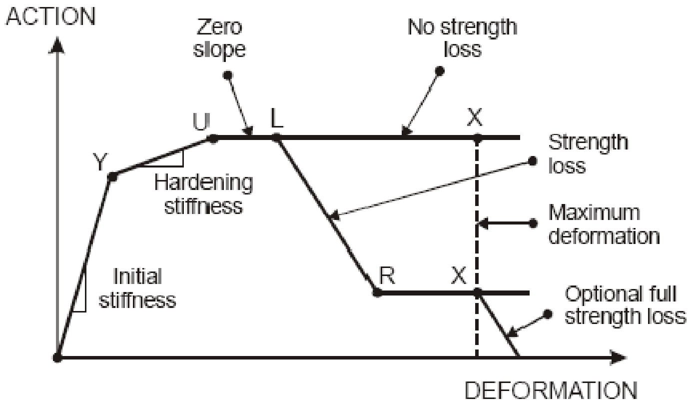
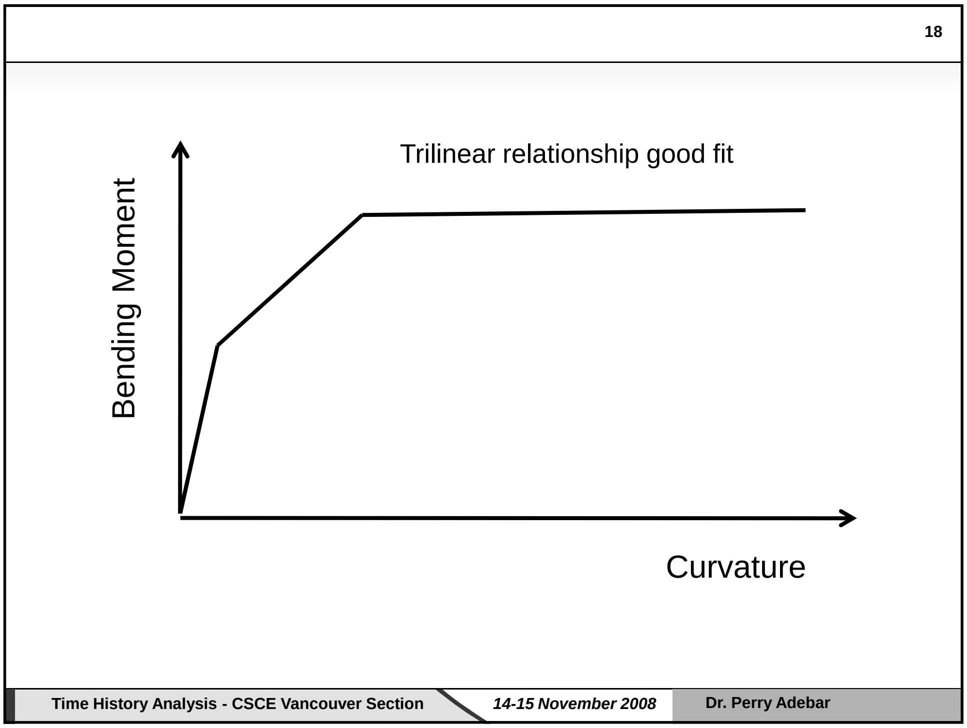


Figure 5.4 PERFORM Action-Deformation Relationship



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Trilinear relationship good fit

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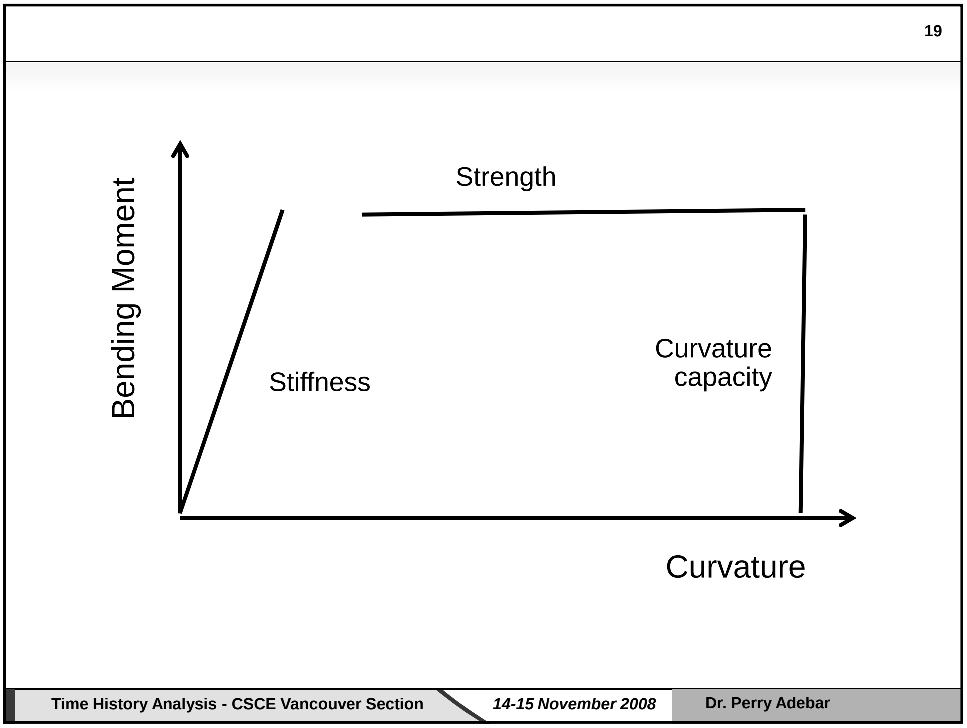

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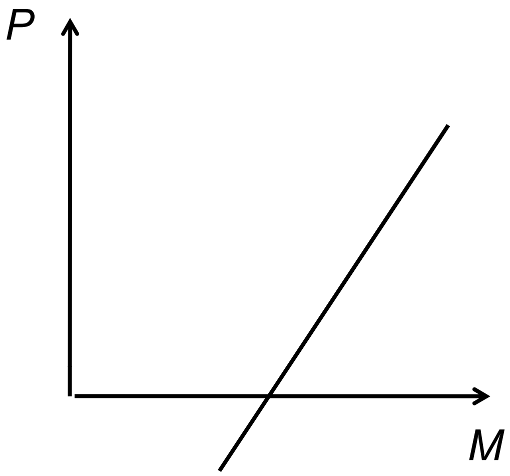
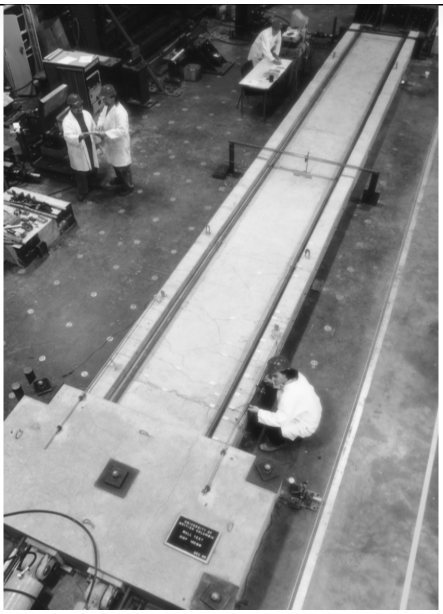
## Flexural Strength

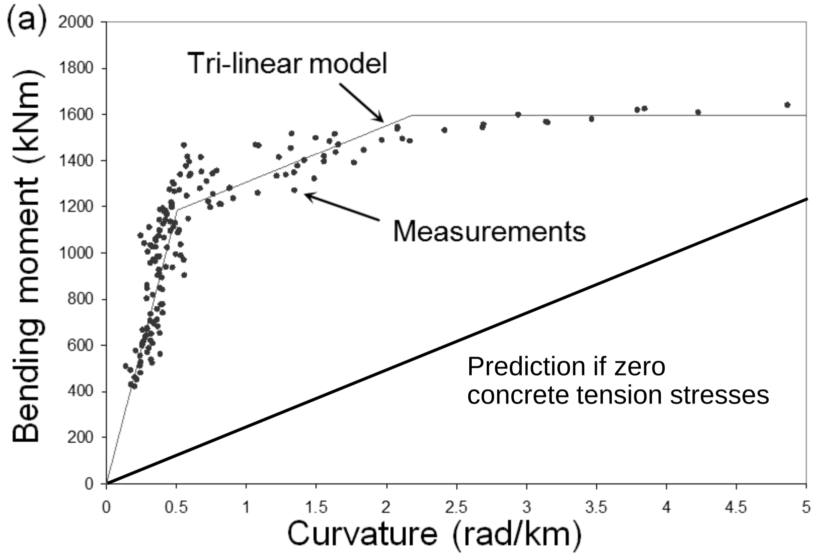


Well known

## Flexural Stiffness



## Flexural Stiffness

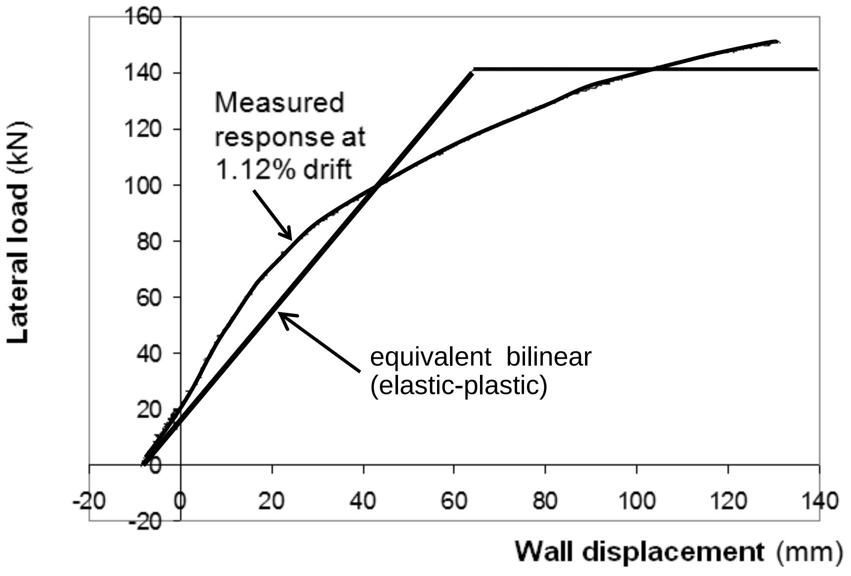


Tri-linear model

Measurements

Prediction if zero
concrete tension stresses

## Flexural Stiffness



Measured
response at
1.12% drift

equivalent bilinear
(elastic-plastic)

## Effective stiffness of wall element:

$$E_c I_e = \alpha E_c I_g$$

Upper-bound stiffness *(previously uncracked wall):*

$$\alpha = 0.6 + \frac{P}{f_c' A_g} \leq 1.0$$

Lower-bound stiffness *(severely cracked wall):*

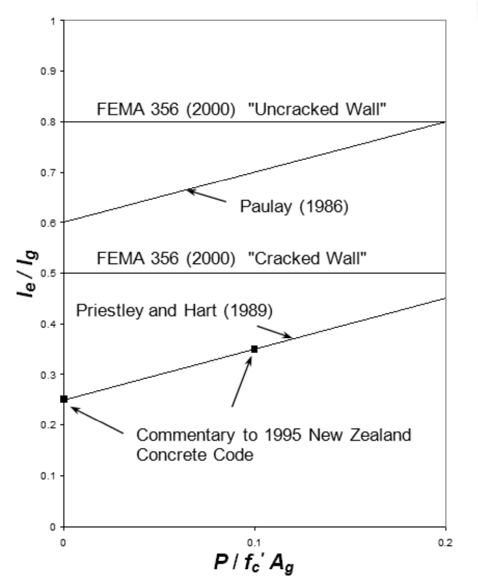
$$\alpha = 0.2 + 2.5 \frac{P}{f_c' A_g} \leq 0.7$$

## Effective stiffness of walls
## for linear seismic analysis:

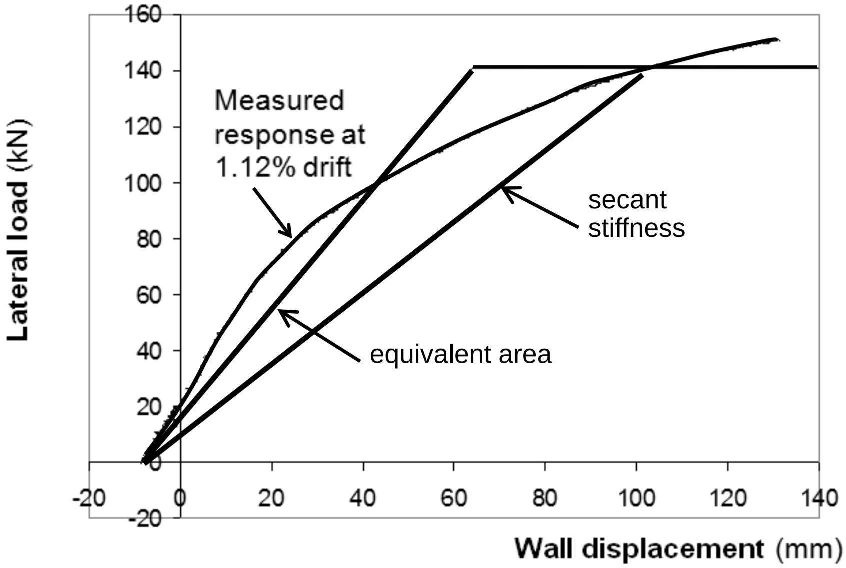
*“Structural modeling shall account for the effect of cracked sections of reinforced concrete”*

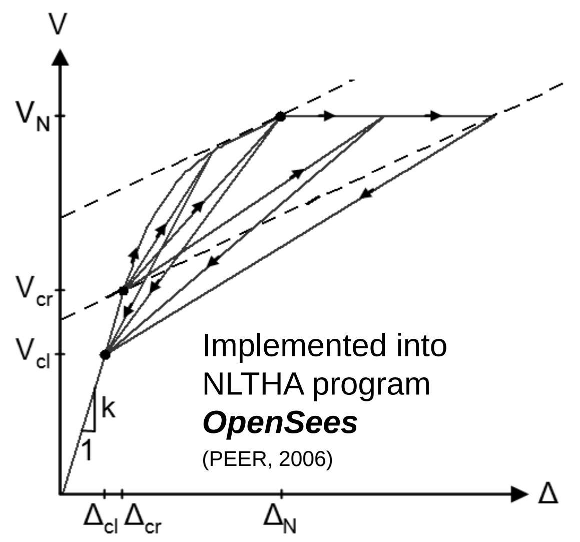
e.g., NBCC 2005 Clause 4.1.8.3 (8)

## Effective Flexural Rigidity



Axial compression





Implemented into NLTHA program ***OpenSees***
(PEER, 2006)

## NLTHA

13 different walls (structures)

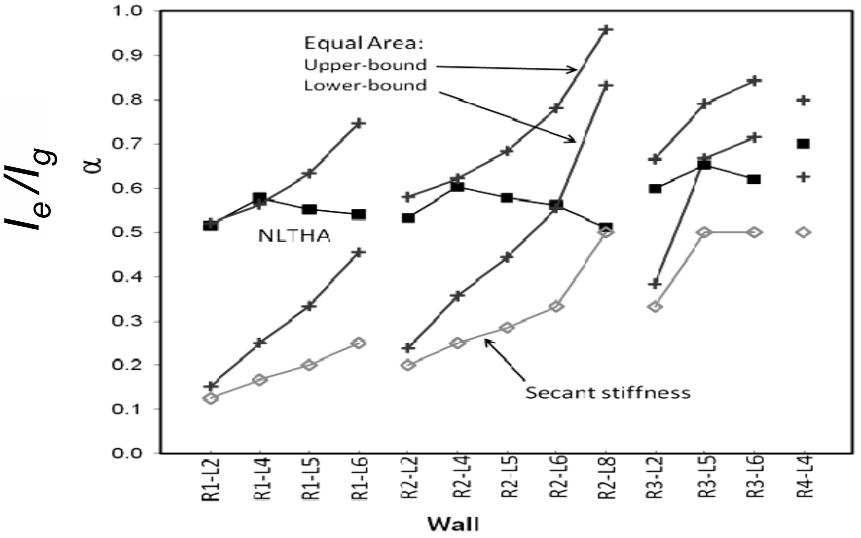
***R*** = 0.5 –> 5.0

ratio of elastic force demand at effective (reduced) stiffness to strength of structure

40 unmodified ground motions and
40 modified ground motions

From ***FEMA 440***:
20 – NEHRP Site Class B
20 – NEHRP Site Class C

WORST CASE: modified ground motions, $R = 5$, $T_i = 3$ s



## Drift capacity of walls

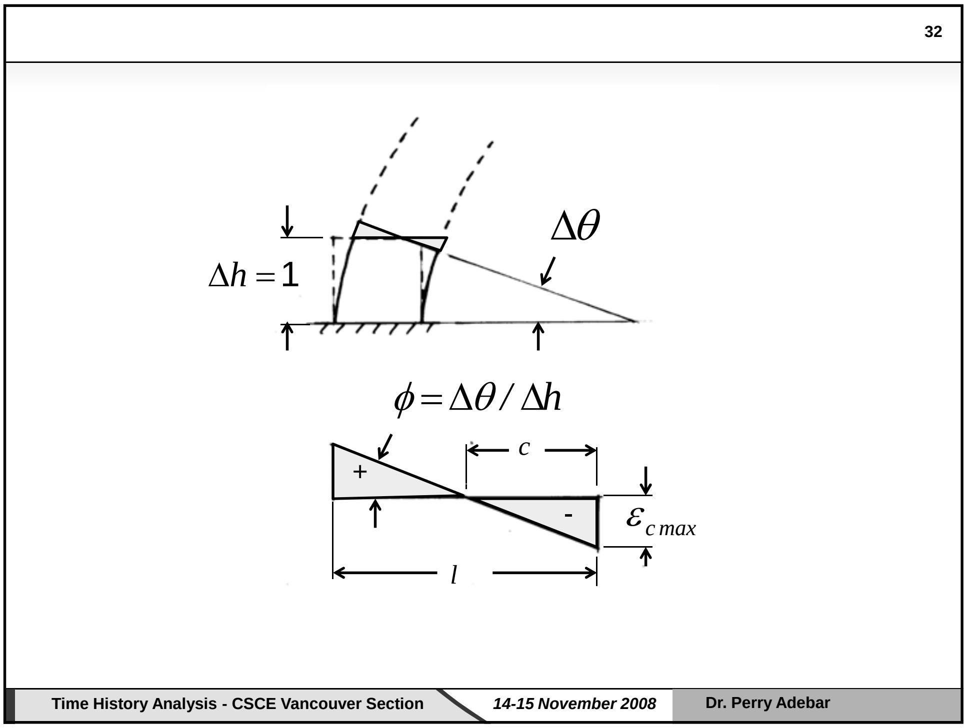
Plastic hinge length

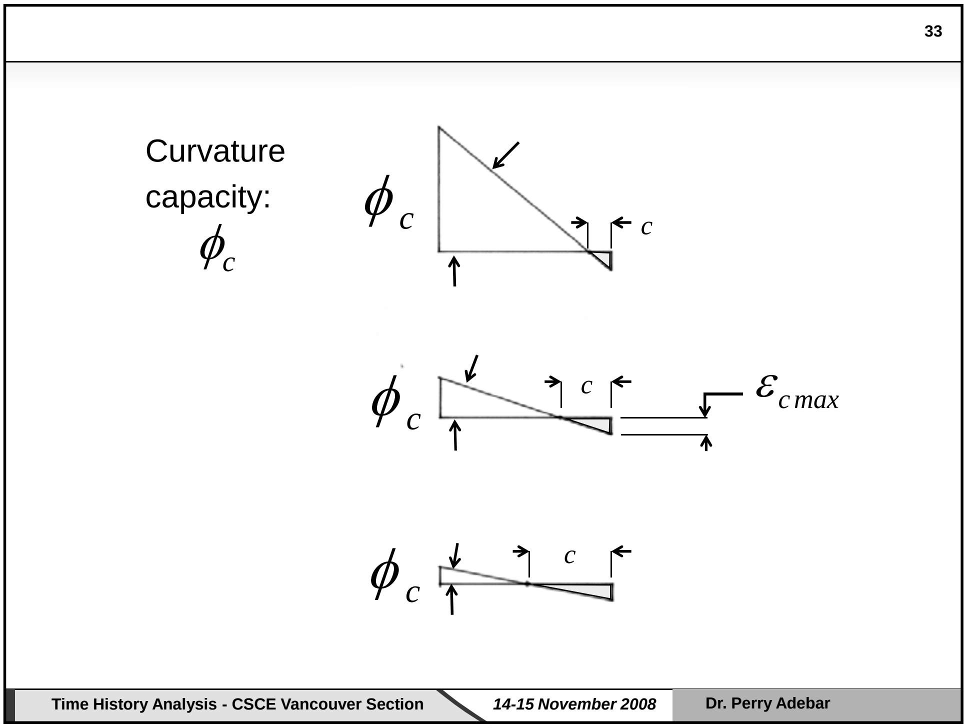
(length of uniform inelastic curvature)

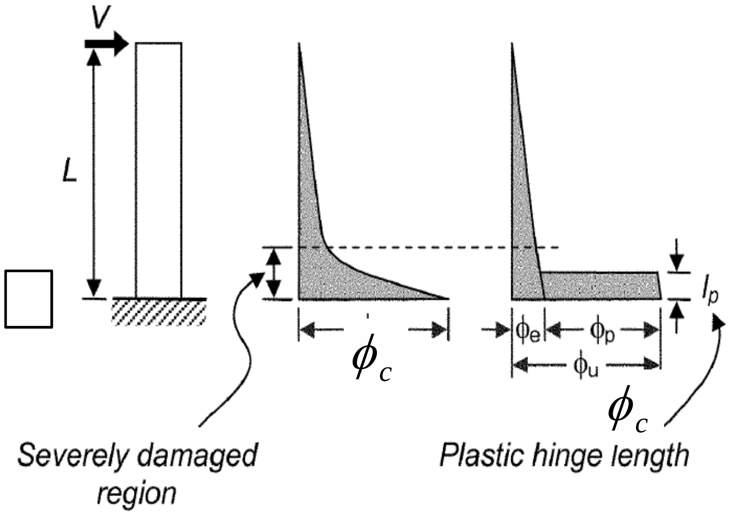
Inelastic rotation (drift)

$$\theta_p = \phi_p \times l_p$$

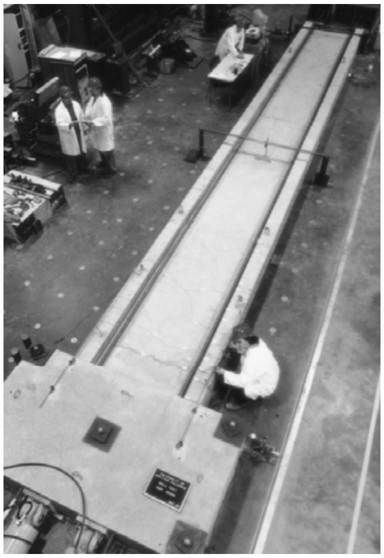
Plastic curvature capacity

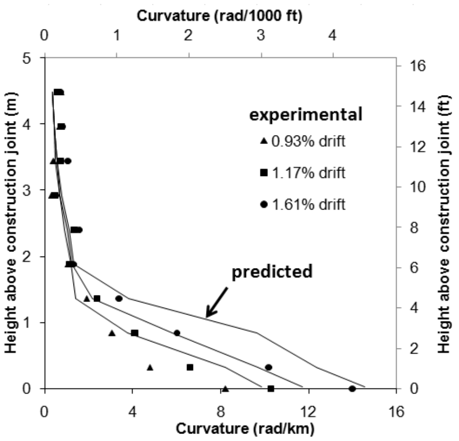


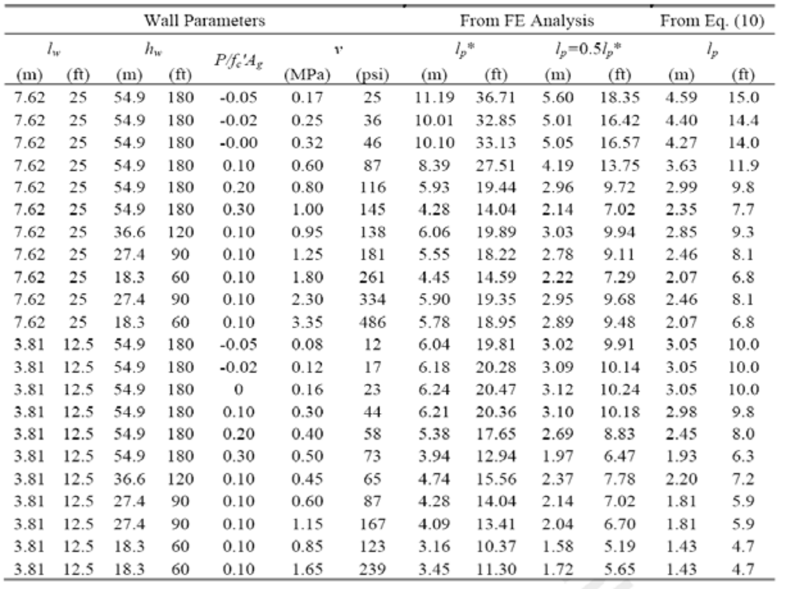

Curvature capacity: $\phi_c$



NLFE predictions of curvatures in wall test







| Wall Parameters | | | | | | | From FE Analysis | | | | From Eq. (10) | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $l_w$ | | $h_w$ | | $P/f_c'A_g$ | $v$ | | $l_p$* | | $l_p$=0.5$l_p$* | | $l_p$ | |
| (m) | (ft) | (m) | (ft) | | (MPa) | (psi) | (m) | (ft) | (m) | (ft) | (m) | (ft) |
| 7.62 | 25 | 54.9 | 180 | -0.05 | 0.17 | 25 | 11.19 | 36.71 | 5.60 | 18.35 | 4.59 | 15.0 |
| 7.62 | 25 | 54.9 | 180 | -0.02 | 0.25 | 36 | 10.01 | 32.85 | 5.01 | 16.42 | 4.40 | 14.4 |
| 7.62 | 25 | 54.9 | 180 | -0.00 | 0.32 | 46 | 10.10 | 33.13 | 5.05 | 16.57 | 4.27 | 14.0 |
| 7.62 | 25 | 54.9 | 180 | 0.10 | 0.60 | 87 | 8.39 | 27.51 | 4.19 | 13.75 | 3.63 | 11.9 |
| 7.62 | 25 | 54.9 | 180 | 0.20 | 0.80 | 116 | 5.93 | 19.44 | 2.96 | 9.72 | 2.99 | 9.8 |
| 7.62 | 25 | 54.9 | 180 | 0.30 | 1.00 | 145 | 4.28 | 14.04 | 2.14 | 7.02 | 2.35 | 7.7 |
| 7.62 | 25 | 36.6 | 120 | 0.10 | 0.95 | 138 | 6.06 | 19.89 | 3.03 | 9.94 | 2.85 | 9.3 |
| 7.62 | 25 | 27.4 | 90 | 0.10 | 1.25 | 181 | 5.55 | 18.22 | 2.78 | 9.11 | 2.46 | 8.1 |
| 7.62 | 25 | 18.3 | 60 | 0.10 | 1.80 | 261 | 4.45 | 14.59 | 2.22 | 7.29 | 2.07 | 6.8 |
| 7.62 | 25 | 27.4 | 90 | 0.10 | 2.30 | 334 | 5.90 | 19.35 | 2.95 | 9.68 | 2.46 | 8.1 |
| 7.62 | 25 | 18.3 | 60 | 0.10 | 3.35 | 486 | 5.78 | 18.95 | 2.89 | 9.48 | 2.07 | 6.8 |
| 3.81 | 12.5 | 54.9 | 180 | -0.05 | 0.08 | 12 | 6.04 | 19.81 | 3.02 | 9.91 | 3.05 | 10.0 |
| 3.81 | 12.5 | 54.9 | 180 | -0.02 | 0.12 | 17 | 6.18 | 20.28 | 3.09 | 10.14 | 3.05 | 10.0 |
| 3.81 | 12.5 | 54.9 | 180 | 0 | 0.16 | 23 | 6.24 | 20.47 | 3.12 | 10.24 | 3.05 | 10.0 |
| 3.81 | 12.5 | 54.9 | 180 | 0.10 | 0.30 | 44 | 6.21 | 20.36 | 3.10 | 10.18 | 2.98 | 9.8 |
| 3.81 | 12.5 | 54.9 | 180 | 0.20 | 0.40 | 58 | 5.38 | 17.65 | 2.69 | 8.83 | 2.45 | 8.0 |
| 3.81 | 12.5 | 54.9 | 180 | 0.30 | 0.50 | 73 | 3.94 | 12.94 | 1.97 | 6.47 | 1.93 | 6.3 |
| 3.81 | 12.5 | 36.6 | 120 | 0.10 | 0.45 | 65 | 4.74 | 15.56 | 2.37 | 7.78 | 2.20 | 7.2 |
| 3.81 | 12.5 | 27.4 | 90 | 0.10 | 0.60 | 87 | 4.28 | 14.04 | 2.14 | 7.02 | 1.81 | 5.9 |
| 3.81 | 12.5 | 27.4 | 90 | 0.10 | 1.15 | 167 | 4.09 | 13.41 | 2.04 | 6.70 | 1.81 | 5.9 |
| 3.81 | 12.5 | 18.3 | 60 | 0.10 | 0.85 | 123 | 3.16 | 10.37 | 1.58 | 5.19 | 1.43 | 4.7 |
| 3.81 | 12.5 | 18.3 | 60 | 0.10 | 1.65 | 239 | 3.45 | 11.30 | 1.72 | 5.65 | 1.43 | 4.7 |

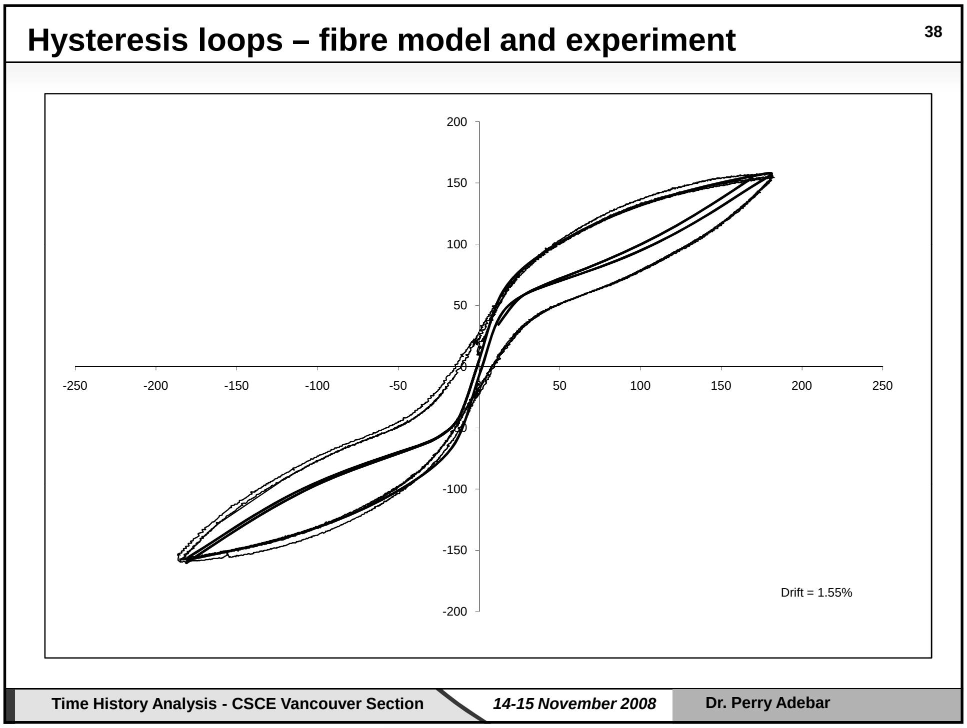
## Length of equivalent uniform inelastic curvature:

$$l_p = \left(0.2l_w + 0.05z\right)\left(1 - \frac{1.5\,P}{f_c'A_g}\right) \le 0.8l_w$$

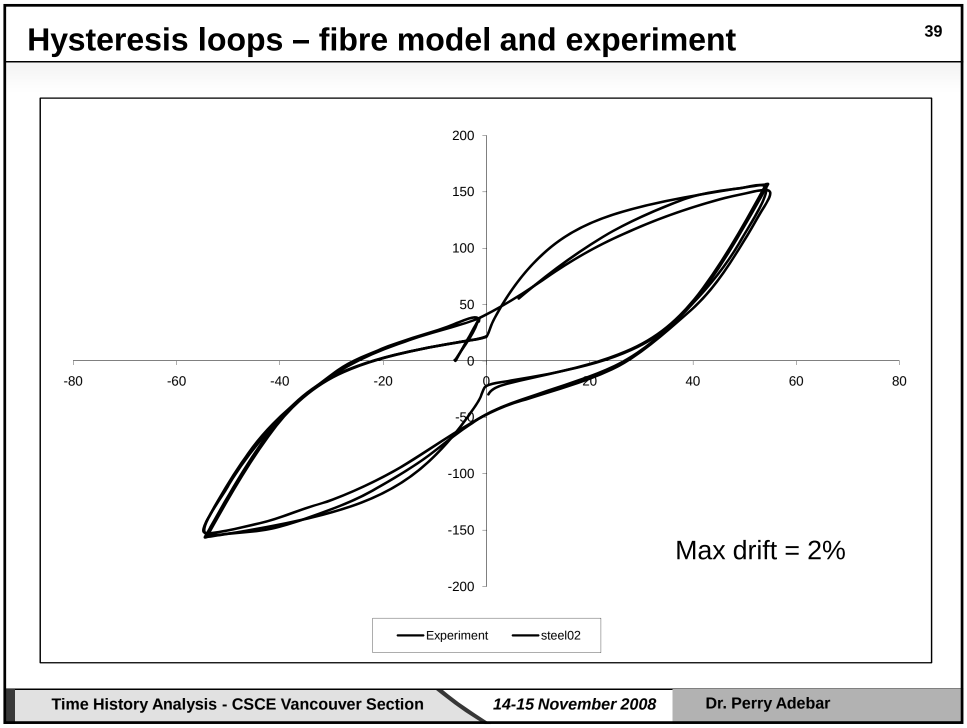
$$z = M/V$$

Use to relate curvature capacity to rotational capacity

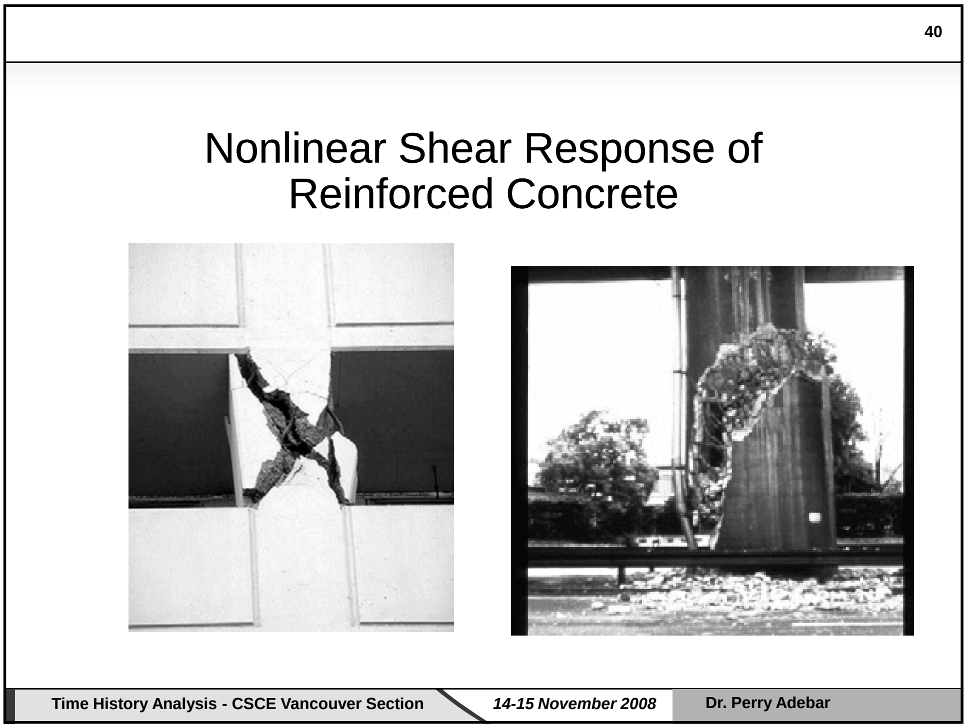
Actual length of inelastic curvature is 2 x $l_p$



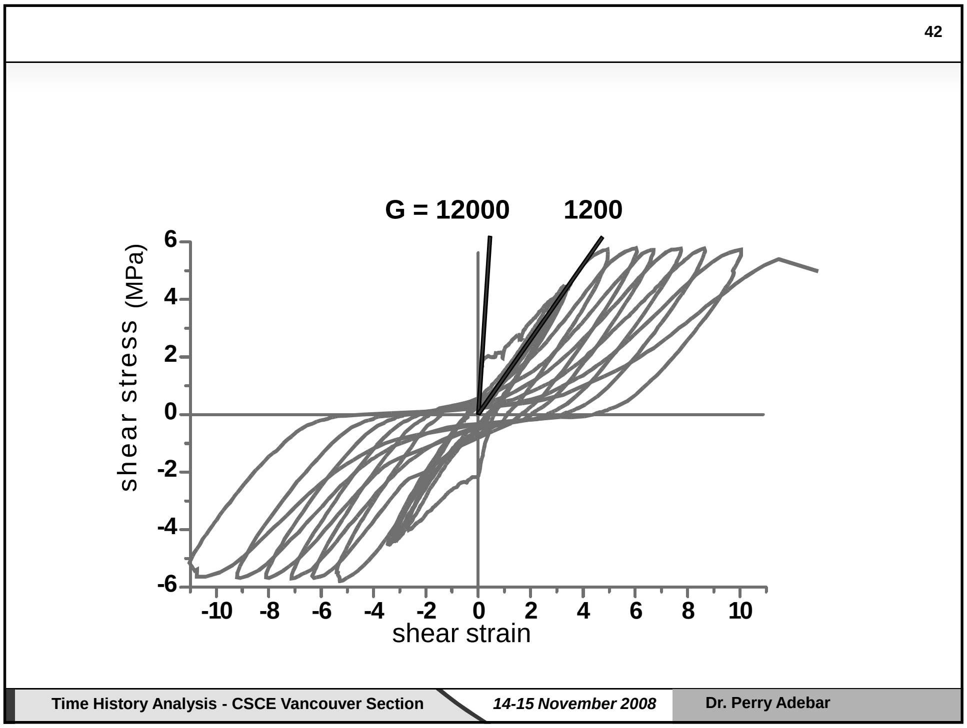
## Hysteresis loops – fibre model and experiment



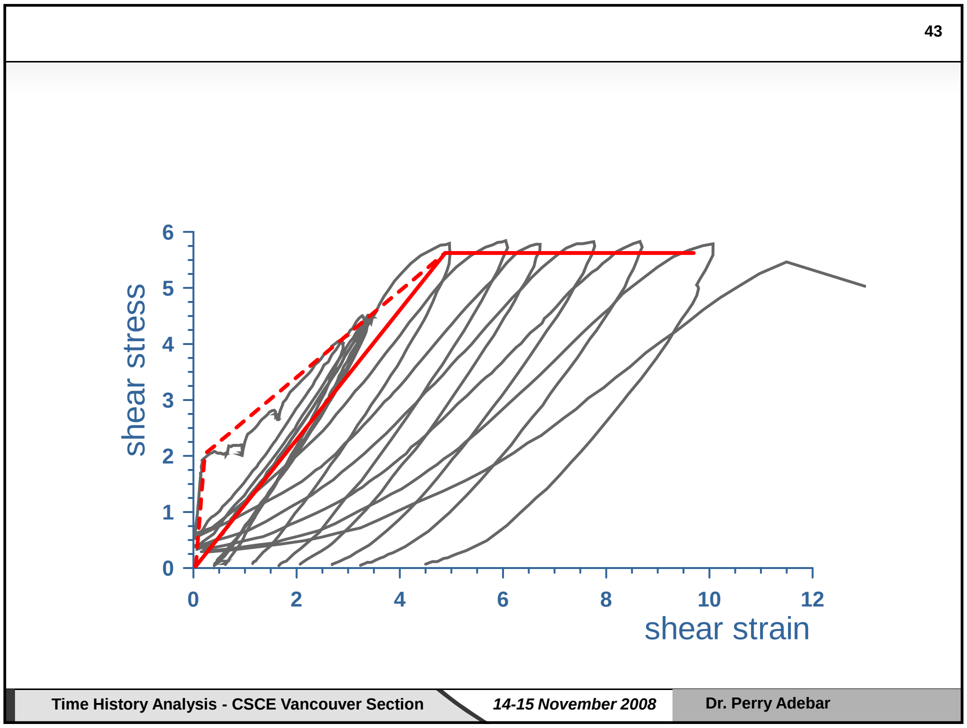
## Hysteresis loops – fibre model and experiment



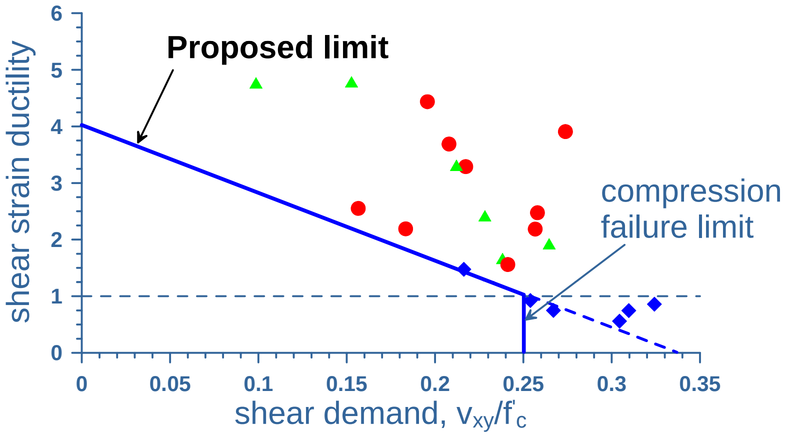
## Nonlinear Shear Response of Reinforced Concrete



G = 12000
1200
shear stress (MPa)
6
4
2
0
-2
-4
-6
-10
-8
-6
-4
-2
0
2
4
6
8
10
shear strain



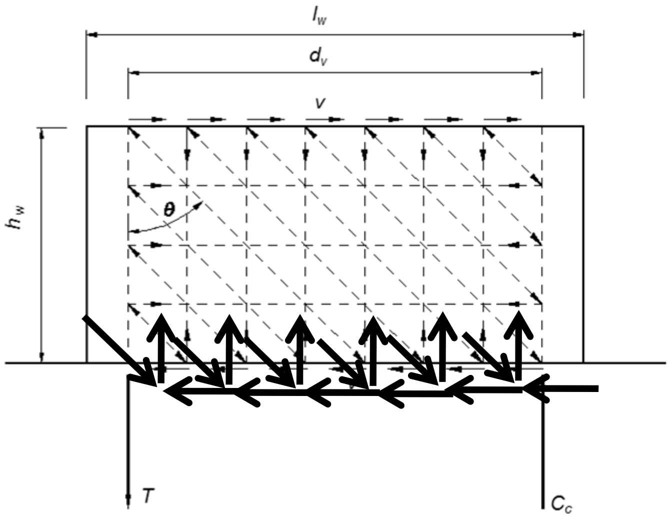
shear stress
6
5
4
3
2
1
0
0
2
4
6
8
10
12
shear strain

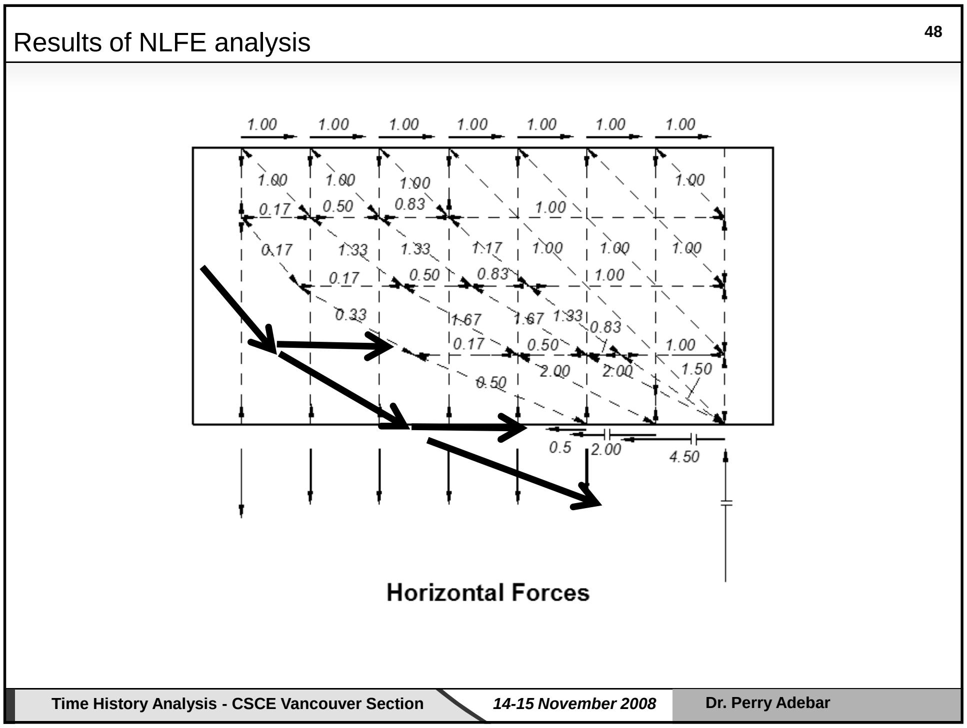


Complete nonlinear shear model to be described here:
(still to come)

Strut-and-tie model in PERFORM is not appropriate for squat walls:

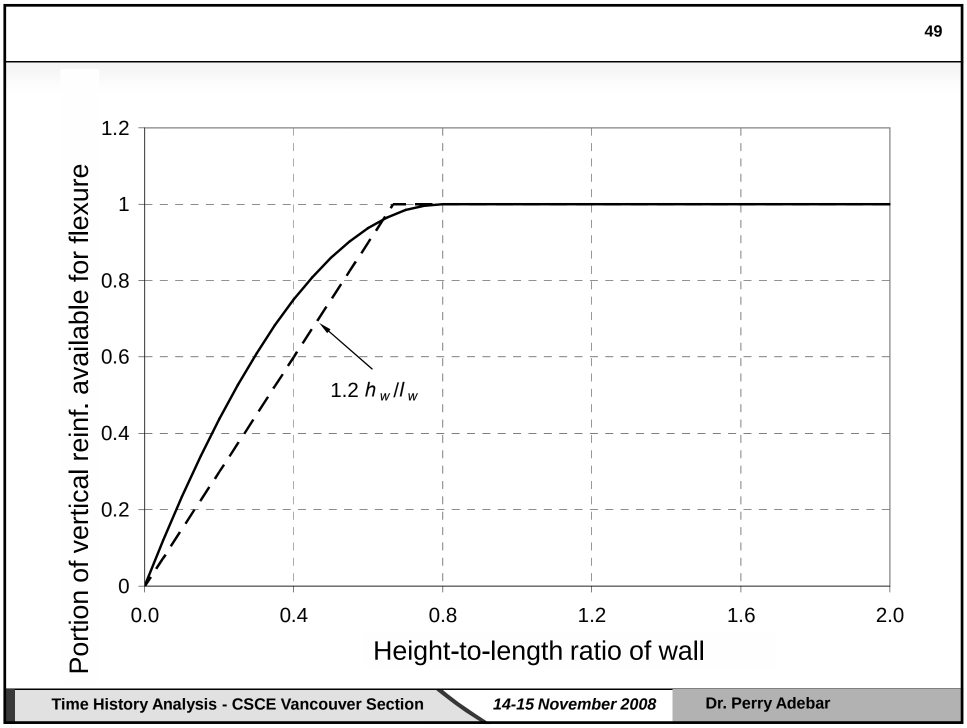
Use shear strength provisions given in CSA A23.3 Clause 21.7.4 except modify Clause 21.7.4.7



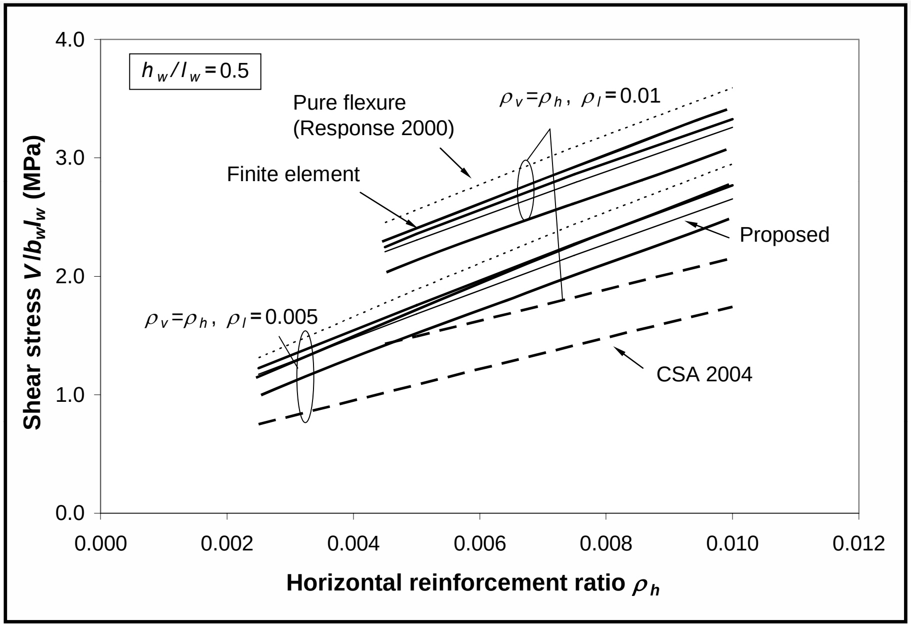


## Results of NLFE analysis

**Horizontal Forces**



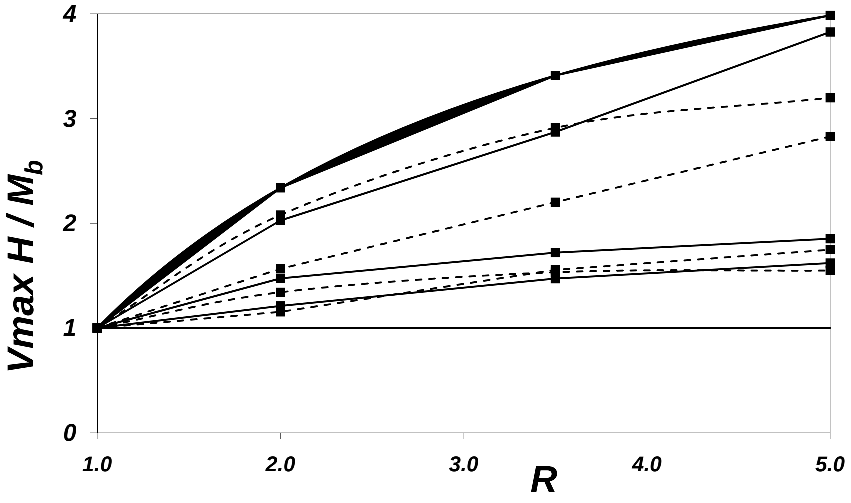
## Wall strength limited by vertical reinf. at base

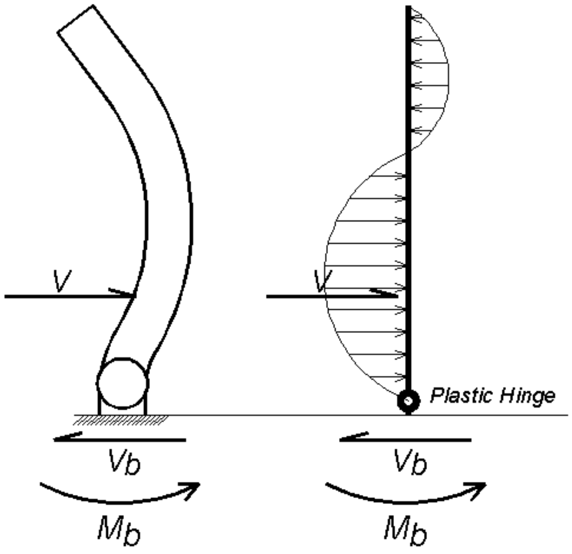


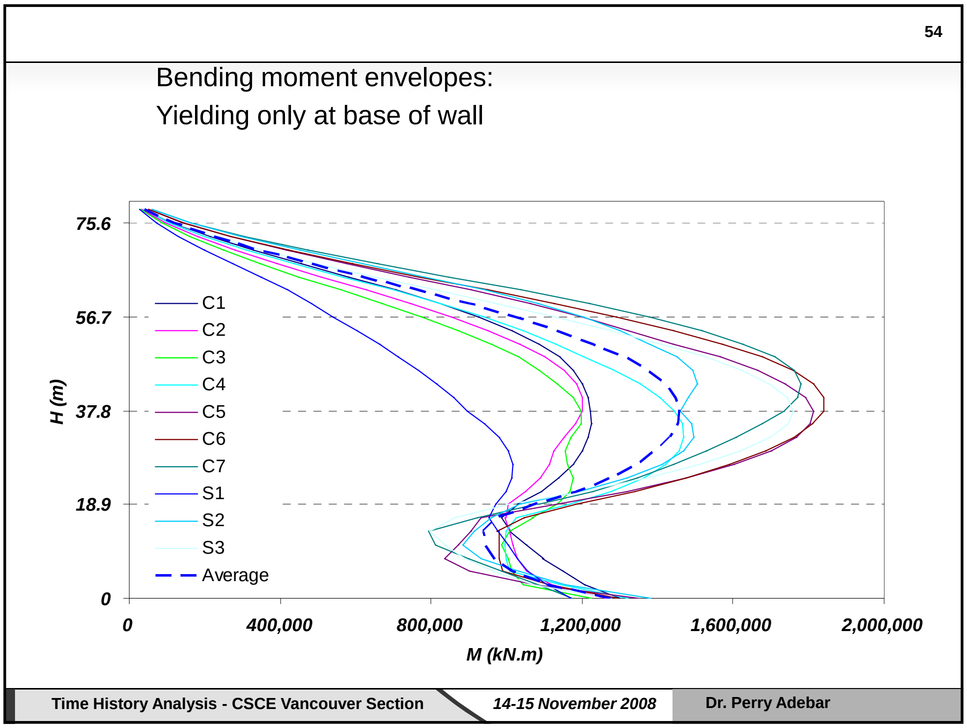
## Example NLTHA – Influence of shear stiffness

The following example of a NLTHA of a tall building demonstrates the importance of the shear stiffness model on the analysis results

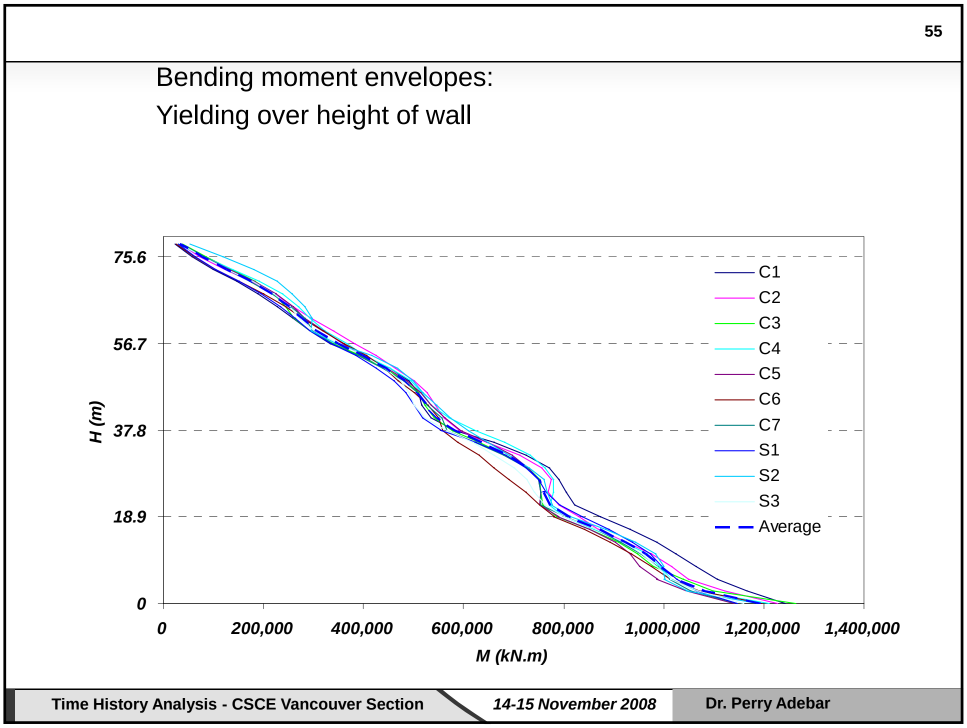
## 2 ground motions, 4 initial periods, flexural yielding only at base





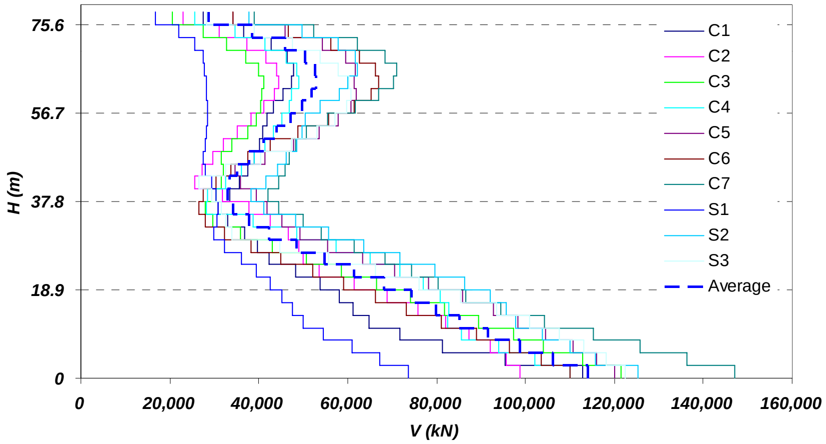


Bending moment envelopes:
Yielding only at base of wall

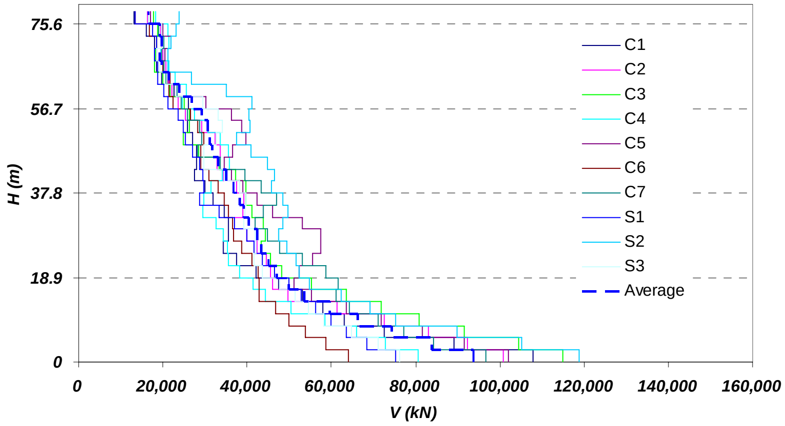


Bending moment envelopes:
Yielding over height of wall

## Shear force envelopes:
## Yielding only at base of wall

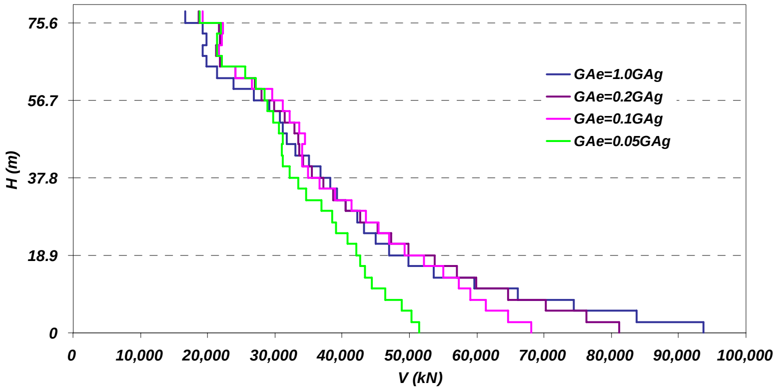


## Shear force envelopes:
## Yielding over height of wall



**Shear force envelopes:**

Yielding over height of wall and
reduced shear stiffness due to cracking



| *R* | **Flexural Yielding** | **Shear Stiffness** *$GA_{ve}/GA_{vg}$* | **Average Dynamic Shear Amplification** |
|---|---|---|---|
| 2.0 | Single hinge at base | 1.0 | 1.48 |
| | Multiple hinges | 1.0 | 1.32 |
| | | 0.2 | 1.06 |
| | | 0.1 | 0.94 |
| | | 0.05 | 0.79 |
| 3.5 | Single hinge at base | 1.0 | 2.34 |
| | Multiple hinges | 1.0 | 1.99 |
| | | 0.2 | 1.66 |
| | | 0.1 | 1.36 |
| | | 0.05 | 1.12 |
| 5.0 | Single hinge at base | 1.0 | 3.09 |
| | Multiple hinges | 1.0 | 2.53 |
| | | 0.2 | 2.20 |
| | | 0.1 | 1.84 |
| | | 0.05 | 1.40 |

# TIME HISTORY ANALYSIS

## LECTURE # 7

## Impact of Foundation Modeling on the Accuracy of Seismic Response History Analysis

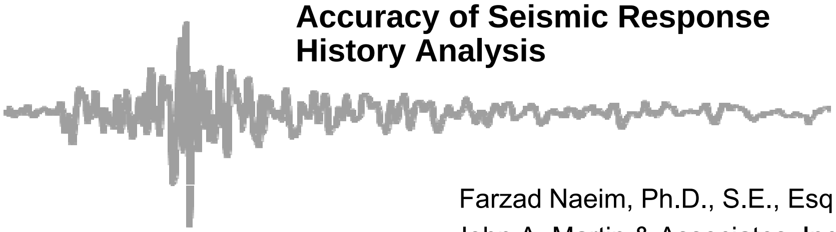
**Farzad Naeim, Ph.D., S.E., Esq.**
**John A. Martin & Associates, Inc.**

Dr. Farzad Naeim is the Vice President and General Counsel for John A. Martin & Associates, Inc., (JAMA) in Los Angeles, one of the largest structural consulting firms in the United States. Farzad is the 2007 recipient of the prestigious Fazlur Khan Medal for life-time achievement from Council on Tall Buildings and Urban Habitat. He has received numerous other awards including the Outstanding Journal Paper Award six times in the past ten years from Los Angeles Tall Buildings Structural Design Council. He just finished his five year term as the editor-in-Chief of *Earthquake Spectra*, the professional journal of the Earthquake Engineering Research Institute (EERI). Farzad is currently the President-Elect of EERI. Dr. Naeim serves as an advisor to several national and state organizations and major universities. He is the editor of *The Seismic Design Handbook*, now in its second edition, and the coauthor of *Design of Seismic Isolated Structures*. He has published more than 120 papers on various aspects of earthquake engineering and has developed more than 45 different software systems for earthquake engineering design and education.

*The Canadian Society for Civil Engineering, Vancouver Section*

# TIME HISTORY ANALYSIS

## Impact of Foundation Modeling on the Accuracy of Seismic Response History Analysis



Farzad Naeim, Ph.D., S.E., Esq.
John A. Martin & Associates, Inc.

*A technical seminar on the use of time histories and site specific response spectra in structural design, and an introduction to linear and non-linear time history analysis.*

***14-15 November 2008 Vancouver, BC***

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## Co-Authors:

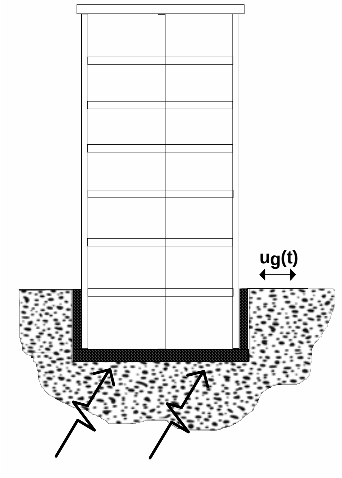
- Prof. Jonathon Stewart, UCLA
- Mr. Salih Tileylioglu, Ph.D. Candidate, UCLA
- Dr. Arzhang Alimoradi, JAMA

## Soil-Foundation-Structure Interaction Outline

- Introduction and project approach
- Buildings considered
- SFSI modeling procedures
  - Ground motions
  - Foundation springs/damping
  - Application to building LA54
- Simplifications to most accurate model
- Implementation issues
- Results and conclusions

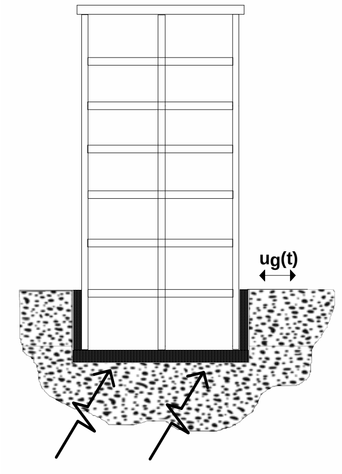
## Introduction

- Subject: Buildings with subterranean levels
- Various methods for evaluating:
  - Input motions
  - Foundation compliance
- *Impact on accuracy of response history analysis results?*



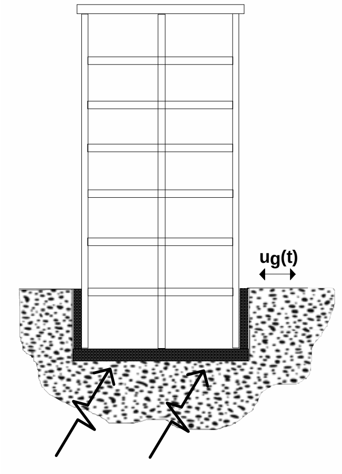
## Project Approach

- Construct “Most Accurate” (MA) model:
  - Realistic ground motions
  - Foundation/soil stiffness & damping
  - Compliant foundation elements (walls, slabs)
  - Elastic structural elements
  - *Verify against recordings*



## Project Approach

- Construct “Most Accurate” (MA) model
- Simplify foundation modeling step-by-step & repeat analyses
- Consider several simple approaches used in practice
- Identify critical components of MA foundation model



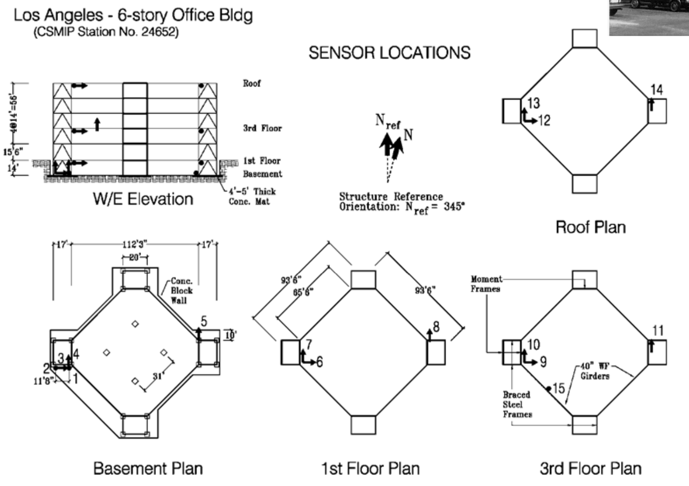
## Buildings Selected

- Require subterranean levels
- Various heights
- Instrumentation inclusive of base verticals preferred

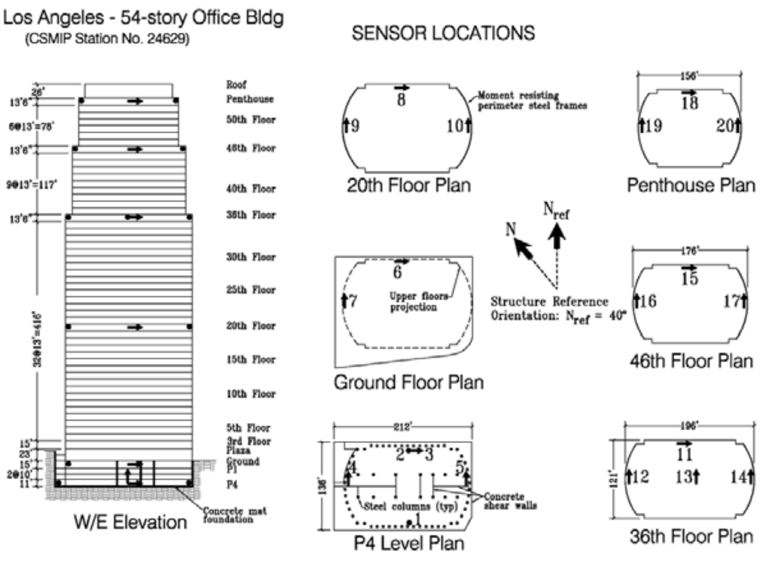
## Buildings Selected

| *No.* | *CSMIP ID* | *Name* | *Recordings* | *Embedment* | **Site Condition** |
|---|---|---|---|---|---|
| 1 | 24652 | Los Angeles 6-Story Office | 1. 1994 Northridge<br>2. 2001 Beverly Hills | 1 level | Deep Alluvium |
| 2 | 24629 | Los Angeles 54 Story Office | 1. 1994 Northridge<br>2. 1999 Hector Mines | 4 levels | Alluvium |
| 3 | 58503 | Richmond 3 Story Gov. Office | 1. 1989 Loma Prieta | 1 level | Deep Alluvium |
| 4 | 24322 | Sherman Oaks 13 Story Office | 1. 1987 Whittier<br>2. 1994 Northridge | 2 levels | Alluvium |

# Building No. 1

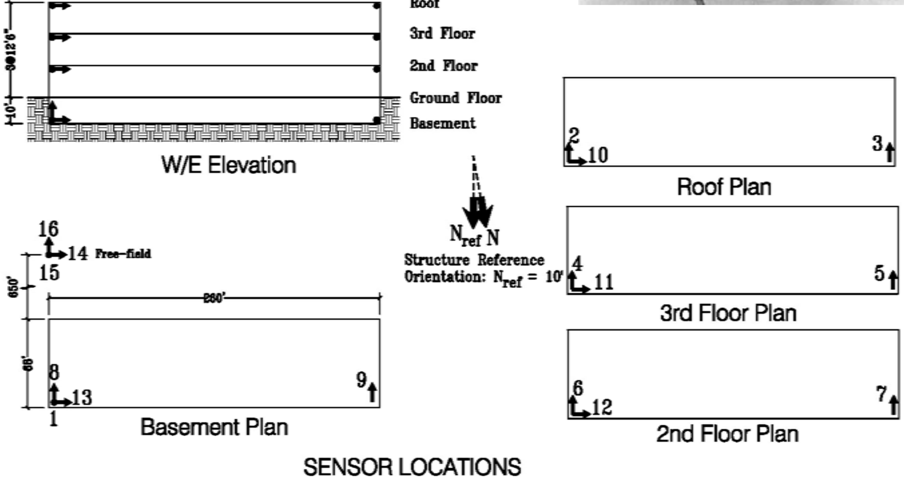


# Building No. 2

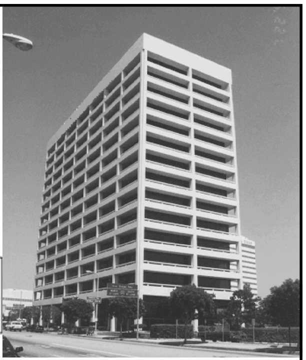


## Building No. 3

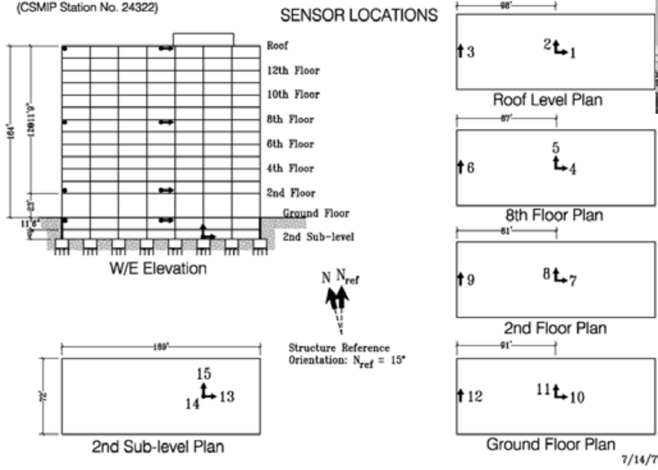
Richmond - 3-story Govt Office Bldg
(CSMIP Station No. 58503)

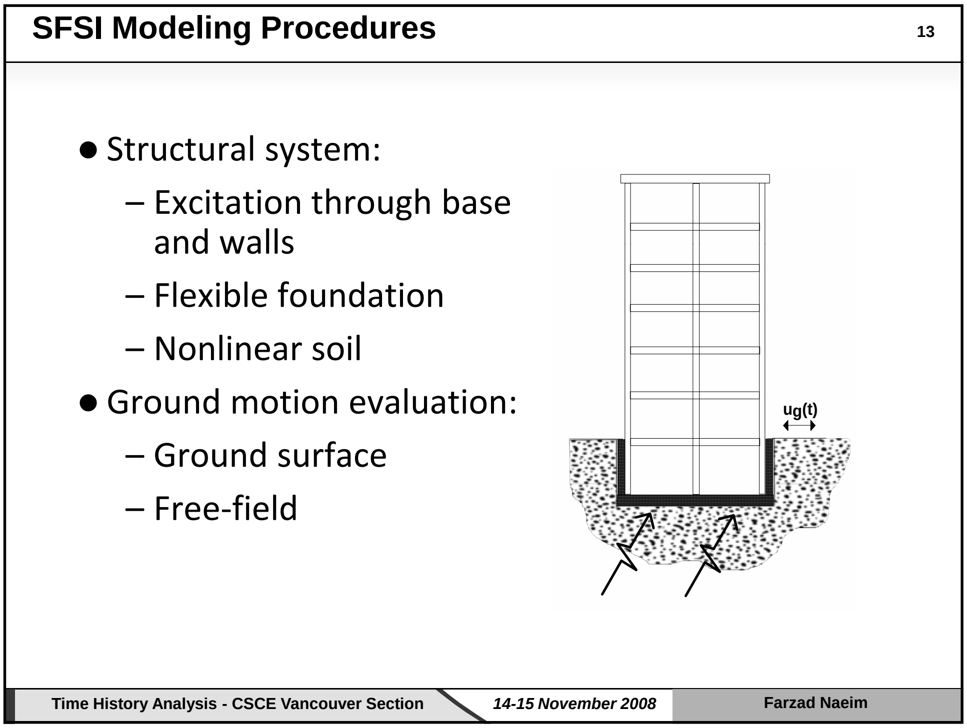


## Building No. 4



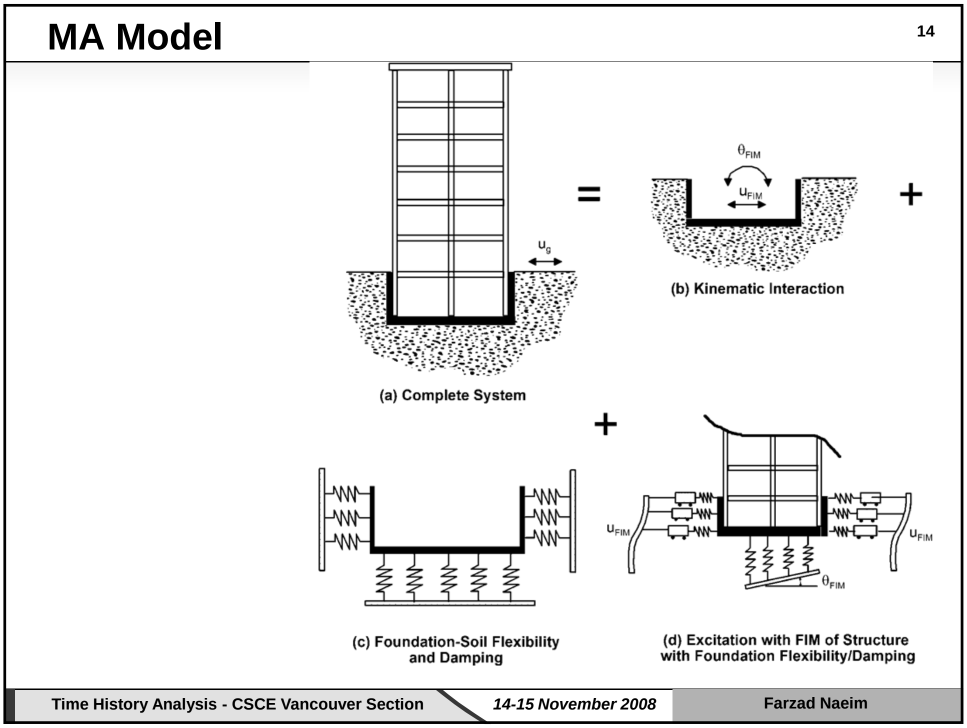
Sherman Oaks - 13-story Commercial Bldg
(CSMIP Station No. 24322)





## SFSI Modeling Procedures

- Structural system:
  - Excitation through base and walls
  - Flexible foundation
  - Nonlinear soil
- Ground motion evaluation:
  - Ground surface
  - Free-field



## MA Model

(a) Complete System

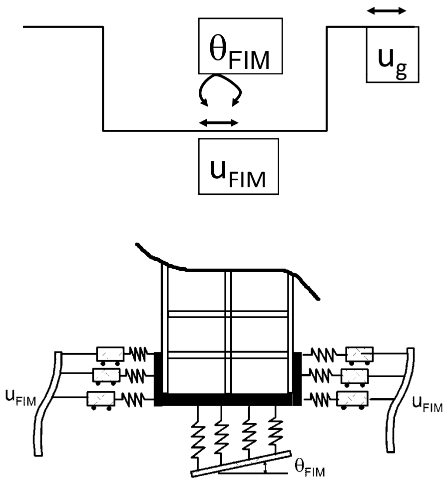
(b) Kinematic Interaction

(c) Foundation-Soil Flexibility and Damping

(d) Excitation with FIM of Structure with Foundation Flexibility/Damping

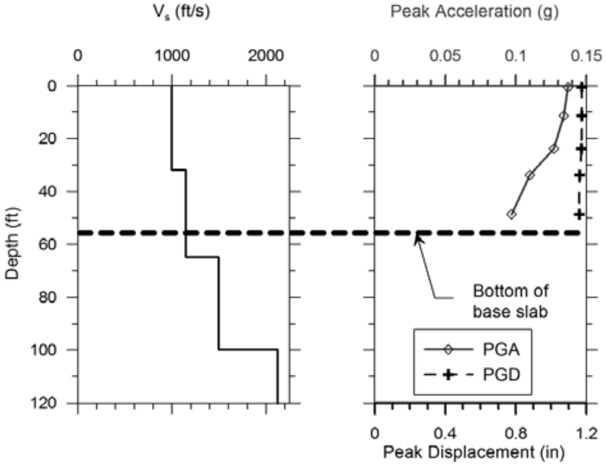
## Ground Motions

- Free-field translation ≠ base slab translation
- Base rotation $\theta_{FIM}$ introduced
- Depth-variable ground motions along basement walls



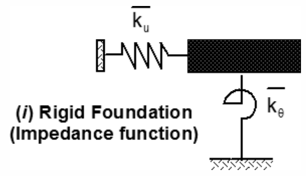
## Application to LA 54

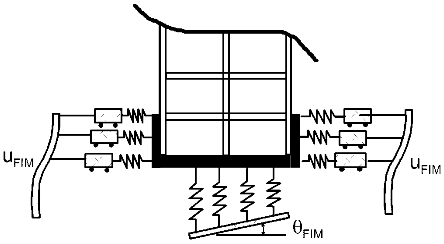
- Ground motion variation with depth



## Soil-Foundation Stiffness and Damping

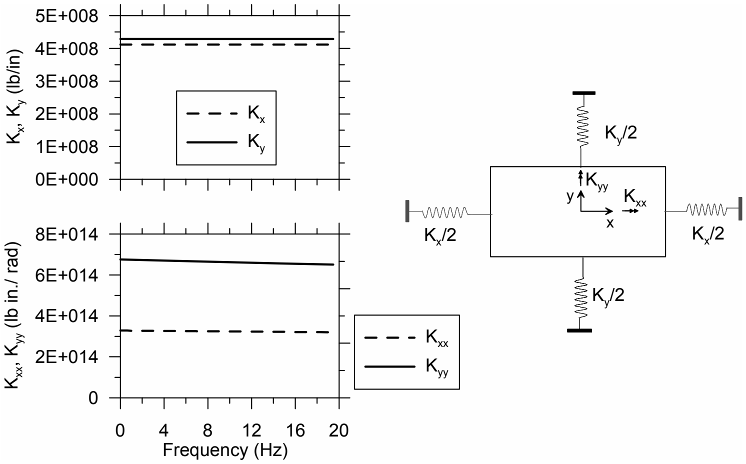
- Calculate impedance functions for rigid foundation
  - Contains springs and dashpots
  - Embedment effect
  - Frequency dependent
- Distribute across foundation
  - Vertical springs ↔ $k_\theta$
  - Horizontal springs ↔ $k_u$

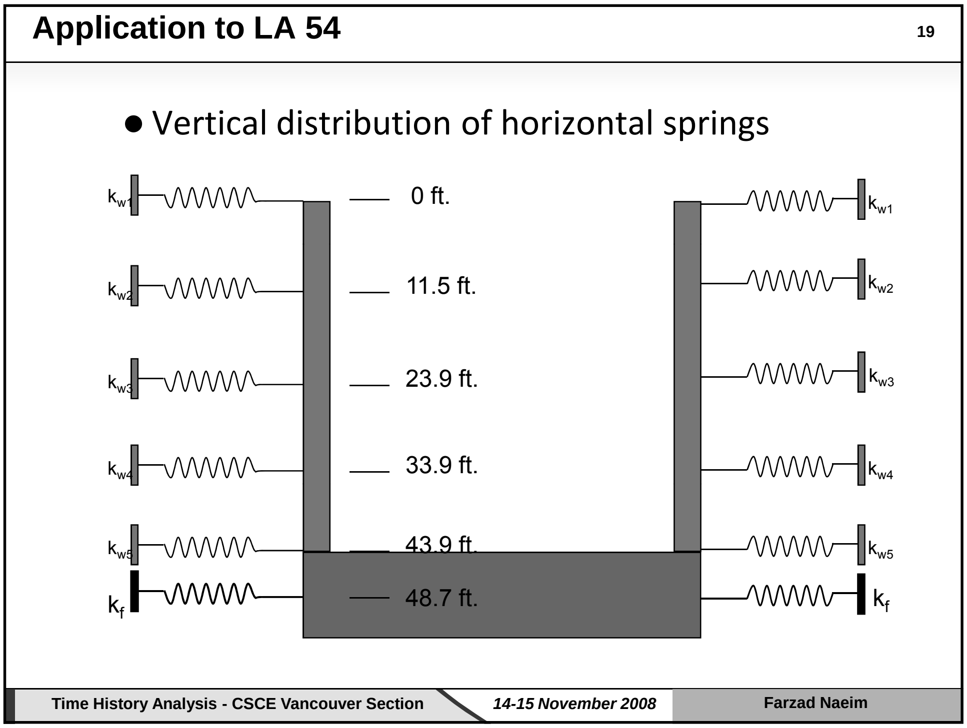




## Application to LA 54

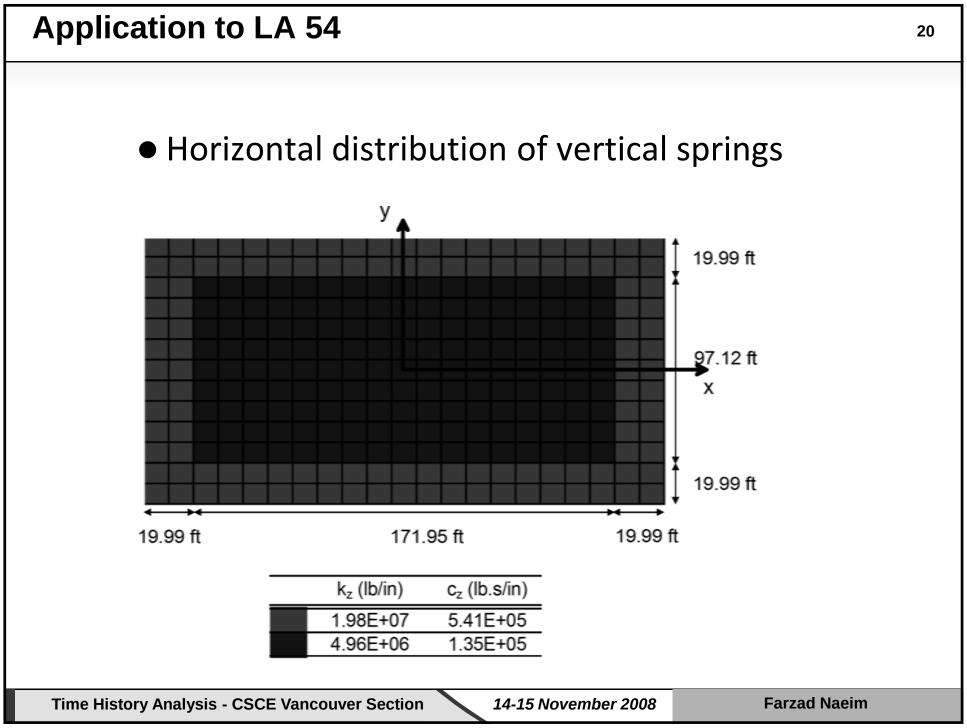
- Foundation impedance (stiffness component)





## Application to LA 54

- Vertical distribution of horizontal springs



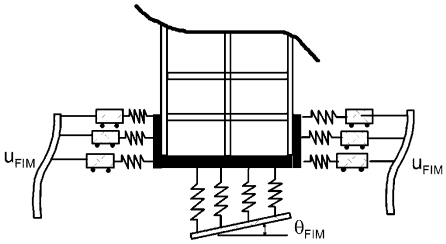
## Application to LA 54

- Horizontal distribution of vertical springs

| | $k_z$ (lb/in) | $c_z$ (lb.s/in) |
|---|---|---|
| ▨ | 1.98E+07 | 5.41E+05 |
| ■ | 4.96E+06 | 1.35E+05 |

## Simplifications to MA Model

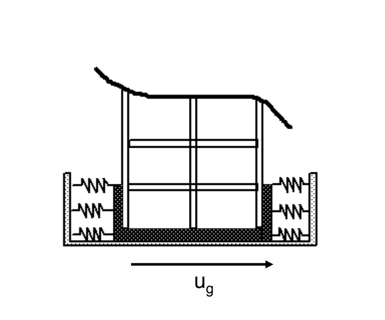
- *Model 1:* Rigid below-ground structural elements
- *Model 3a:* Tension allowed at spring-foundation interface

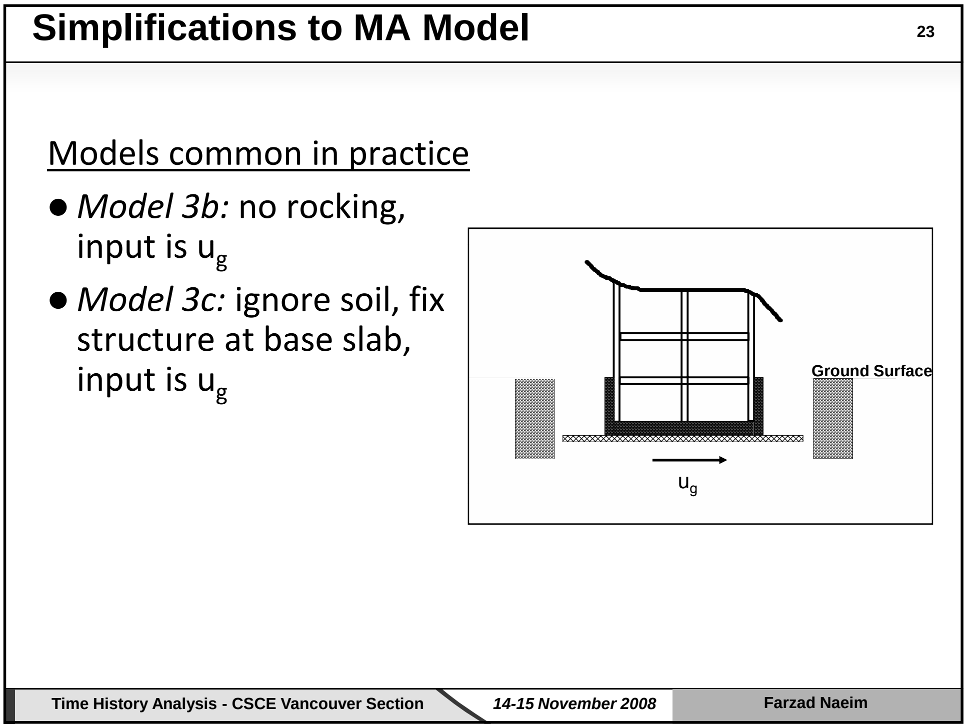


## Simplifications to MA Model

Models common in practice

- *Model 3b:* no rocking, input is $u_g$

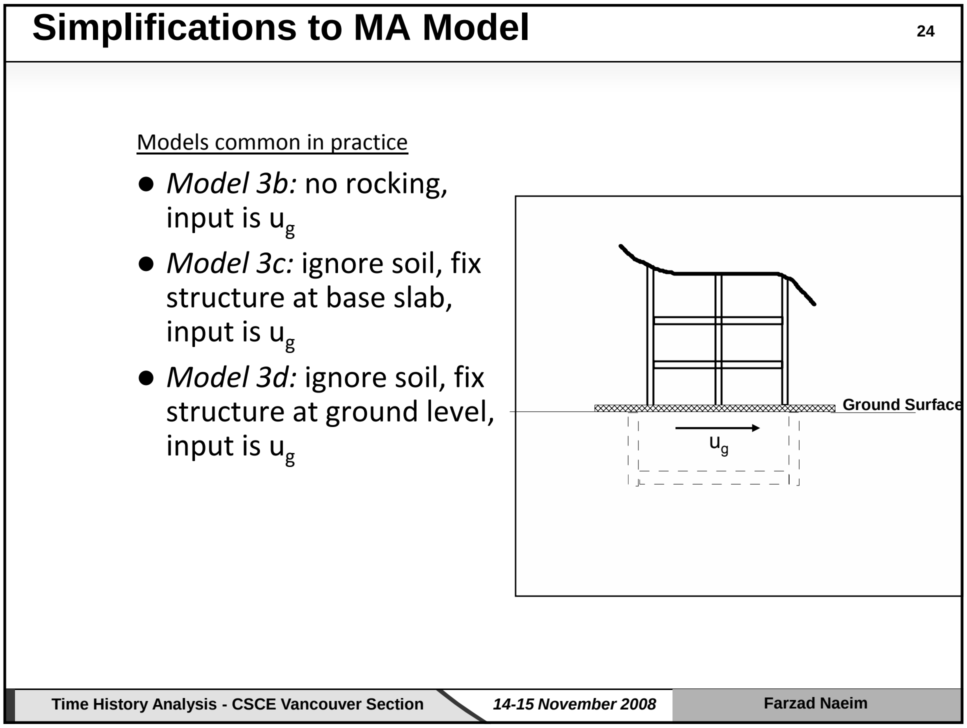




## Simplifications to MA Model

Models common in practice

- *Model 3b:* no rocking, input is $u_g$
- *Model 3c:* ignore soil, fix structure at base slab, input is $u_g$



## Simplifications to MA Model

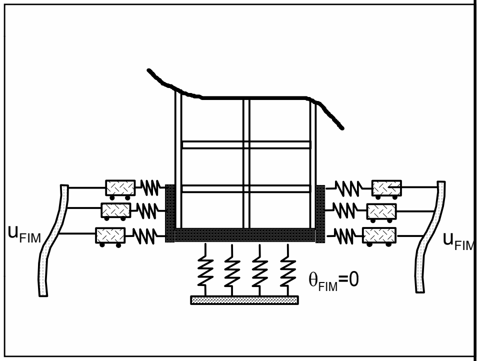
Models common in practice

- *Model 3b:* no rocking, input is $u_g$
- *Model 3c:* ignore soil, fix structure at base slab, input is $u_g$
- *Model 3d:* ignore soil, fix structure at ground level, input is $u_g$

## Simplifications to MA Model

Ground motion issues:

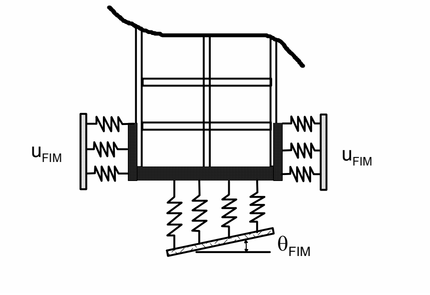
- *Model 2a:* Remove $\theta_{FIM}$



## Simplifications to MA Model

Ground motion issues:

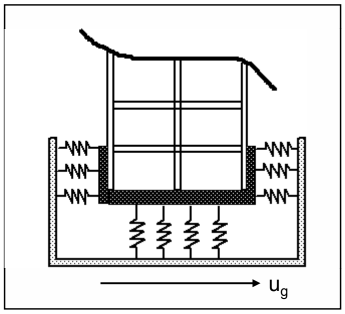
- *Model 2a:* Remove $\theta_{FIM}$
- *Model 2b:* Depth-invariant ground motion



## Simplifications to MA Model

<u>Ground motion issues</u>:

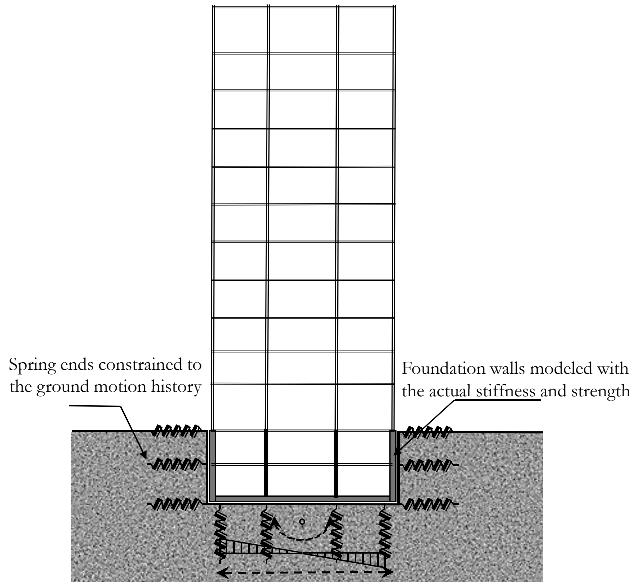
- *Model 2a:* Remove $\theta_{FIM}$
- *Model 2b:* Depth-invariant ground motion
- Model 2c: ignore kinematic interaction



## Choice of Software (nonlinear capable)

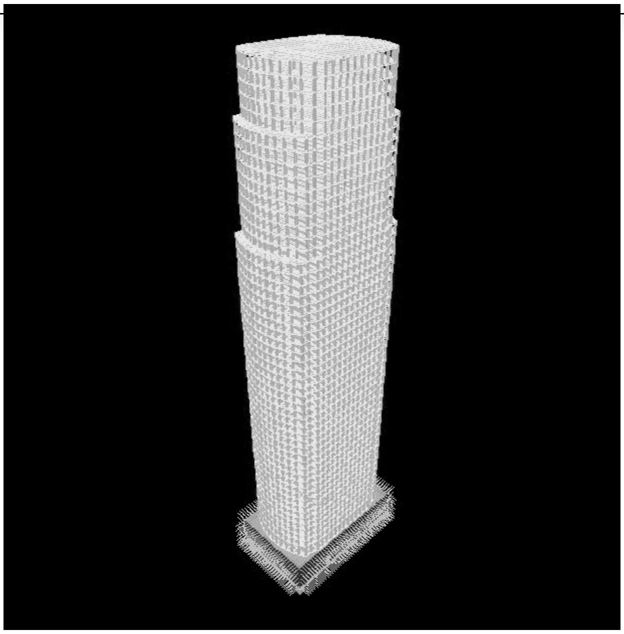
- Commonly used for seismic analysis and design
  - – ETABS
  - – SAP2000
  - – Perfrom-3D
- Public-domain (not user friendly)
  - – OpenSees
- General F.E. (if you are suicidal!)
  - – Adina
  - – Abaqus
  - – Ansys
  - – and more

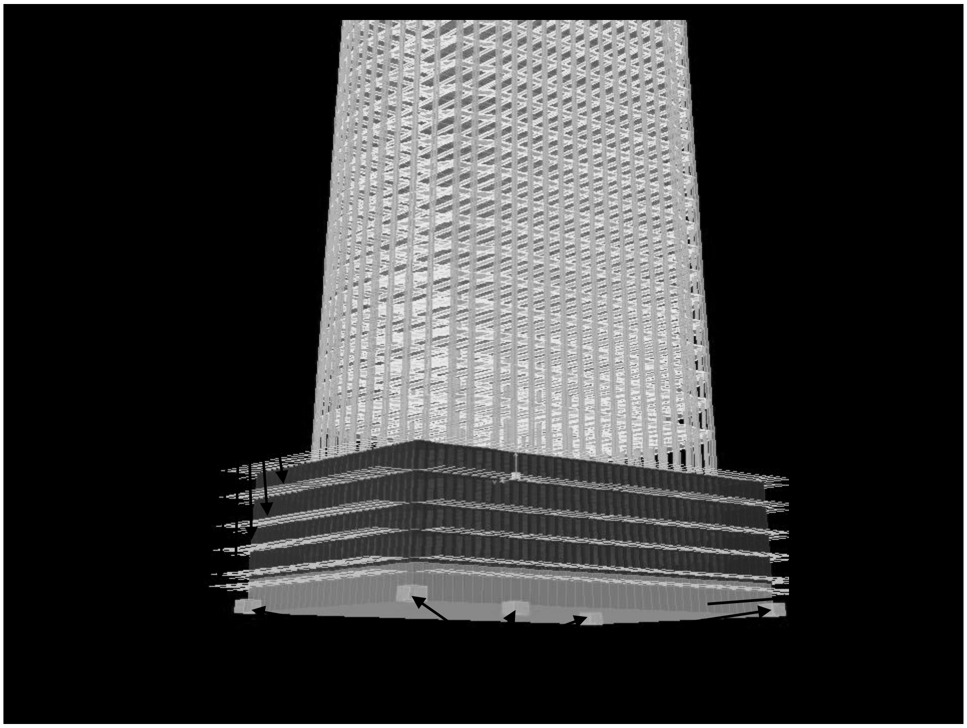
# MA Model

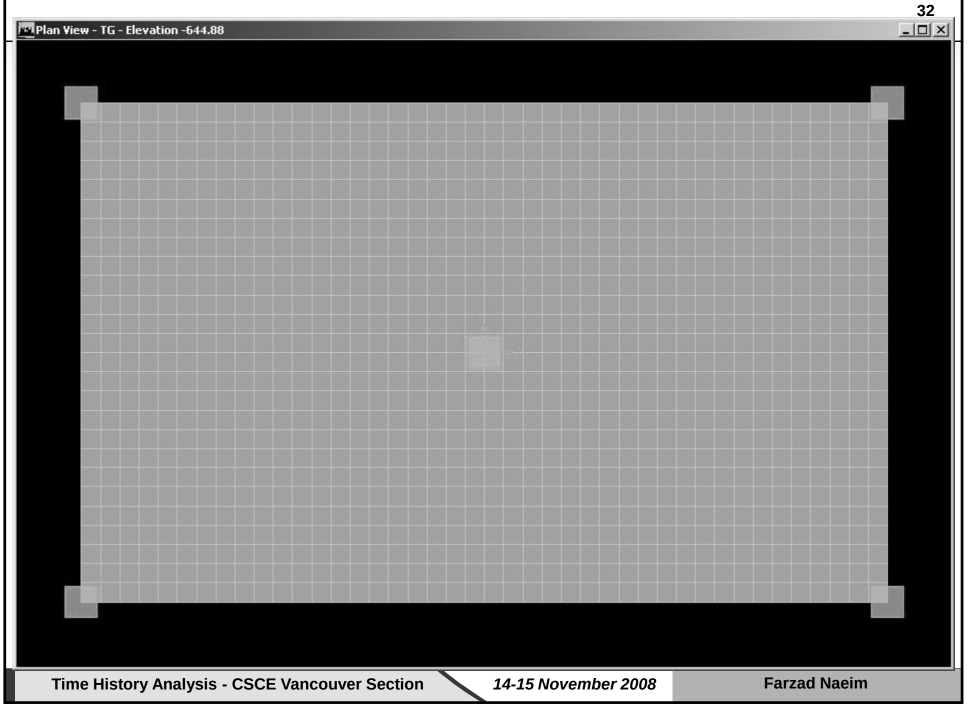


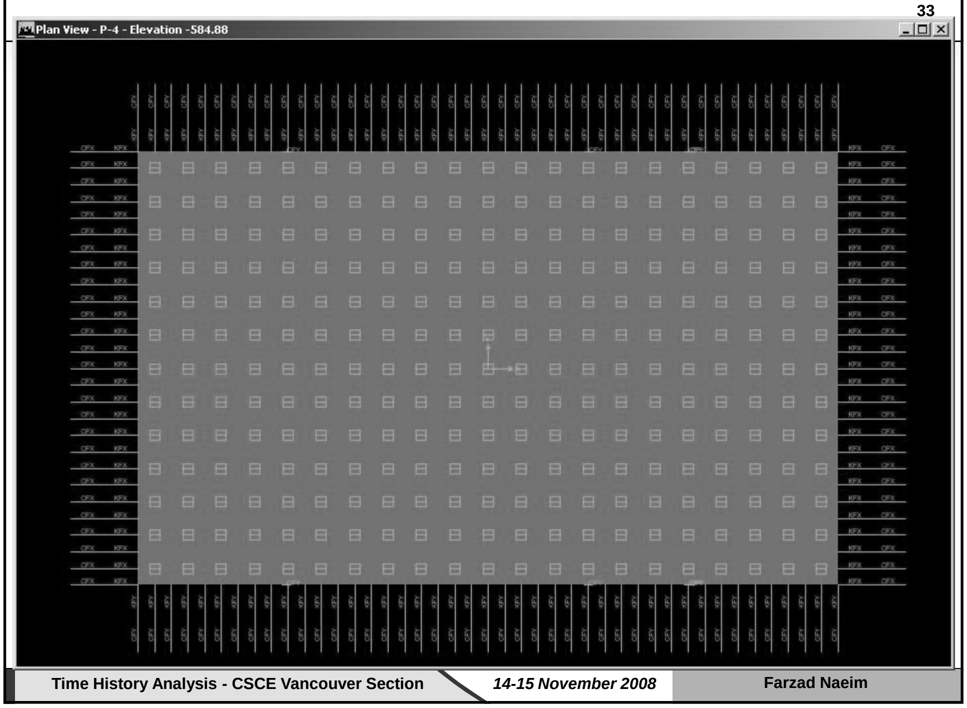
Spring ends constrained to the ground motion history

Foundation walls modeled with the actual stiffness and strength

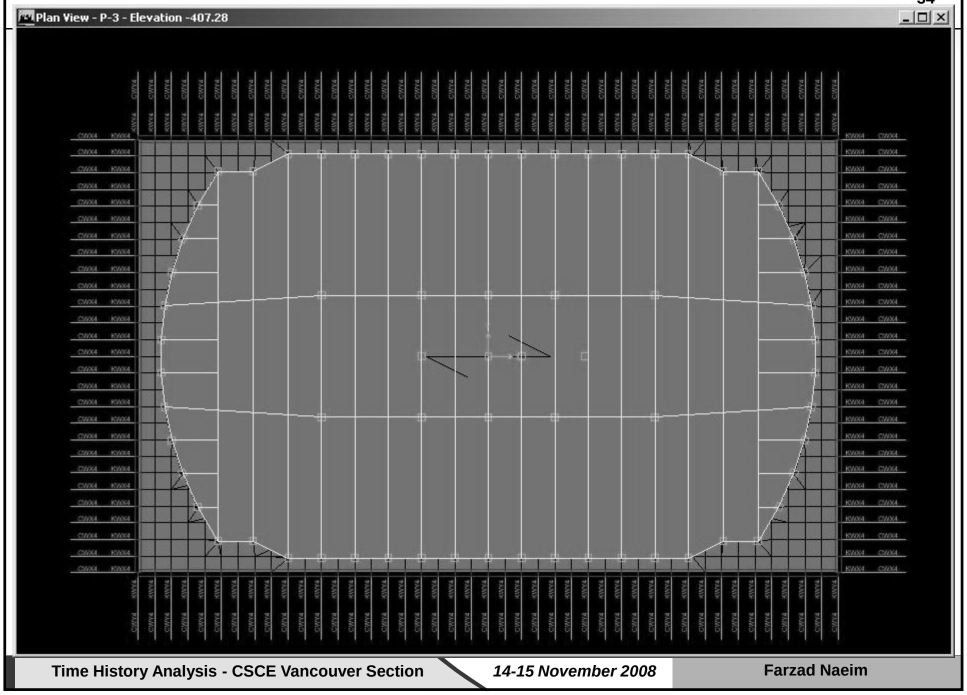




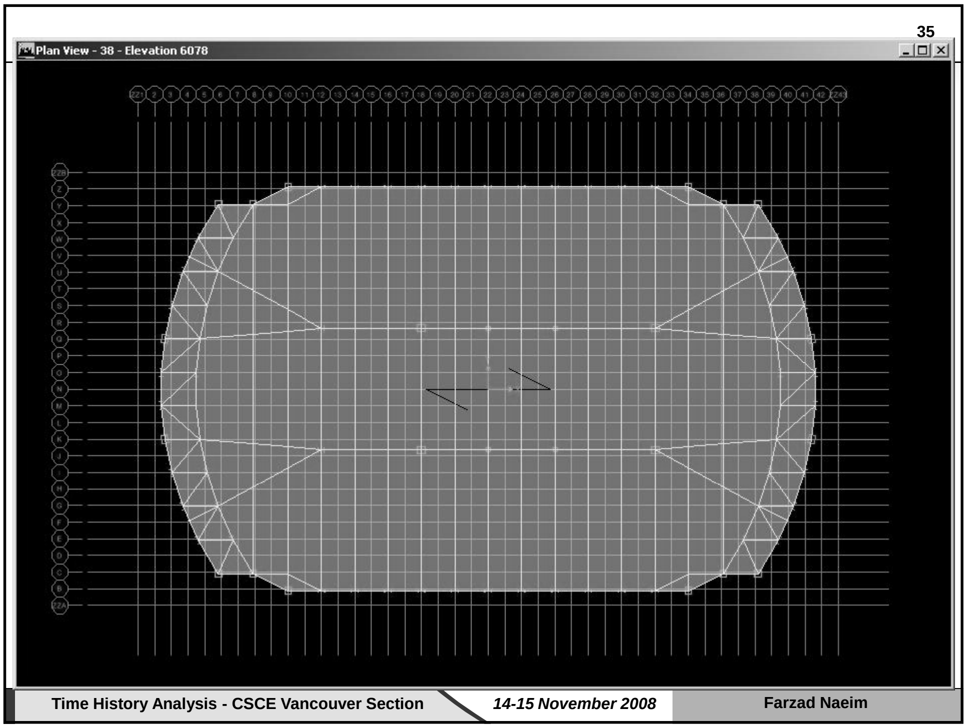

Plan View - TG - Elevation -644.88



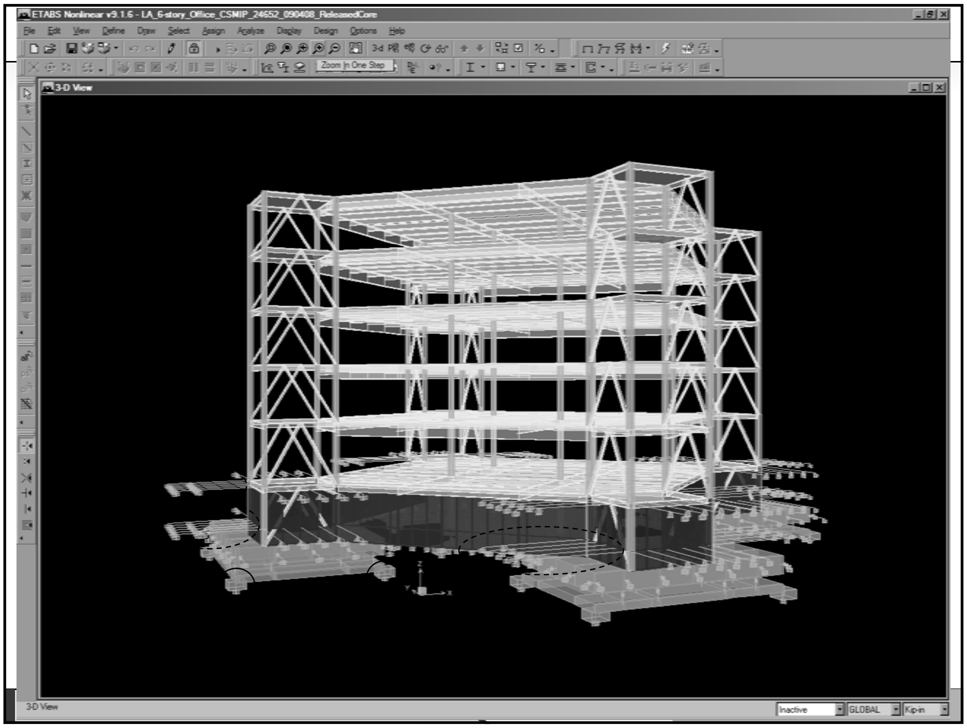
Plan View - P-4 - Elevation -584.88

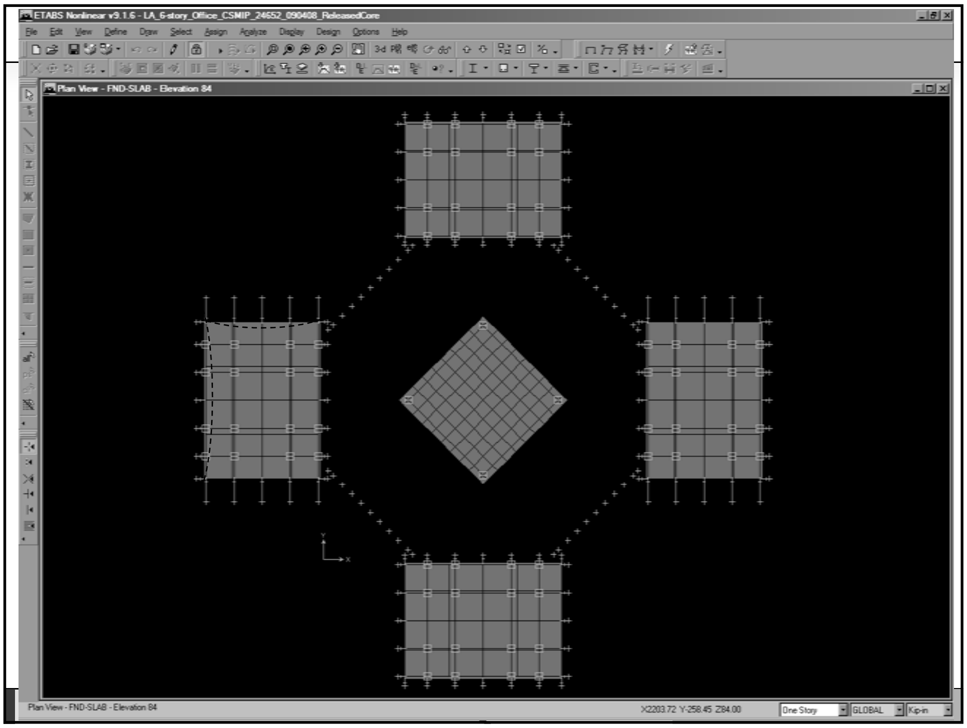


Plan View - P-3 - Elevation -407.28



Time History Analysis - CSCE Vancouver Section | *14-15 November 2008* | Farzad Naeim



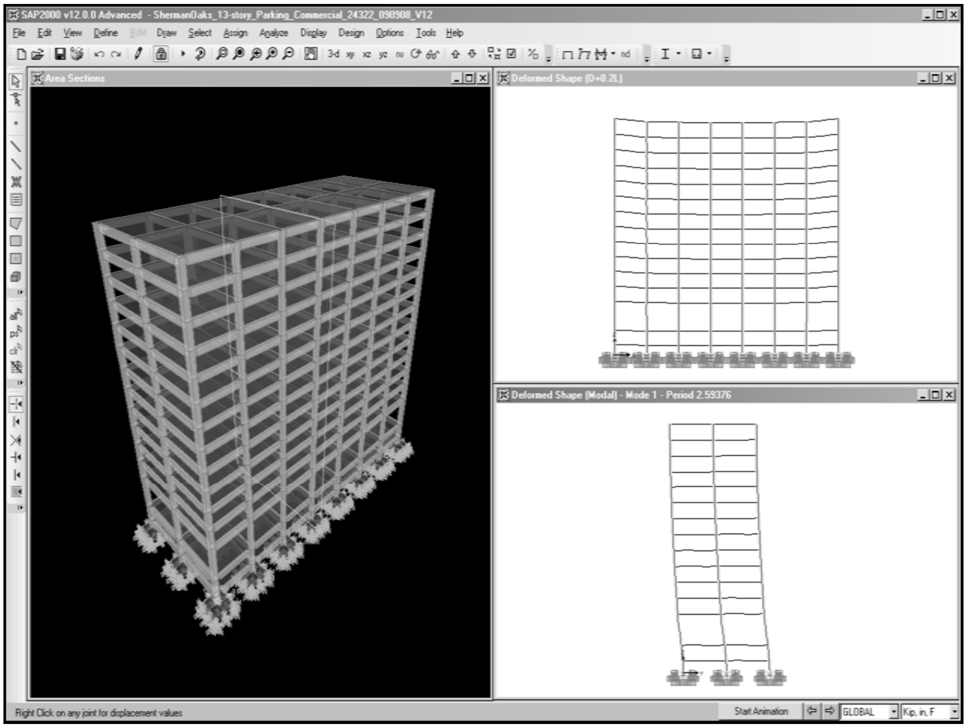


ETABS Nonlinear v9.1.6 - LA_6-story_Office_CSMIP_24652_090408_ReleasedCore

Plan View - FND-SLAB - Elevation 84

Plan View - FND-SLAB - Elevation 84

X2203.72 Y-258.45 Z84.00



SAP2000 v12.0.0 Advanced - ShermanOaks_13-story_Parking_Commercial_24322_090908_V12

Area Sections

Deformed Shape (D+0.2L)

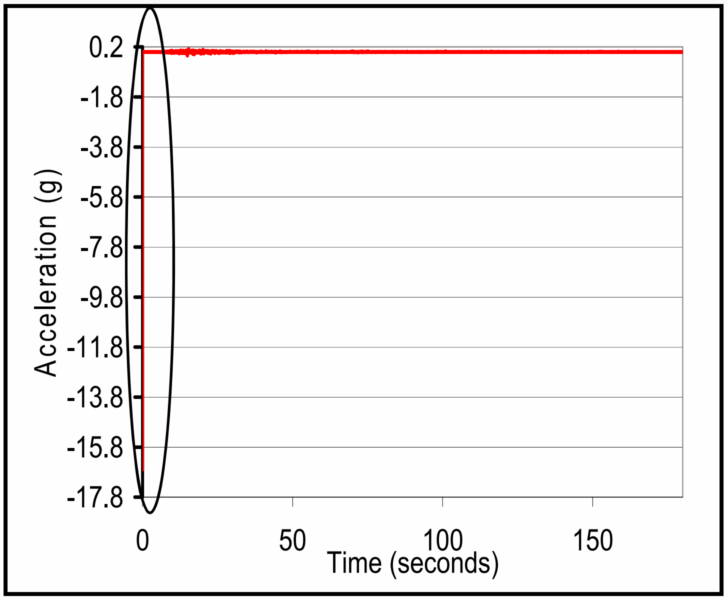
Deformed Shape (Modal) - Mode 1 - Period 2.59376

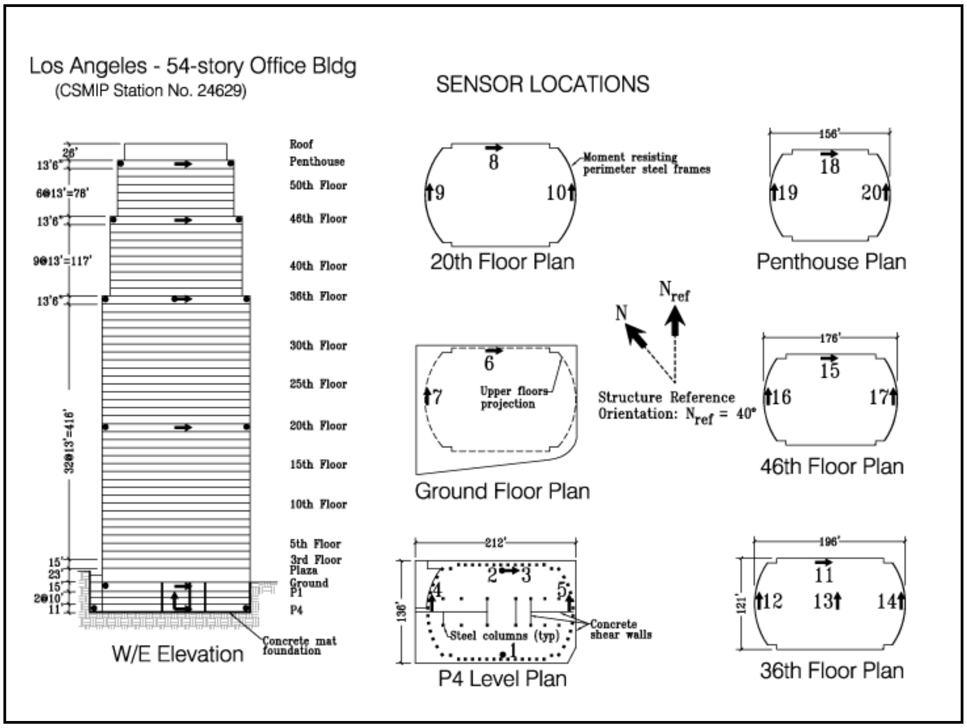
Right Click on any joint for displacement values

## Nonlinear ETABS Model (MA)

- Vertical masses included
- Eigenvalue analysis does not work
- Ritz versus eigenvalue analysis
- 50 Ritz vectors are utilized.
  - The first 12 mode shapes used as Ritz vectors
  - Subbasement deformations used as Ritz vectors
- The gravity load was imposed as a ramp function followed by imposed horizontal and vertical ground displacements
- Damping: 1% critical, except for modes 1 and 4 (1.8%).

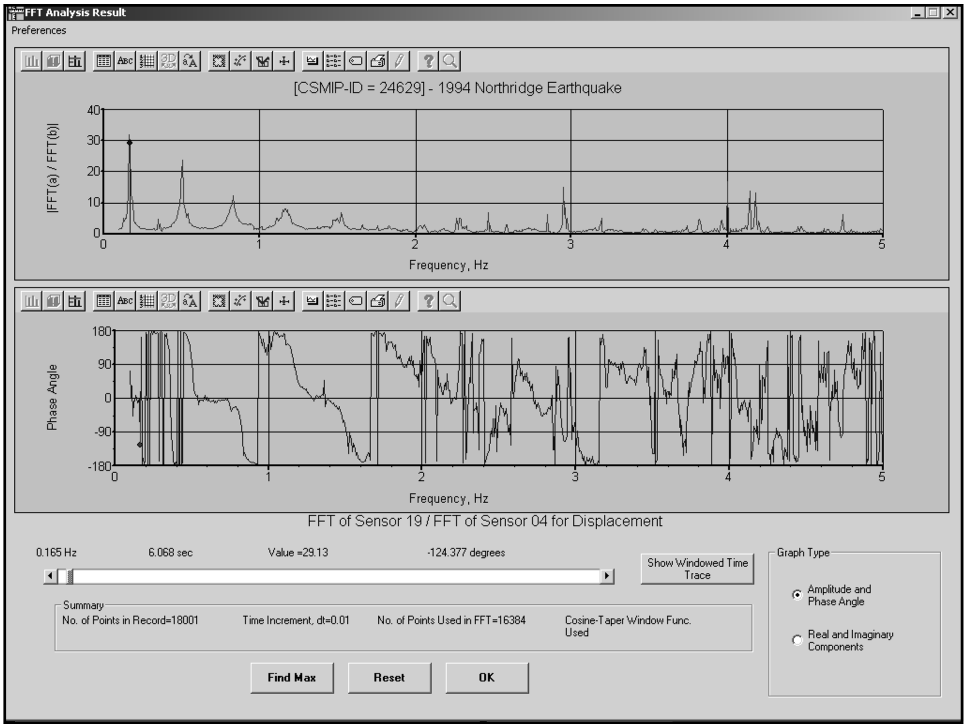
---





Los Angeles - 54-story Office Bldg
(CSMIP Station No. 24629)

SENSOR LOCATIONS



FFT of Sensor 19 / FFT of Sensor 04 for Displacement

0.165 Hz 6.068 sec Value =29.13 -124.377 degrees

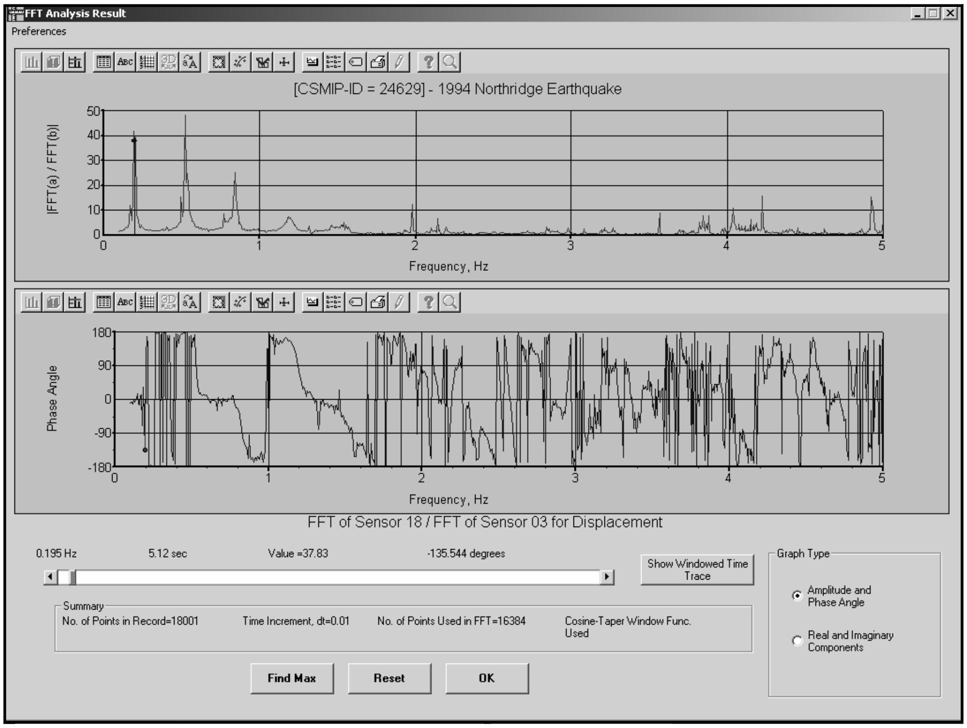
Graph Type

● Amplitude and Phase Angle

○ Real and Imaginary Components

Summary

No. of Points in Record=18001 Time Increment, dt=0.01 No. of Points Used in FFT=16384 Cosine-Taper Window Func. Used



**FFT Analysis Result**

Preferences

[CSMIP-ID = 24629] - 1994 Northridge Earthquake

FFT of Sensor 18 / FFT of Sensor 03 for Displacement

0.195 Hz    5.12 sec    Value =37.83    -135.544 degrees

Graph Type

● Amplitude and Phase Angle

○ Real and Imaginary Components

Summary

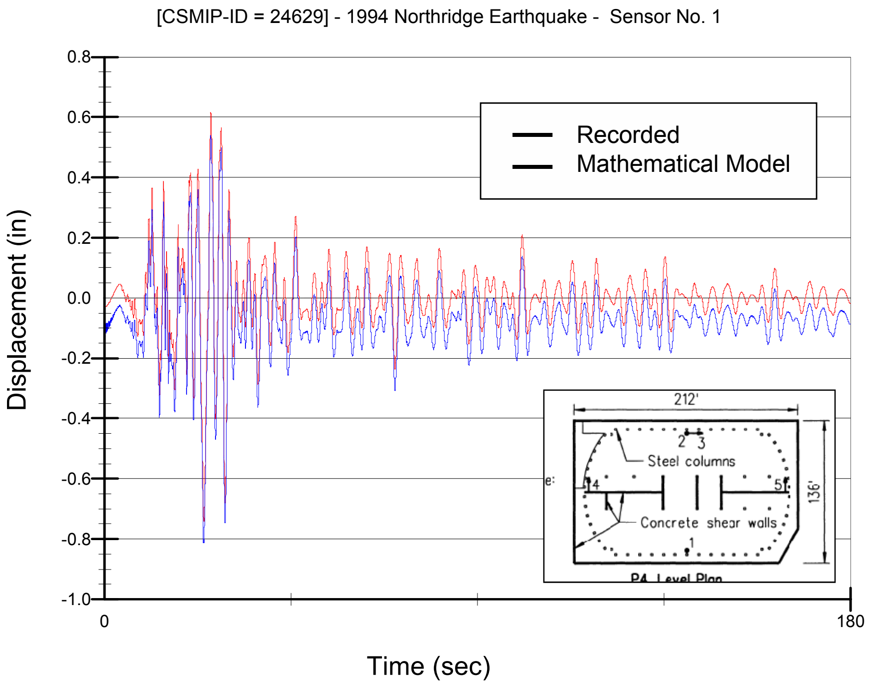
No. of Points in Record=18001    Time Increment, dt=0.01    No. of Points Used in FFT=16384    Cosine-Taper Window Func. Used

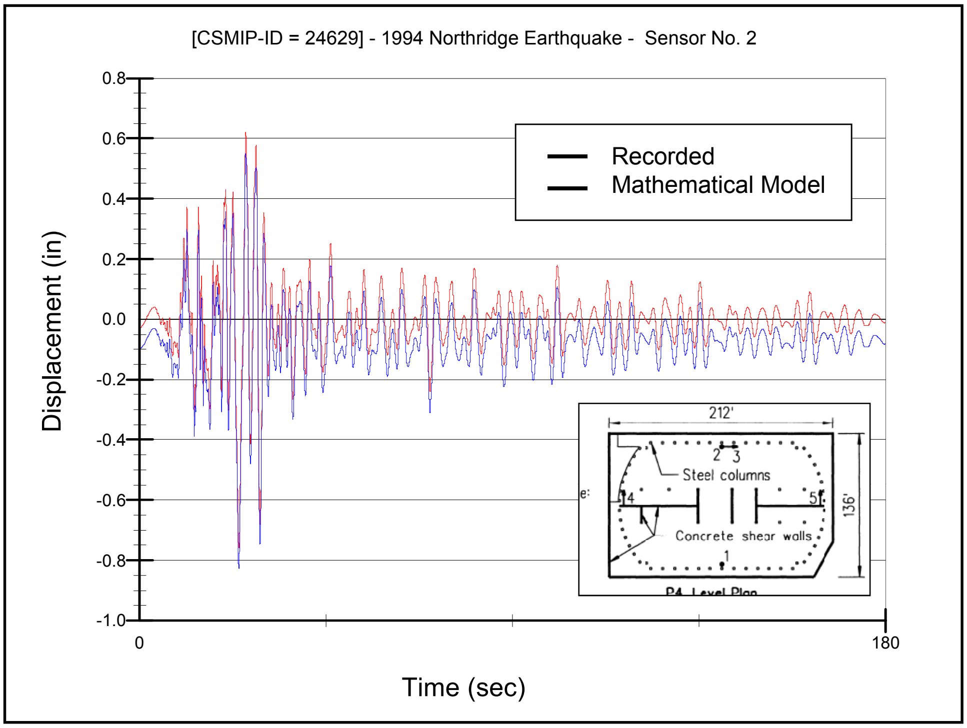
## Comparison with system identification results

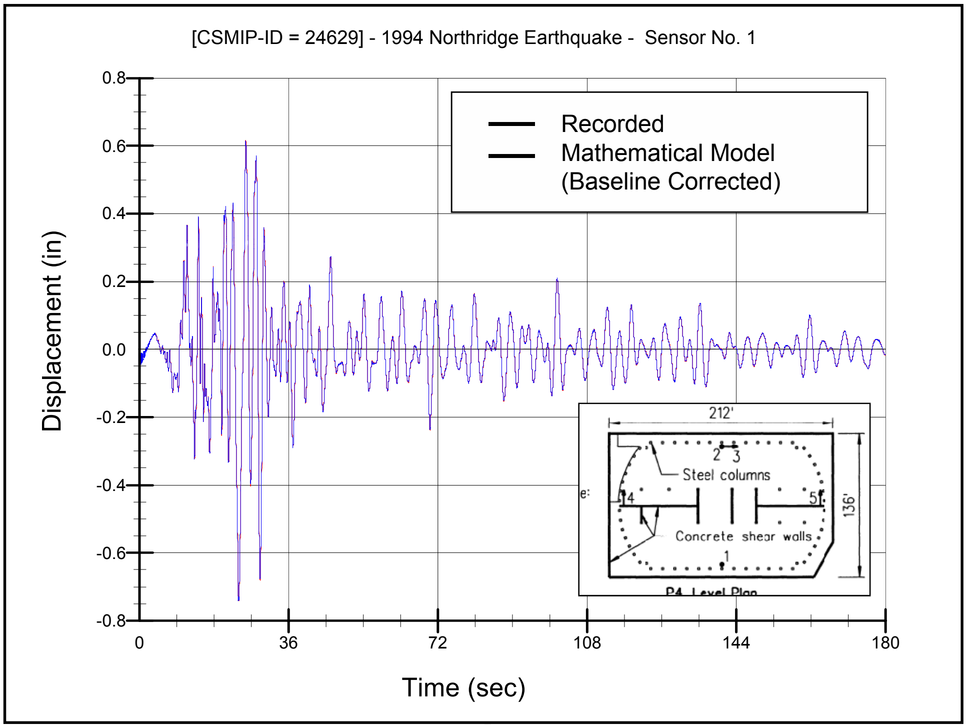
| Direction | Identified Periods (sec.) | | MA Model Periods (sec.) | |
|---|---|---|---|---|
| | Mode 1 | Mode 2 | Mode 1 | Mode 2 |
| E-W | 6.07 | 1.95 | 6.06 | 1.92 |
| N-S | 5.12 | 1.86 | 5.18 | 1.81 |
| Torsional | 2.78 | | 2.76 | |

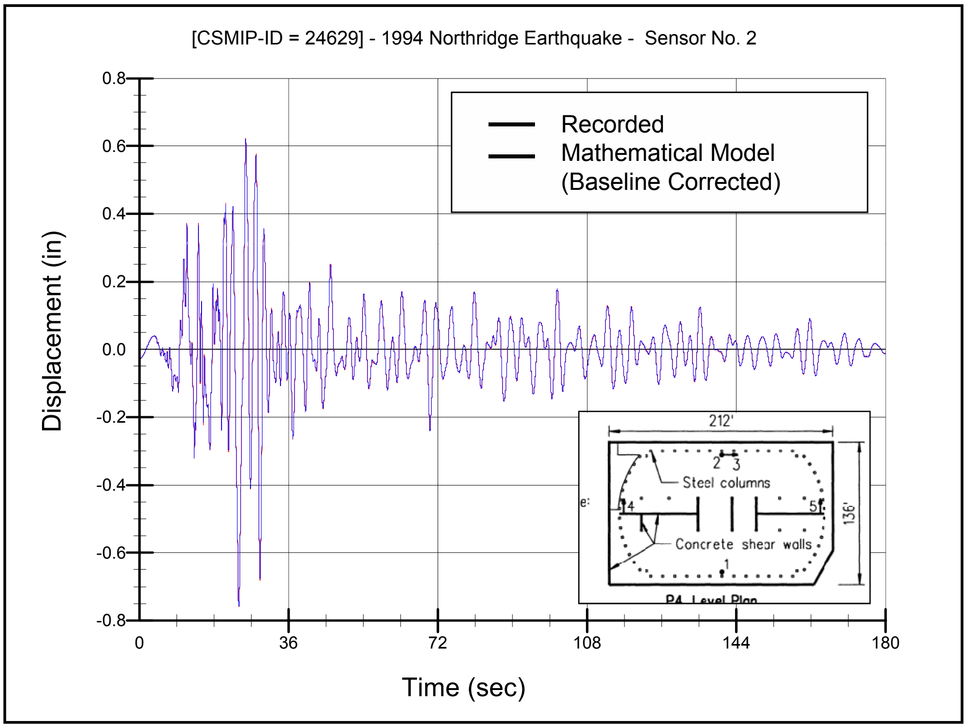
# Period Comparisons

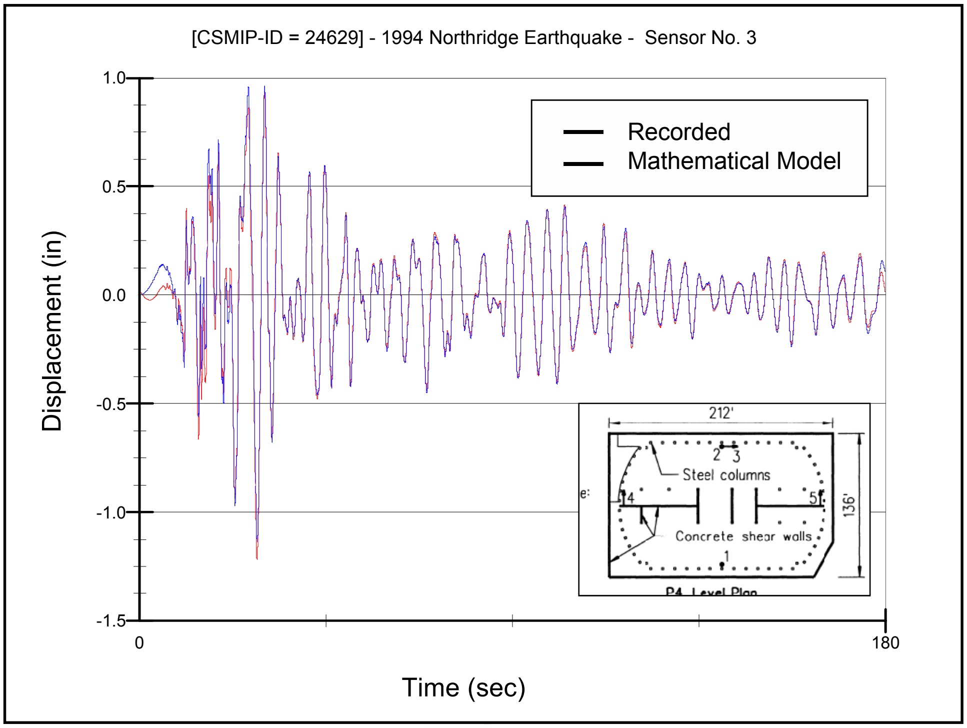
| **Model** | **Reported vibration periods for first five Ritz vectors (sec.)** | | | | |
|---|---|---|---|---|---|
| | **1** | **2** | **3** | **4** | **5** |
| MA* | 6.06 | 5.18 | 2.76 | 1.92 | 1.81 |
| 1 | 6.03 | 5.15 | 2.75 | 1.91 | 1.81 |
| 2A | 6.06 | 5.18 | 2.76 | 1.92 | 1.81 |
| 2B | 6.06 | 5.18 | 2.76 | 1.92 | 1.81 |
| 2C | 6.06 | 5.18 | 2.76 | 1.92 | 1.81 |
| 3A | 6.04 | 5.18 | 2.78 | 1.92 | 1.82 |
| 3B | 5.79 | 4.99 | 2.76 | 1.92 | 1.82 |
| 3C | 5.79 | 4.99 | 2.76 | 1.92 | 1.82 |
| 3D | 5.63 | 4.90 | 2.74 | 1.89 | 1.80 |

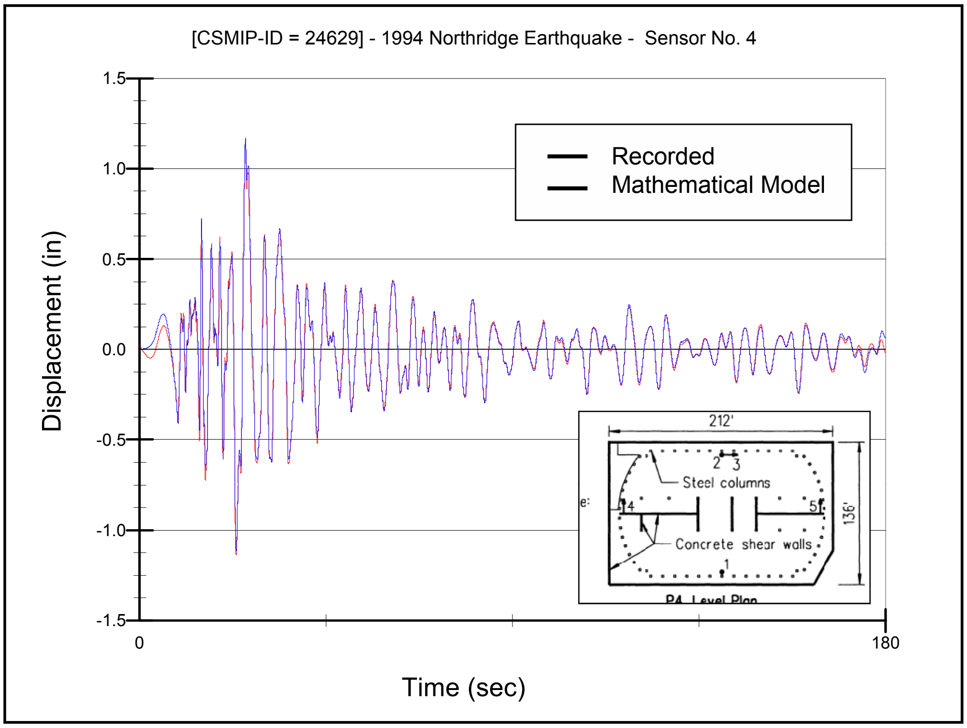


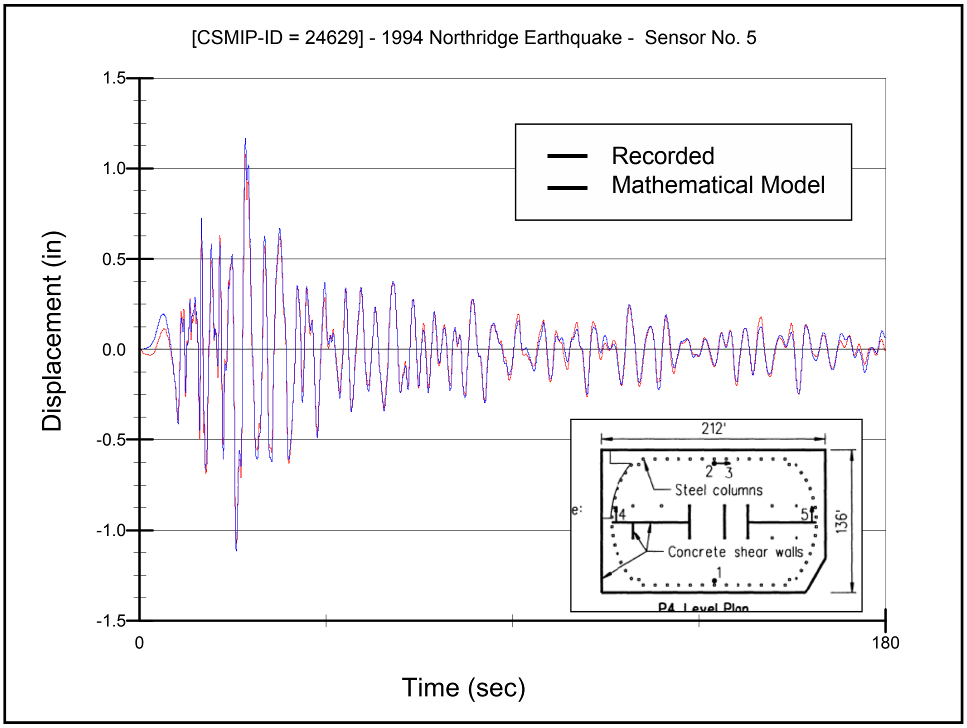

[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 2
Displacement (in)
0.8
0.6
0.4
0.2
0.0
-0.2
-0.4
-0.6
-0.8
-1.0
0
180
Time (sec)
Recorded
Mathematical Model
212'
136'
2
3
Steel columns
4
5
Concrete shear walls
1
P4 Level Plan

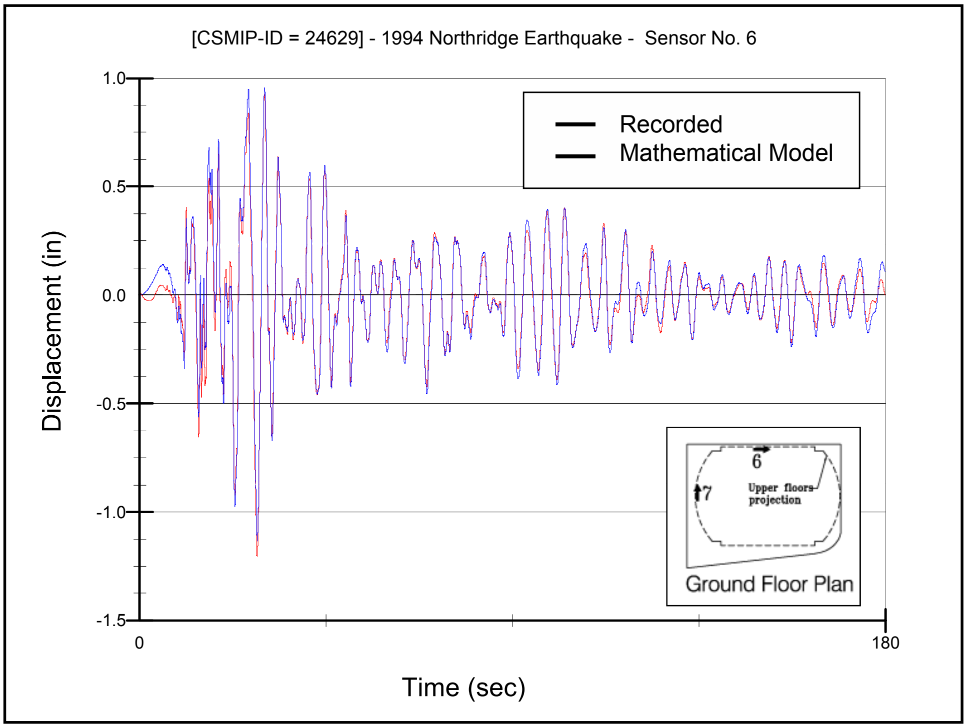

[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 1
Displacement (in)
0.8
0.6
0.4
0.2
0.0
-0.2
-0.4
-0.6
-0.8
0
36
72
108
144
180
Time (sec)
Recorded
Mathematical Model
(Baseline Corrected)
212'
136'
2
3
Steel columns
4
5
Concrete shear walls
1
P4 Level Plan

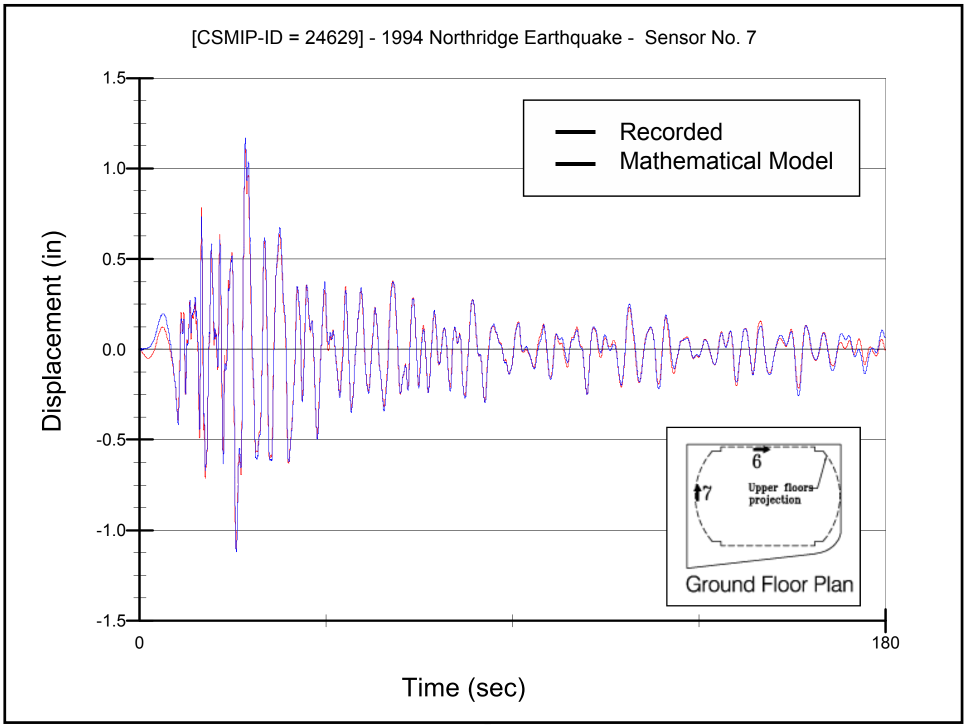

[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 2
Displacement (in)
Time (sec)
0.8
0.6
0.4
0.2
0.0
-0.2
-0.4
-0.6
-0.8
0
36
72
108
144
180
Recorded
Mathematical Model
(Baseline Corrected)
212'
136'
2 3
Steel columns
Concrete shear walls
4
5
1
P4 Level Plan

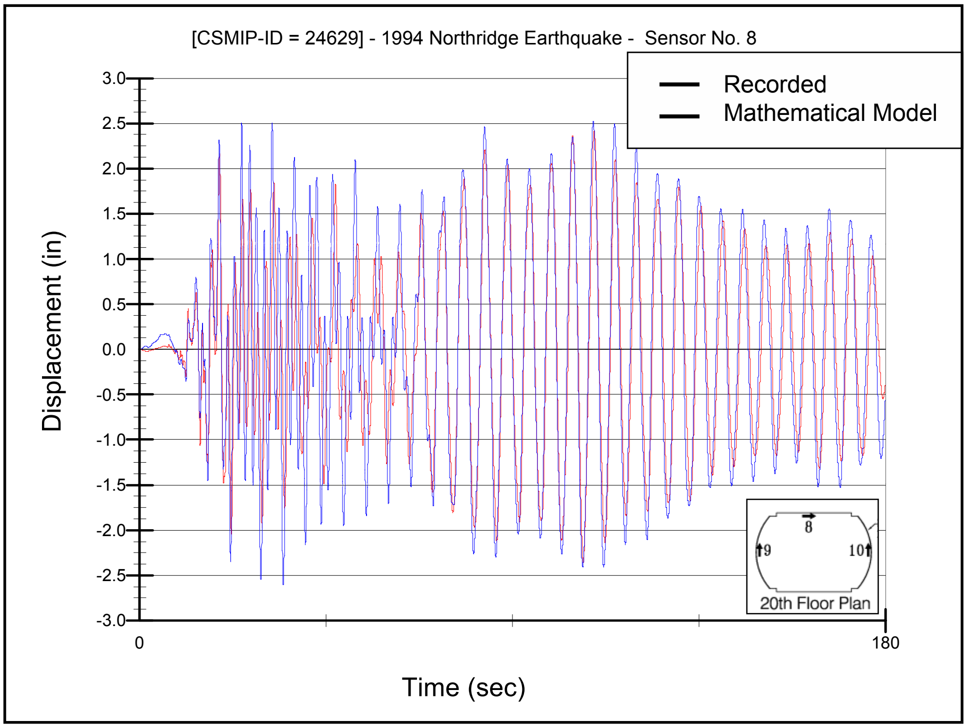

[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 3
Displacement (in)
Time (sec)
1.0
0.5
0.0
-0.5
-1.0
-1.5
0
180
Recorded
Mathematical Model
212'
136'
2 3
Steel columns
Concrete shear walls
4
5
1
P4 Level Plan

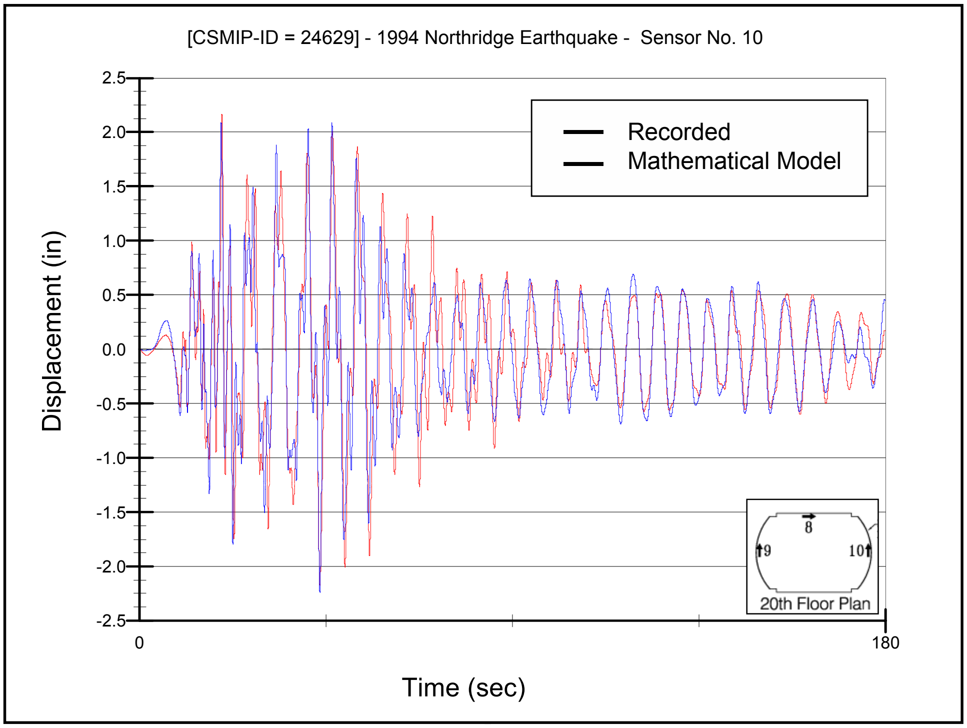

[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 4
Displacement (in)
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
0
180
Time (sec)
Recorded
Mathematical Model
212'
136'
2 3
Steel columns
4
5
Concrete shear walls
1
P4 Level Plan

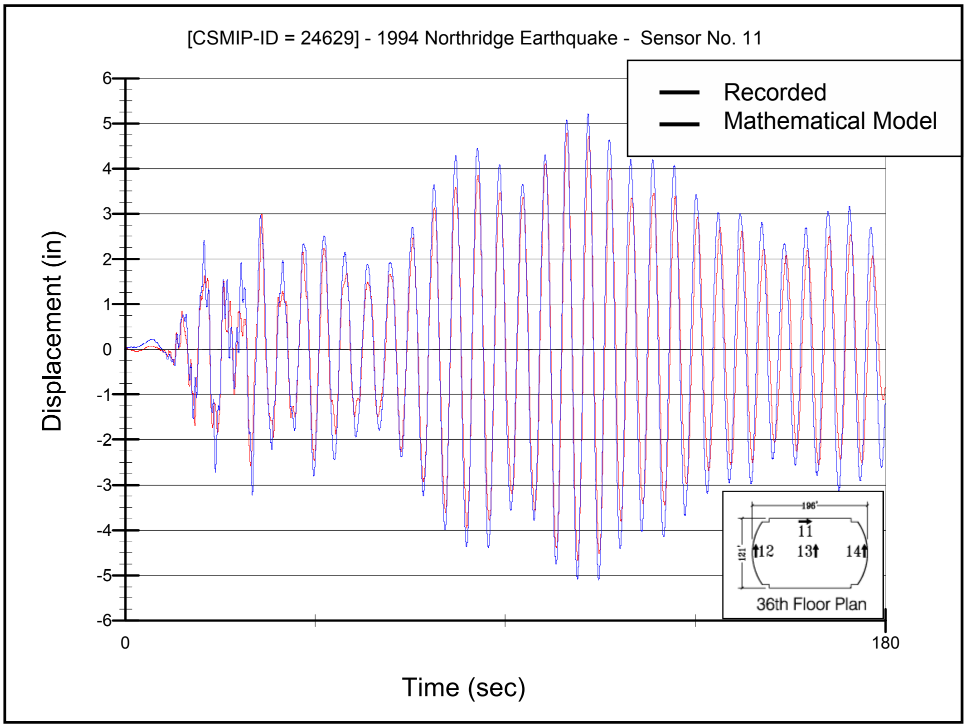

[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 5
Displacement (in)
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
0
180
Time (sec)
Recorded
Mathematical Model
212'
136'
2 3
Steel columns
4
5
Concrete shear walls
1
P4 Level Plan

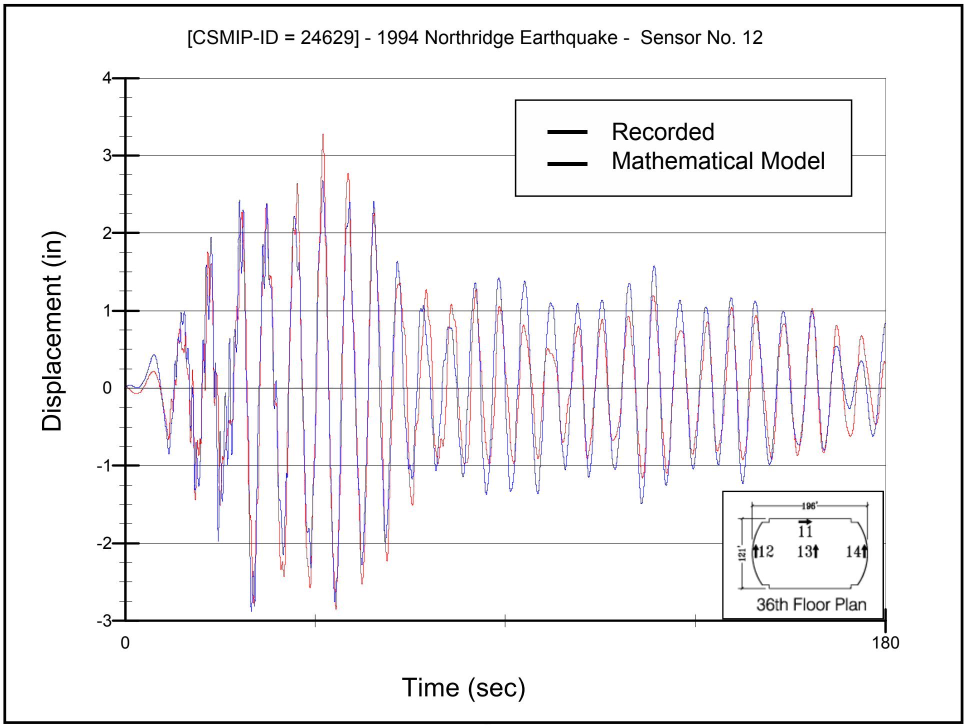

[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 6
Recorded
Mathematical Model
Displacement (in)
1.0
0.5
0.0
-0.5
-1.0
-1.5
0
180
Time (sec)
6
7
Upper floors projection
Ground Floor Plan

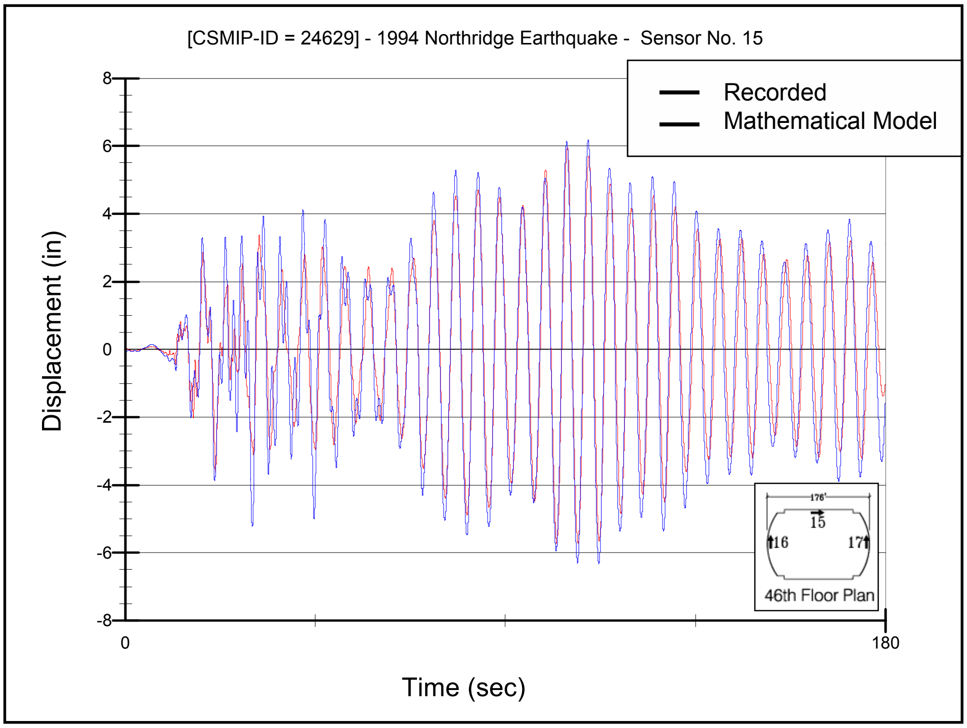

[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 7
Recorded
Mathematical Model
Displacement (in)
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
0
180
Time (sec)
6
7
Upper floors projection
Ground Floor Plan

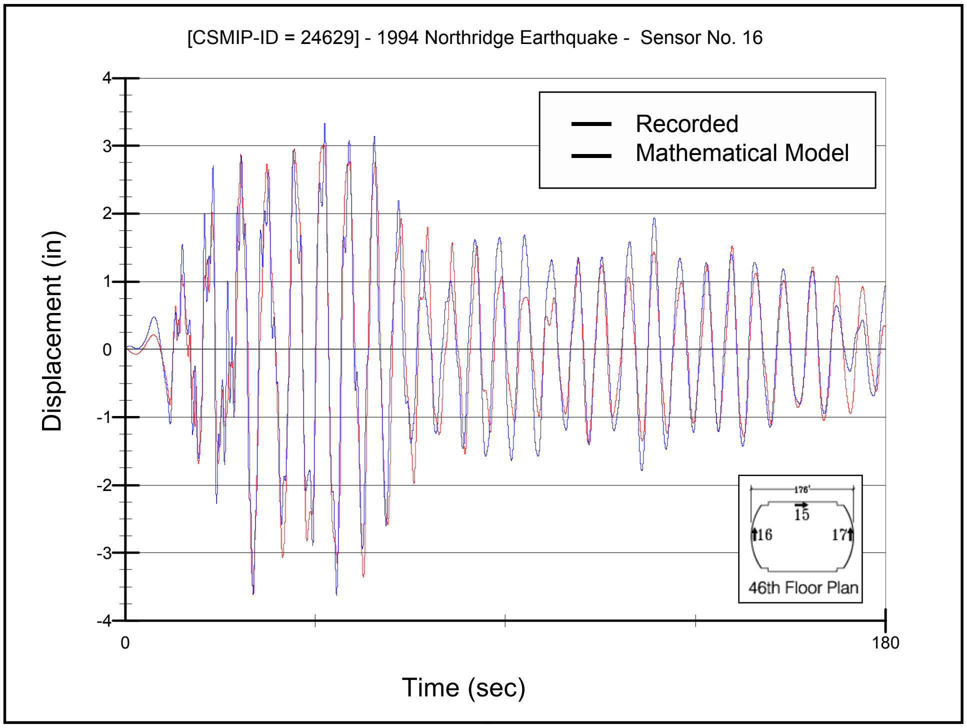

[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 8
Recorded
Mathematical Model
Displacement (in)
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
-2.0
-2.5
-3.0
0
180
Time (sec)
8
9
10
20th Floor Plan

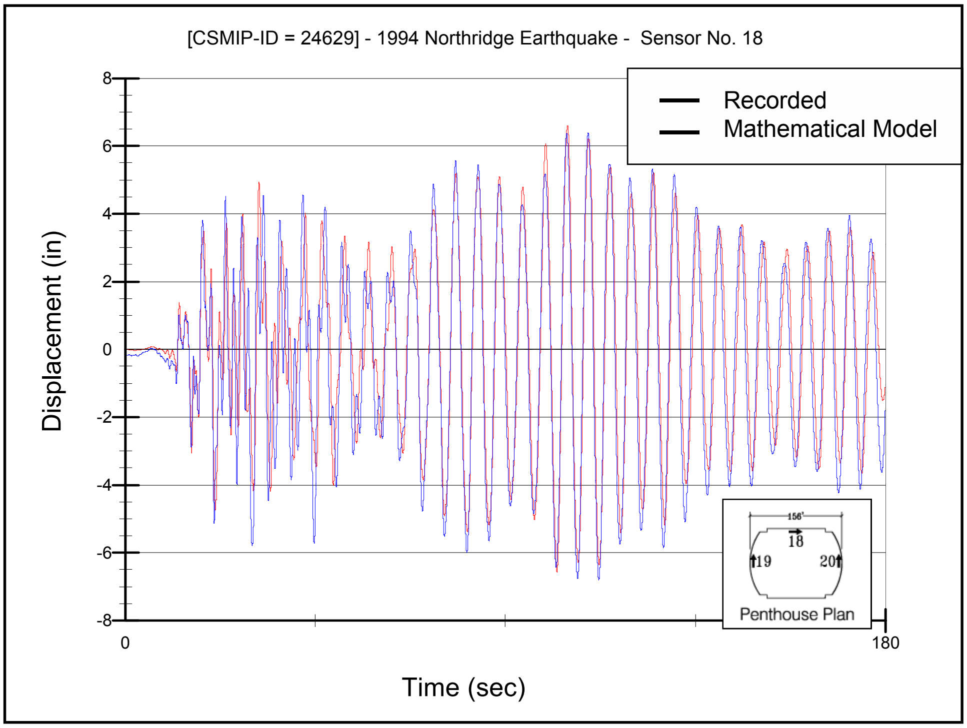

[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 10
Recorded
Mathematical Model
Displacement (in)
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
-2.0
-2.5
0
180
Time (sec)
8
9
10
20th Floor Plan

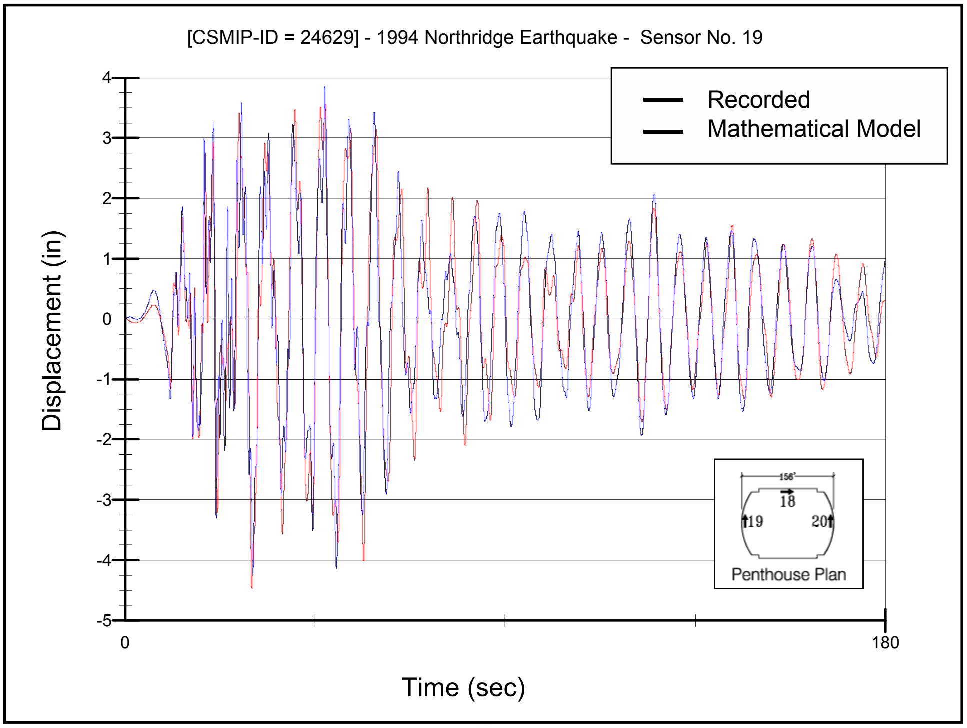

[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 11
Recorded
Mathematical Model
Displacement (in)
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
0
180
Time (sec)
196'
121'
11
12
13
14
36th Floor Plan

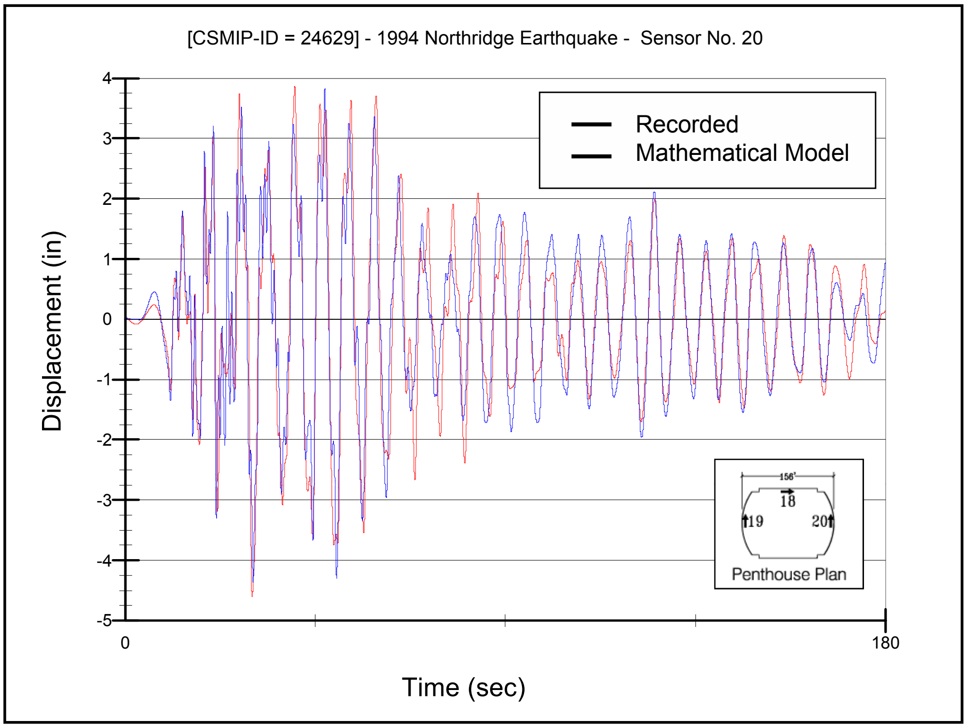

[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 12
Recorded
Mathematical Model
Displacement (in)
4
3
2
1
0
-1
-2
-3
0
180
Time (sec)
196'
121'
11
12
13
14
36th Floor Plan


[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 15
Recorded
Mathematical Model
Displacement (in)
8
6
4
2
0
-2
-4
-6
-8
0
180
Time (sec)
176'
15
16
17
46th Floor Plan


[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 16
Recorded
Mathematical Model
Displacement (in)
4
3
2
1
0
-1
-2
-3
-4
0
180
Time (sec)
176'
15
16
17
46th Floor Plan


[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 18
Displacement (in)
8
6
4
2
0
-2
-4
-6
-8
0
180
Time (sec)
Recorded
Mathematical Model
156'
18
19
20
Penthouse Plan


[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 19
Displacement (in)
4
3
2
1
0
-1
-2
-3
-4
-5
0
180
Time (sec)
Recorded
Mathematical Model
156'
18
19
20
Penthouse Plan

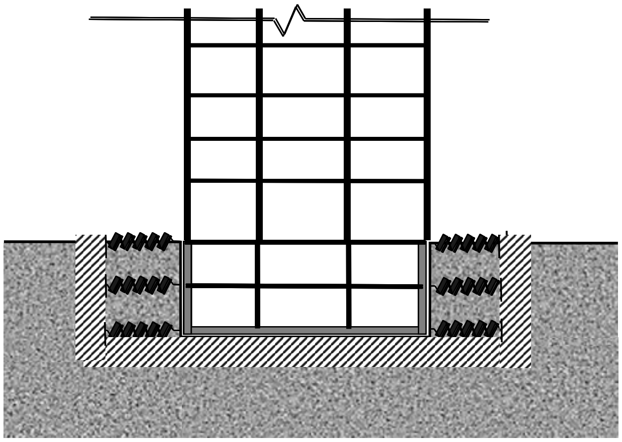


## Period Comparisons

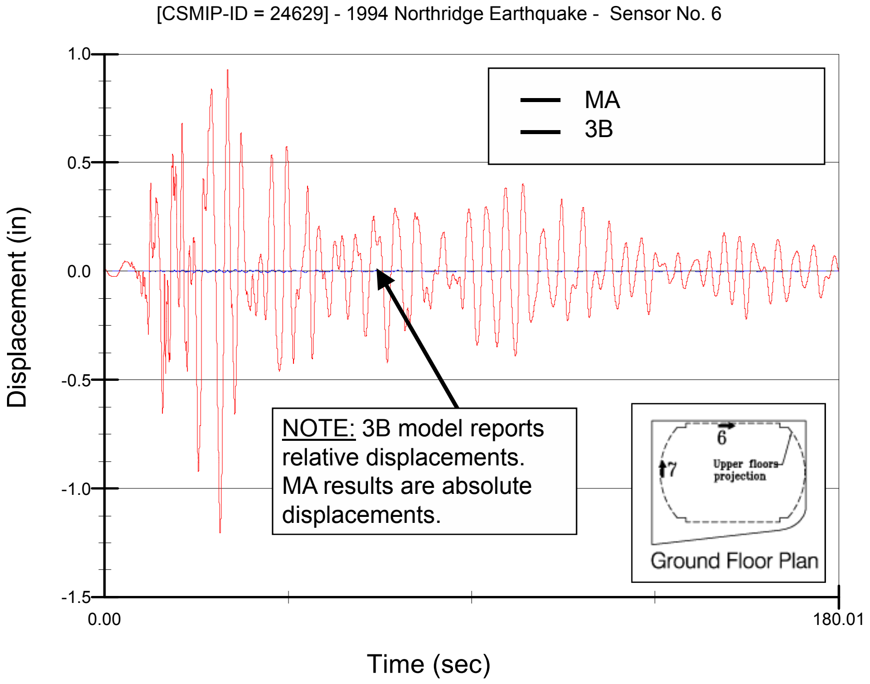
<table>
<tr><th rowspan="2">Model</th><th colspan="5">Reported vibration periods for first five Ritz vectors (sec.)</th></tr>
<tr><th>1</th><th>2</th><th>3</th><th>4</th><th>5</th></tr>
<tr><td>MA*</td><td>6.06</td><td>5.18</td><td>2.76</td><td>1.92</td><td>1.81</td></tr>
<tr><td>1</td><td>6.03</td><td>5.15</td><td>2.75</td><td>1.91</td><td>1.81</td></tr>
<tr><td>2A</td><td>6.06</td><td>5.18</td><td>2.76</td><td>1.92</td><td>1.81</td></tr>
<tr><td>2B</td><td>6.06</td><td>5.18</td><td>2.76</td><td>1.92</td><td>1.81</td></tr>
<tr><td>2C</td><td>6.06</td><td>5.18</td><td>2.76</td><td>1.92</td><td>1.81</td></tr>
<tr><td>3A</td><td>6.04</td><td>5.18</td><td>2.78</td><td>1.92</td><td>1.82</td></tr>
<tr><td>3B</td><td>5.79</td><td>4.99</td><td>2.76</td><td>1.92</td><td>1.82</td></tr>
<tr><td>3C</td><td>5.79</td><td>4.99</td><td>2.76</td><td>1.92</td><td>1.82</td></tr>
<tr><td>3D</td><td>5.63</td><td>4.90</td><td>2.74</td><td>1.89</td><td>1.80</td></tr>
</table>

## Approximation #3b:

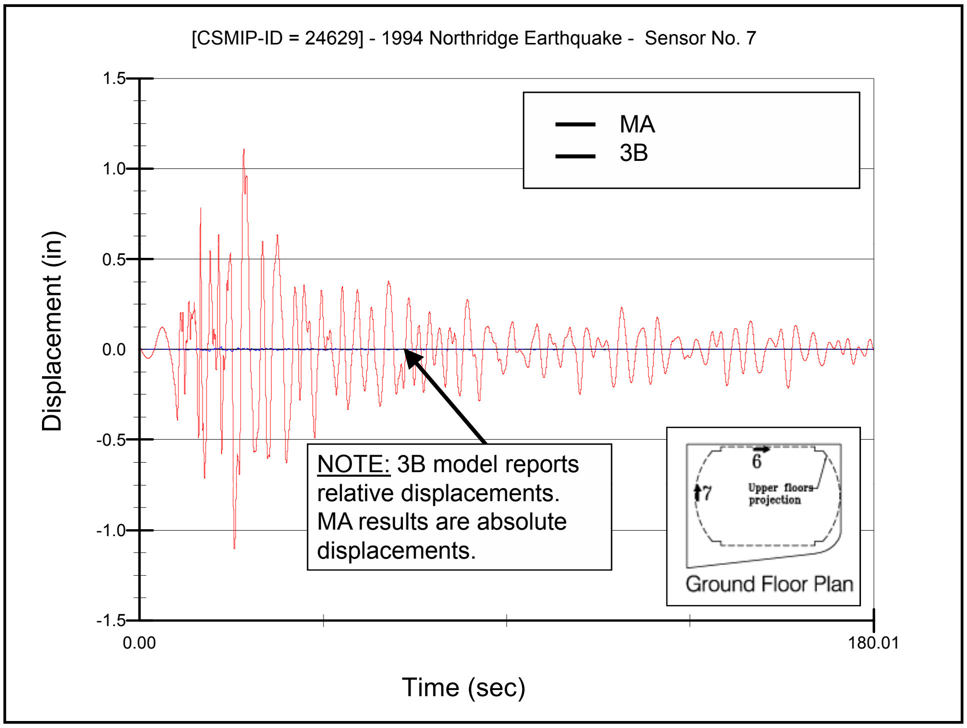
**Rigid soil beneath base slab and basement wall springs (tension allowed) with fixed ends**



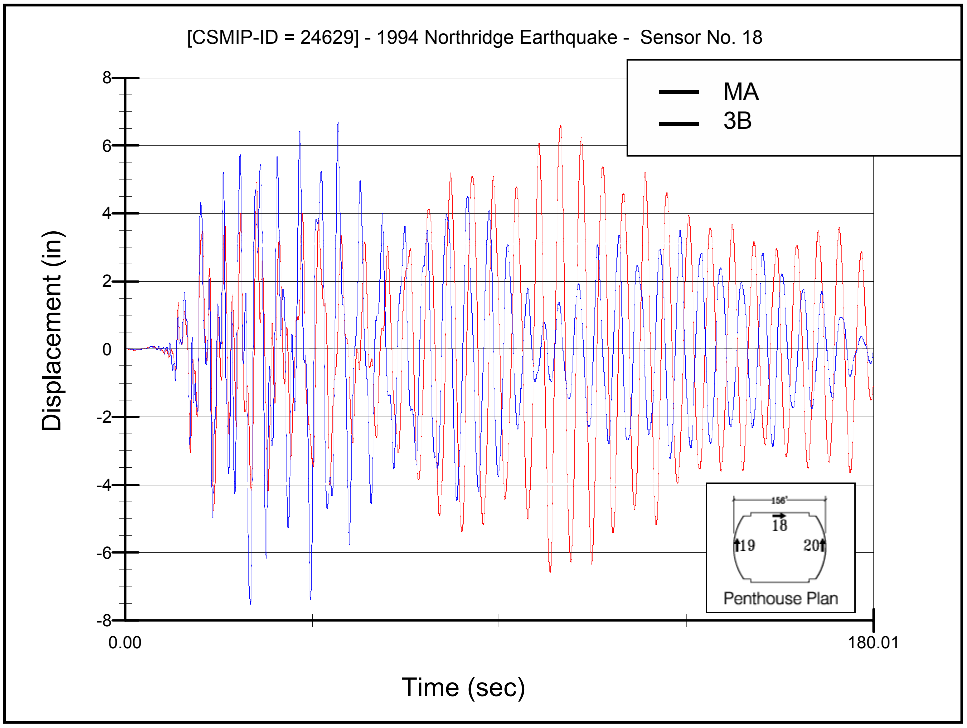
**INPUT MOTIONS: Free-Field Accelerations applied at the base**

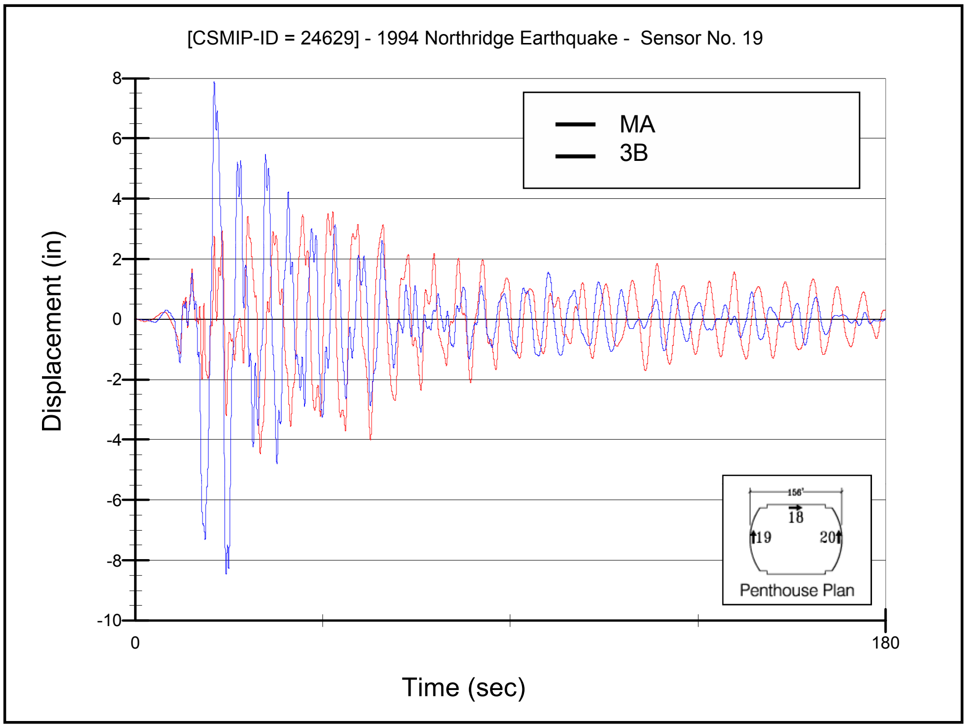

[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 6

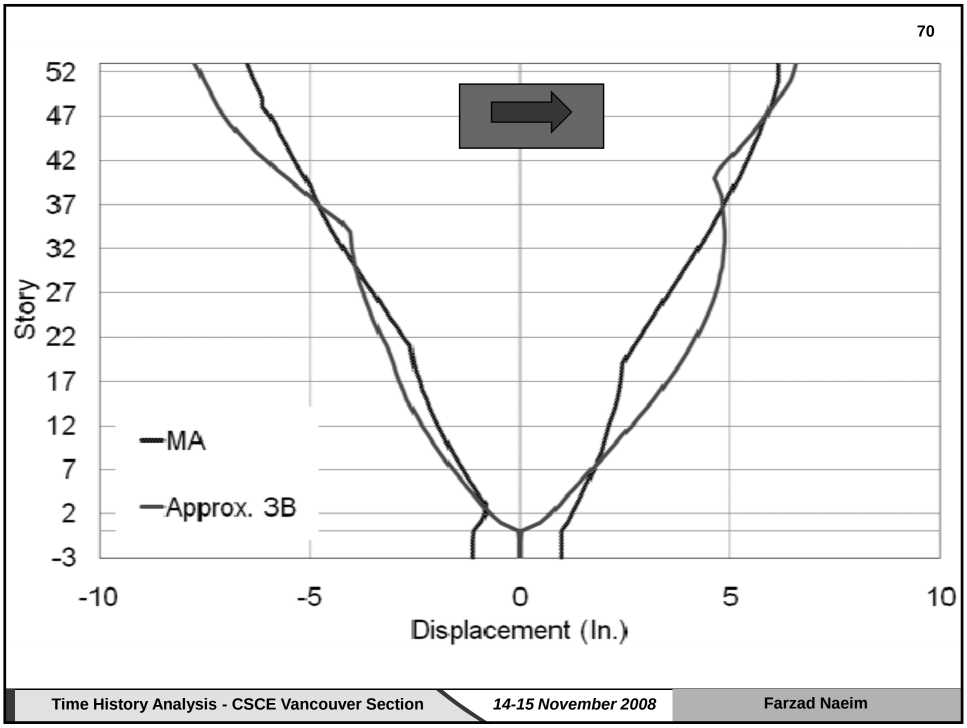
NOTE: 3B model reports relative displacements. MA results are absolute displacements.

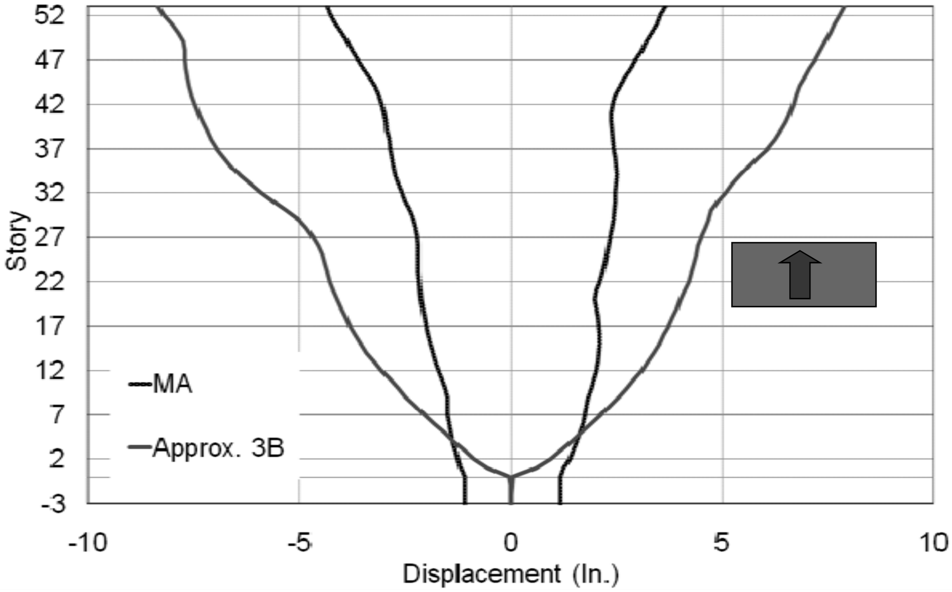


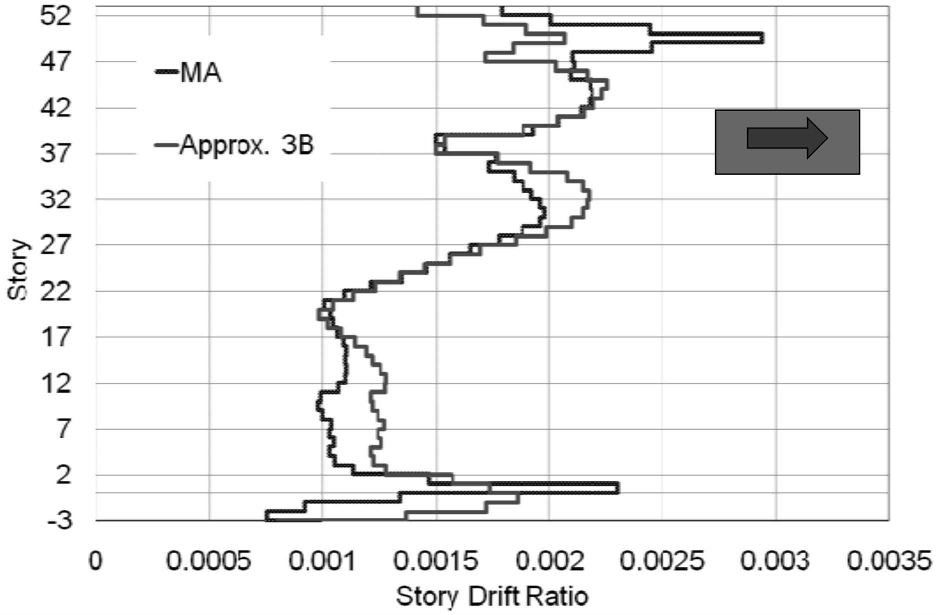
NOTE: 3B model reports relative displacements. MA results are absolute displacements.

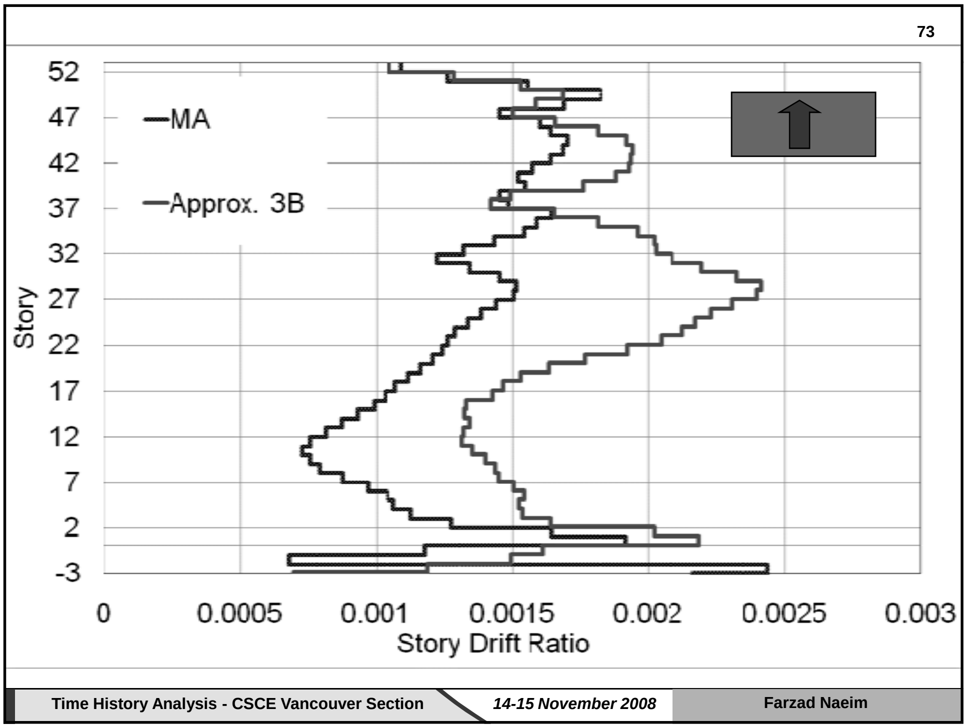


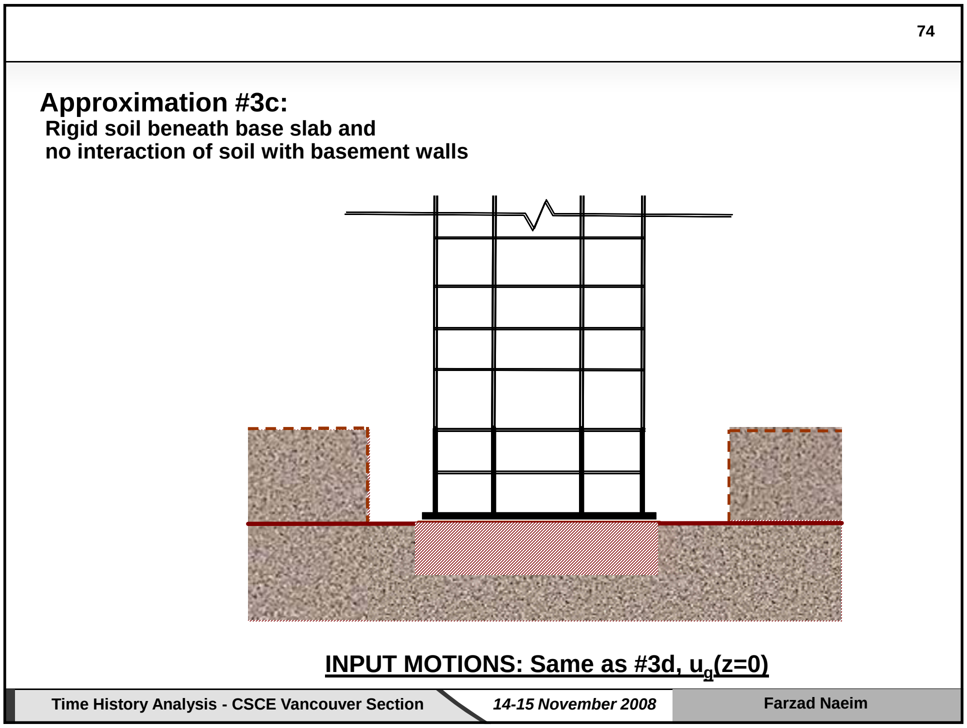

[CSMIP-ID = 24629] - 1994 Northridge Earthquake - Sensor No. 19
Displacement (in)
8
6
4
2
0
-2
-4
-6
-8
-10
0
180
Time (sec)
MA
3B
156'
18
19
20
Penthouse Plan


70
Story
52
47
42
37
32
27
22
17
12
7
2
-3
-10
-5
0
5
10
Displacement (In.)
MA
Approx. 3B
Time History Analysis - CSCE Vancouver Section
14-15 November 2008
Farzad Naeim


52
47
42
37
32
27
22
17
12
7
2
-3
Story
-10
-5
0
5
10
Displacement (In.)
MA
Approx. 3B


52
47
42
37
32
27
22
17
12
7
2
-3
Story
0
0.0005
0.001
0.0015
0.002
0.0025
0.003
0.0035
Story Drift Ratio
MA
Approx. 3B

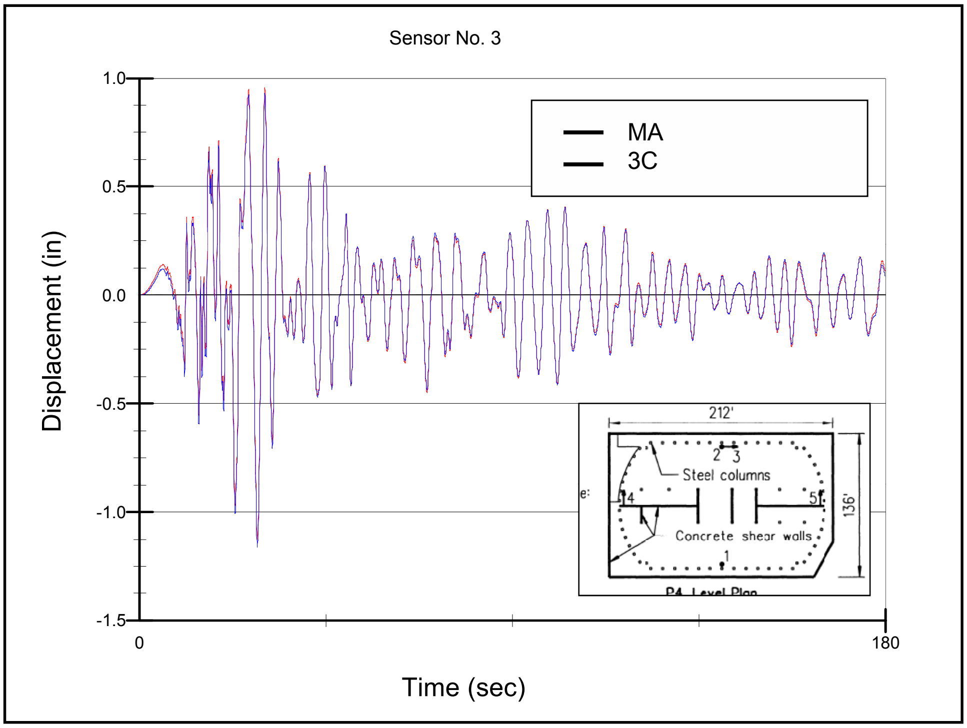


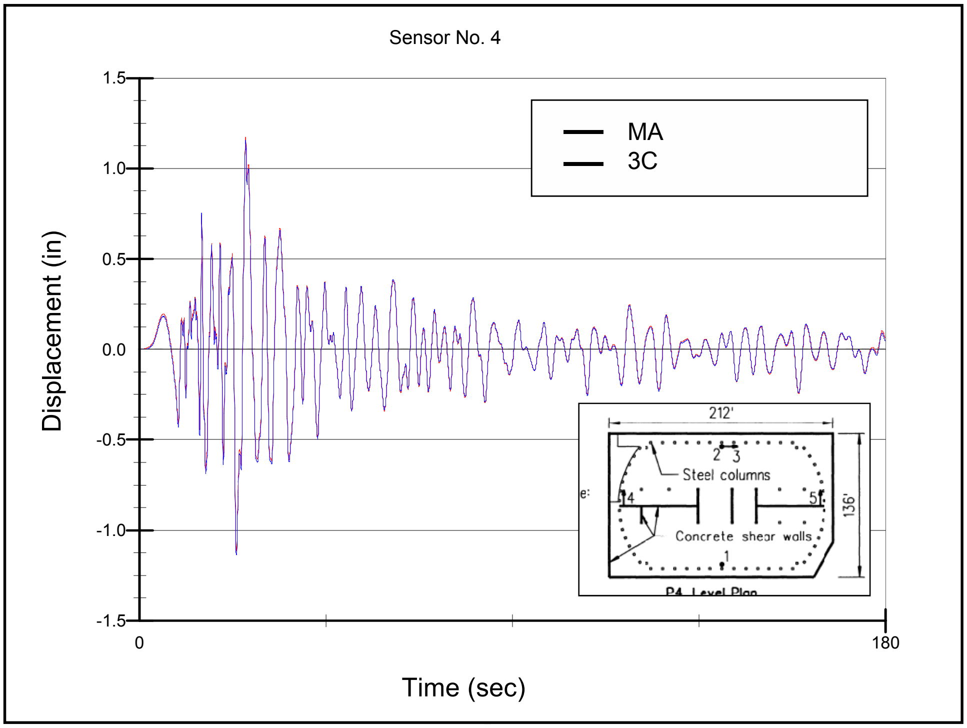


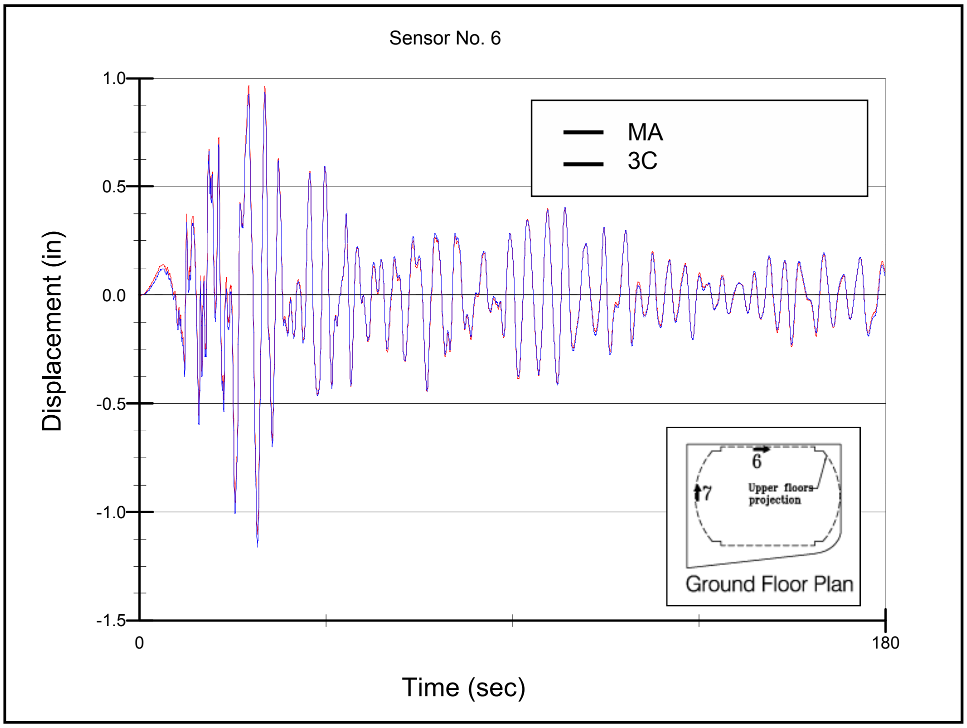
### Approximation #3c:

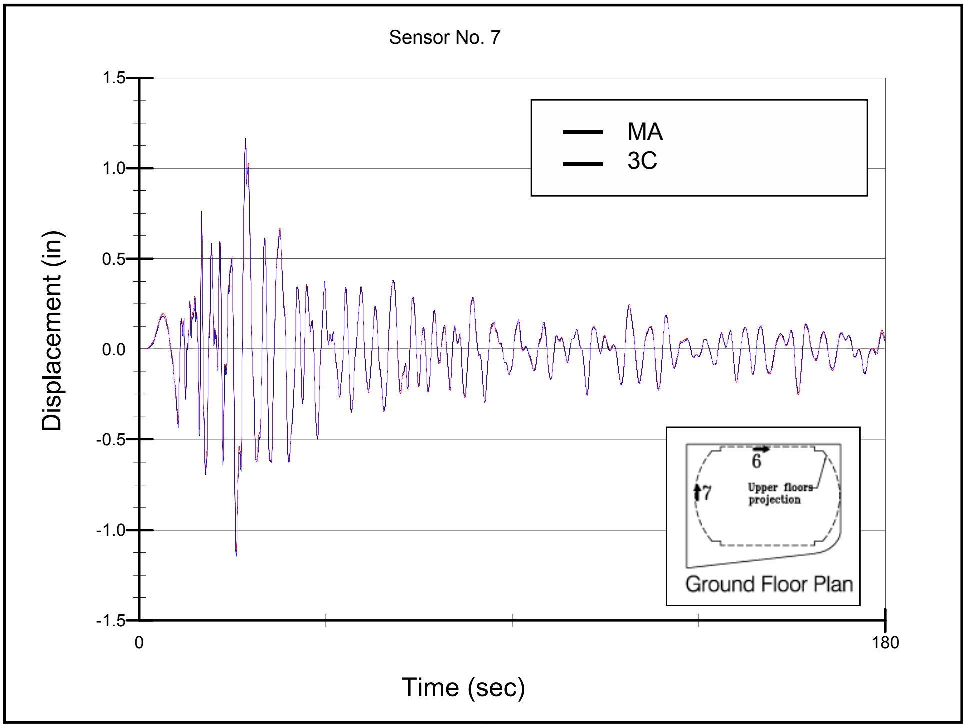
**Rigid soil beneath base slab and no interaction of soil with basement walls**

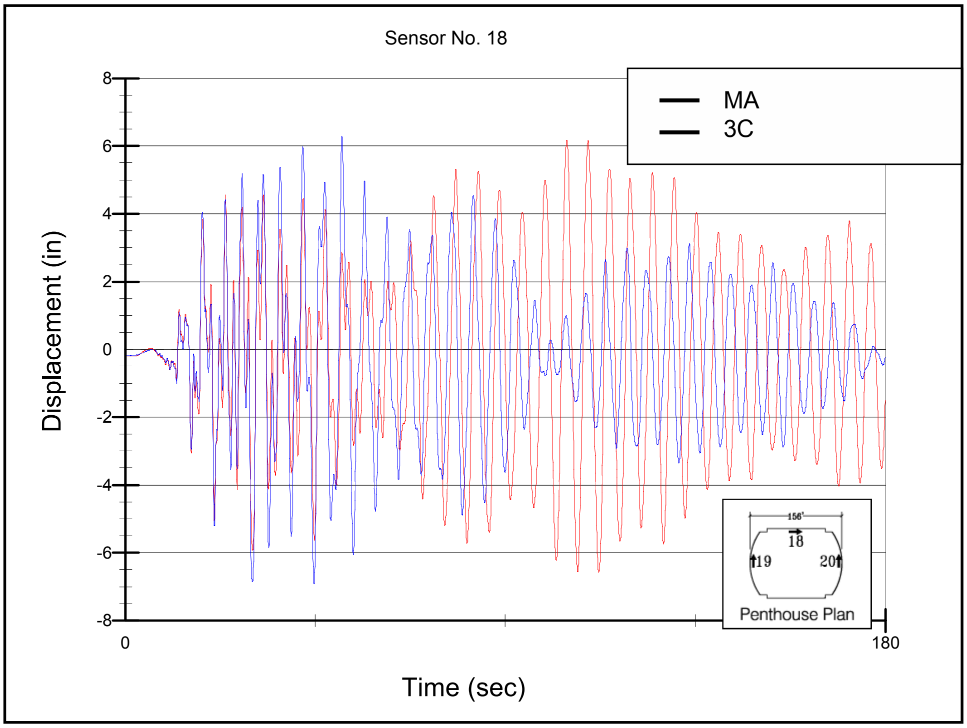
**INPUT MOTIONS: Same as #3d, $u_g(z=0)$**

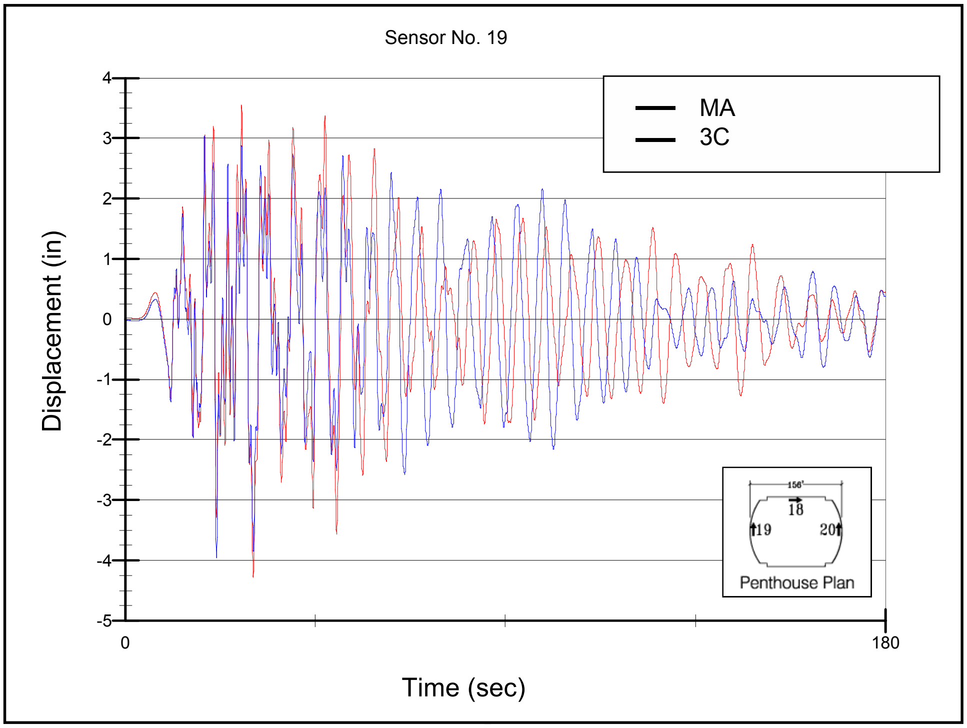

Sensor No. 3
1.0
0.5
0.0
-0.5
-1.0
-1.5
Displacement (in)
0
180
Time (sec)
MA
3C
212'
136'
2
3
Steel columns
4
5
Concrete shear walls
1
P4 Level Plan

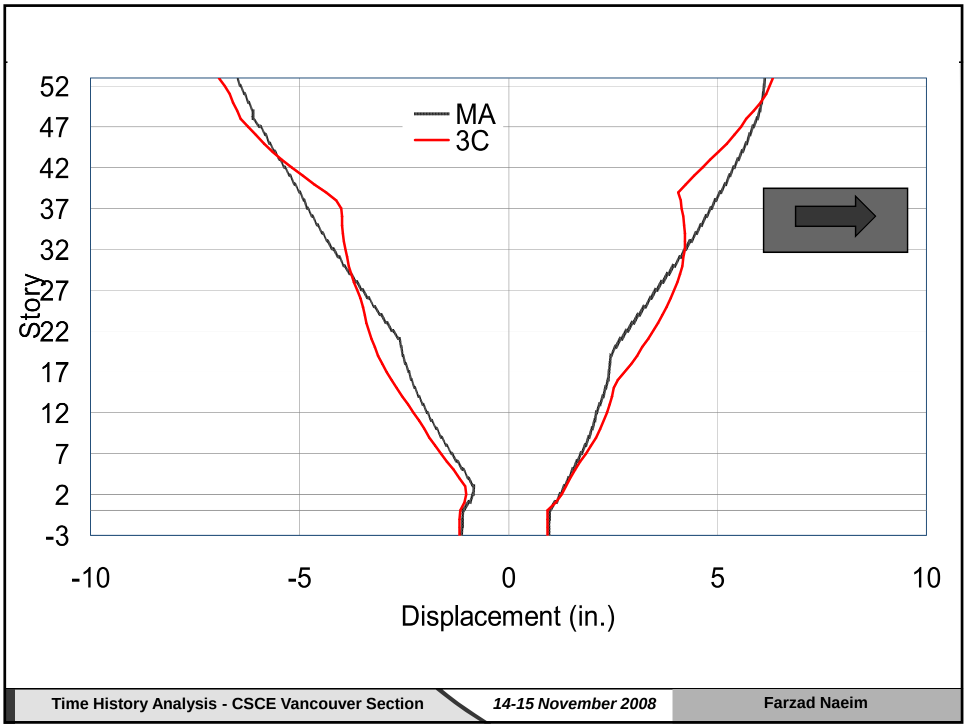

Sensor No. 4
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
Displacement (in)
0
180
Time (sec)
MA
3C
212'
136'
2
3
Steel columns
4
5
Concrete shear walls
1
P4 Level Plan

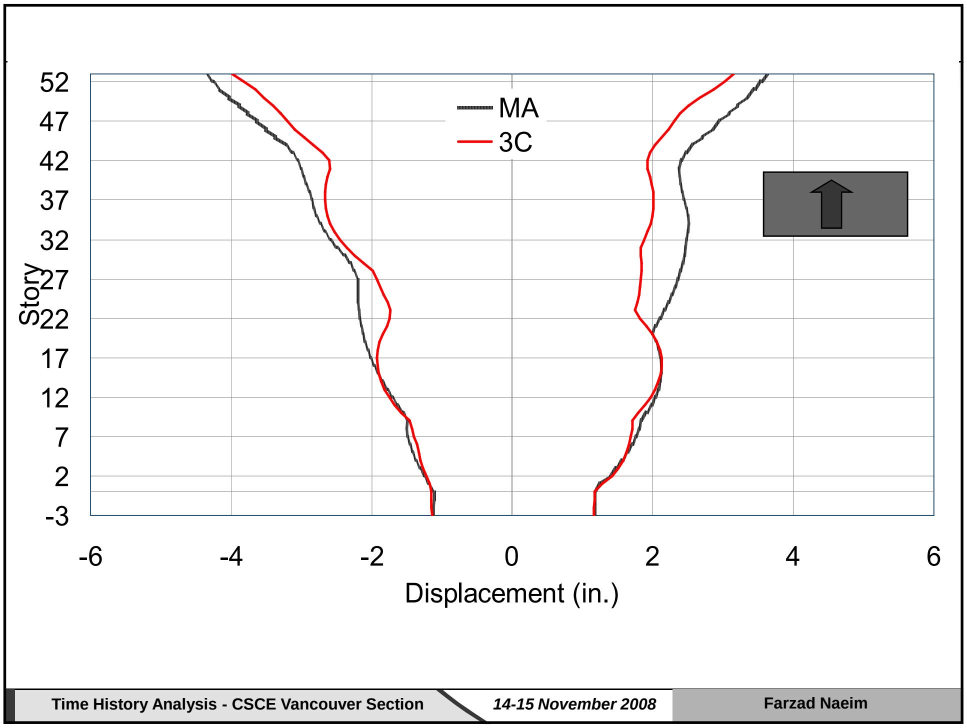

Sensor No. 6
Displacement (in)
1.0
0.5
0.0
-0.5
-1.0
-1.5
0
180
Time (sec)
MA
3C
6
7
Upper floors projection
Ground Floor Plan

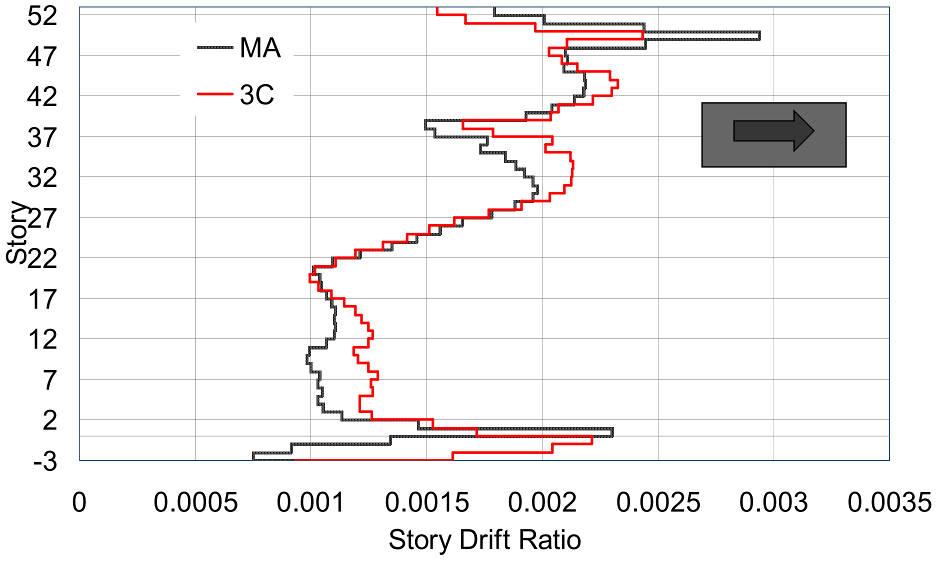

Sensor No. 7
Displacement (in)
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
0
180
Time (sec)
MA
3C
6
7
Upper floors projection
Ground Floor Plan

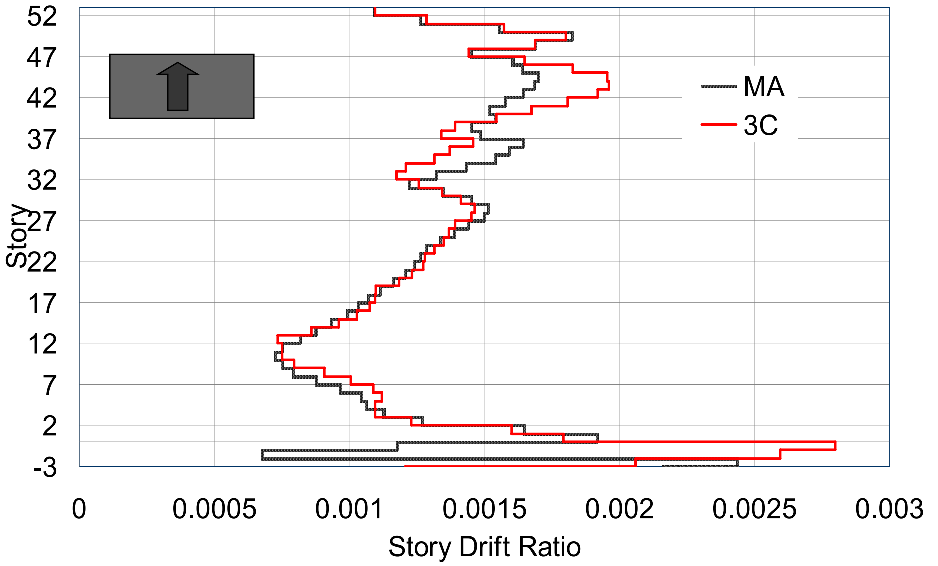

Sensor No. 18
MA
3C
Displacement (in)
8
6
4
2
0
-2
-4
-6
-8
0
180
Time (sec)
156'
18
19
20
Penthouse Plan


Sensor No. 19
MA
3C
Displacement (in)
4
3
2
1
0
-1
-2
-3
-4
-5
0
180
Time (sec)
156'
18
19
20
Penthouse Plan


52
47
42
37
32
27
22
17
12
7
2
-3
Story
MA
3C
-10
-5
0
5
10
Displacement (in.)

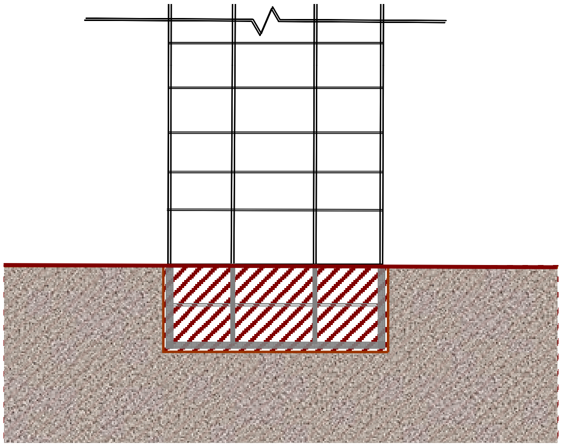

52
47
42
37
32
27
22
17
12
7
2
-3
Story
MA
3C
-6
-4
-2
0
2
4
6
Displacement (in.)


52
47
42
37
32
27
22
17
12
7
2
-3
Story
MA
3C
0
0.0005
0.001
0.0015
0.002
0.0025
0.003
0.0035
Story Drift Ratio


52
47
42
37
32
27
22
17
12
7
2
-3
Story
MA
3C
0
0.0005
0.001
0.0015
0.002
0.0025
0.003
Story Drift Ratio

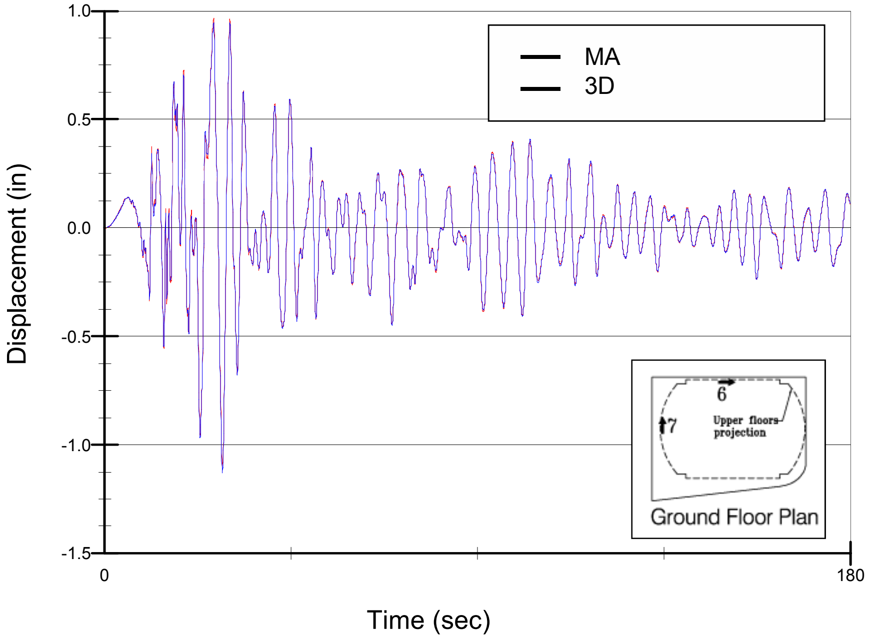
## Approximation #3d:

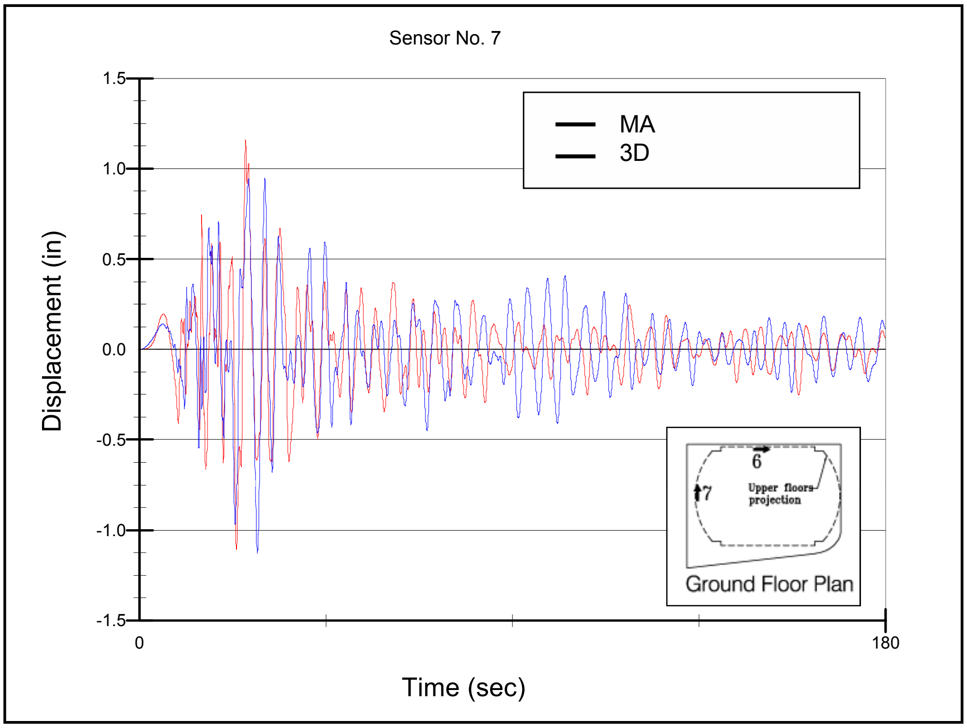
**Embedded portion of structure neglected and fixed base assumed at ground level**

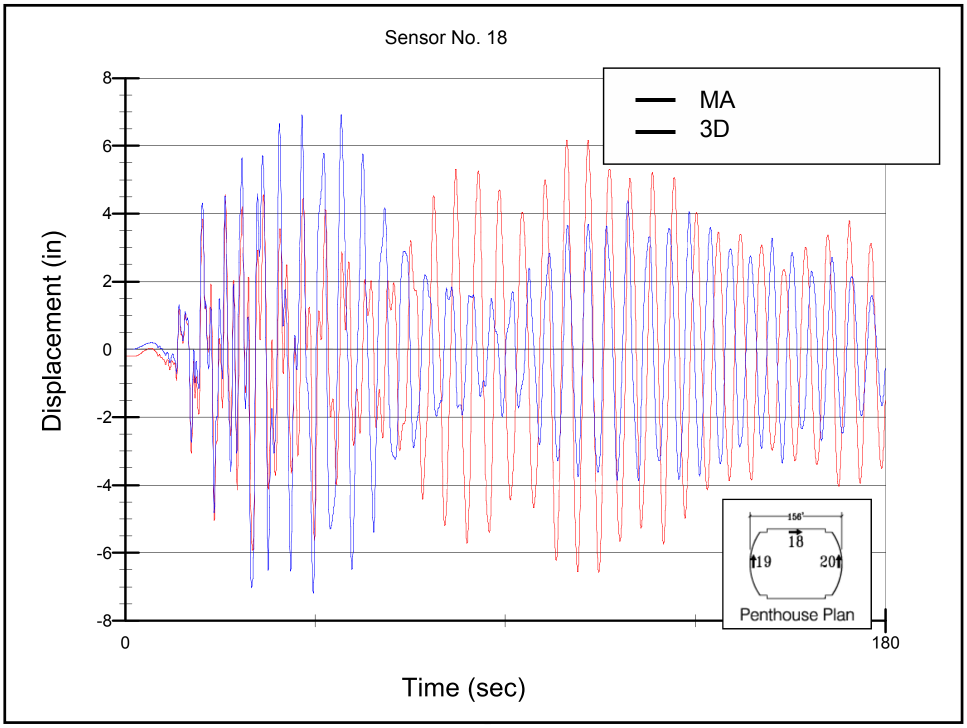


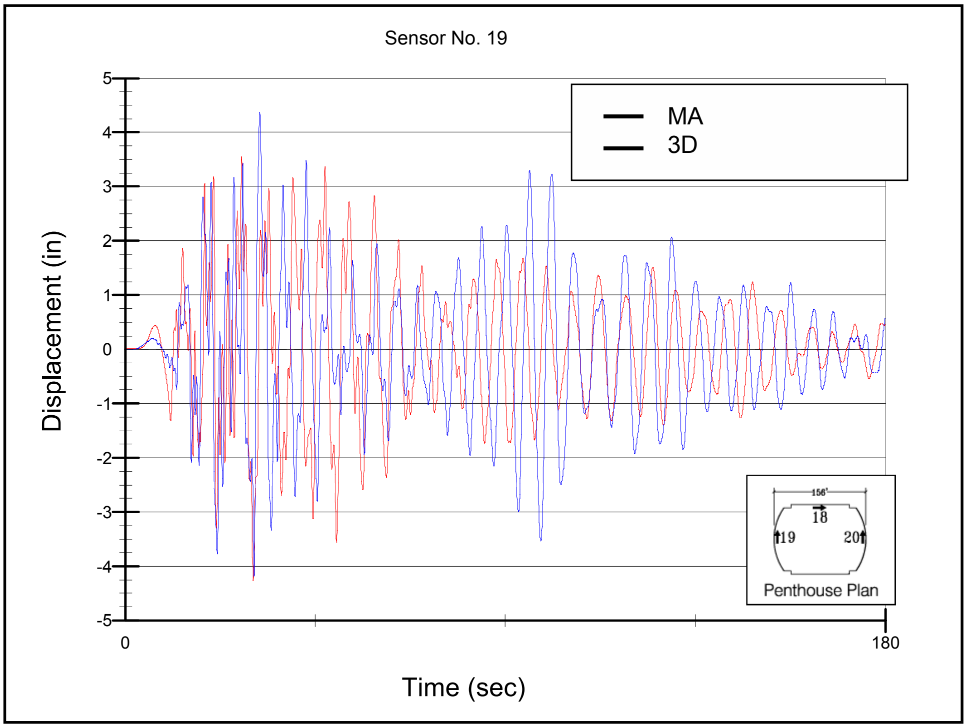
**INPUT MOTIONS: Free-field ground surface, $u_g(z=0)$; $\theta_f=0$**

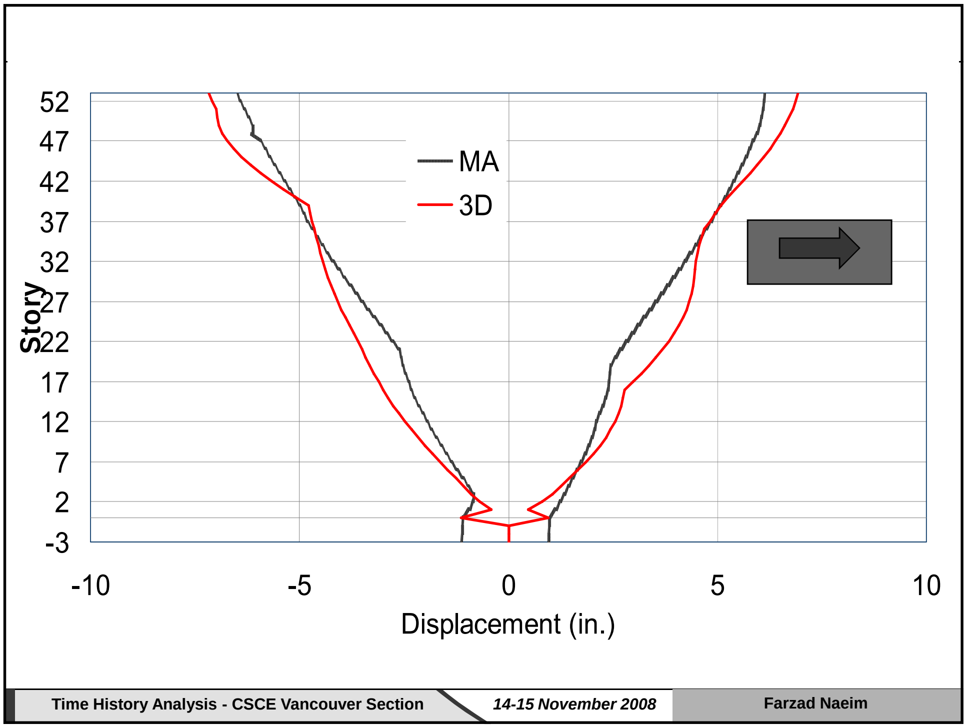
Sensor No. 6

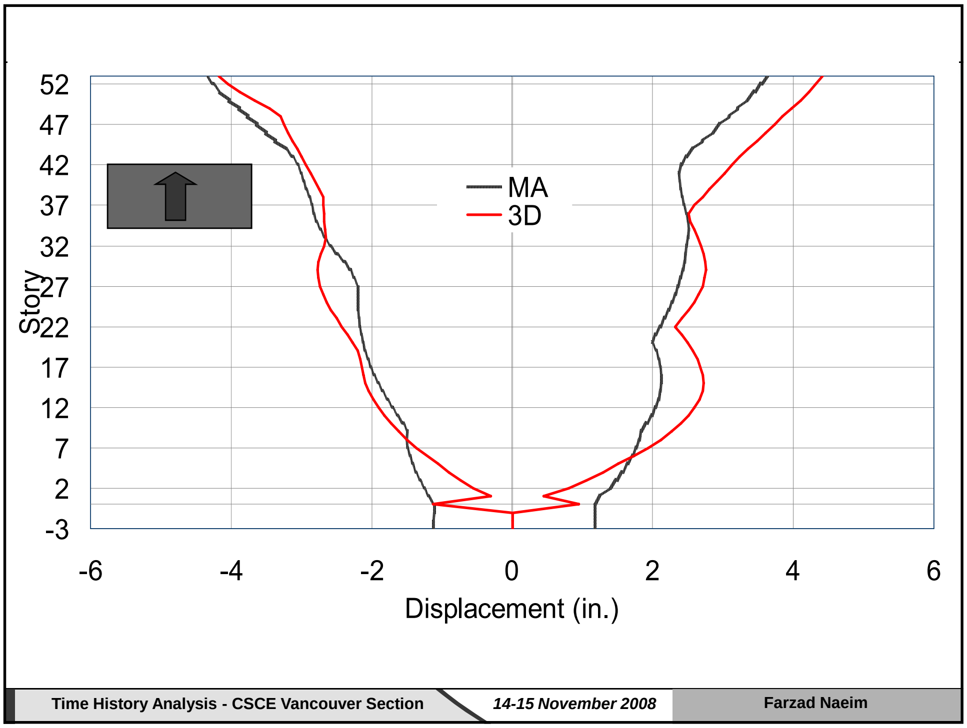


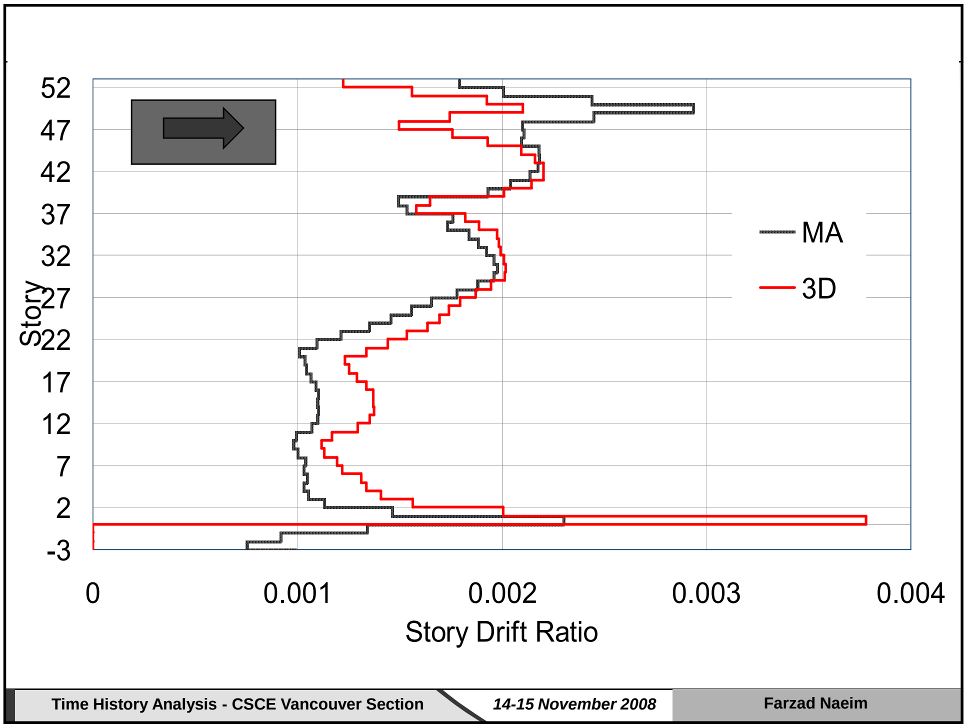

Sensor No. 7
Displacement (in)
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
0
180
Time (sec)
MA
3D
6
7
Upper floors projection
Ground Floor Plan

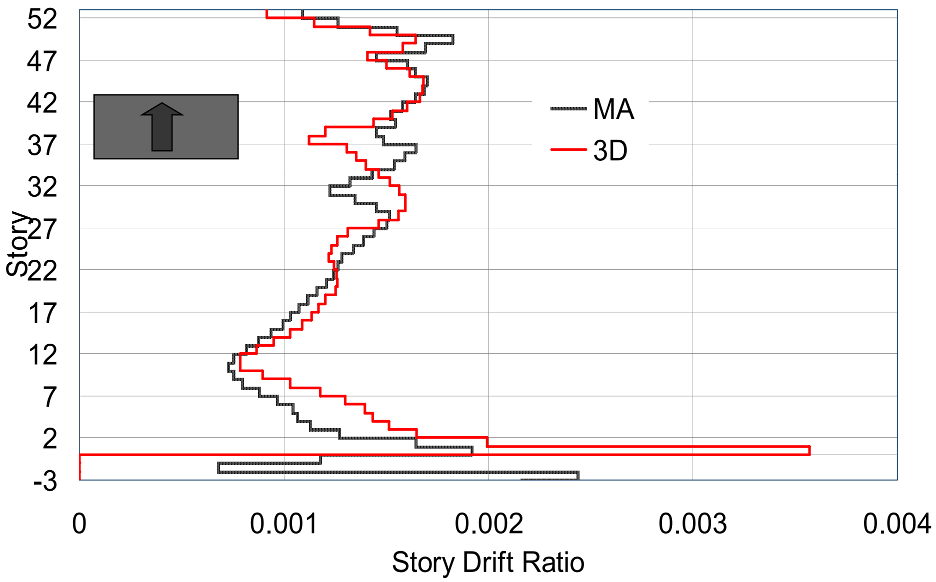

Sensor No. 18
Displacement (in)
8
6
4
2
0
-2
-4
-6
-8
0
180
Time (sec)
MA
3D
156"
18
19
20
Penthouse Plan


Sensor No. 19
MA
3D
Displacement (in)
5
4
3
2
1
0
-1
-2
-3
-4
-5
0
180
Time (sec)
156'
18
19
20
Penthouse Plan


52
47
42
37
32
27
22
17
12
7
2
-3
Story
MA
3D
-10
-5
0
5
10
Displacement (in.)
Time History Analysis - CSCE Vancouver Section
14-15 November 2008
Farzad Naeim


52
47
42
37
32
27
22
17
12
7
2
-3
Story
MA
3D
-6
-4
-2
0
2
4
6
Displacement (in.)
Time History Analysis - CSCE Vancouver Section
14-15 November 2008
Farzad Naeim


52
47
42
37
32
27
22
17
12
7
2
-3
Story
MA
3D
0
0.001
0.002
0.003
0.004
Story Drift Ratio
Time History Analysis - CSCE Vancouver Section
14-15 November 2008
Farzad Naeim



## Preliminary Findings

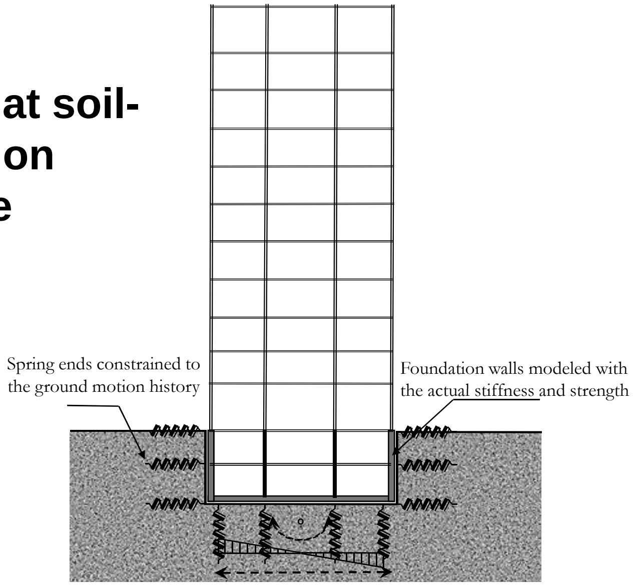
- Effects on modal properties are not that great
- Significant effect on drift distribution over height of structure
- Two models do a particularly poor job:
  - 3B model: $u_g$ applied at base and fixed-end horizontal springs
  - 3D model: Fixed base at ground level
- Not so bad (for this building): fixed base at base level of structure

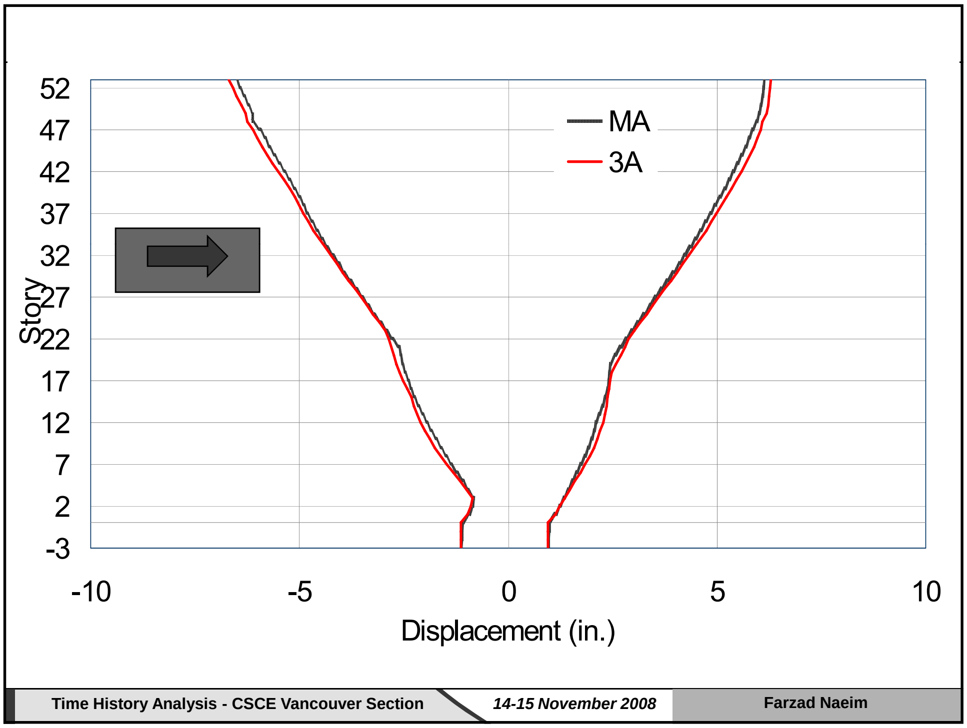
## Period Comparisons

| Model | Reported vibration periods for first five Ritz vectors (sec.) | | | | |
|---|---|---|---|---|---|
| | **1** | **2** | **3** | **4** | **5** |
| MA* | 6.06 | 5.18 | 2.76 | 1.92 | 1.81 |
| 1 | 6.03 | 5.15 | 2.75 | 1.91 | 1.81 |
| 2A | 6.06 | 5.18 | 2.76 | 1.92 | 1.81 |
| 2B | 6.06 | 5.18 | 2.76 | 1.92 | 1.81 |
| 2C | 6.06 | 5.18 | 2.76 | 1.92 | 1.81 |
| 3A | 6.04 | 5.18 | 2.78 | 1.92 | 1.82 |
| 3B | 5.79 | 4.99 | 2.76 | 1.92 | 1.82 |
| 3C | 5.79 | 4.99 | 2.76 | 1.92 | 1.82 |
| 3D | 5.63 | 4.90 | 2.74 | 1.89 | 1.80 |

## Approximation 3a

**Tension allowed at soil-foundation interface**

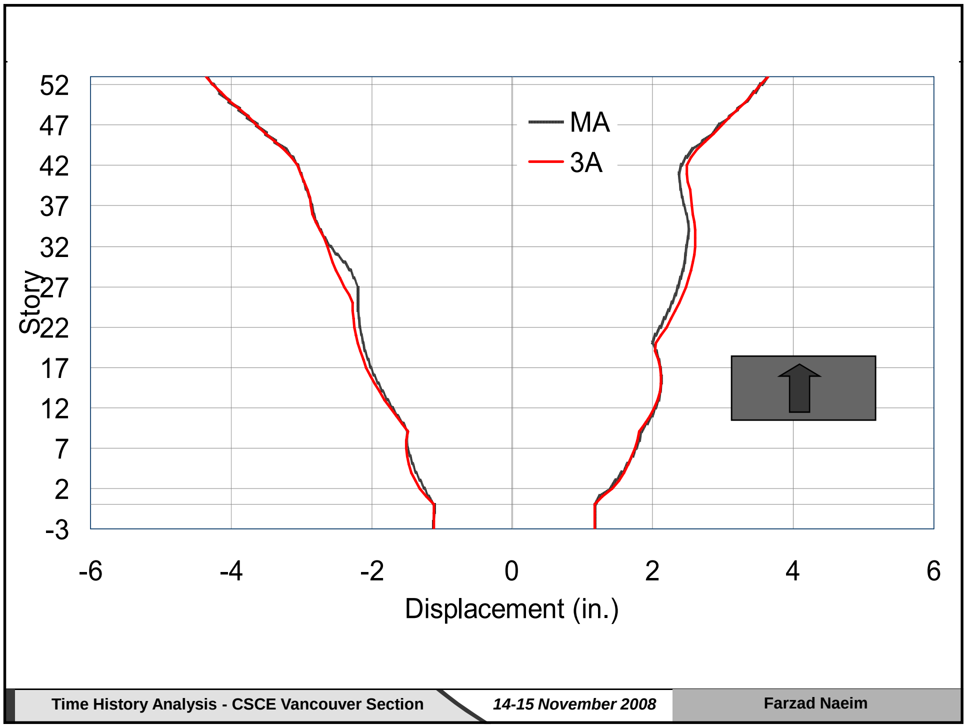




Time History Analysis - CSCE Vancouver Section

*14-15 November 2008*

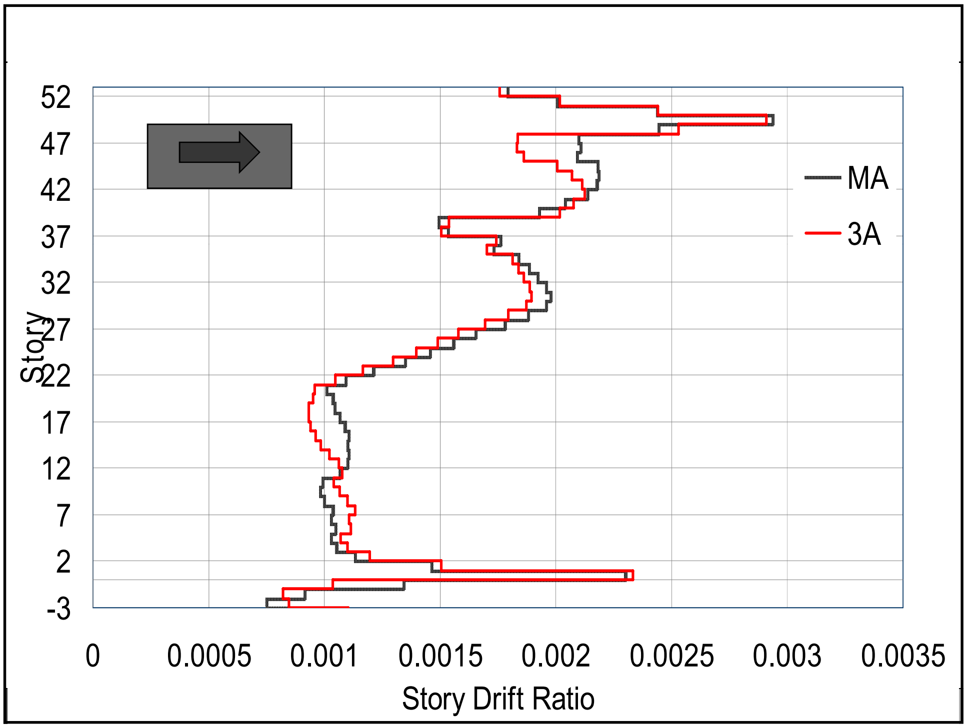
Farzad Naeim

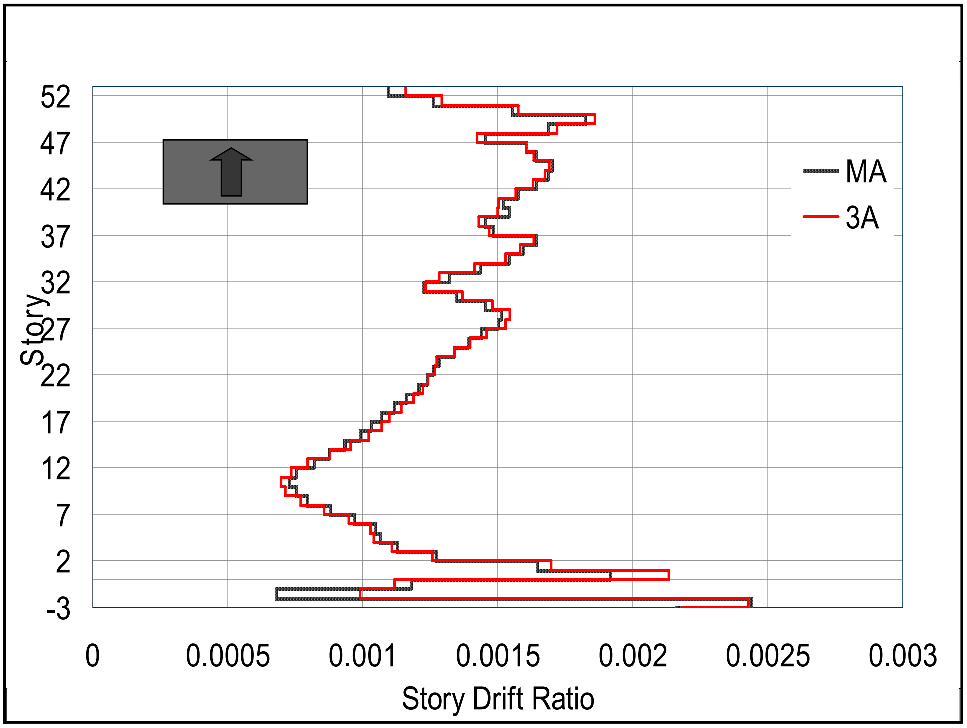


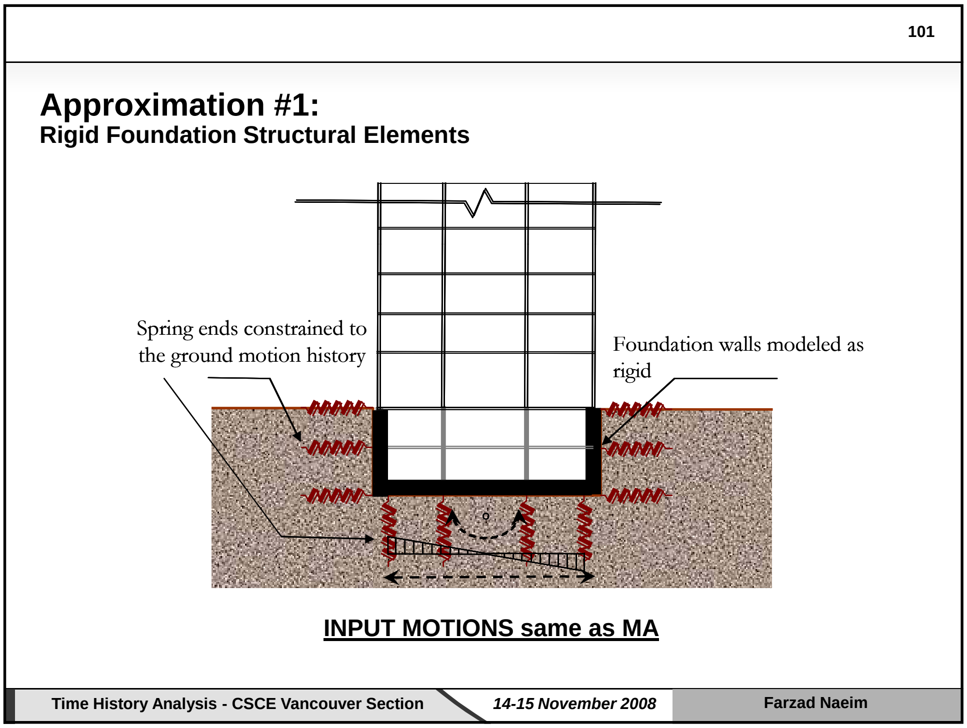
Time History Analysis - CSCE Vancouver Section

*14-15 November 2008*

Farzad Naeim


52
47
42
37
32
27
22
17
12
7
2
-3
Story
0
0.0005
0.001
0.0015
0.002
0.0025
0.003
0.0035
Story Drift Ratio
MA
3A


52
47
42
37
32
27
22
17
12
7
2
-3
Story
0
0.0005
0.001
0.0015
0.002
0.0025
0.003
Story Drift Ratio
MA
3A

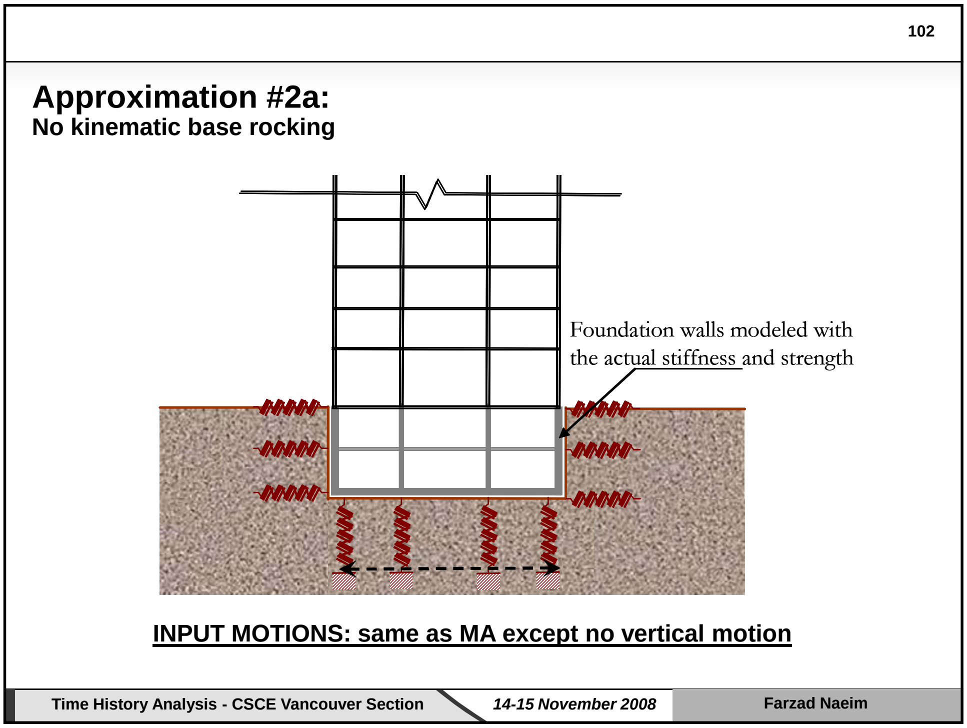


## Approximation #1:
**Rigid Foundation Structural Elements**

Spring ends constrained to the ground motion history

Foundation walls modeled as rigid

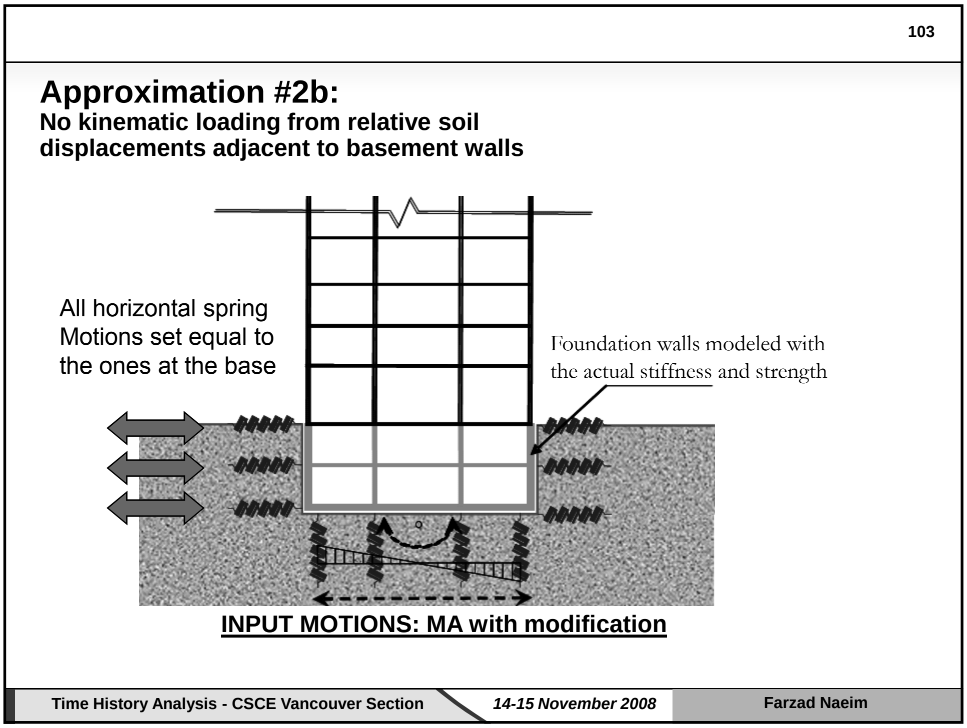
**INPUT MOTIONS same as MA**



## Approximation #2a:
**No kinematic base rocking**

Foundation walls modeled with the actual stiffness and strength

**INPUT MOTIONS: same as MA except no vertical motion**



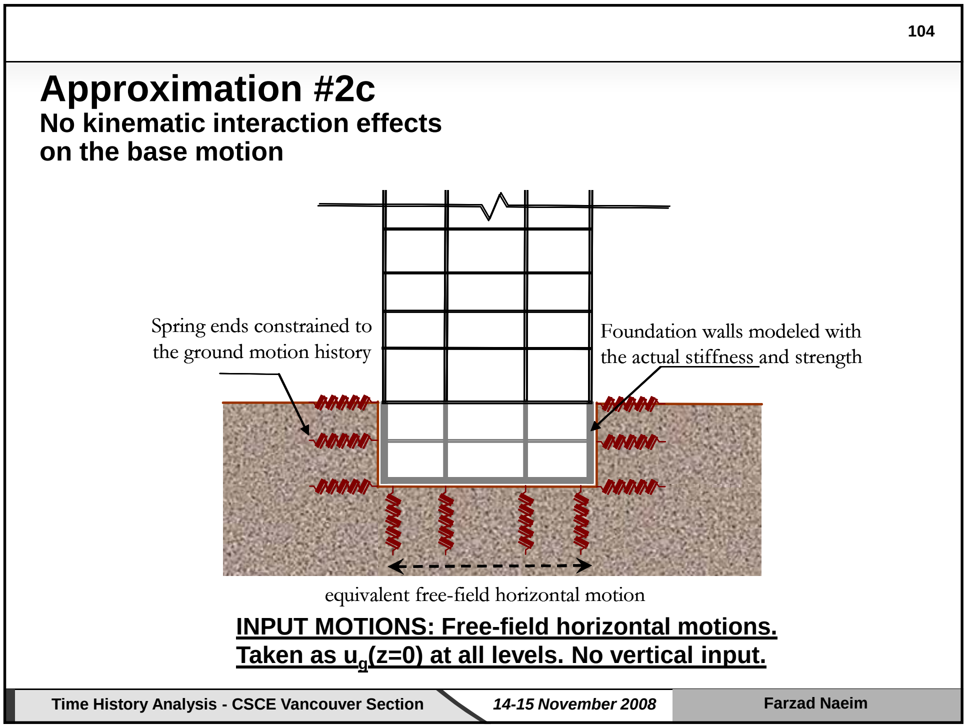
## Approximation #2b:

**No kinematic loading from relative soil displacements adjacent to basement walls**

All horizontal spring Motions set equal to the ones at the base

Foundation walls modeled with the actual stiffness and strength

**INPUT MOTIONS: MA with modification**



## Approximation #2c

**No kinematic interaction effects on the base motion**

Spring ends constrained to the ground motion history

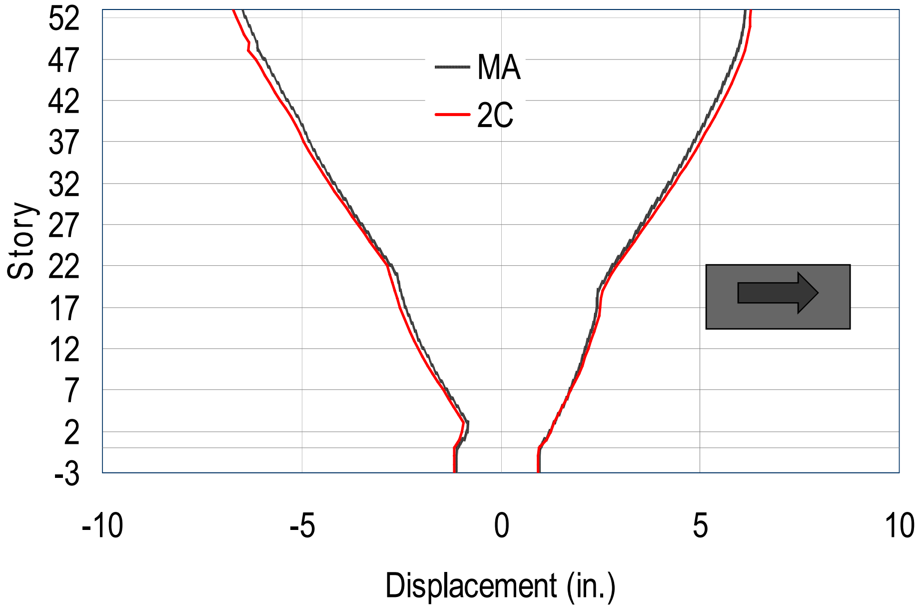
Foundation walls modeled with the actual stiffness and strength

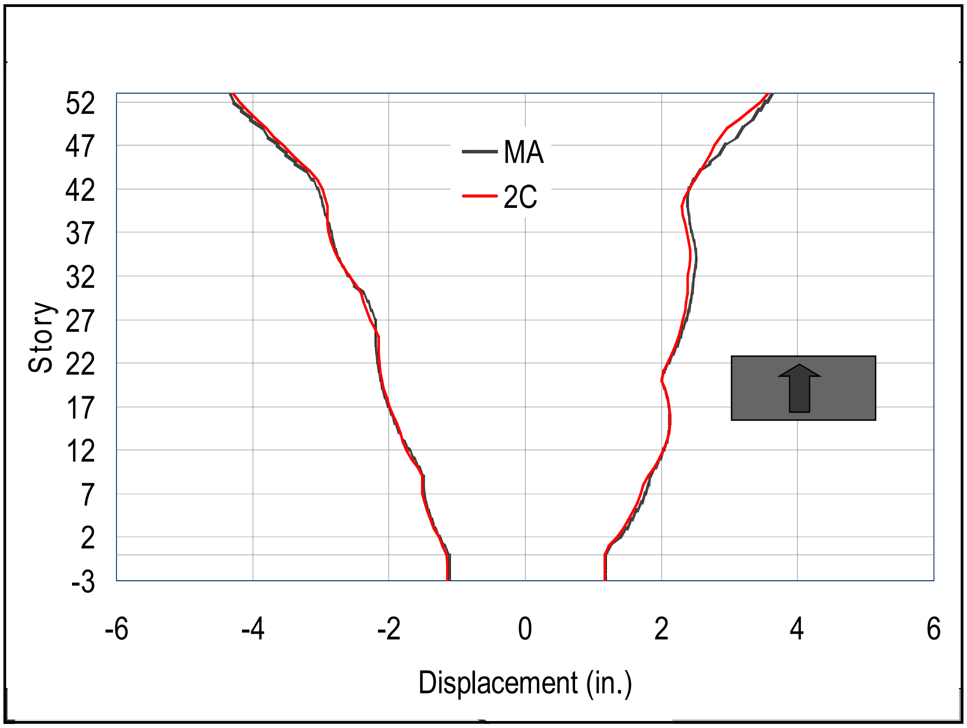
equivalent free-field horizontal motion

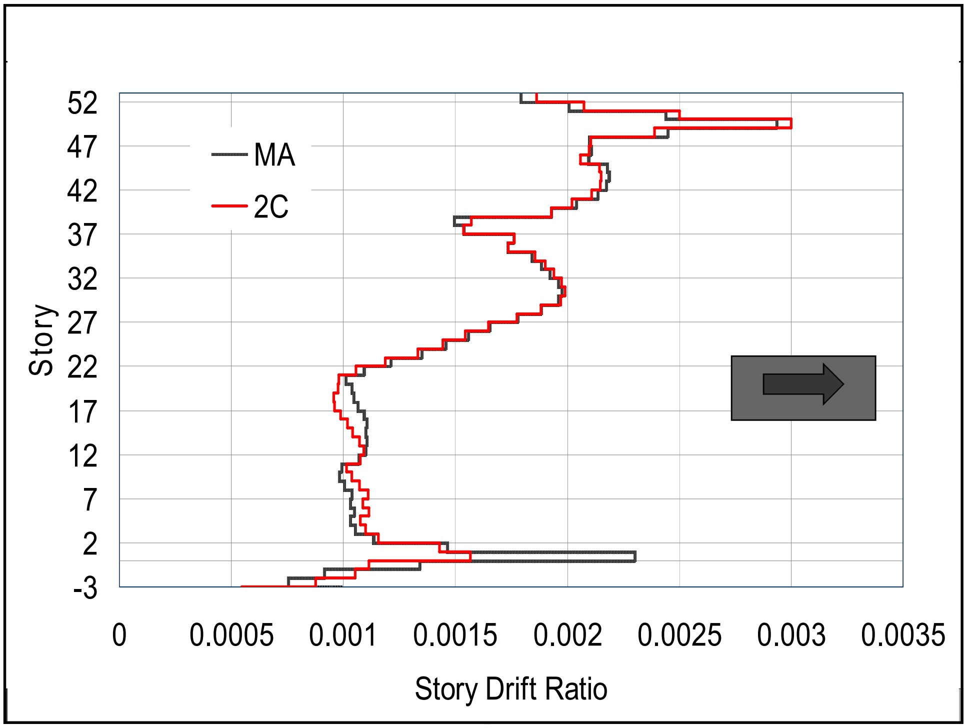
**INPUT MOTIONS: Free-field horizontal motions. Taken as $u_g$(z=0) at all levels. No vertical input.**

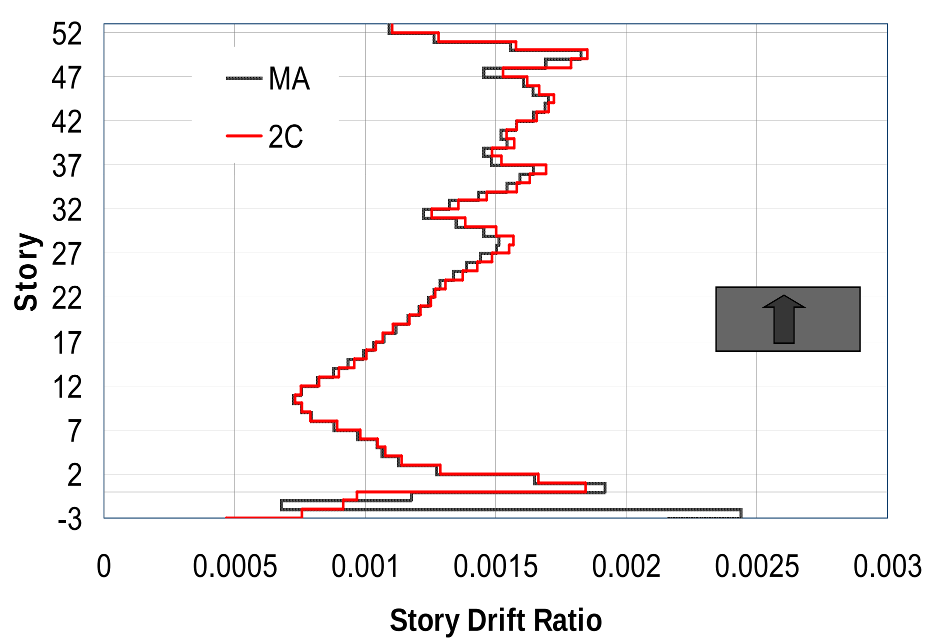
## Preliminary Findings

- Effects on modal properties are negligible
- Displacements and drifts at level above ground are very close to those obtained from the MA model
- Significant errors are present in estimates of drift for the subterranean levels
- As an example, we will show you the results for Approximation 2C
- Results for others are contained in the paper.




52
47
42
37
32
27
22
17
12
7
2
-3
Story
MA
2C
-6
-4
-2
0
2
4
6
Displacement (in.)


52
47
42
37
32
27
22
17
12
7
2
-3
Story
MA
2C
0
0.0005
0.001
0.0015
0.002
0.0025
0.003
0.0035
Story Drift Ratio



## Conclusions

- Soil-structure interaction can affect the response of buildings with subterranean levels
- While procedures are available to account for these effects, they are seldom utilized in engineering practice
- With reasonable tuning of superstructure damping, the MA model accurately reproduces the observed response to the 1994 Northridge earthquake.
- There are hurdles to the implementation of SSI in building design.
  - Multiple support excitations
  - Lack of direct integration (ETABS)
  - Acceleration spikes (ETABS)
- We anticipate these hurdles to go away real soon

## Conclusions (continued)

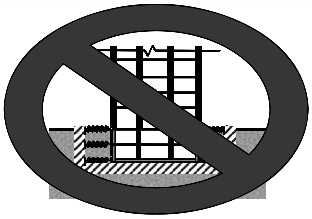
- Factors found to generally have a modest effect on building response above ground level:
  - compliance of structural foundation elements
  - kinematic interaction effects (on translation or rocking)
  - depth-variable ground motions applied to the ends of horizontal soil springs/dashpots.
- However, these factors did generally affect below-ground response as measured by interstory drift

## Conclusions (continued)

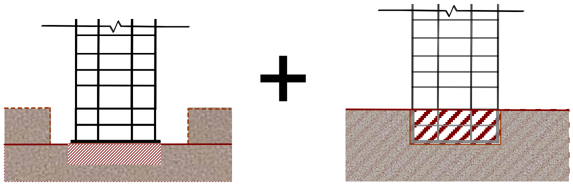
- Properly accounting for foundation/soil deformations does not significantly affect vibration periods for this tall building (which is expected),
- It does impact significantly the distribution of inter-story drifts over the height of the structure.
- To our knowledge, the latter observation is new to this study.

## Conclusions (continued)

- One of the approximations commonly used in practice is shown to provide particularly poor results.



- Two other approximations commonly used in practice, if used together, do a decent job of enveloping drift and displacement demands for the above ground stories (in the reverse roles assumed in practice).



# Thank you!

# TIME HISTORY ANALYSIS

## LECTURE # 7

## SSI Modeling issues

**Farzad Naeim, Ph.D., S.E., Esq.**
**John A. Martin & Associates, Inc.**

# NOTES

For soil-foundation-structure interaction modeling issues see:

> Naeim, F., Tileylioglu, S., Alimoradi, A. and Stewart, J.P. (2008), "Impact of Foundation Modeling on the Accuracy of Response History Analysis of a Tall Building," Proceedings of SMIP-08 Seminar, California Geological Survey, Los Angeles, September.

This article can be downloaded from:

> ftp://ftp.johnmartin.com/ATC58/BCTH08/FN-SMIP-08.pdf

# TIME HISTORY ANALYSIS

## LECTURE # 8

## Software Options for Structural Time History Analysis

**Mahmoud Rezai**
**EQ-Tec Engineering Ltd.**

Dr. Rezai specializes in the analysis/design and understanding of non-linear behaviour of structures and their components. He has successfully incorporated ”innovative technologies” in various projects including using Ballast Water Tanks to increase the overall damping and thus minimizing the effect of wave motions, Fibre Reinforced Polymers (FRP), passive energy dissipation devices such as viscous dampers as well as base isolation system. He has carried out seismic assessment and design of a number of buildings and bridges in the past decade. He has provided peer reviews and design checks of numerous upgrade projects including analysis/design and construction field services for a number of concrete high-rise buildings, the Pattullo Bridge, Lions Gate Bridge and upgrade and assessment services for many different structures including Vancouver schools and hospitals. He has authored more than 50 papers and reports on structural analysis/design and behaviour/response of structural systems. Over the past ten years he has taught courses related to seismic analysis and design and retrofit of existing structures as a lecturer for UBC’s Certificate Program to the practicing engineers.

The Canadian Society for Civil Engineering, Vancouver Section

# TIME HISTORY ANALYSIS

## Software Options for Structural Time History Analysis

Mahmoud Rezai, Ph.D., P.Eng., Struct.Eng.

EQ-Tec Engineering Ltd.

*A technical seminar on the use of time histories and site specific response spectra in structural design, and an introduction to linear and non-linear time history analysis.*

***14-15 November 2008 Vancouver, BC***

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## Software Options

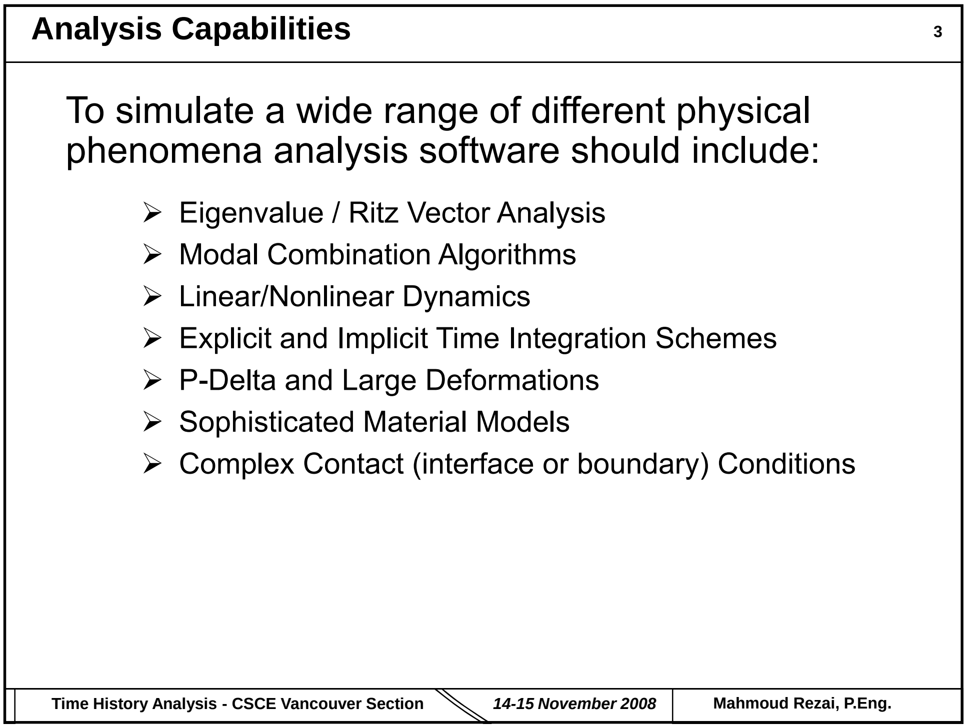
Commonly Used List of Software for Structural Analysis:

- CSI Software (SAP2000, ETABS, Perform3D)
- CSC (S-Frame, Orion, Fastrak)
- Bentley (RAM Structural System and STAAD Pro)
- GT STRUDL (Georgia Tech Research Corporation)
- RISA-3D (Risa Technologies)
- Visual Tools, Multiframe,
- OpenSees, SeismoStruct, Nonlin & Nonlin-Pro

General Purpose Finite Element Analysis Software:

- ABAQUS, ANSYS, ALGOR, LS-DYNA, ADINA, NASTRAN, DIANA, COSMOS

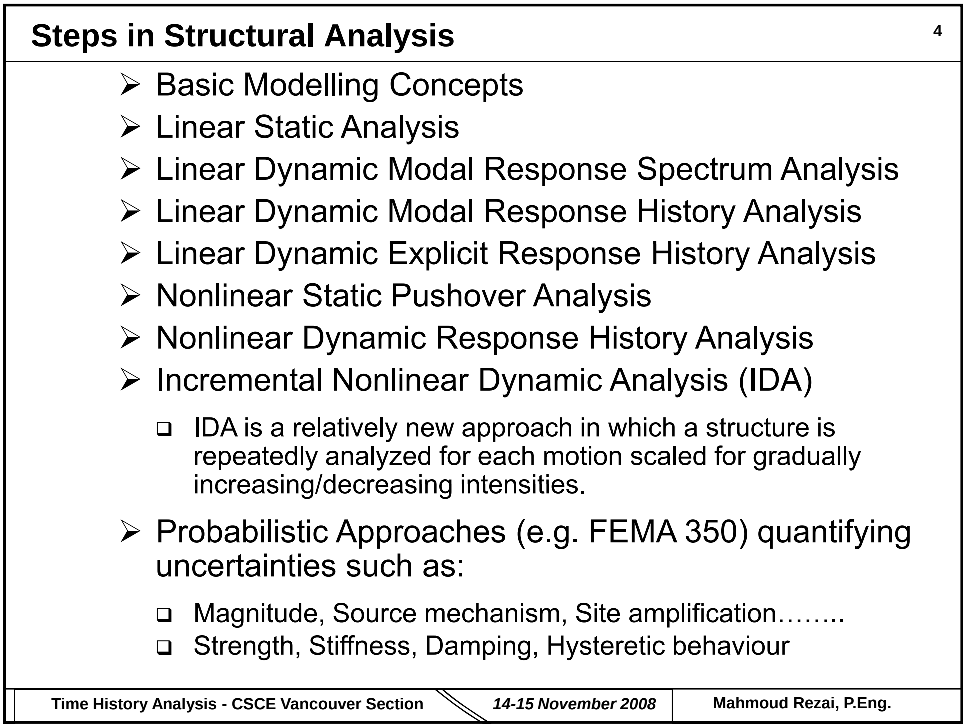
  These are extremely powerful FEA programs but are not very practical for analysis of building and bridge structures.



## Analysis Capabilities

To simulate a wide range of different physical phenomena analysis software should include:

- Eigenvalue / Ritz Vector Analysis
- Modal Combination Algorithms
- Linear/Nonlinear Dynamics
- Explicit and Implicit Time Integration Schemes
- P-Delta and Large Deformations
- Sophisticated Material Models
- Complex Contact (interface or boundary) Conditions



## Steps in Structural Analysis

- Basic Modelling Concepts
- Linear Static Analysis
- Linear Dynamic Modal Response Spectrum Analysis
- Linear Dynamic Modal Response History Analysis
- Linear Dynamic Explicit Response History Analysis
- Nonlinear Static Pushover Analysis
- Nonlinear Dynamic Response History Analysis
- Incremental Nonlinear Dynamic Analysis (IDA)
  - IDA is a relatively new approach in which a structure is repeatedly analyzed for each motion scaled for gradually increasing/decreasing intensities.
- Probabilistic Approaches (e.g. FEMA 350) quantifying uncertainties such as:
  - Magnitude, Source mechanism, Site amplification……..
  - Strength, Stiffness, Damping, Hysteretic behaviour

## Dynamic Time History Analysis

In general, a three-dimensional model is necessary for TH Analysis. However, due to limitations in available software, 3-D inelastic time history analysis may not be practical (except for very special and important structures).

### Main Concerns in Nonlinear Dynamic Analysis:

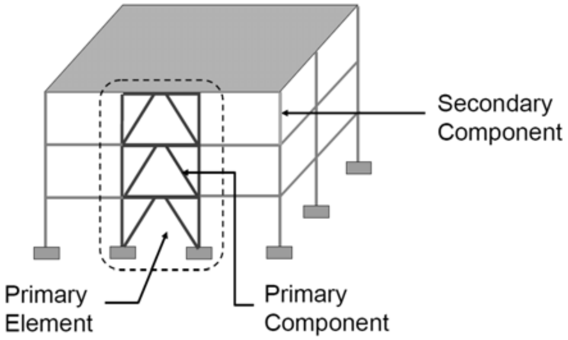
- Modelling of hysteretic behaviour
- Modelling inherent damping
- Selection and scaling of ground motions
- Interpretation of results
- Results may be quite sensitive to seemingly minor perturbations

Due to the fact that some of these concerns may be insurmountable in the framework of a deterministic analysis, a probabilistic framework is being developed.

## Basic Modelling Concepts

In general, a model should include the following:

- Structural (Primary) Components and Elements
- Soil-Structure-Foundation System
- Structural (Secondary) Components and Elements



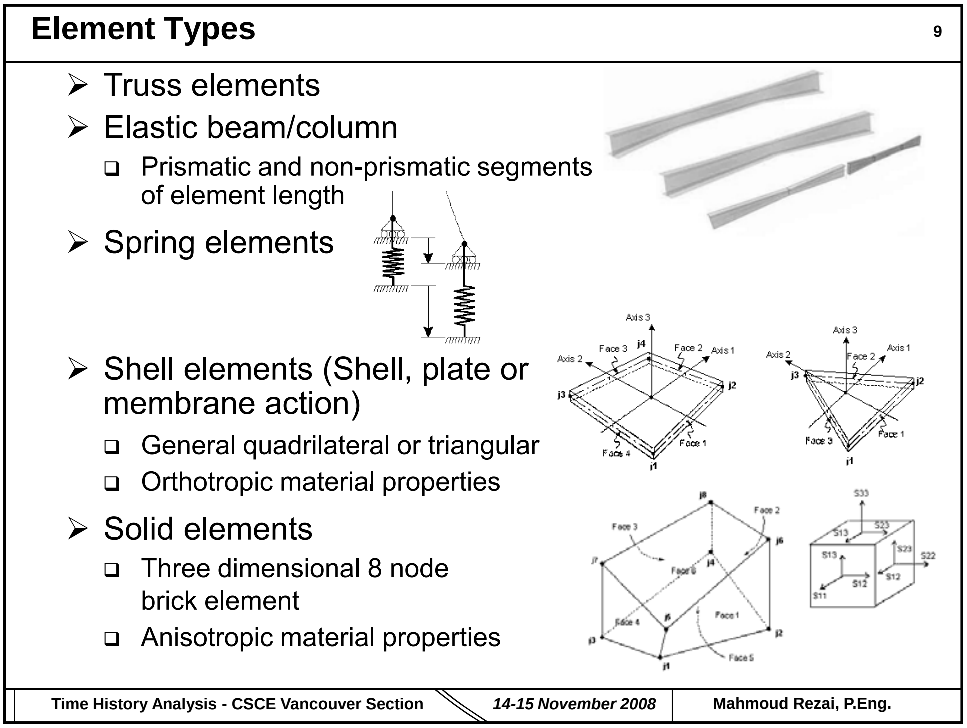
## Basic Modelling Concepts

…………a model should include the following:

- Mechanical Systems (if performance of such systems is being assessed)
- Reasonable Distribution and Sequencing of gravity loads
- P-Delta (Second Order) Effects
- Reasonable Representation of Inherent Damping
- Realistic Representation of Inelastic Behaviour
- Realistic Representation of Ground Shaking

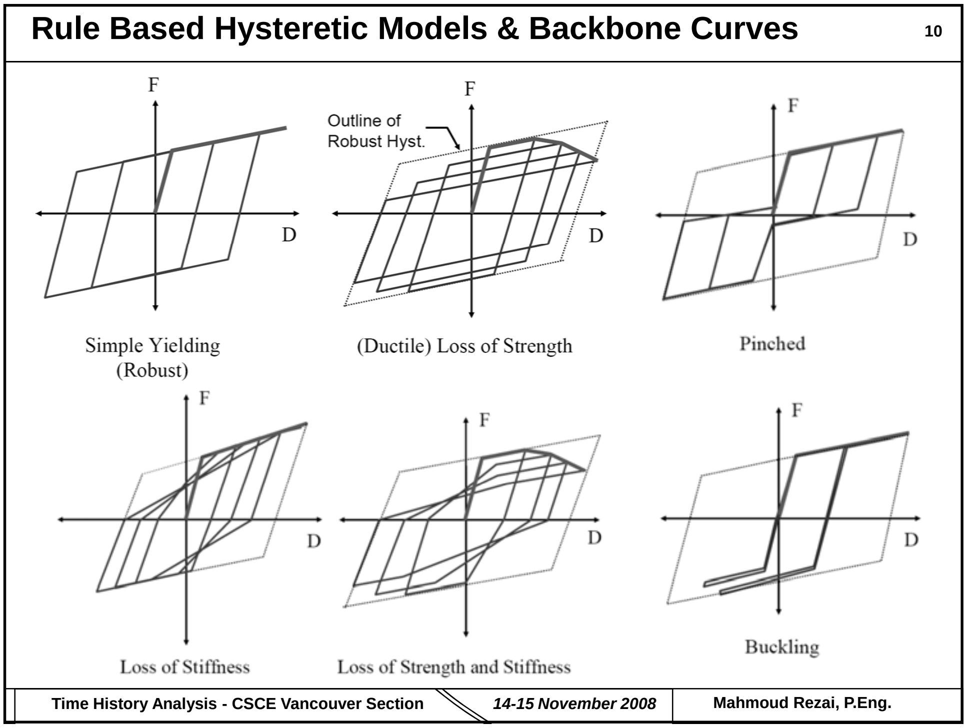
## Few Modelling Tips

- An analytical model should not feature an excessive number of elements, section fibres, load increments or iterations, all of which, together with too-stringent convergence criteria, will cause the analysis to slow down quite considerably.
- Run sensitivity studies of similar but smaller models to find out the optimum values of the aforementioned modelling parameters that will lead to the attainment of accurate results but at a lower computational cost, before embarking on time-consuming analyses of very large models.
- Also if you are, for instance, interested in predicting the top displacement of a building (i.e. global response) subjected to monotonic loading, you are most likely not to require the same level of mesh/fibre refinement that you would need if trying to predict the failure strain of a column section (local response) subjected to cyclic loading.



## Element Types

- Truss elements
- Elastic beam/column
  - ❑ Prismatic and non-prismatic segments of element length
- Spring elements
- Shell elements (Shell, plate or membrane action)
  - ❑ General quadrilateral or triangular
  - ❑ Orthotropic material properties
- Solid elements
  - ❑ Three dimensional 8 node brick element
  - ❑ Anisotropic material properties



## Rule Based Hysteretic Models & Backbone Curves

Simple Yielding (Robust)

(Ductile) Loss of Strength

Pinched

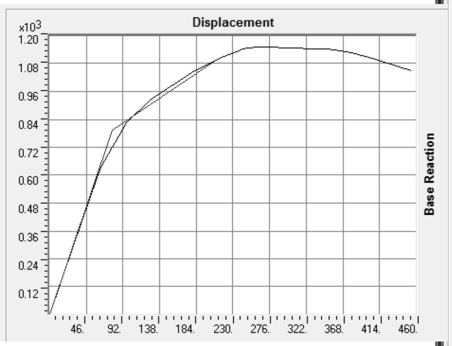
Loss of Stiffness

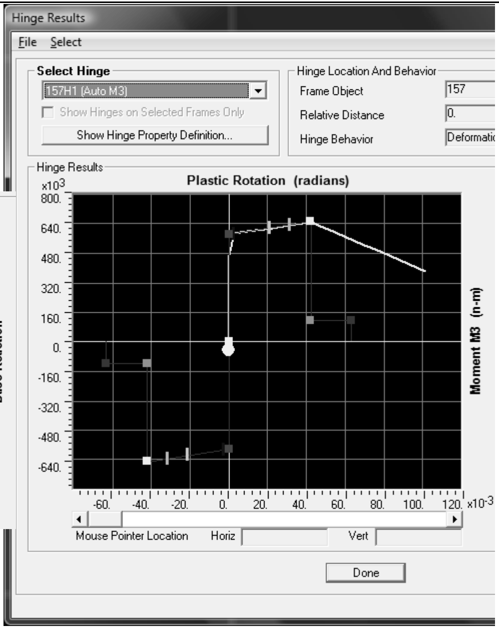
Loss of Strength and Stiffness

Buckling

## Beam-Column – Backbone/hysteretic Curve – SAP2000

- Frame plastic hinge element for use with static nonlinear analysis





Select Hinge: 157H1 (Auto M3)

☐ Show Hinges on Selected Frames Only

Show Hinge Property Definition...

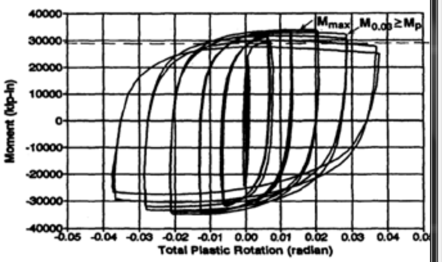
Frame Object: 157

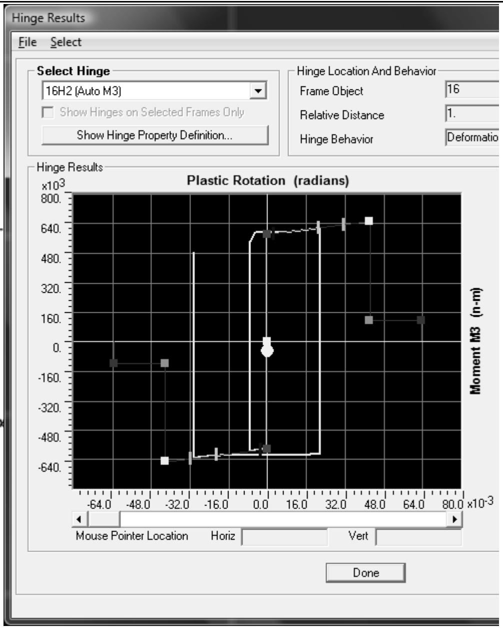
Relative Distance: 0.

Hinge Behavior: Deformatio

## Beam-Column – Backbone/hysteretic Curve – SAP2000

- Frame plastic hinge element for use with dynamic nonlinear analysis





Select Hinge: 16H2 (Auto M3)

☐ Show Hinges on Selected Frames Only

Show Hinge Property Definition...

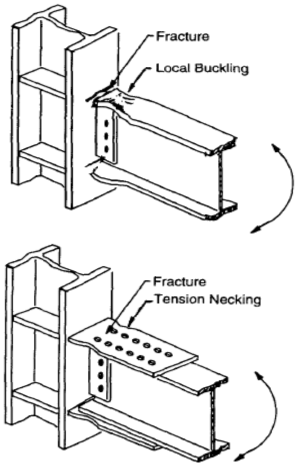
Frame Object: 16

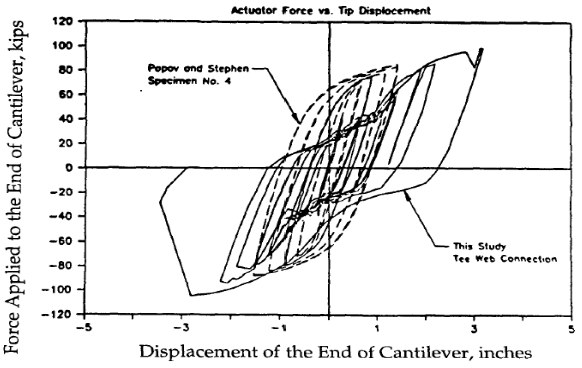
Relative Distance: 1.

Hinge Behavior: Deformatio

## Welded and Bolted Moment Connections

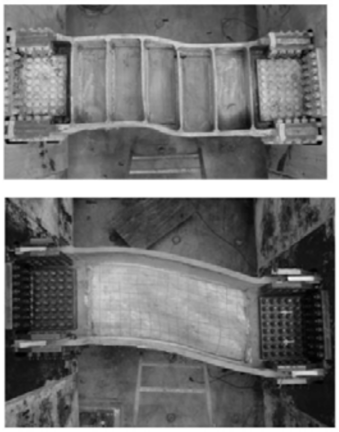
- Comparison of the moment-rotation behaviour of a bolted connection (Astaneh-Asl et al., 1991) and a comparable fully welded connection from the tests conducted by Popov and Bertero (1973).

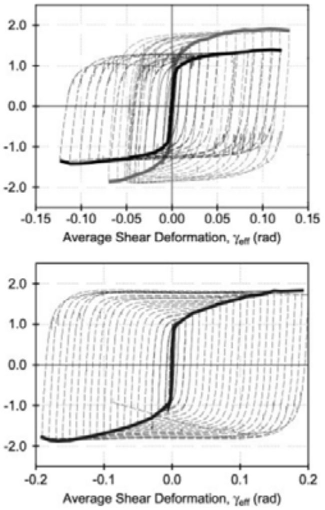




## Eccentric Braced Frames

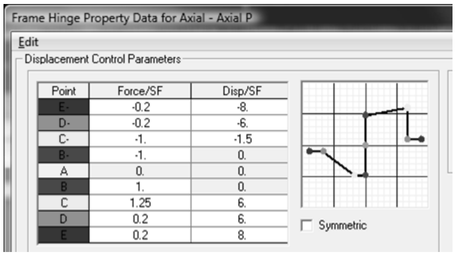
- Deformation near end of test cyclic test results for shear links with high performance steel (top) and low-yield steel (bottom), after Dusicka P., Itani AM & Buckle IG (2004)





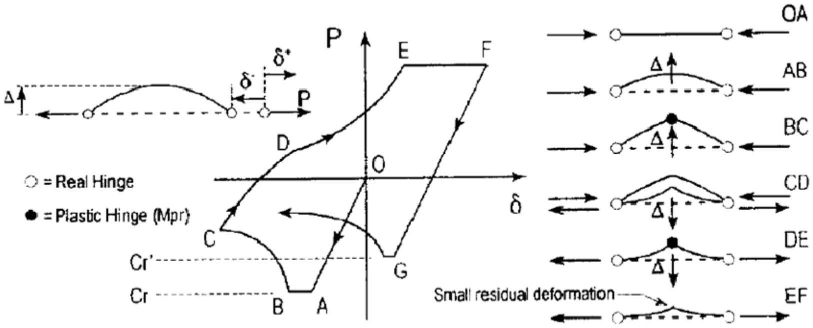
## Brace – Backbone/Hysteretic Curve – SAP2000

- Brace axial element for use with static and dynamic nonlinear analysis

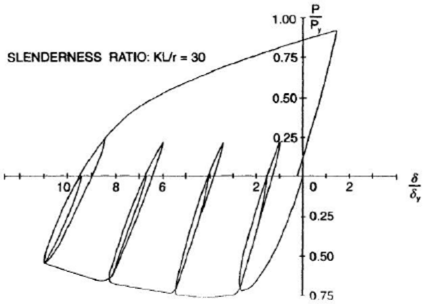


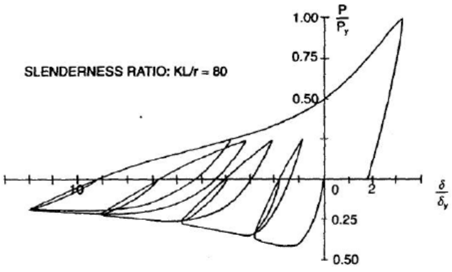
Displacement Control Parameters

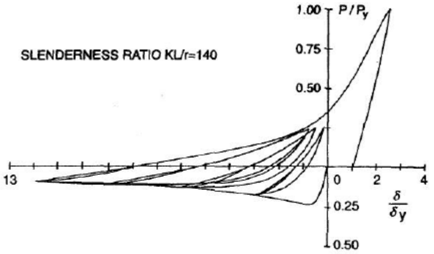
| Point | Force/SF | Disp/SF |
|---|---|---|
| E- | -0.2 | -8. |
| D- | -0.2 | -6. |
| C- | -1. | -1.5 |
| B- | -1. | 0. |
| A | 0. | 0. |
| B | 1. | 0. |
| C | 1.25 | 6. |
| D | 0.2 | 6. |
| E | 0.2 | 8. |

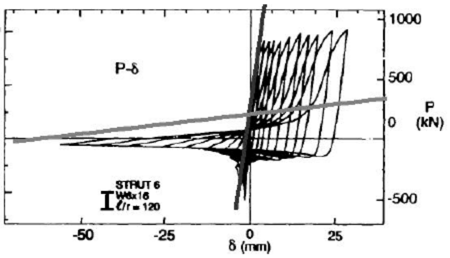


## Effect of Brace Slenderness Ratio



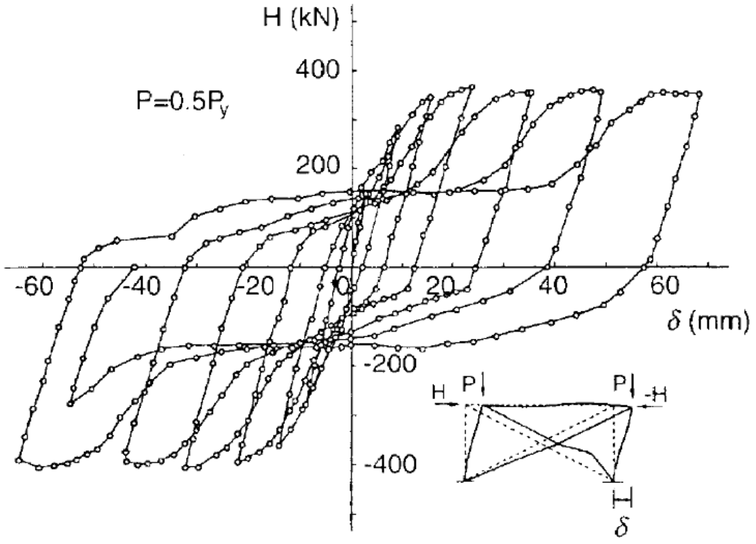






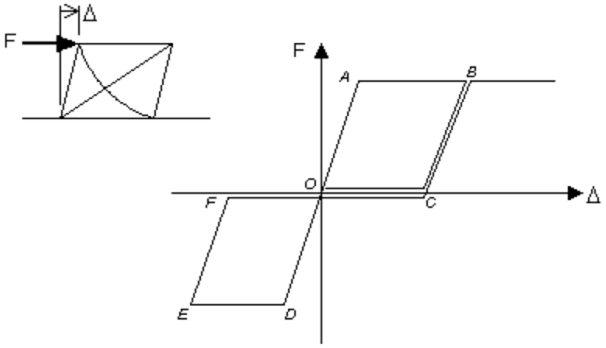
## Hysteresis Loops of a Slender Braced Frame

- Brace axial tension-only element



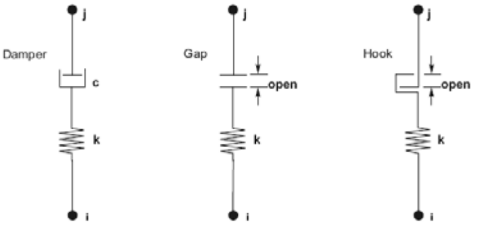
## Brace Tension Only – Backbone/hysteretic Curve

- Brace axial tension-only element for use with static and dynamic nonlinear analysis
  - Comprised of slender rods that buckle under any practical compressive load

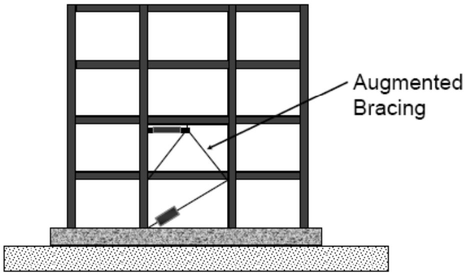


## Link elements

- Nonlinear Viscous Damper, Gap, and Hook for Axial Deformations

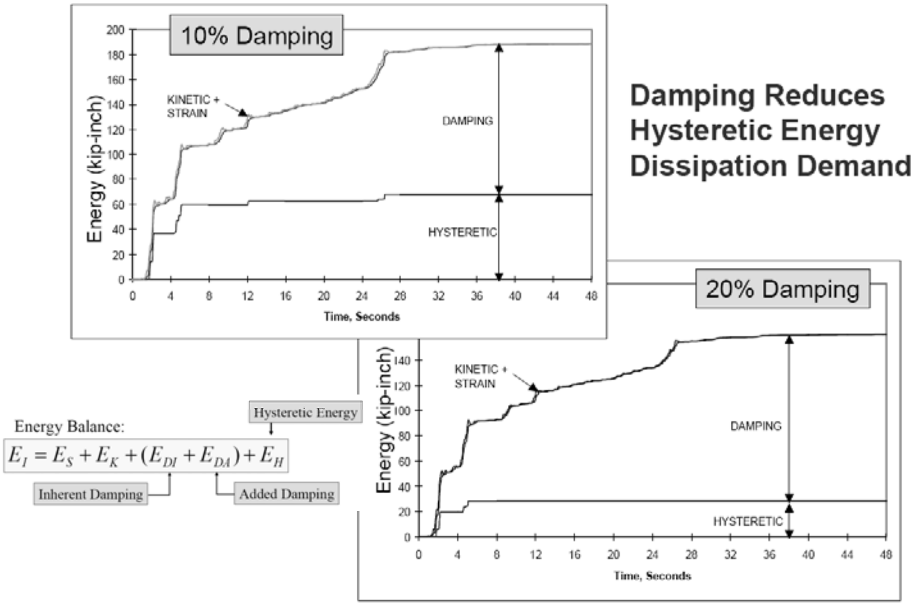


Comp. only Tension only



These nonlinear link elements can model structural gapping/pounding, expansion joints, deck restrainers or simply used for modelling tension-only braces.

## Benefit of Added Damping



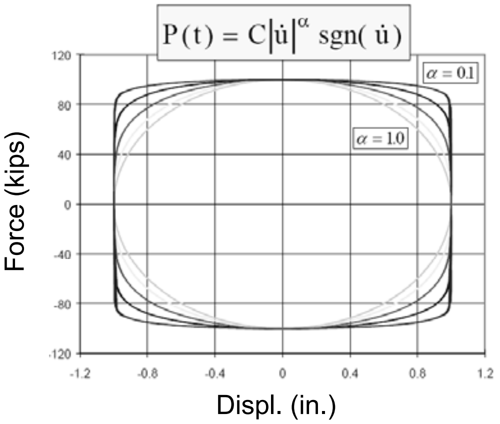
**Damping Reduces Hysteretic Energy Dissipation Demand**

Energy Balance:

$$E_I = E_S + E_K + (E_{DI} + E_{DA}) + E_H$$

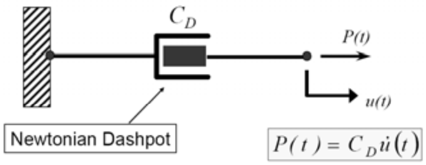
## Link Elements – Nonlinear Viscous Damper

- Link elements
  - ❑ Nonlinear Viscous Dampers are velocity dependent.
    - ❑ Viscous fluid or viscoelastic solid dampers



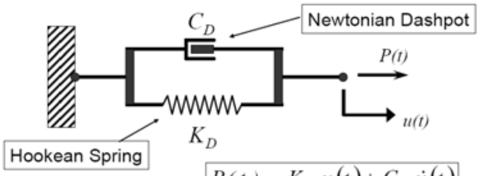
## Link Elements – Nonlinear Viscous Dampers

### Simple Dashpot



$$P(t) = C_D\dot{u}(t)$$

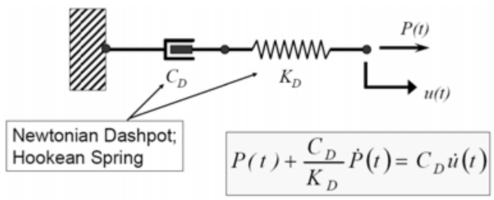
### Dampers: Kelvin Model



$$P(t) = K_D u(t) + C_D\dot{u}(t)$$

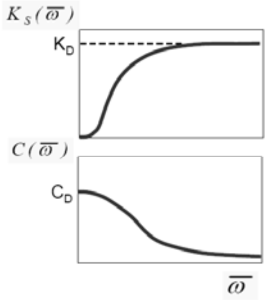
## Link Elements – Nonlinear Viscous Dampers

### Dampers: Maxwell Model



Newtonian Dashpot; Hookean Spring

$$P(t)+\frac{C_D}{K_D}\dot{P}(t)=C_D\dot{u}(t)$$



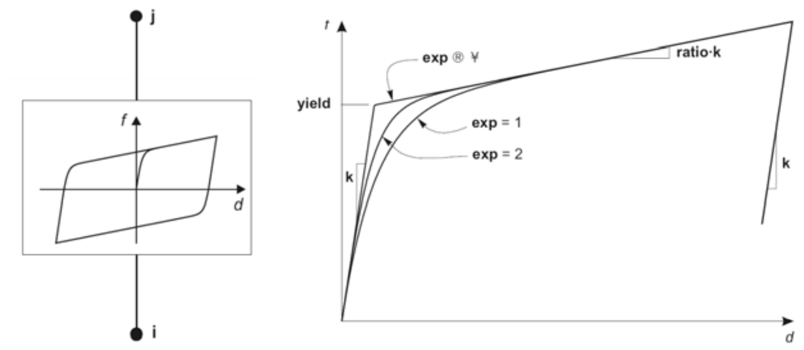
- To model a linear viscous dashpot, $K_D$ must be set to a large value, but not too large or convergence will not be achieved. To achieve this, it is recommended that the ratio $C_D/K_D$ be an order of magnitude less than the loading time step Δt.
- For example, let $K_D = 100C_D/\Delta t$. Sensitivity to $K_D$ should be checked.
- SAP2000 often has difficulty converging when nonlinear dampers are used and the velocity exponent is less than 0.4.

## Link Elements – Dampers

- Recommendations Related to Nonlinear Viscous Dampers
  - ☐ Use discrete damper elements and explicitly include these dampers in the system damping matrix. Explicitly model inelastic behaviour in superstructure. Perform response history analysis of full system.
  - ☐ Do NOT attempt to linearize the problem when nonlinear viscous dampers are used. Perform the analysis with discrete nonlinear viscous dampers.
  - ☐ Do NOT attempt to calculate effective damping in terms of a damping ratio (ξ) when using nonlinear viscous dampers.
  - ☐ DO NOT attempt to use a free vibration analysis to determine equivalent viscous damping when nonlinear viscous dampers are used.

## Link Elements

- ❑ Uniaxial plasticity (all six degrees of freedom)



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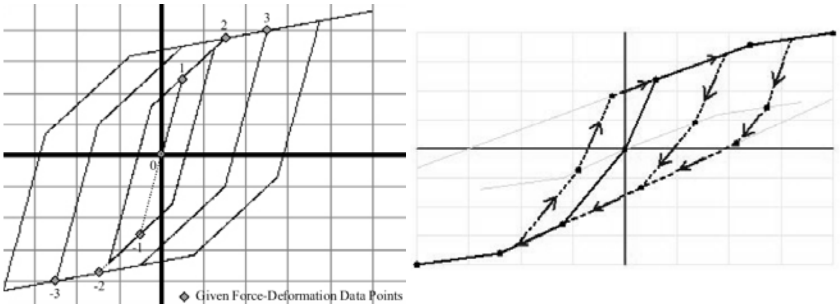
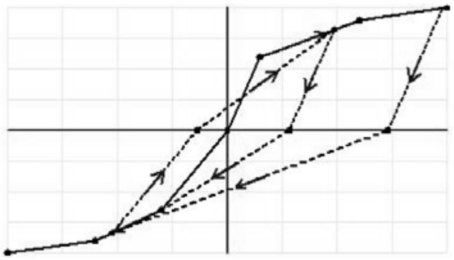
## Link Elements

- ❑ Multi-linear Kinematic Plasticity Property for Uniaxial Deformation





Takeda model includes:

(a) Stiffness changes at flexural cracking and yielding,

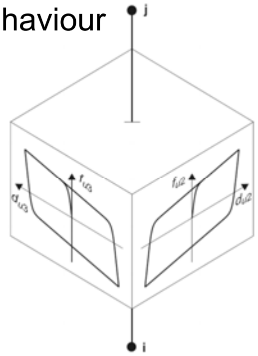
(b) Hysteresis rules for inner hysteresis loops inside the outer loop,

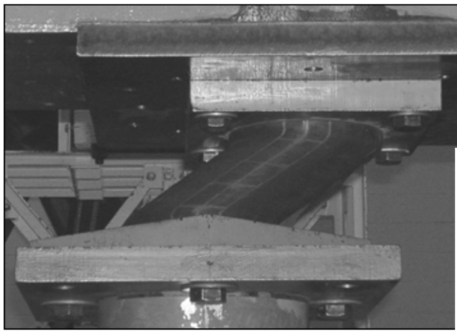
(c) Unloading stiffness degradation with deformation

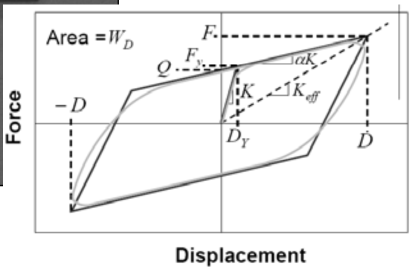
## Link Elements

- ❑ Base isolator with biaxial plasticity behaviour

Coupled plasticity properties for the two shear deformations, and linear effective-stiffness properties for the remaining four deformations.

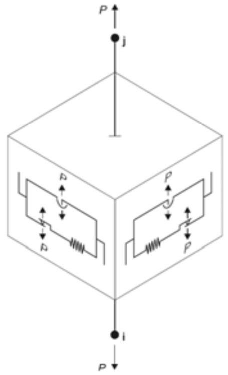




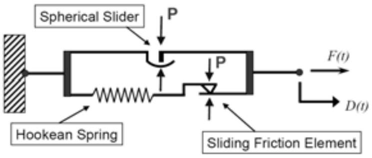


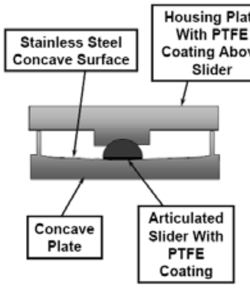
## Link Elements

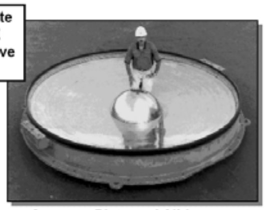
- ❑ Base isolator with friction and/or pendulum behaviour



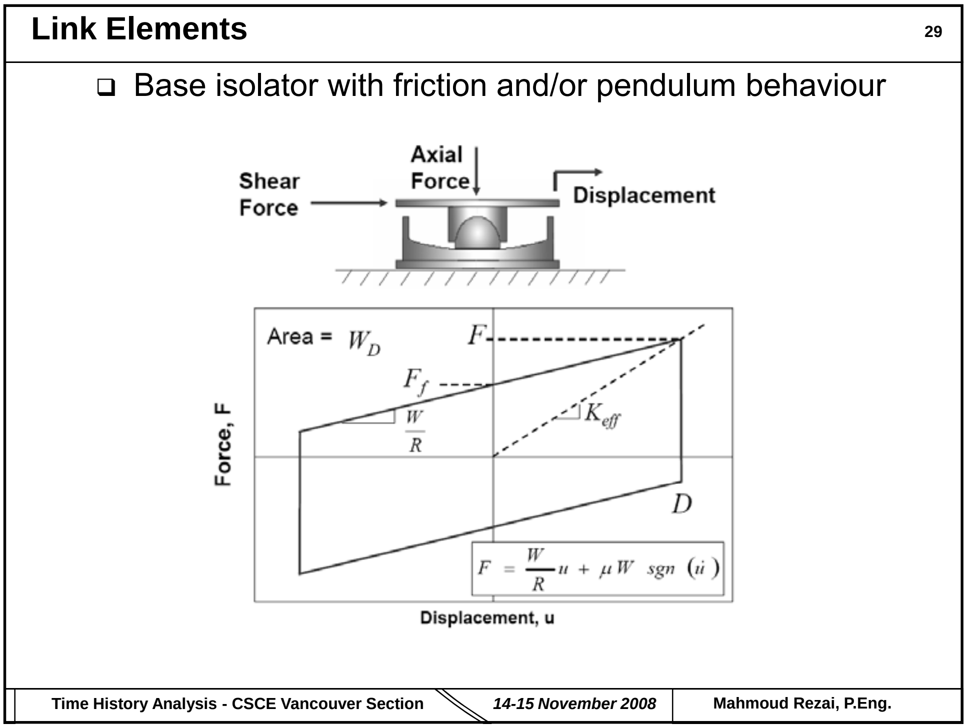
coupled friction properties for the two shear deformations, post-slip stiffness in the shear directions due the pendulum radii of the slipping surfaces, gap behaviour in the axial direction, and linear effective-stiffness properties for the three moment deformations.







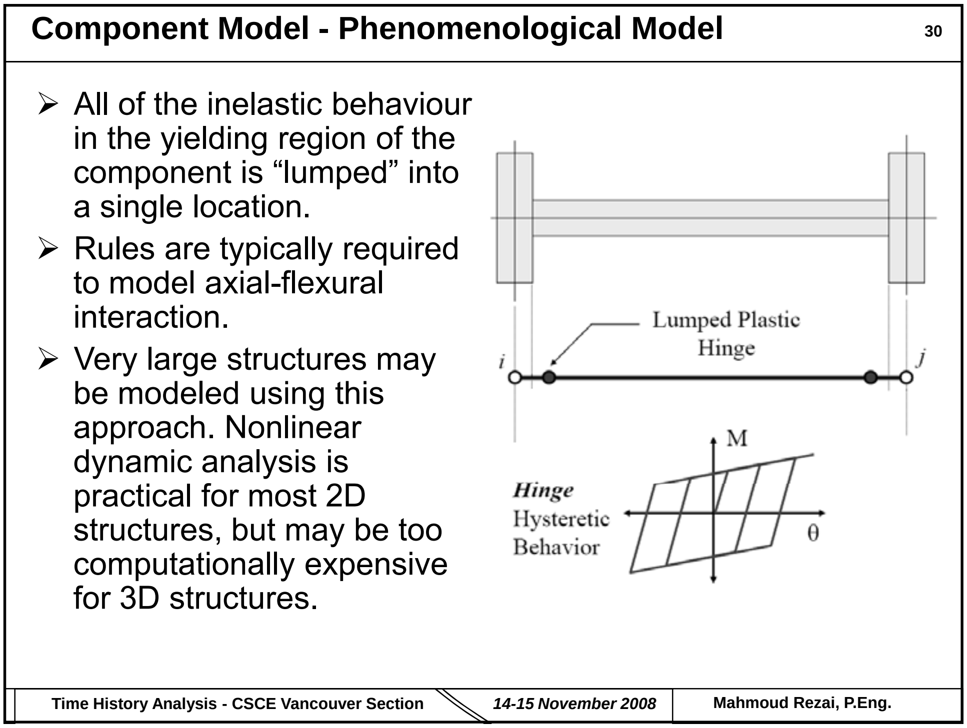
Concave Plate and Slider for FPS Bridge Bearing
- Seismic retrofit of Benicia-Martinez Bridge, San Francisco, CA
- 7.5 to 13 ft diameters
- Displ. Capacity of 13 ft bearings = +/- 4.3 ft



## Link Elements

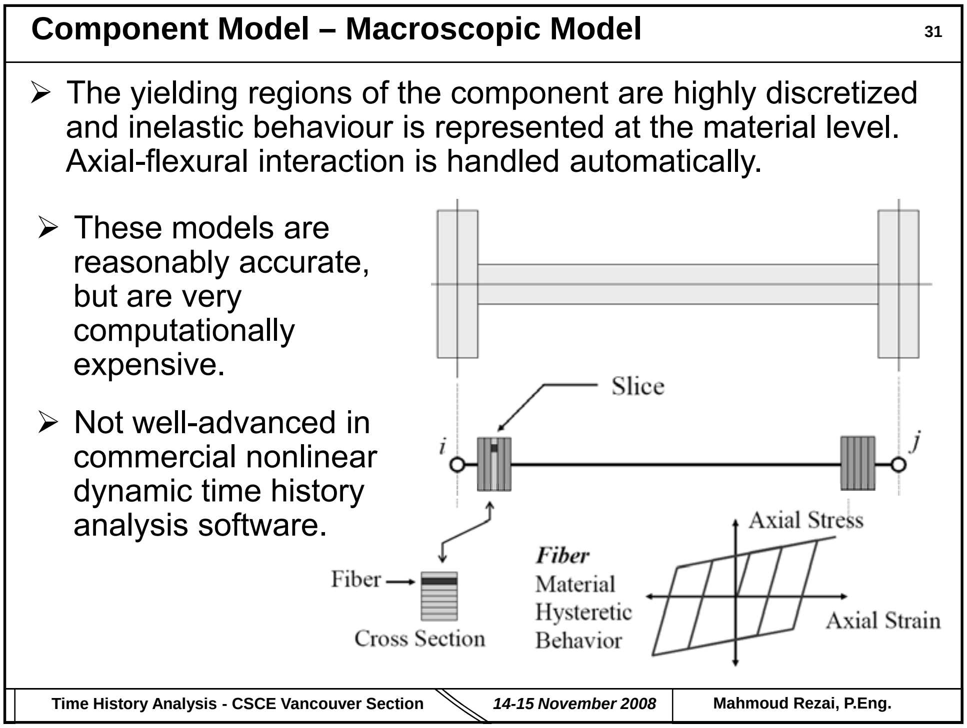
- Base isolator with friction and/or pendulum behaviour

$$F = \frac{W}{R}u + \mu W \; sgn\;(\dot{u})$$



## Component Model - Phenomenological Model

- All of the inelastic behaviour in the yielding region of the component is “lumped” into a single location.
- Rules are typically required to model axial-flexural interaction.
- Very large structures may be modeled using this approach. Nonlinear dynamic analysis is practical for most 2D structures, but may be too computationally expensive for 3D structures.

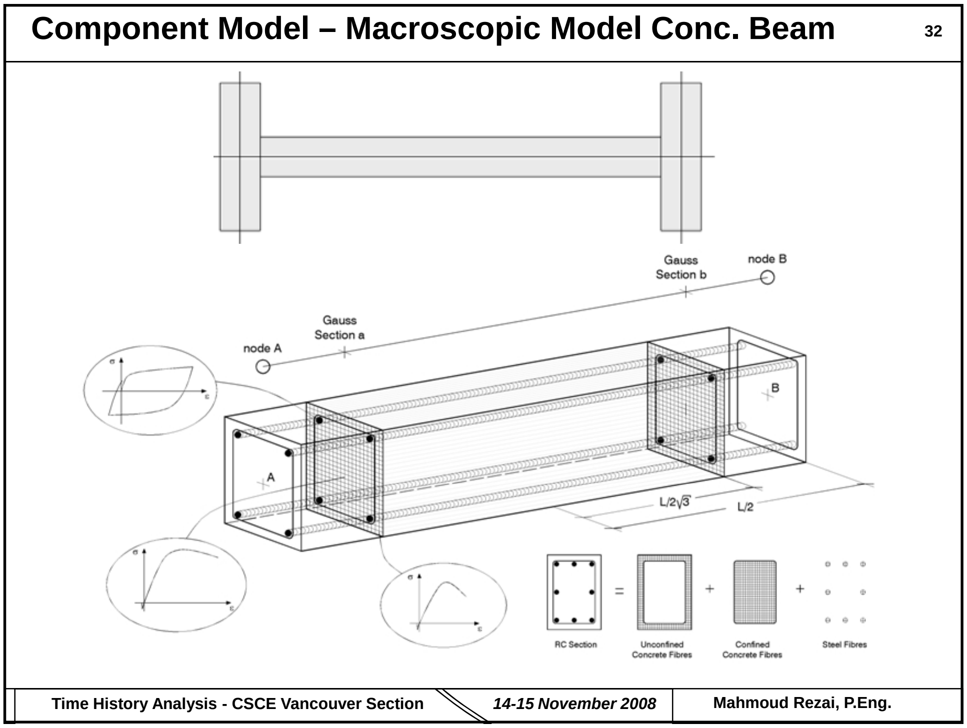


## Component Model – Macroscopic Model

- The yielding regions of the component are highly discretized and inelastic behaviour is represented at the material level. Axial-flexural interaction is handled automatically.

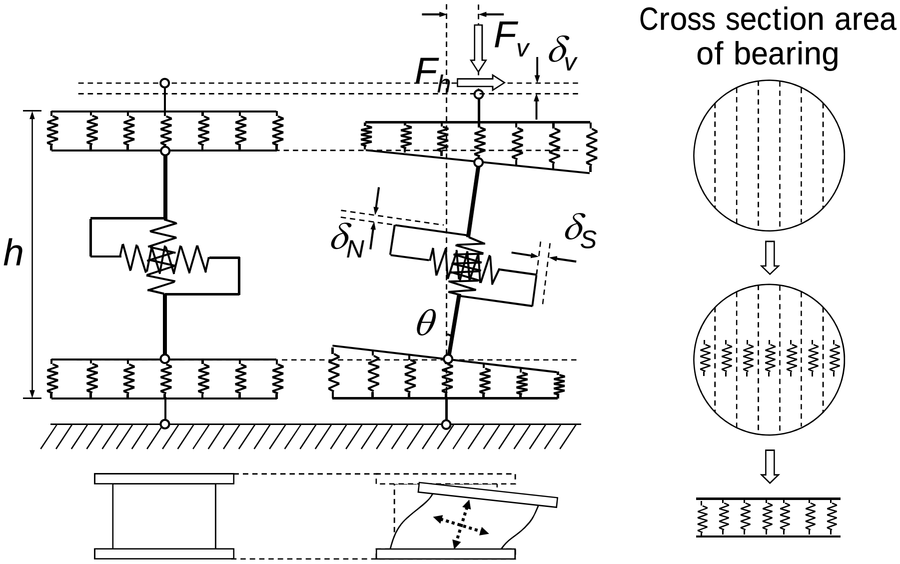
- These models are reasonably accurate, but are very computationally expensive.

- Not well-advanced in commercial nonlinear dynamic time history analysis software.



## Component Model – Macroscopic Model Conc. Beam

## Component Model – Macroscopic Model Isolator



Cross section area of bearing

---

## Component Model – Isolator Modelling – Shear Behaviour

P = 0 kN

**SAP2000**

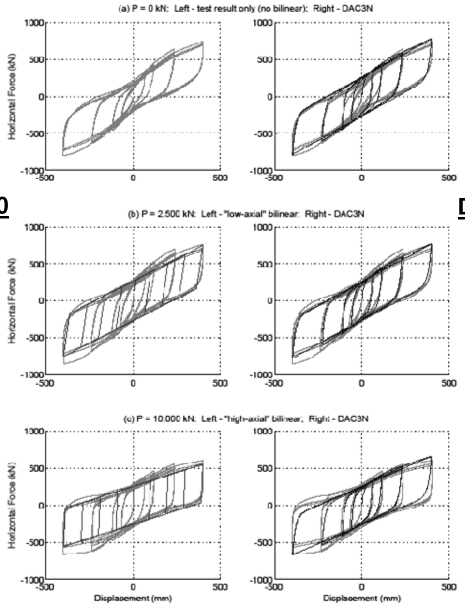
**DAC3N**

P = 2,500 kN

**Low Axial Load**

**High Axial Load**

P = 10,000 kN



## Performing Time History Analysis

1) Develop Linear Elastic Model, without P-Delta Effects
    a) Mode Shapes and Periods (Animate!)
    b) Independent Gravity Load Analysis
    c) Independent Lateral Load Analysis
2) Repeat Analysis but include P-Delta Effects
3) Revise model to include Inelastic Effects. Disable P-Delta
    a) Mode Shapes and Periods (Animate!)
    b) Independent Gravity Load Analysis
    c) Independent Lateral Load (Pushover) Analysis
    d) Gravity Load followed by Lateral Load
    e) Check effect of variable load step
4) Repeat Analysis but include P-Delta Effects

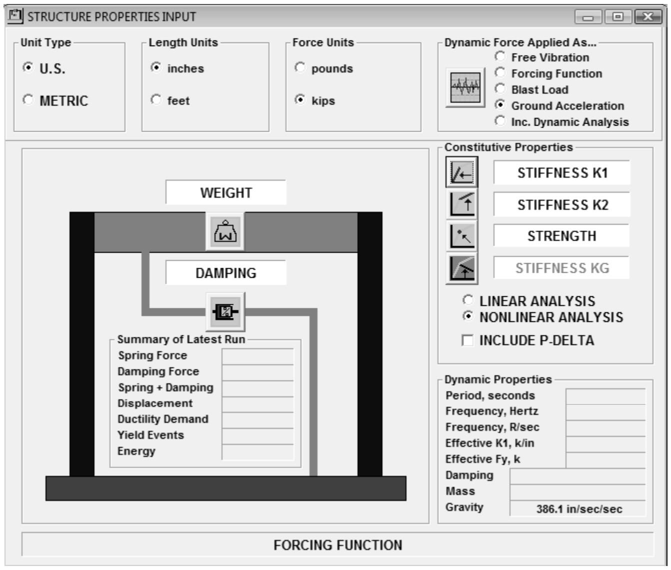
## …..Performing Time History Analysis

5) Run Linear Response History Analysis, disable P-Delta
    a) Harmonic Pulse followed by Free Vibration
    b) Full Ground Motion
    c) Check effect of variable time step
6) Repeat Analysis but include P-Delta Effects
7) Run Nonlinear Response History Analysis, disable P-Delta
    a) Harmonic Pulse followed by Free Vibration
    b) Full Ground Motion
    c) Check effect of variable time step
8) Repeat Analysis but include P-Delta Effects

## Nonlin Structural Analysis Software

- NONLIN is a program for computing the nonlinear dynamic response to simple structural systems. Capabilities include:
  - Inelastic response history analysis of SDOF and MDOF systems with a variety of hysteretic behaviours.
  - The program allows for easy input of ground motions, and provides a suite of ground motion analysis tools.
  - Systems may be analyzed incrementally for several ground motions, or may be analyzed for a single ground motion but with varying system parameters.
  - Other options include blast loading analysis, analysis under user specified dynamic loads, and evaluation of modal response characteristics for proportionally and nonproportionally damped systems.

## Nonlin Structural Analysis Software


STRUCTURE PROPERTIES INPUT

Unit Type: ● U.S. ○ METRIC

Length Units: ● inches ○ feet

Force Units: ○ pounds ● kips

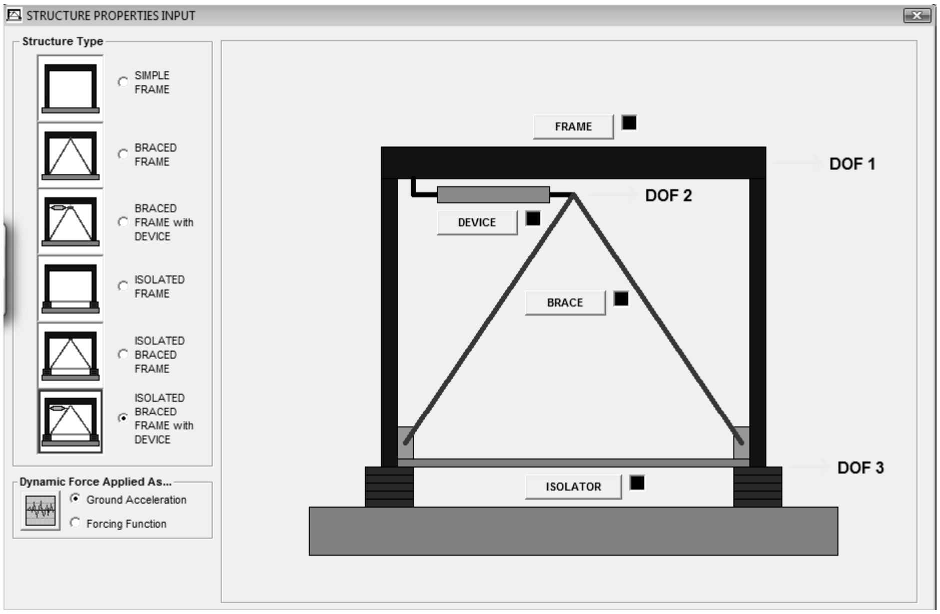
Dynamic Force Applied As...
○ Free Vibration
○ Forcing Function
○ Blast Load
● Ground Acceleration
○ Inc. Dynamic Analysis

Constitutive Properties
○ LINEAR ANALYSIS
● NONLINEAR ANALYSIS
☐ INCLUDE P-DELTA

Summary of Latest Run
Spring Force
Damping Force
Spring + Damping
Displacement
Ductility Demand
Yield Events
Energy

Dynamic Properties
Period, seconds
Frequency, Hertz
Frequency, R/sec
Effective K1, k/in
Effective Fy, k
Damping
Mass
Gravity 386.1 in/sec/sec

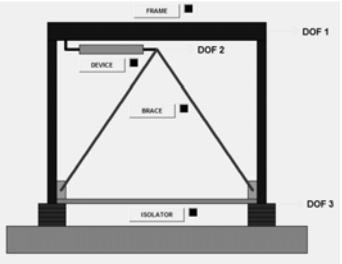
## Nonlin Structural Analysis Software

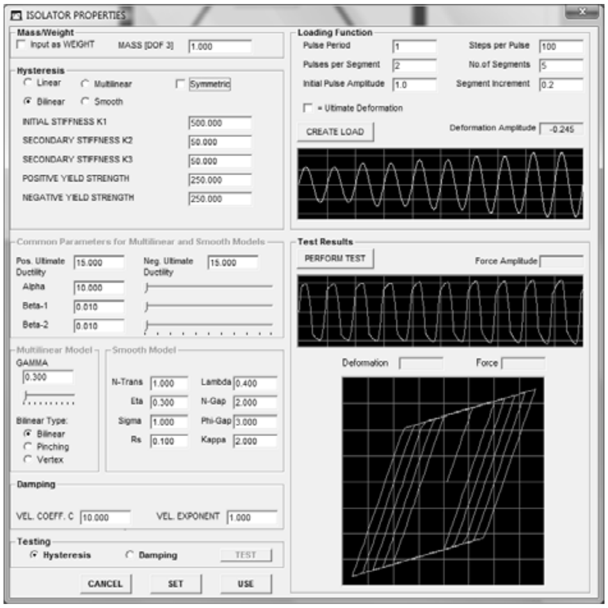


**Structure Type**
- ○ SIMPLE FRAME
- ○ BRACED FRAME
- ○ BRACED FRAME with DEVICE
- ○ ISOLATED FRAME
- ○ ISOLATED BRACED FRAME
- ● ISOLATED BRACED FRAME with DEVICE

**Dynamic Force Applied As...**
- ● Ground Acceleration
- ○ Forcing Function

## Nonlin – Multilinear Degrading Model





**Mass/Weight**
☐ Input as WEIGHT
MASS [DOF 3] 1.000

**Hysteresis**
○ Linear ○ Multilinear ☐ Symmetric
● Bilinear ○ Smooth

| | |
|---|---|
| INITIAL STIFFNESS K1 | 500.000 |
| SECONDARY STIFFNESS K2 | 50.000 |
| SECONDARY STIFFNESS K3 | 50.000 |
| POSITIVE YIELD STRENGTH | 250.000 |
| NEGATIVE YIELD STRENGTH | 250.000 |

**Common Parameters for Multilinear and Smooth Models**
Pos. Ultimate Ductility 15.000
Neg. Ultimate Ductility 15.000
Alpha 10.000
Beta-1 0.010
Beta-2 0.010

**Multilinear Model**
GAMMA 0.300
Bilinear Type:
- ● Bilinear
- ○ Pinching
- ○ Vertex

**Smooth Model**

| | | | |
|---|---|---|---|
| N-Trans | 1.000 | Lambda | 0.400 |
| Eta | 0.300 | N-Gap | 2.000 |
| Sigma | 1.000 | Phi-Gap | 3.000 |
| Rs | 0.100 | Kappa | 2.000 |

**Damping**
VEL. COEFF. C 10.000
VEL. EXPONENT 1.000

**Testing**
● Hysteresis ○ Damping

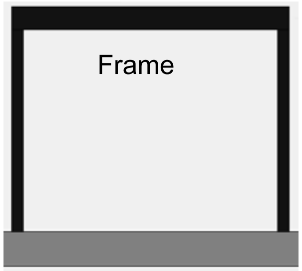
**Loading Function**

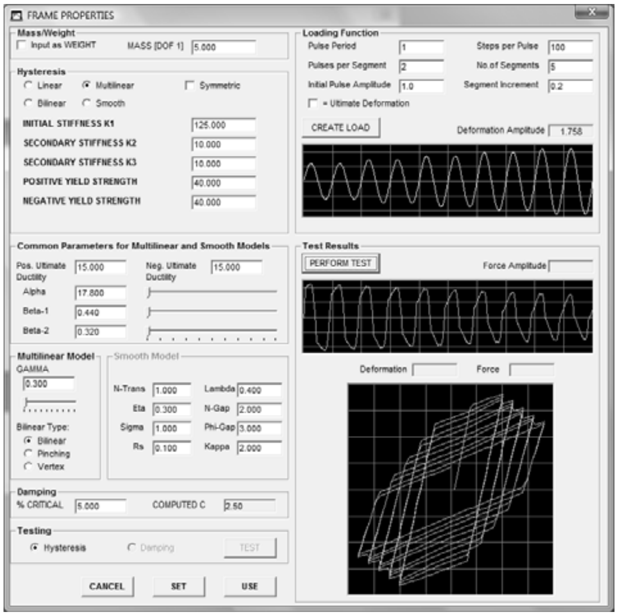
| | | | |
|---|---|---|---|
| Pulse Period | 1 | Steps per Pulse | 100 |
| Pulses per Segment | 2 | No.of Segments | 5 |
| Initial Pulse Amplitude | 1.0 | Segment Increment | 0.2 |

☐ = Ultimate Deformation
Deformation Amplitude -0.245

**Test Results**
Force Amplitude
Deformation
Force

## Nonlin – Multilinear Degrading Model





**Mass/Weight**
☐ Input as WEIGHT
MASS [DOF 1] 5.000

**Hysteresis**
○ Linear ● Multilinear ☐ Symmetric
○ Bilinear ○ Smooth
INITIAL STIFFNESS K1 125.000
SECONDARY STIFFNESS K2 10.000
SECONDARY STIFFNESS K3 10.000
POSITIVE YIELD STRENGTH 40.000
NEGATIVE YIELD STRENGTH 40.000

**Common Parameters for Multilinear and Smooth Models**
Pos. Ultimate Ductility 15.000
Neg. Ultimate Ductility 15.000
Alpha 17.800
Beta-1 0.440
Beta-2 0.320

**Multilinear Model**
GAMMA 0.300
Bilinear Type:
● Bilinear
○ Pinching
○ Vertex

**Smooth Model**
N-Trans 1.000
Lambda 0.400
Eta 0.300
N-Gap 2.000
Sigma 1.000
Phi-Gap 3.000
Rs 0.100
Kappa 2.000

**Damping**
% CRITICAL 5.000
COMPUTED C 2.50

**Testing**
● Hysteresis ○ Damping

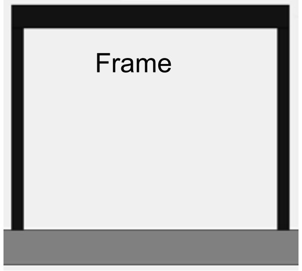
**Loading Function**
Pulse Period 1
Steps per Pulse 100
Pulses per Segment 2
No.of Segments 5
Initial Pulse Amplitude 1.0
Segment Increment 0.2
☐ = Ultimate Deformation

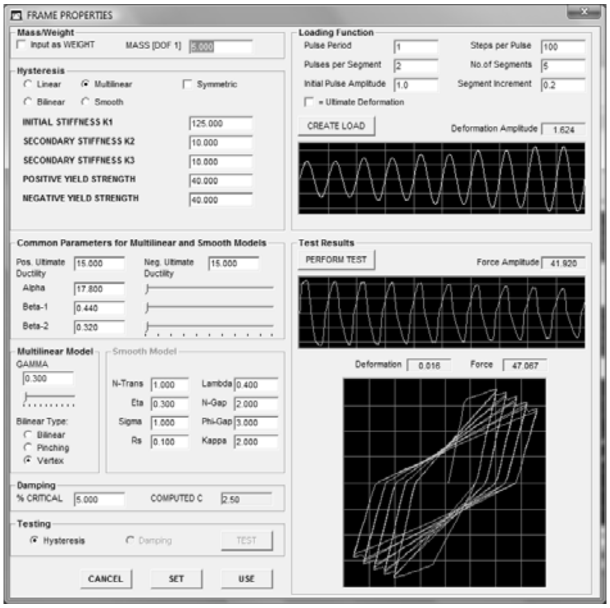
Deformation Amplitude 1.758

**Test Results**

Force Amplitude
Deformation
Force

## Nonlin – Multilinear Degrading Model with Vertex





**Mass/Weight**
☐ Input as WEIGHT
MASS [DOF 1] 5.000

**Hysteresis**
○ Linear ● Multilinear ☐ Symmetric
○ Bilinear ○ Smooth
INITIAL STIFFNESS K1 125.000
SECONDARY STIFFNESS K2 10.000
SECONDARY STIFFNESS K3 10.000
POSITIVE YIELD STRENGTH 40.000
NEGATIVE YIELD STRENGTH 40.000

**Common Parameters for Multilinear and Smooth Models**
Pos. Ultimate Ductility 15.000
Neg. Ultimate Ductility 15.000
Alpha 17.800
Beta-1 0.440
Beta-2 0.320

**Multilinear Model**
GAMMA 0.300
Bilinear Type:
○ Bilinear
○ Pinching
● Vertex

**Smooth Model**
N-Trans 1.000
Lambda 0.400
Eta 0.300
N-Gap 2.000
Sigma 1.000
Phi-Gap 3.000
Rs 0.100
Kappa 2.000

**Damping**
% CRITICAL 5.000
COMPUTED C 2.50

**Testing**
● Hysteresis ○ Damping

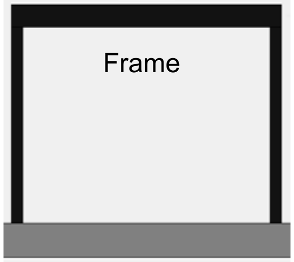
**Loading Function**
Pulse Period 1
Steps per Pulse 100
Pulses per Segment 2
No.of Segments 5
Initial Pulse Amplitude 1.0
Segment Increment 0.2
☐ = Ultimate Deformation

Deformation Amplitude 1.624

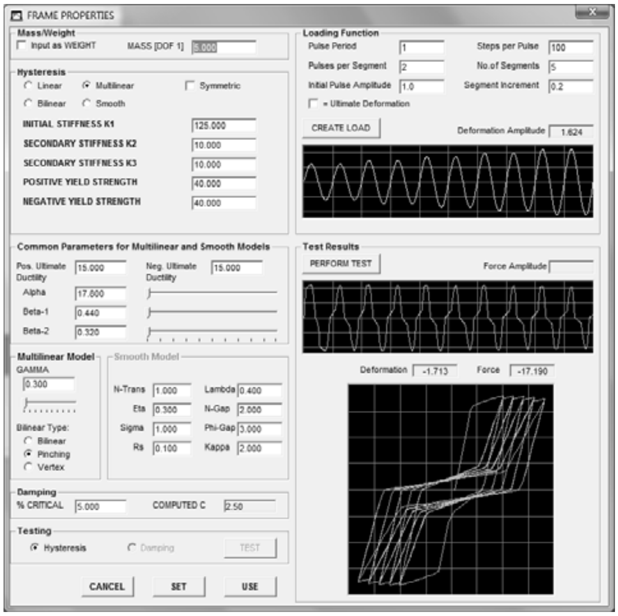
**Test Results**

Force Amplitude 41.920
Deformation 0.016
Force 47.067

## Nonlin – Multilinear Pinching Model



Pinching model is used for example if a reinforced concrete section is subjected to high shear stress reversals, or if the slippage of the reinforcement within the anchorage area occurs; as a result the force-deflection curve exhibits a pronounced pinching.



FRAME PROPERTIES

**Mass/Weight**
☐ Input as WEIGHT
MASS [DOF 1]: 5.000

**Hysteresis**
○ Linear ● Multilinear ☐ Symmetric
○ Bilinear ○ Smooth
INITIAL STIFFNESS K1: 125.000
SECONDARY STIFFNESS K2: 10.000
SECONDARY STIFFNESS K3: 10.000
POSITIVE YIELD STRENGTH: 40.000
NEGATIVE YIELD STRENGTH: 40.000

**Common Parameters for Multilinear and Smooth Models**
Pos. Ultimate Ductility: 15.000
Neg. Ultimate Ductility: 15.000
Alpha: 17.000
Beta-1: 0.440
Beta-2: 0.320

**Multilinear Model**
GAMMA: 0.300
Bilinear Type:
○ Bilinear
● Pinching
○ Vertex

**Smooth Model**
N-Trans: 1.000
Eta: 0.300
Sigma: 1.000
Rs: 0.100
Lambda: 0.400
N-Gap: 2.000
Phi-Gap: 3.000
Kappa: 2.000

**Damping**
% CRITICAL: 5.000
COMPUTED C: 2.50

**Testing**
● Hysteresis ○ Damping

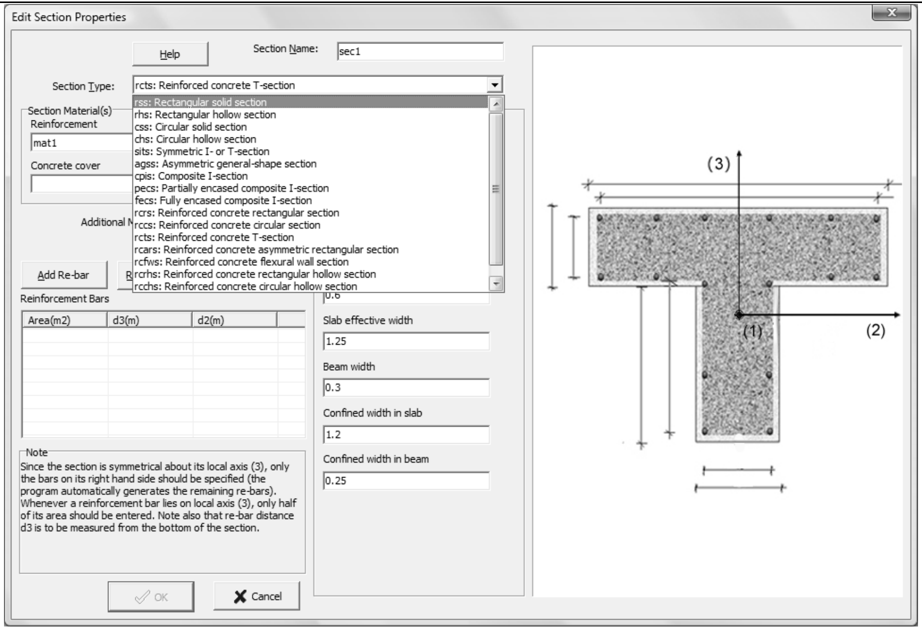
**Loading Function**
Pulse Period: 1
Steps per Pulse: 100
Pulses per Segment: 2
No.of Segments: 5
Initial Pulse Amplitude: 1.0
Segment Increment: 0.2
☐ = Ultimate Deformation
Deformation Amplitude: 1.624

**Test Results**
Force Amplitude:
Deformation: -1.713
Force: -17.190

## SeismoStruct Structural Analysis Software

- SeismoStruct is a Finite Elements package capable of predicting the large displacement behaviour of space frames under static or dynamic loading, taking into account both geometric nonlinearities and material inelasticity. Some of its analytical features are:
  - 7 Analysis Types, such as Pushover Analysis, Nonlinear Dynamic Analysis, Incremental Dynamic Analysis, Displacement-based Adaptive Pushover, etc…
  - 8 Element Types, such as nonlinear fibre beam-column element, nonlinear truss element, nonlinear infill panel element, nonlinear link elements, etc…
  - 11 Material Models, such as nonlinear concrete models, high-strength nonlinear concrete model, nonlinear steel models, FRP-confined nonlinear concrete model, SMA nonlinear model, etc…
  - 16 hysteretic models, such as linear/bilinear/trilinear kinematic hardening response models, gap-hook models, soil-structure interaction model, Takeda model, Ramberg-Osgood model, etc…

## Section Properties



Section Name: sec1

Section Type: rcts: Reinforced concrete T-section

Section Material(s)
Reinforcement: mat1
Concrete cover:

0.6

Slab effective width: 1.25

Beam width: 0.3

Confined width in slab: 1.2

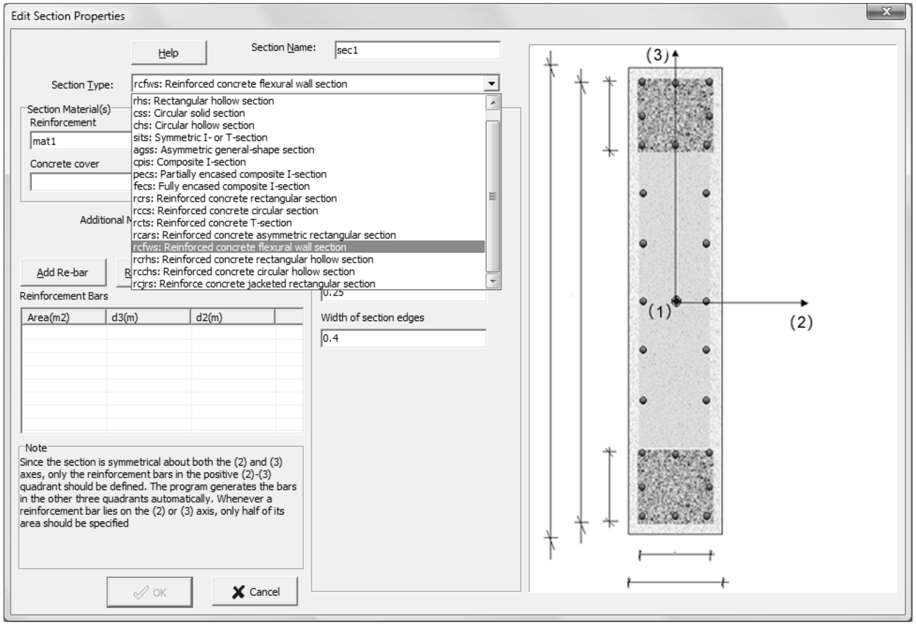
Confined width in beam: 0.25

Reinforcement Bars

| Area(m2) | d3(m) | d2(m) |
|---|---|---|

Note
Since the section is symmetrical about its local axis (3), only the bars on its right hand side should be specified (the program automatically generates the remaining re-bars). Whenever a reinforcement bar lies on local axis (3), only half of its area should be entered. Note also that re-bar distance d3 is to be measured from the bottom of the section.

## Section Properties



Section Name: sec1

Section Type: rcfws: Reinforced concrete flexural wall section

Section Material(s)
Reinforcement: mat1
Concrete cover:

0.25

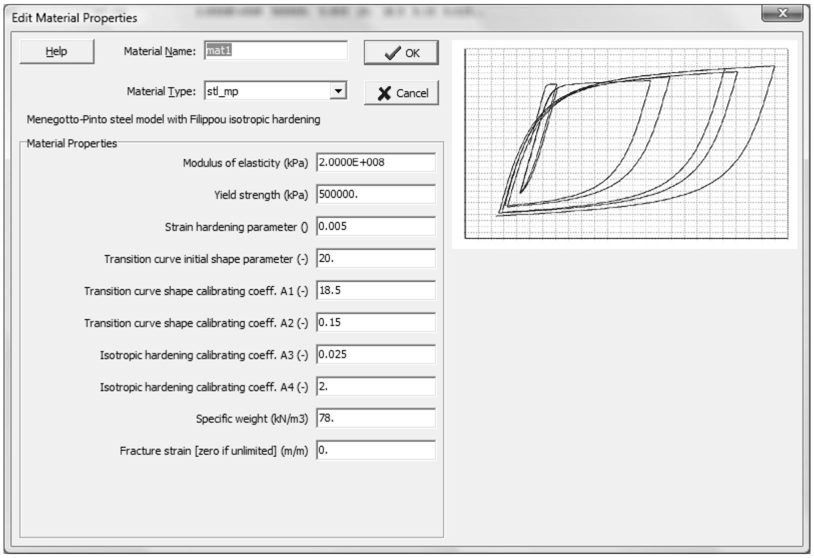
Width of section edges: 0.4

Reinforcement Bars

| Area(m2) | d3(m) | d2(m) |
|---|---|---|

Note
Since the section is symmetrical about both the (2) and (3) axes, only the reinforcement bars in the positive (2)-(3) quadrant should be defined. The program generates the bars in the other three quadrants automatically. Whenever a reinforcement bar lies on the (2) or (3) axis, only half of its area should be specified

## Material Model: Steel with Isotropic Hardening



Material Name: mat1

Material Type: stl_mp

Menegotto-Pinto steel model with Filippou isotropic hardening

Modulus of elasticity (kPa) 2.0000E+008

Yield strength (kPa) 500000.

Strain hardening parameter () 0.005

Transition curve initial shape parameter (-) 20.

Transition curve shape calibrating coeff. A1 (-) 18.5

Transition curve shape calibrating coeff. A2 (-) 0.15

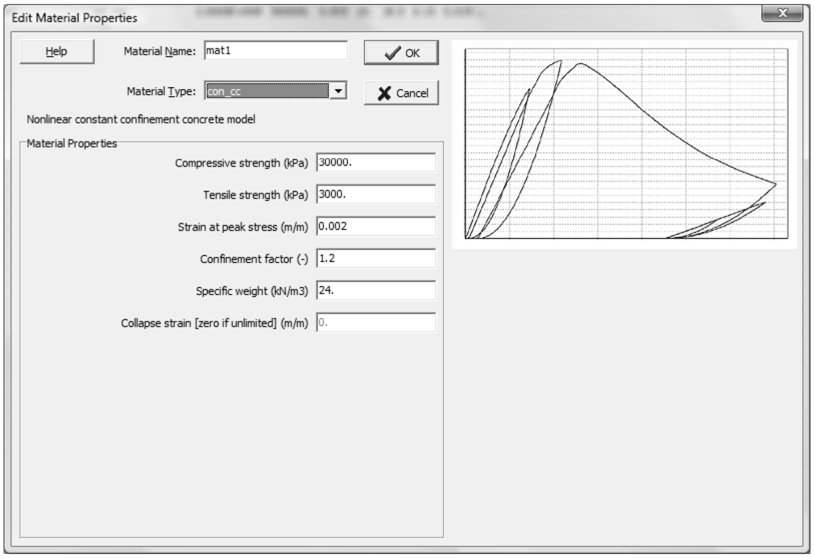
Isotropic hardening calibrating coeff. A3 (-) 0.025

Isotropic hardening calibrating coeff. A4 (-) 2.

Specific weight (kN/m3) 78.

Fracture strain [zero if unlimited] (m/m) 0.

## Material Model: Constant Confinement Concrete



Material Name: mat1

Material Type: con_cc

Nonlinear constant confinement concrete model

Compressive strength (kPa) 30000.

Tensile strength (kPa) 3000.

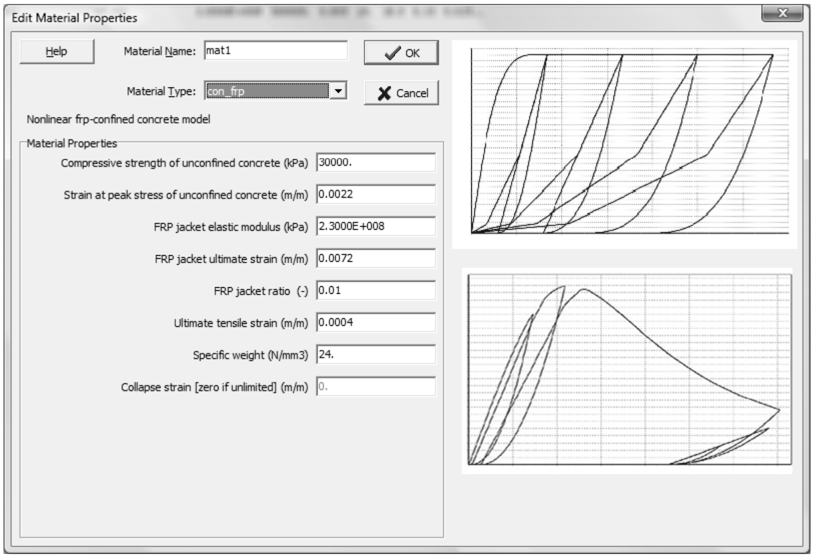
Strain at peak stress (m/m) 0.002

Confinement factor (-) 1.2

Specific weight (kN/m3) 24.

Collapse strain [zero if unlimited] (m/m) 0.

## Material Model: FRP-Confined Concrete



Material Name: mat1

Material Type: con_frp

Nonlinear frp-confined concrete model

Material Properties

Compressive strength of unconfined concrete (kPa) 30000.

Strain at peak stress of unconfined concrete (m/m) 0.0022

FRP jacket elastic modulus (kPa) 2.3000E+008

FRP jacket ultimate strain (m/m) 0.0072

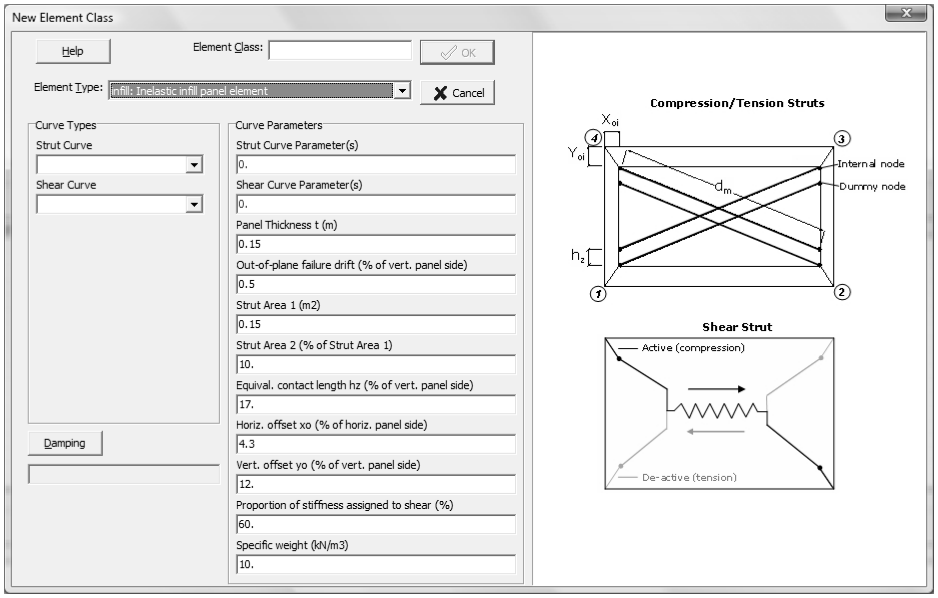
FRP jacket ratio (-) 0.01

Ultimate tensile strain (m/m) 0.0004

Specific weight (N/mm3) 24.

Collapse strain [zero if unlimited] (m/m) 0.

## Inelastic Infill Panel Element



Element Class:

Element Type: infill: Inelastic infill panel element

Curve Parameters

Strut Curve Parameter(s) 0.

Shear Curve Parameter(s) 0.

Panel Thickness t (m) 0.15

Out-of-plane failure drift (% of vert. panel side) 0.5

Strut Area 1 (m2) 0.15

Strut Area 2 (% of Strut Area 1) 10.

Equival. contact length hz (% of vert. panel side) 17.

Horiz. offset xo (% of horiz. panel side) 4.3

Vert. offset yo (% of vert. panel side) 12.

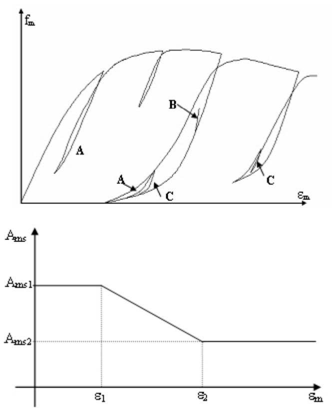
Proportion of stiffness assigned to shear (%) 60.

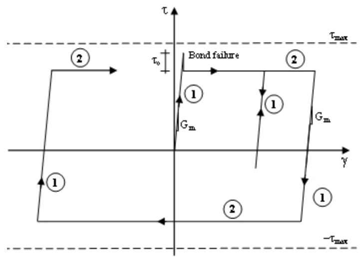
Specific weight (kN/m3) 10.

## Infill Panel Element – Hysteretic Models

Axial

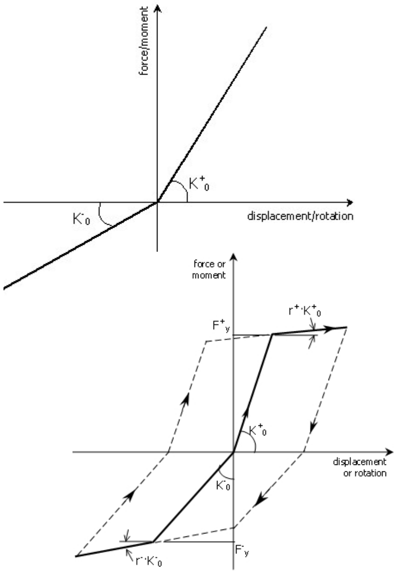
Shear





This is the masonry infill strut model, developed and implemented in SeismoStruct to be used (almost exclusively) in association with the infill panel element.

## Other Types of Hysteretic Models



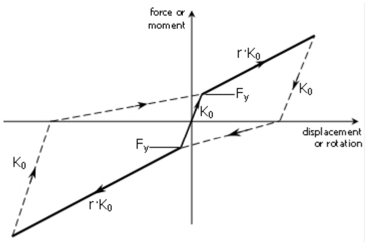
Asymmetric Linear Curve:

This is a curve employed to model idealized linear asymmetric behaviour, e.g. soil/foundation flexibility.

Asymmetric Bilinear Curve:

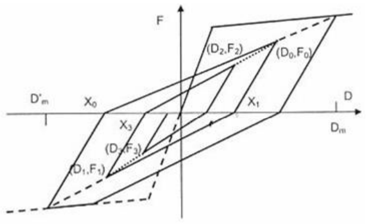
This is a curve frequently employed to model idealized asymmetric elastic-plastic behaviour. An isotropic hardening rule is adopted.

## Other Types of Hysteretic Models



Symmetric Bilinear Curve:

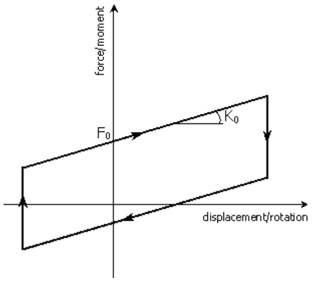
This is a curve frequently employed to model idealised symmetric elastic-plastic behaviour. An isotropic hardening rule is adopted.



Simplified Bilinear Takeda Curve:

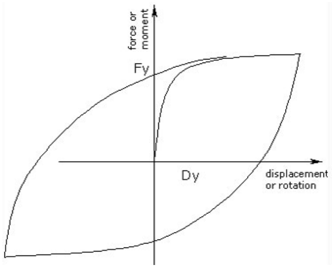
This model consists of a bilinear simplification of the original trilinear model proposed by Takeda. Used to model flexural hysteretic behaviour of reinforced concrete by changing stiffness parameters.

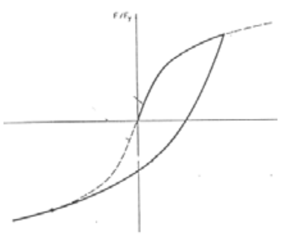
## Other Types of Hysteretic Models



Plastic Curve:

This is a curve frequently employed to model idealized rigid-plastic behaviour, sliding bearings, FPS (friction pendulum system) isolating devices, hydraulic or lead-extrusion dampers, and so on. A kinematic hardening rule is adopted.

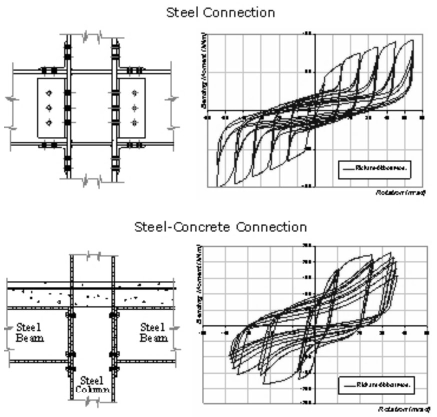




Ramberg-Osgood Curve:

This model dissipates energy even if the ductility factor is less than one.

## Other Types of Hysteretic Models

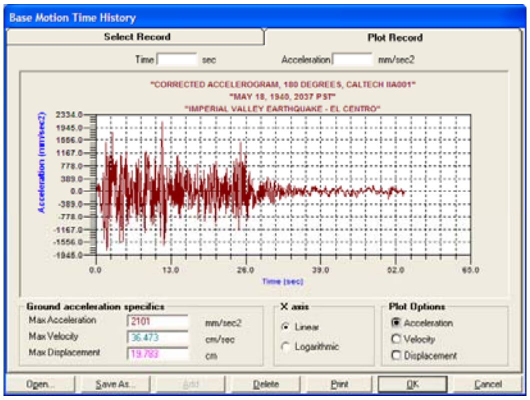


<u>Modified Richard-Abbott Curve:</u>

The model is very flexible, being capable of modelling all sorts of steel and composite connections (e.g. welded-flange bolted-web connection, extended end-plate connection, flush end-plate connection, angle connection, etc.), for as long as the model parameters are calibrated accordingly.

## S-FRAME Structural Analysis Software

- S-FRAME Supports two types of Time History Analysis
  - ☐ Nodal excitation; e.g. cyclic loading from machinery, wind, etc…
  - ☐ Base motion; for modeling earthquake loads. A library of earthquake records is available which is installed with the program, or the user may define their own synthetic curve(s).



Ground acceleration specifics
Max Acceleration 2101 mm/sec2
Max Velocity 36.473 cm/sec
Max Displacement 19.793 cm

X axis
● Linear
○ Logarithmic

Plot Options
● Acceleration
○ Velocity
○ Displacement

## S-FRAME Structural Analysis Software

- S-FRAME's Time History Analysis uses the direct integration method.
- Both Constant and Variable time-step integration are supported.
- All time steps can be combined with a static loadcase or combination.
- Viscous dampers are supported.

## Modal Damping

- Assuming the damping matrix is of classical form, the coupled equations can be uncoupled via a modal transformation that employs the mode shape matrix.

$$M\ddot{v} + C\dot{v} + Kv = -MR\ddot{v}_g$$

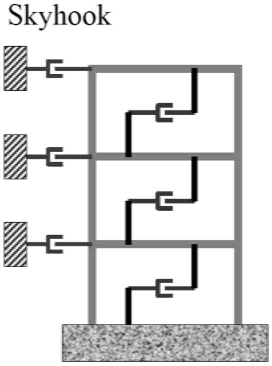
$$v = \Phi y$$

$$\ddot{y}_i + 2\underset{\uparrow}{\xi_i}\omega_i\dot{y}_i + \omega_i^{\ 2} y_i = \Gamma_i\ddot{v}_g$$

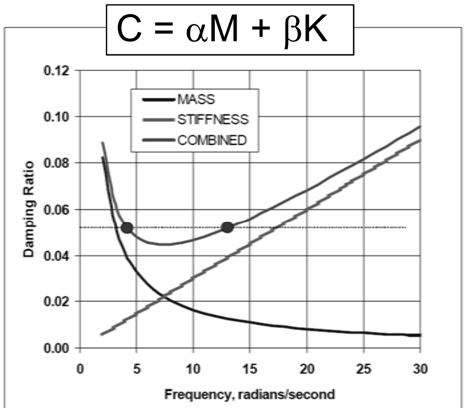
Specify modal damping values directly

## Rayleigh Proportional Damping

- The physical interpretation of Rayleigh damping is that it corresponds to both “skyhook” and interstorey dampers. Note that Rayleigh damping formulations are commonly available in structural analysis software and thus it can be a very convenient, although not necessarily accurate, approach to accounting for added damping.



$C = \alpha M + \beta K$



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## Rayleigh Proportional Damping

Select Damping value in two modes, $\xi_k$ and $\xi_n$

Compute Coefficients $\alpha$ and $\beta$:

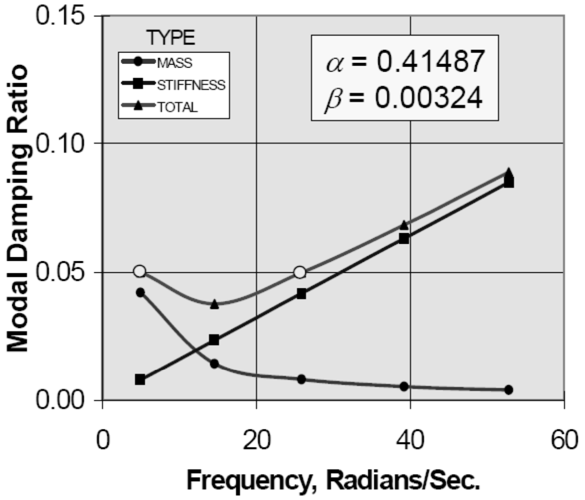
$$\begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = 2\frac{\omega_k \omega_n}{\omega_n^2 - \omega_k^2} \begin{bmatrix} \omega_n & -\omega_k \\ -1/\omega_n & 1/\omega_k \end{bmatrix} \begin{Bmatrix} \xi_k \\ \xi_n \end{Bmatrix}$$

Form Damping Matrix $C = \alpha M + \beta K$

Damping in any other Mode $m$:

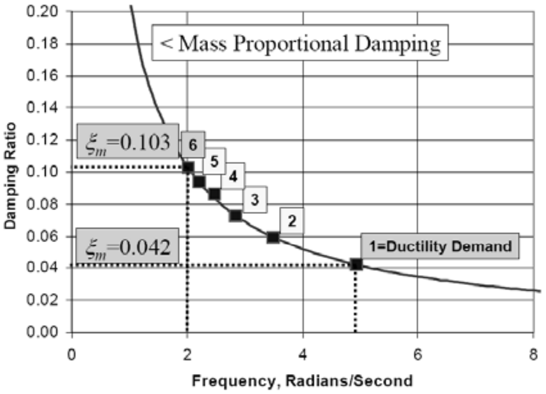
$$\xi_m = 0.5 \begin{bmatrix} \frac{1}{\omega_m} & \omega_m \end{bmatrix} \begin{Bmatrix} \alpha \\ \beta \end{Bmatrix}$$

## Rayleigh Proportional Damping



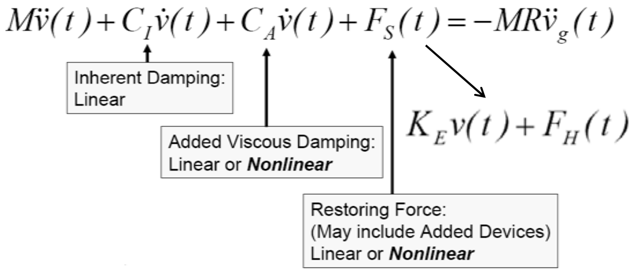
## Rayleigh Proportional Damping

- Loss of stiffness results in frequency shift, and thus higher mass proportional damping.
- Also, the higher the ductility demand, the greater the apparent increase in damping.



## Fast Nonlinear Analysis (FNA)

- The FNA Method is designed for the static and dynamic analysis of nonlinear structures with a limited number of predefined nonlinear elements.
- Advantages of The FNA Method:
  - ❑ The method can be used for both static and dynamic nonlinear analyses.
  - ❑ The method is very efficient and requires a small amount of additional computer time as compared to linear analysis.



$$M\ddot{v}(t)+C_I\dot{v}(t)+C_A\dot{v}(t)+F_S(t)=-MR\ddot{v}_g(t)$$

$$K_E v(t)+F_H(t)$$

Inherent Damping: Linear

Added Viscous Damping: Linear or ***Nonlinear***

Restoring Force: (May include Added Devices) Linear or ***Nonlinear***

## Fast Nonlinear Analysis (FNA)

- First, the added nonlinear force vector and the restoring force vector (if it is nonlinear) are moved to the right-hand side.
- The physical coordinates are then transformed to a new set of coordinates through a transformation that employs stiffness and mass orthogonal load-dependent Ritz (LDR) vectors.
- Note that the LDR Vectors are a linear combination of the exact Eigenvectors plus, the static displacement vectors.

$$\underbrace{M\ddot{v}(t)+C_I\dot{v}(t)+K_E v(t)}_{\text{Linear Terms}}=-MR\ddot{v}_g(t)\underbrace{-F_H(t)-C_A\dot{v}(t)}_{\text{Nonlinear Terms}}$$

Transform Coordinates: $v(t)=\Phi y(t)$

## Fast Nonlinear Analysis (FNA)

- The left-hand side of the resulting equation will be uncoupled. In the transformation process, the inherent damping is represented by modal damping ratios and the forces associated with the discrete added nonlinear element are included in the right-hand side nonlinear force vector. Finally, iteration is performed on the unbalanced right-hand side forces.

Transform Coordinates: $v(t) = \Phi y(t)$

Orthogonal basis of Ritz vectors:
Number of vectors << $N$

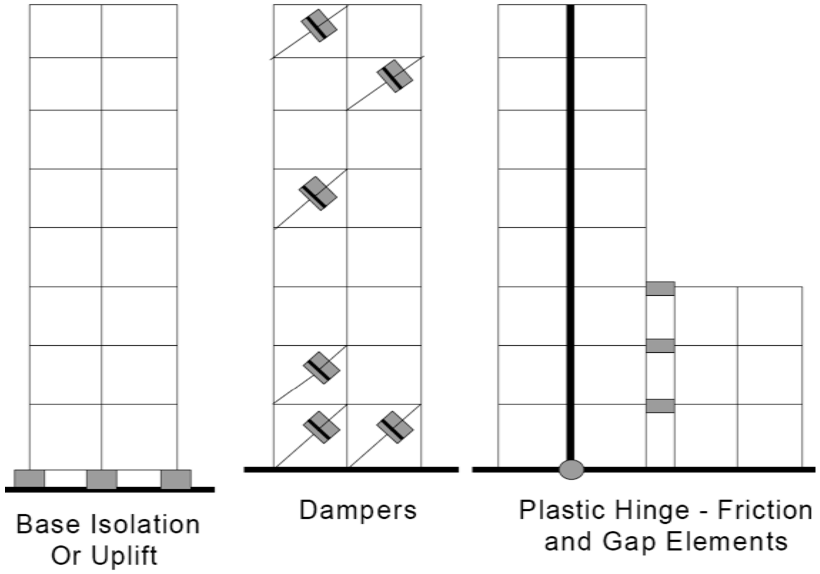
Apply Transformation:

$$\underbrace{\widetilde{M}\ddot{y}(t)+\widetilde{C}_I\dot{y}(t)+\widetilde{K}_E y(t)}_{\text{Uncoupled}} = \underbrace{-\Phi^T MR\ddot{v}_g(t)-\Phi^T F_H(t)-\widetilde{C}_A\dot{y}(t)}_{\text{Coupled}}$$

## Fast Nonlinear Analysis (FNA)

- It is up to analyst to determine if the Modes calculated by the program are adequate to represent the time-history response to the applied load. You should check:
  - That enough Modes have been computed
  - That the Modes cover an adequate frequency range
  - That the dynamic load (mass) participation mass ratios are adequate for the load cases and/or Acceleration Loads being applied
  - That the modes shapes adequately represent all desired deformations

## Examples Where FNA Works – Localized Plasticity



## Direct Integration/Explicit Nonlinear Analysis

- One method to solve dynamic equation of motion is to perform a step-by-step analysis in which the fully coupled system of N equations is explicitly integrated (e.g., using a Newmark solver). In this case, the inherent damping matrix can be represented by a Rayleigh formulation.
- A variety of common methods are available for performing direct-integration time-history analysis, as well documented in standard textbooks.

$$M\ddot{v}(t)+C_I\dot{v}(t)+C_A\dot{v}(t)+F_S(t)=-MR\ddot{v}_g(t)$$

**Explicit integration of fully coupled equations:**

## Linear/Nonlinear Dynamic Response History Analysis

- FNA vs. Direct-integration: These are two different solution methods, each with advantages and disadvantages. Under ideal circumstances, both methods should yield the same results to a given problem.
- Time Steps:
  - ❑ FNA: time increment may be any sampling value that is deemed fine enough to capture the maximum response values. One-tenth of the time period of the highest mode is usually recommended; however, a larger value may give an equally accurate sampling if the contribution of the higher modes is small.
  - ❑ Direct Integration: Direct integration results are extremely sensitive to time-step size in a way that is not true for modal superposition. You should always run your direct-integration analyses with decreasing time-step sizes until the step size is small enough that results are no longer affected by it.

    For best results, use the smallest time step practical

## Acknowledgements

- Some of the images presented in these slides were obtained from web pages of the:
  - ❑ FEMA

    Instructional Material Complementing FEMA 451, Design Examples
  - ❑ Softwares presented,
  - ❑ Ductile Design of Steel Structures by M. Bruneau, C.M. Uang and A. Whittaker

# TIME HISTORY ANALYSIS

## LECTURE # 9

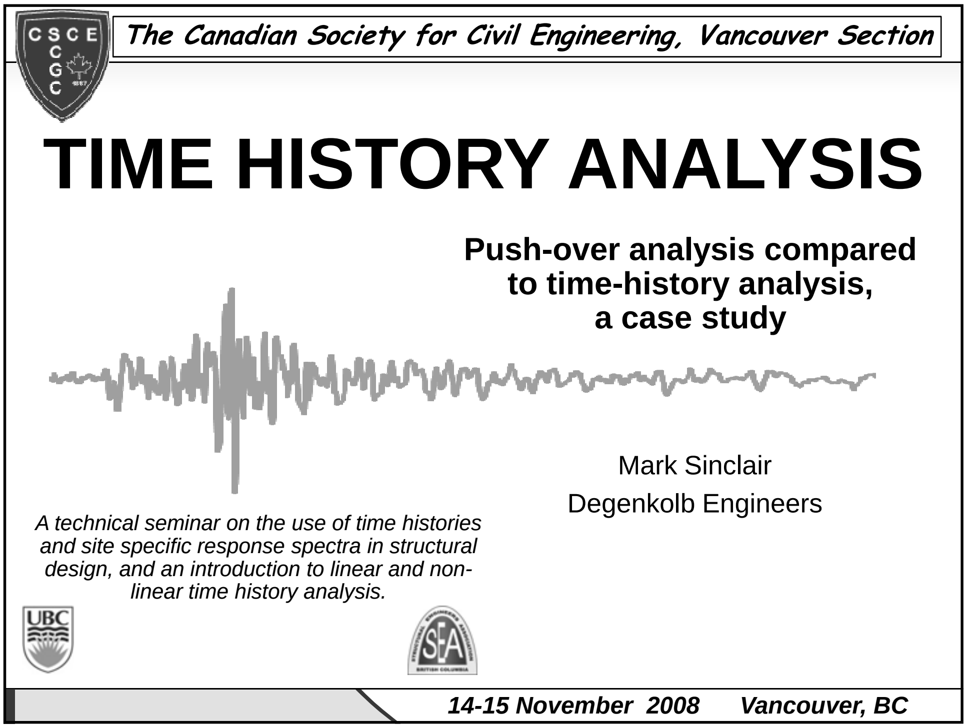
## Pushover Analysis-Compared to Time History Analysis, a Case of Study

**Mark Sinclair, SE**
**Degenkolb Engineers.**

Mark Sinclair is a registered Structural Engineer and Associate Principal at Degenkolb Engineers. He arrived in California in 1993 after first completing his undergraduate and graduate studies at Canterbury University in Christchurch, New Zealand.

He joined Degenkolb in 1999 after working for one of the major US-based suppliers of seismic isolation bearings. Mark is advancing Degenkolb's use of the latest engineering technology such as seismic isolation, energy dissipation, building instrumentation, and isolated products for data centers. His projects typically include application of these technologies in combination with analysis techniques such as nonlinear time-history analysis.

He serves as an internal company consultant to a wide variety of projects across the firm, and has 14 years of experience in research, design, testing, and supply of seismic isolation and energy dissipation systems.



*The Canadian Society for Civil Engineering, Vancouver Section*

# TIME HISTORY ANALYSIS

## Push-over analysis compared to time-history analysis, a case study

Mark Sinclair
Degenkolb Engineers

*A technical seminar on the use of time histories and site specific response spectra in structural design, and an introduction to linear and non-linear time history analysis.*

***14-15 November 2008 Vancouver, BC***

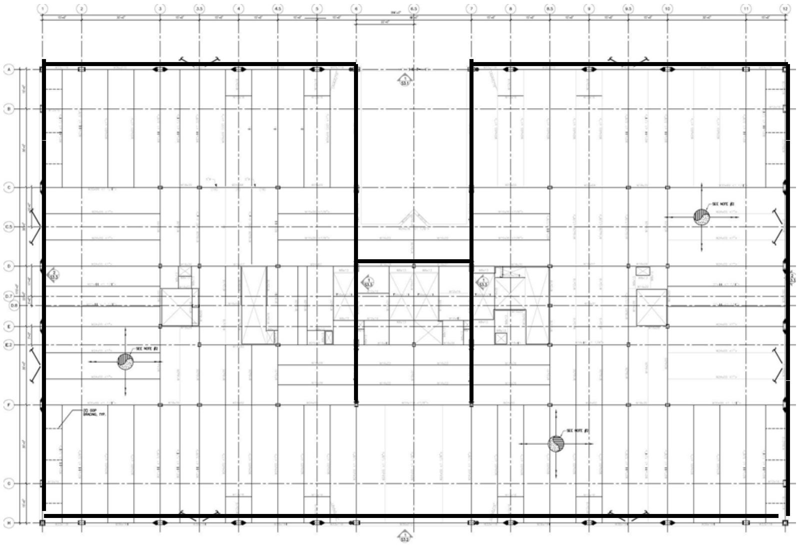
## Outline

- Building Description
- Design Criteria
- Existing Building Connections
- Study Phase Schemes
- Study Phase Pushover Analysis
- Selected Scheme
- Selected Scheme Analysis
- Retrofit Scheme Connections
- Connection Test Program
- Conclusions

## Building Description

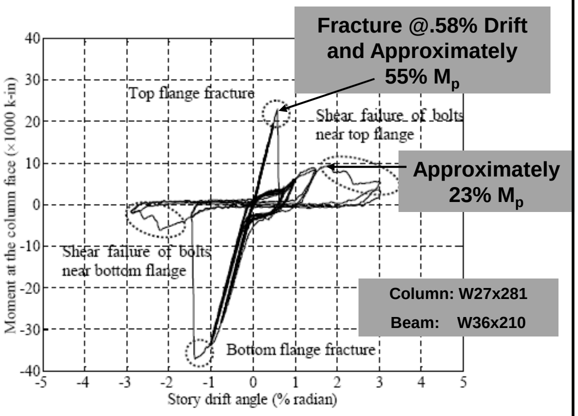
- 15 Stories Plus Basement
- Designed to 1988 UBC
- Steel SMRF
- Constructed in 1991
- Parking Levels 2-5
- Atrium 6th Level and up
- GFRC Exterior
- Design Build Delivery

## Building Description- Continued



## Issues/ Challenges

- Poor Existing Connections
- W27 & Square Columns
- Heavy W36x Beams
- Drift Criteria
- Large Atrium
- Occupied Facility (TMC)
- Limited As-Builts (MEP)
- Budget



**Fracture @.58% Drift and Approximately 55% $M_p$**

**Approximately 23% $M_p$**

**Column: W27x281**
**Beam: W36x210**

Figure 3-37: Moment at the column face versus story drift angle for Specimen EC03

## Design Criteria

- State Risk Level III Definition
  - Building: Minor structural damage, repairable.
    Moderate non-structural damage, extensive repair
  - Systems: Disruption of systems for days to months.
  - Occupancy: Return within weeks with minor disruptions
  - **FEMA 351 Default Drift Levels**
    - Global = 1.8%
    - Local = 1.1% to 1.3%
  - **FEMA 351 Appendix A calculations**
    - Global = 2.4%
    - Local = 1.8% to 2.0%
  - **Peer Review limit 1.5% +/-**

## Previous Testing Existing Connection (UCB)

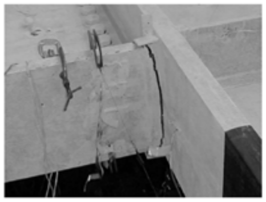


Figure 3-31: Beam top flange fracture during 0.75% drift cycle in Specimen EC03

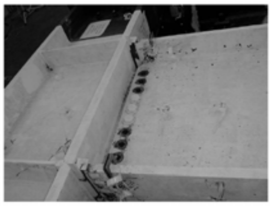
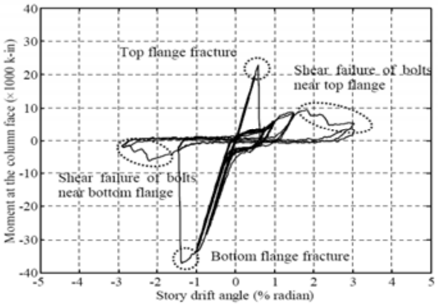


Figure 3-36: Bolts failure at 3% drift cycle in Specimen EC03

Figure 2-9: Photograph of test fixture for Specimen EC03

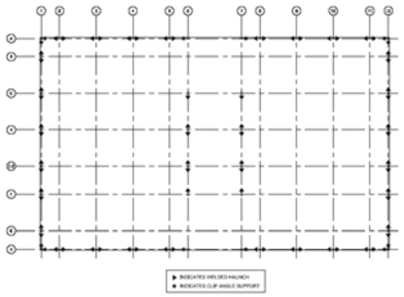
**Beam:** W36x210
**Column**: W27x281

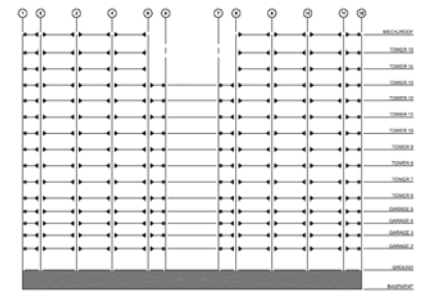


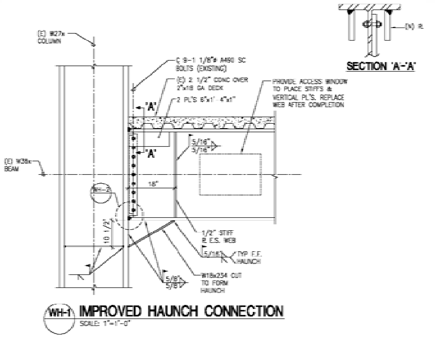
## Study Phase: Four Schemes Evaluated

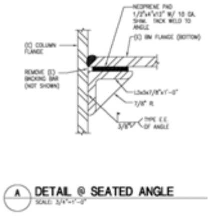
- (1) Connection Modification
- (2) Connection Modification plus Dampers
- (3) Buckling Restrained Braced Frames
- (4) Seismic Isolation

## Scheme (1): Connection Modification

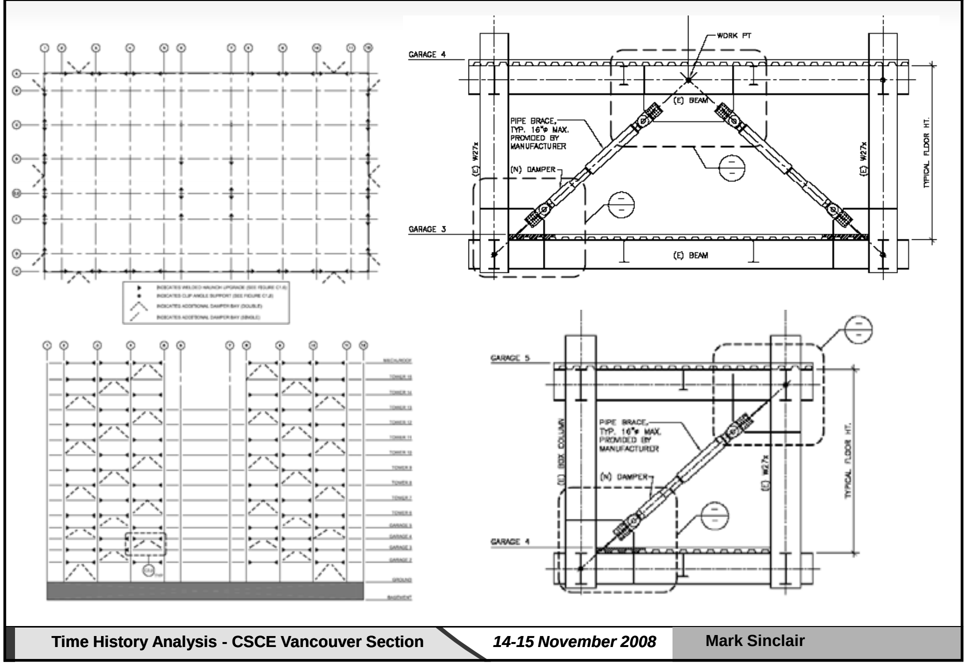




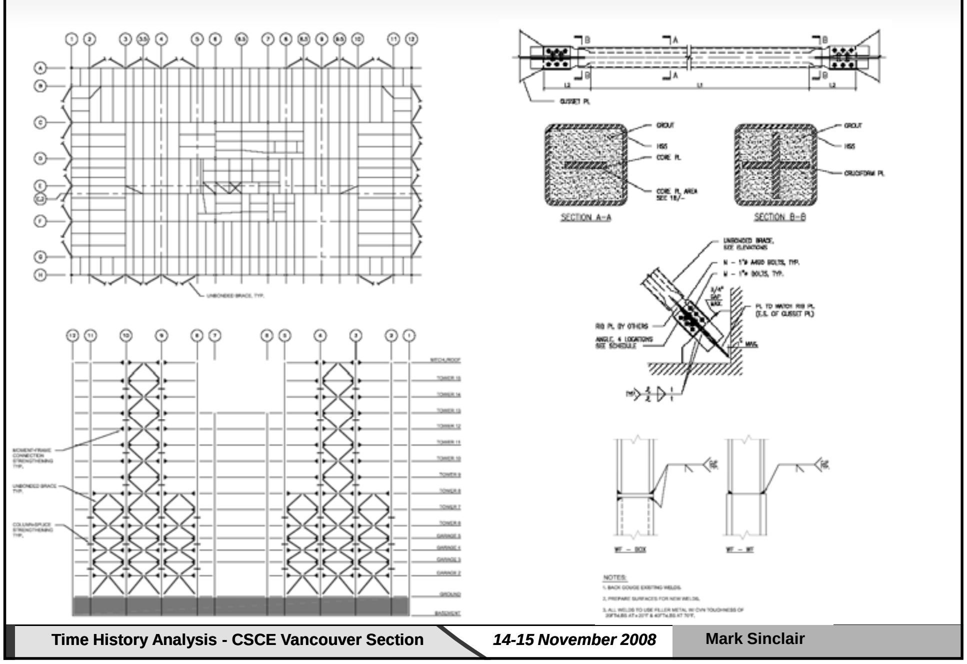




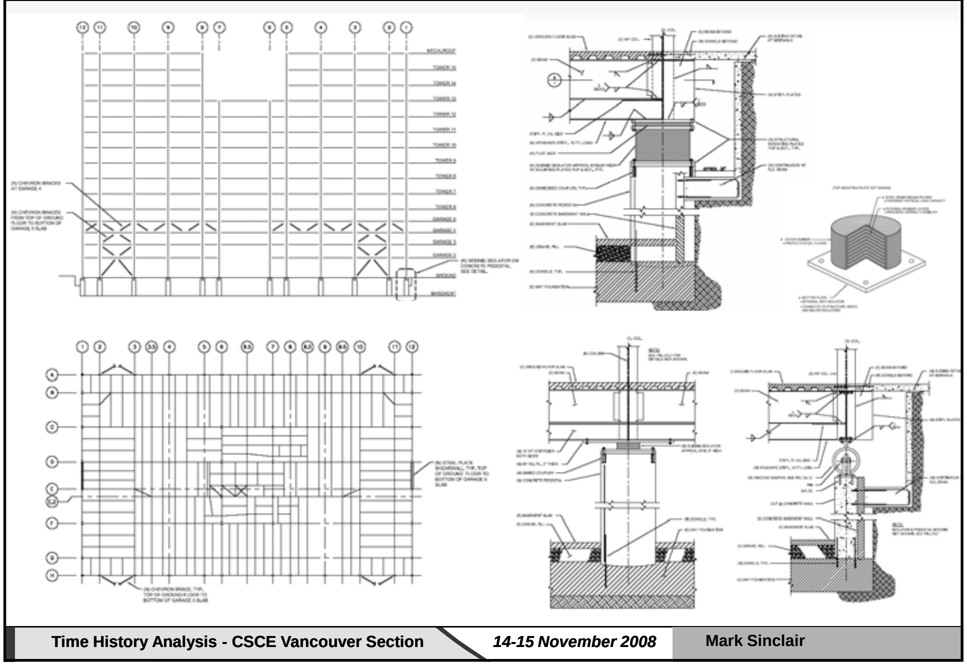
## Scheme (2): Connection Modification + Dampers



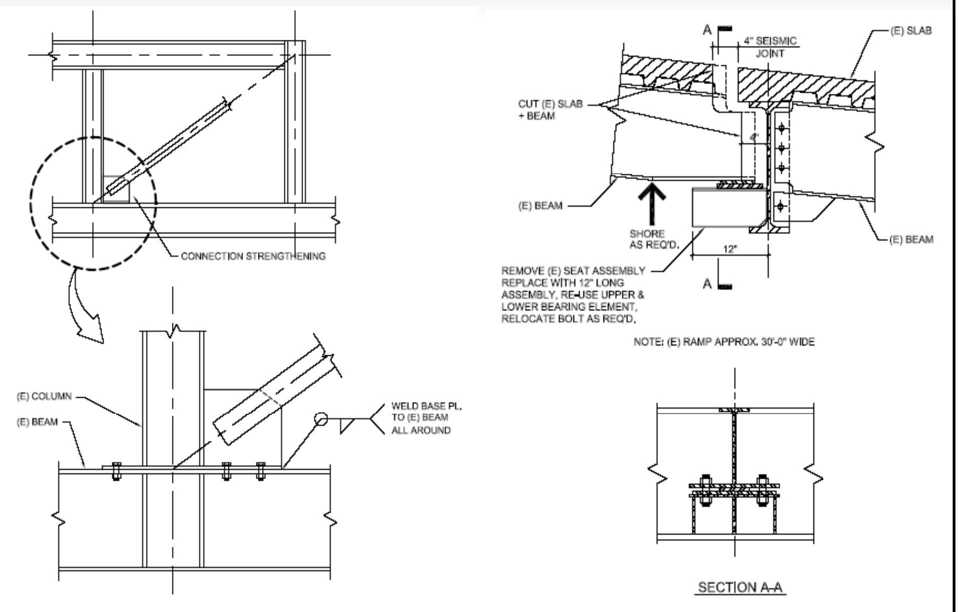
## Scheme (3): Buckling Restrained Braced Frames



## Scheme (4): Base Isolation



## All Schemes: Misc Strengthening



## Decision Matrix-Study Phase - 2003

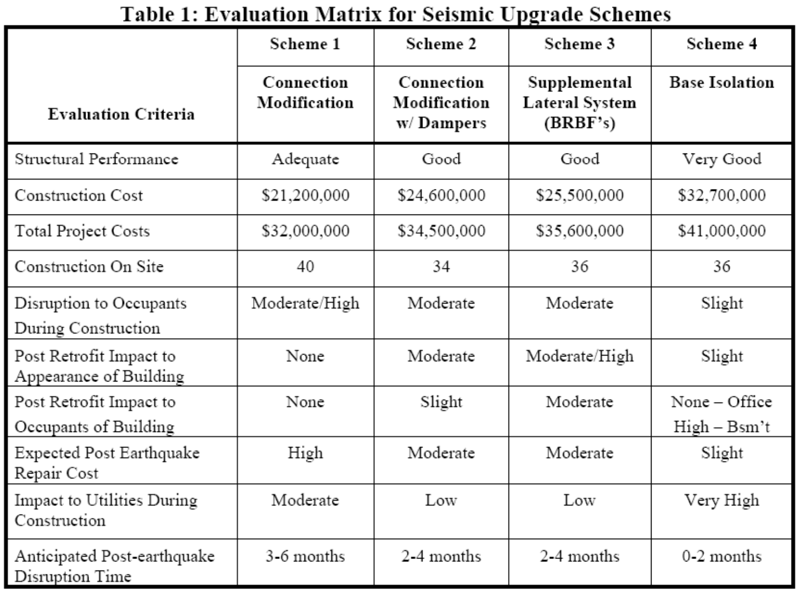
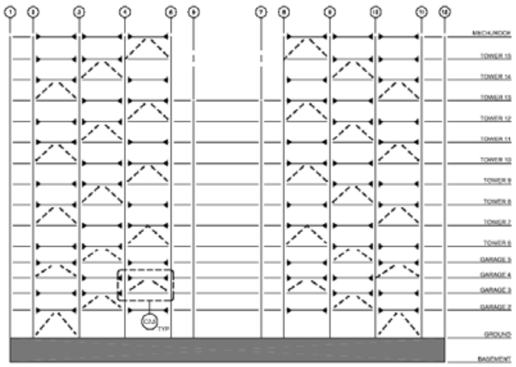


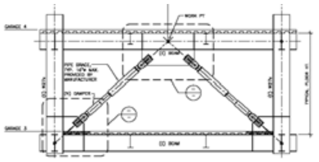
Table 1: Evaluation Matrix for Seismic Upgrade Schemes

| Evaluation Criteria | Scheme 1 Connection Modification | Scheme 2 Connection Modification w/ Dampers | Scheme 3 Supplemental Lateral System (BRBF's) | Scheme 4 Base Isolation |
|---|---|---|---|---|
| Structural Performance | Adequate | Good | Good | Very Good |
| Construction Cost | $21,200,000 | $24,600,000 | $25,500,000 | $32,700,000 |
| Total Project Costs | $32,000,000 | $34,500,000 | $35,600,000 | $41,000,000 |
| Construction On Site | 40 | 34 | 36 | 36 |
| Disruption to Occupants During Construction | Moderate/High | Moderate | Moderate | Slight |
| Post Retrofit Impact to Appearance of Building | None | Moderate | Moderate/High | Slight |
| Post Retrofit Impact to Occupants of Building | None | Slight | Moderate | None – Office<br>High – Bsm't |
| Expected Post Earthquake Repair Cost | High | Moderate | Moderate | Slight |
| Impact to Utilities During Construction | Moderate | Low | Low | Very High |
| Anticipated Post-earthquake Disruption Time | 3-6 months | 2-4 months | 2-4 months | 0-2 months |

## Selected Scheme (2)

- Combination of connection Modifications and dampers
- Connection strengthening and dampers installed at same locations to minimize work locations
- Maximum practical damper sizes used to limit work locations
- Work at building perimeter and interior lines to limit MEP
- Column Splice Strengthening
- Collector Element Strengthening

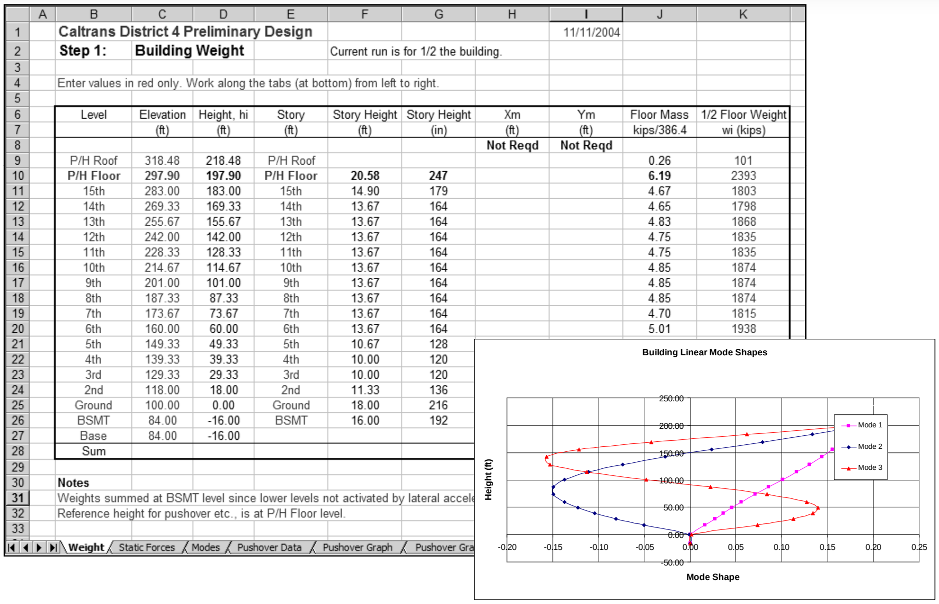




# Study Phase Analysis

- No time history records available at this stage of project
- Account for higher mode response
  - Story drift contribution
  - Dampers are velocity sensitive
- Account for existing connection fracture and frame yielding
- Multimode Pushover analysis (SAP 2000)
- Model displacements computed using ATC55/FEMA440 Improved Linearization (Capacity Spectrum)
- Spreadsheet analysis

# P/O Analysis: Step 1, Masses, Mode Shapes



**Caltrans District 4 Preliminary Design** 11/11/2004

**Step 1:** **Building Weight** Current run is for 1/2 the building.

Enter values in red only. Work along the tabs (at bottom) from left to right.

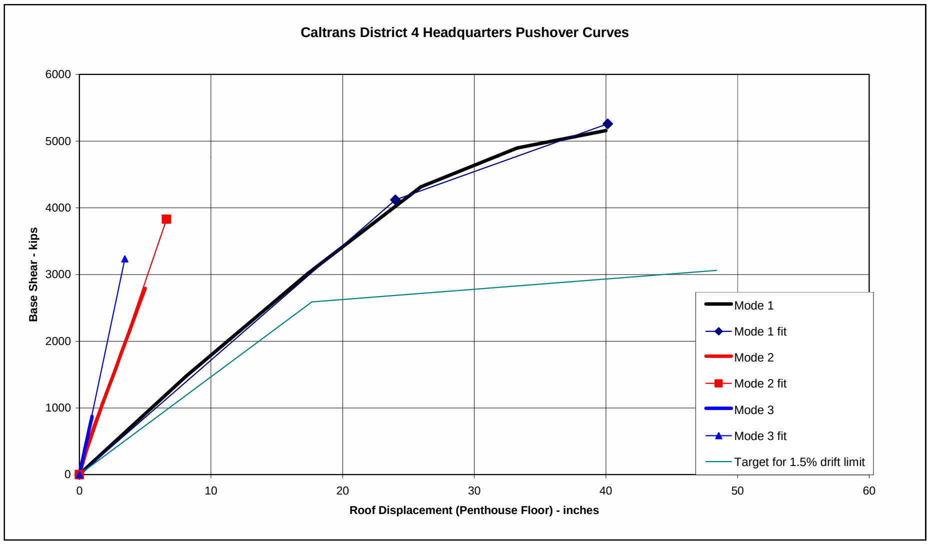
| Level | Elevation (ft) | Height, hi (ft) | Story (ft) | Story Height (ft) | Story Height (in) | Xm (ft) | Ym (ft) | Floor Mass kips/386.4 | 1/2 Floor Weight wi (kips) |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | **Not Reqd** | **Not Reqd** | | |
| P/H Roof | 318.48 | 218.48 | P/H Roof | | | | | 0.26 | 101 |
| **P/H Floor** | **297.90** | **197.90** | **P/H Floor** | **20.58** | **247** | | | **6.19** | 2393 |
| 15th | 283.00 | 183.00 | 15th | 14.90 | 179 | | | 4.67 | 1803 |
| 14th | 269.33 | 169.33 | 14th | 13.67 | 164 | | | 4.65 | 1798 |
| 13th | 255.67 | 155.67 | 13th | 13.67 | 164 | | | 4.83 | 1868 |
| 12th | 242.00 | 142.00 | 12th | 13.67 | 164 | | | 4.75 | 1835 |
| 11th | 228.33 | 128.33 | 11th | 13.67 | 164 | | | 4.75 | 1835 |
| 10th | 214.67 | 114.67 | 10th | 13.67 | 164 | | | 4.85 | 1874 |
| 9th | 201.00 | 101.00 | 9th | 13.67 | 164 | | | 4.85 | 1874 |
| 8th | 187.33 | 87.33 | 8th | 13.67 | 164 | | | 4.85 | 1874 |
| 7th | 173.67 | 73.67 | 7th | 13.67 | 164 | | | 4.70 | 1815 |
| 6th | 160.00 | 60.00 | 6th | 13.67 | 164 | | | 5.01 | 1938 |
| 5th | 149.33 | 49.33 | 5th | 10.67 | 128 | | | | |
| 4th | 139.33 | 39.33 | 4th | 10.00 | 120 | | | | |
| 3rd | 129.33 | 29.33 | 3rd | 10.00 | 120 | | | | |
| 2nd | 118.00 | 18.00 | 2nd | 11.33 | 136 | | | | |
| Ground | 100.00 | 0.00 | Ground | 18.00 | 216 | | | | |
| BSMT | 84.00 | -16.00 | BSMT | 16.00 | 192 | | | | |
| Base | 84.00 | -16.00 | | | | | | | |
| Sum | | | | | | | | | |

**Notes**

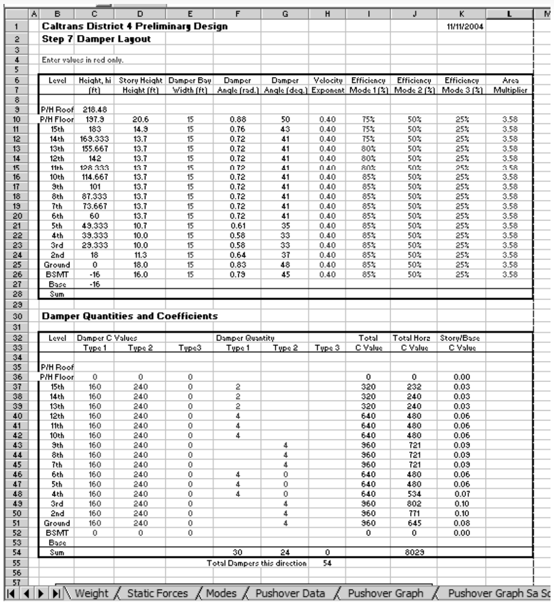
Weights summed at BSMT level since lower levels not activated by lateral accele

Reference height for pushover etc., is at P/H Floor level.

## Step 2, Run Pushover in 3 Modes E/W



## P/O Analysis: Step 3, Trial Damper Layout



**Caltrans District 4 Preliminary Design** 11/11/2004

**Step 7 Damper Layout**

Enter values in red only.

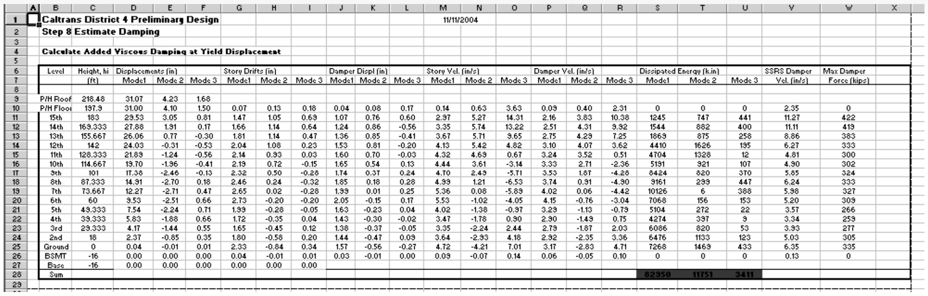
| Level | Height, hi (ft) | Story Height Height (ft) | Damper Bay Width (ft) | Damper Angle (rad.) | Damper Angle (deg.) | Velocity Exponent | Efficiency Mode 1 (%) | Efficiency Mode 2 (%) | Efficiency Mode 3 (%) | Area Multiplier |
|---|---|---|---|---|---|---|---|---|---|---|
| P/H Roof | 218.48 | | | | | | | | | |
| P/H Floor | 197.9 | 20.6 | 15 | 0.88 | 50 | 0.40 | 75% | 50% | 25% | 3.58 |
| 15th | 183 | 14.9 | 15 | 0.76 | 43 | 0.40 | 75% | 50% | 25% | 3.58 |
| 14th | 169.333 | 13.7 | 15 | 0.72 | 41 | 0.40 | 75% | 50% | 25% | 3.58 |
| 13th | 155.667 | 13.7 | 15 | 0.72 | 41 | 0.40 | 80% | 50% | 25% | 3.58 |
| 12th | 142 | 13.7 | 15 | 0.72 | 41 | 0.40 | 80% | 50% | 25% | 3.58 |
| 11th | 128.333 | 13.7 | 15 | 0.72 | 41 | 0.40 | 80% | 50% | 25% | 3.58 |
| 10th | 114.667 | 13.7 | 15 | 0.72 | 41 | 0.40 | 85% | 50% | 25% | 3.58 |
| 9th | 101 | 13.7 | 15 | 0.72 | 41 | 0.40 | 85% | 50% | 25% | 3.58 |
| 8th | 87.333 | 13.7 | 15 | 0.72 | 41 | 0.40 | 85% | 50% | 25% | 3.58 |
| 7th | 73.667 | 13.7 | 15 | 0.72 | 41 | 0.40 | 85% | 50% | 25% | 3.58 |
| 6th | 60 | 13.7 | 15 | 0.72 | 41 | 0.40 | 85% | 50% | 25% | 3.58 |
| 5th | 49.333 | 10.7 | 15 | 0.61 | 35 | 0.40 | 85% | 50% | 25% | 3.58 |
| 4th | 39.333 | 10.0 | 15 | 0.58 | 33 | 0.40 | 85% | 50% | 25% | 3.58 |
| 3rd | 29.333 | 10.0 | 15 | 0.58 | 33 | 0.40 | 85% | 50% | 25% | 3.58 |
| 2nd | 18 | 11.3 | 15 | 0.64 | 37 | 0.40 | 85% | 50% | 25% | 3.58 |
| Ground | 0 | 18.0 | 15 | 0.83 | 48 | 0.40 | 85% | 50% | 25% | 3.58 |
| BSMT | -16 | 16.0 | 15 | 0.79 | 45 | 0.40 | 85% | 50% | 25% | 3.58 |
| Base | -16 | | | | | | | | | |
| Sum | | | | | | | | | | |

**Damper Quantities and Coefficients**

| Level | Damper C Values | | | Damper Quantity | | | Total | Total Horz | Story/Base |
|---|---|---|---|---|---|---|---|---|---|
| | Type 1 | Type 2 | Type3 | Type 1 | Type 2 | Type 3 | C Value | C Value | C Value |
| P/H Roof | | | | | | | | | |
| P/H Floor | 0 | 0 | 0 | | | | 0 | 0 | 0.00 |
| 15th | 160 | 240 | 0 | 2 | | | 320 | 232 | 0.03 |
| 14th | 160 | 240 | 0 | 2 | | | 320 | 240 | 0.03 |
| 13th | 160 | 240 | 0 | 2 | | | 320 | 240 | 0.03 |
| 12th | 160 | 240 | 0 | 4 | | | 640 | 480 | 0.06 |
| 11th | 160 | 240 | 0 | 4 | | | 640 | 480 | 0.06 |
| 10th | 160 | 240 | 0 | 4 | | | 640 | 480 | 0.06 |
| 9th | 160 | 240 | 0 | | 4 | | 960 | 721 | 0.09 |
| 8th | 160 | 240 | 0 | | 4 | | 960 | 721 | 0.09 |
| 7th | 160 | 240 | 0 | | 4 | | 960 | 721 | 0.09 |
| 6th | 160 | 240 | 0 | 4 | 0 | | 640 | 480 | 0.06 |
| 5th | 160 | 240 | 0 | 4 | 0 | | 640 | 480 | 0.06 |
| 4th | 160 | 240 | 0 | 4 | 0 | | 640 | 534 | 0.07 |
| 3rd | 160 | 240 | 0 | | 4 | | 960 | 802 | 0.10 |
| 2nd | 160 | 240 | 0 | | 4 | | 960 | 771 | 0.10 |
| Ground | 160 | 240 | 0 | | 4 | | 960 | 645 | 0.08 |
| BSMT | 0 | 0 | 0 | | | | 0 | 0 | 0.00 |
| Base | | | | | | | | | |
| Sum | | | | 30 | 24 | 0 | | 8029 | |

Total Dampers this direction 54

## Step 4, Guess Modal Displacements, Calc. Damping



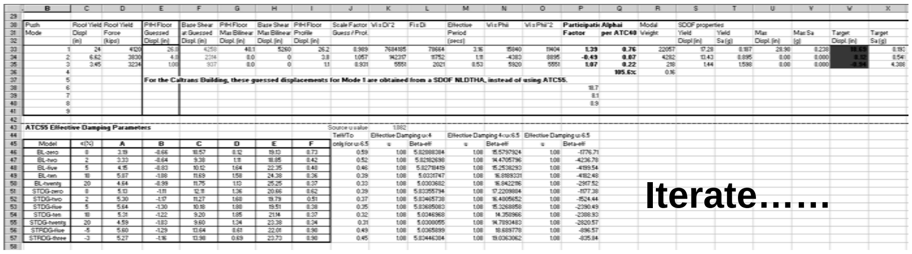
**Caltrans District 4 Preliminary Design** 11/11/2004

**Step 8 Estimate Damping**

**Calculate Added Viscous Damping at Yield Displacement**

| Level | Height, hi (ft) | Displacements (in) Mode1 | Mode 2 | Mode 3 | Story Drifts (in) Mode1 | Mode 2 | Mode 3 | Damper Displ (in) Mode1 | Mode 2 | Mode 3 | Story Vel. (in/s) Mode1 | Mode 2 | Mode 3 | Damper Vel. (in/s) Mode1 | Mode 2 | Mode 3 | Dissipated Energy (k.in) Mode1 | Mode 2 | Mode 3 | SSRS Damper Vel. (in/s) | Max Damper Force (kips) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| P/H Roof | 218.48 | 31.07 | 4.23 | 1.68 | | | | | | | | | | | | | | | | | |
| P/H Floor | 197.9 | 31.00 | 4.10 | 1.50 | 0.07 | 0.13 | 0.18 | 0.04 | 0.08 | 0.17 | 0.14 | 0.63 | 3.63 | 0.09 | 0.40 | 2.31 | 0 | 0 | 0 | 2.35 | 0 |
| 15th | 183 | 29.53 | 3.05 | 0.81 | 1.47 | 1.05 | 0.69 | 1.07 | 0.76 | 0.60 | 2.97 | 5.27 | 14.31 | 2.16 | 3.83 | 10.38 | 1245 | 747 | 441 | 11.27 | 422 |
| 14th | 169.333 | 27.88 | 1.91 | 0.17 | 1.66 | 1.14 | 0.64 | 1.24 | 0.86 | -0.56 | 3.35 | 5.74 | 13.22 | 2.51 | 4.31 | 9.92 | 1544 | 882 | 400 | 11.11 | 419 |
| 13th | 155.667 | 26.06 | 0.77 | -0.30 | 1.81 | 1.14 | 0.47 | 1.36 | 0.85 | -0.41 | 3.67 | 5.71 | 9.65 | 2.75 | 4.29 | 7.25 | 1869 | 875 | 258 | 8.86 | 383 |
| 12th | 142 | 24.03 | -0.31 | -0.53 | 2.04 | 1.08 | 0.23 | 1.53 | 0.81 | -0.20 | 4.13 | 5.42 | 4.82 | 3.10 | 4.07 | 3.62 | 4410 | 1626 | 195 | 6.27 | 333 |
| 11th | 128.333 | 21.89 | -1.24 | -0.56 | 2.14 | 0.93 | 0.03 | 1.60 | 0.70 | -0.03 | 4.32 | 4.69 | 0.67 | 3.24 | 3.52 | 0.51 | 4704 | 1328 | 12 | 4.81 | 300 |
| 10th | 114.667 | 19.70 | -1.96 | -0.41 | 2.19 | 0.72 | -0.15 | 1.65 | 0.54 | 0.13 | 4.44 | 3.61 | -3.14 | 3.33 | 2.71 | -2.36 | 5191 | 921 | 107 | 4.90 | 302 |
| 9th | 101 | 17.36 | -2.46 | -0.13 | 2.32 | 0.50 | -0.26 | 1.74 | 0.37 | 0.24 | 4.70 | 2.49 | -5.71 | 3.53 | 1.67 | -4.26 | 6424 | 620 | 370 | 5.65 | 324 |
| 8th | 87.333 | 14.91 | -2.70 | 0.18 | 2.46 | 0.24 | -0.32 | 1.85 | 0.18 | 0.28 | 4.99 | 1.21 | -6.53 | 3.74 | 0.91 | -4.90 | 9161 | 299 | 447 | 6.24 | 333 |
| 7th | 73.667 | 12.27 | -2.71 | 0.47 | 2.65 | 0.02 | -0.28 | 1.99 | 0.01 | 0.25 | 5.36 | 0.08 | -5.89 | 4.02 | 0.06 | -4.42 | 10126 | 6 | 388 | 5.98 | 327 |
| 6th | 60 | 9.53 | -2.51 | 0.66 | 2.73 | -0.20 | -0.20 | 2.05 | -0.15 | 0.17 | 5.53 | -1.02 | -4.05 | 4.15 | -0.76 | -3.04 | 7068 | 156 | 153 | 5.20 | 309 |
| 5th | 49.333 | 7.54 | -2.24 | 0.71 | 1.99 | -0.28 | -0.05 | 1.63 | -0.23 | 0.04 | 4.02 | -1.38 | -0.97 | 3.29 | -1.13 | -0.79 | 5104 | 272 | 22 | 3.57 | 266 |
| 4th | 39.333 | 5.83 | -1.88 | 0.66 | 1.72 | -0.35 | 0.04 | 1.43 | -0.30 | -0.02 | 3.47 | -1.78 | 0.90 | 2.90 | -1.49 | 0.75 | 4274 | 397 | 9 | 3.34 | 259 |
| 3rd | 29.333 | 4.17 | -1.44 | 0.55 | 1.65 | -0.45 | 0.12 | 1.38 | -0.37 | -0.05 | 3.35 | -2.24 | 2.44 | 2.79 | -1.87 | 2.03 | 6086 | 820 | 53 | 3.93 | 277 |
| 2nd | 18 | 2.37 | -0.85 | 0.35 | 1.80 | -0.58 | 0.20 | 1.44 | -0.47 | 0.09 | 3.64 | -2.93 | 4.18 | 2.92 | -2.35 | 3.36 | 6476 | 1133 | 123 | 5.03 | 305 |
| Ground | 0 | 0.04 | -0.01 | 0.01 | 2.33 | -0.84 | 0.34 | 1.57 | -0.56 | -0.27 | 4.72 | -4.21 | 7.01 | 3.17 | -2.83 | 4.71 | 7268 | 1469 | 433 | 6.35 | 335 |
| BSMT | -16 | 0.00 | 0.00 | 0.00 | 0.04 | -0.01 | 0.01 | 0.03 | -0.01 | 0.00 | 0.09 | -0.07 | 0.14 | 0.06 | -0.05 | 0.10 | 0 | 0 | 0 | 0.13 | 0 |
| Base | -16 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | | | | | | | | | | | | | | |
| Sum | | | | | | | | | | | | | | | | | 82950 | 11751 | 3411 | | |

## Step 5, Add Hysteretic Total Damping, Calc. Displ.



Push Mode | Roof Yield Displ (in) | Roof Yield Force (kips) | P/H Floor Guessed Displ. (in) | Base Shear at Guessed Displ. (in) | P/H Floor Max Bilinear Displ. (in) | Base Shear Max Bilinear Displ. (in) | P/H Floor Profile Displ. (in) | Scale Factor Guess / Prof. | Wi x Di^2 | Fi x Di | Effective Period (secs) | Wi x Phii | Wi x Phii^2 | **Participation Factor** | **Alphai per ATC40** | Modal Weight | SDOF properties Yield Displ (in) | Yield Sa (g) | Max Displ. (in) | Max Sa (g) | Target Displ. (in) | Target Sa (g)

**For the Caltrans Building, these guessed displacements for Mode 1 are obtained from a SDOF NLDTHA, instead of using ATC55.**

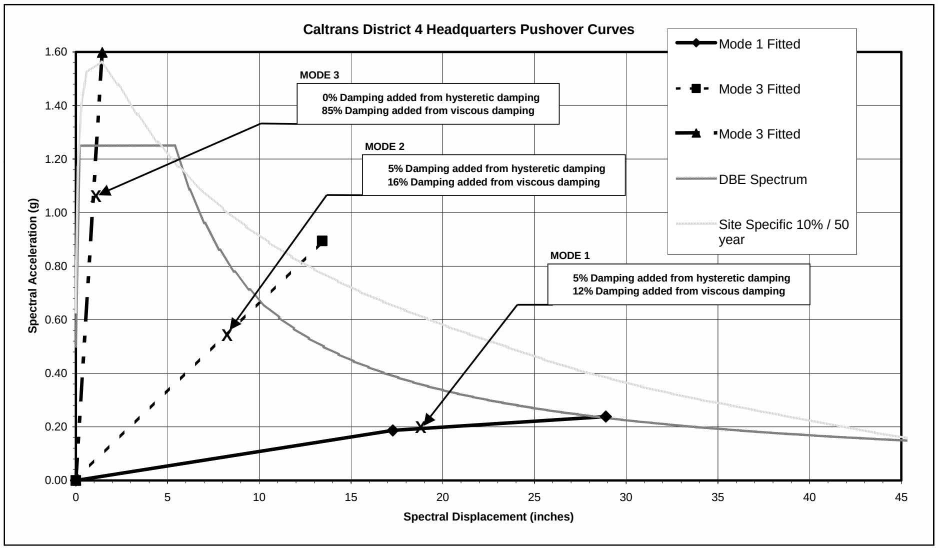
**ATC55 Effective Damping Parameters**

Model | α(%) | A | B | C | D | E | F | Teff/To only for u>6.5 | Effective Damping u<4 u | Beta-eff | Effective Damping 4<u<6.5 u | Beta-eff | Effective Damping u>6.5 u | Beta-eff

BL-zero, BL-two, BL-five, BL-ten, BL-twenty, STDG-zero, STDG-two, STDG-five, STDG-ten, STDG-twenty, STRDG-five, STRDG-three

## Iterate……

## Step 6, Converged Modal Displacements



Caltrans District 4 Headquarters Pushover Curves

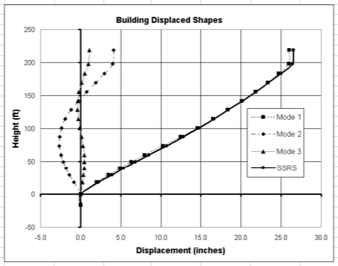
MODE 3
0% Damping added from hysteretic damping
85% Damping added from viscous damping

MODE 2
5% Damping added from hysteretic damping
16% Damping added from viscous damping

MODE 1
5% Damping added from hysteretic damping
12% Damping added from viscous damping

## Iterate……

## Step 7, Combine Modal Responses



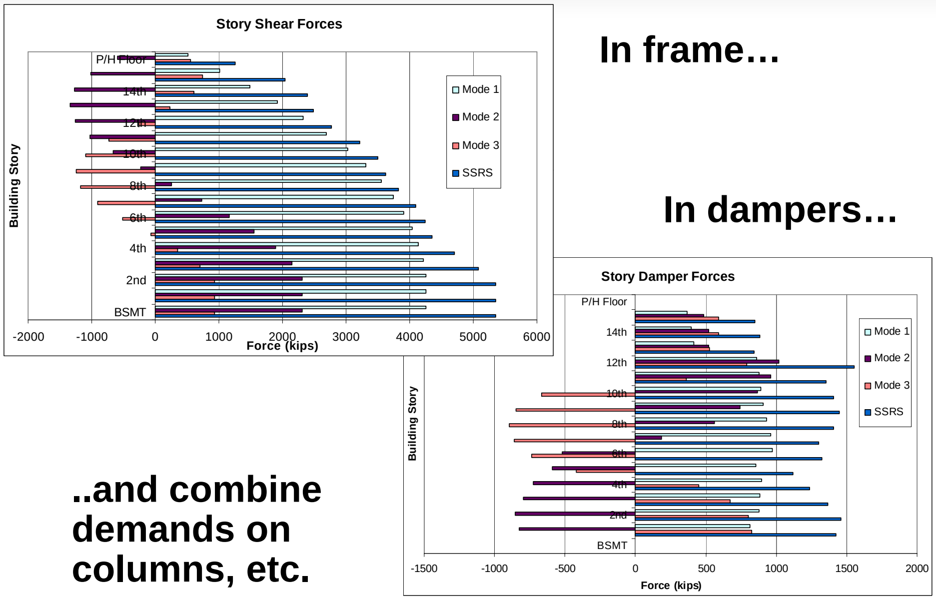
| Floor | Dampers | | Effciency | DBE Drifts |
|---|---|---|---|---|
| 15th | 2 - | 450k Dampers | 75% | 1.08% |
| 14th | 2 - | 450k Dampers | 75% | 1.26% |
| 13th | 2 - | 450k Dampers | 80% | 1.27% |
| 12th | 4 - | 450k Dampers | 80% | 1.28% |
| 11th | 4 - | 450k Dampers | 80% | 1.23% |
| 10th | 4 - | 450k Dampers | 85% | 1.22% |
| 9th | 4 - | 450k Dampers | 85% | 1.25% |
| 8th | 4 - | 450k Dampers | 85% | 1.29% |
| 7th | 4 - | 450k Dampers | 85% | 1.36% |
| 6th | 4 - | 450k Dampers | 85% | 1.41% |
| 5th | 4 - | 450k Dampers | 85% | 1.32% |
| 4th | 4 - | 450k Dampers | 85% | 1.24% |
| 3rd | 4 - | 450k Dampers | 85% | 1.23% |
| 2nd | 4 - | 450k Dampers | 85% | 1.22% |
| Ground | 4 - | 450k Dampers | 85% | 1.03% |

**By hand……**

**Caltrans District 4 Preliminary Design**
**Step 6: Pushover Displaced Shape**
11/11/2004

| Level | Height, hi | Floor Weight | Scaled to Actual Displacement | | | Drifts | | | SSRS Drifts and Displacements | | | | Pushover Shears at Actual Displacement | | | SSRS Shears |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | PushMode1 | PushMode2 | PushMode3 | PushMode1 | PushMode2 | PushMode3 | Displ. | Drifts | Dr fm Disp | **Drift Ratios** | PushMode1 | PushMode | PushMode3 | |
| | (ft) | wi (kips) | (in) | (in) | (in) | (in) | (in) | (in) | (in) | (in) | (in) | | (kips) | (kips) | (kips) | (kips) |
| P/H Roof | 218.48 | 101 | 26.0 | 4.1 | 1.1 | | | | 26.54 | | | | | | | |
| P/H Floor | 197.9 | 2393 | 26.0 | 4.0 | 1.0 | 0.06 | 0.12 | 0.12 | 26.44 | 0.25 | 0.10 | **0.10%** | 520 | -590 | 555 | 1258 |
| 15th | 183 | 1803 | 24.7 | 3.0 | 0.5 | 1.23 | 1.02 | 0.46 | 24.98 | 1.93 | 1.46 | **1.08%** | 1015 | -1016 | 748 | 2036 |
| 14th | 169.333 | 1798 | 23.3 | 1.9 | 0.1 | 1.39 | 1.12 | 0.43 | 23.43 | 2.07 | 1.55 | **1.26%** | 1484 | -1269 | 614 | 2397 |
| 13th | 155.667 | 1868 | 21.8 | 0.8 | -0.2 | 1.52 | 1.11 | 0.31 | 21.85 | 2.08 | 1.58 | **1.27%** | 1922 | -1344 | 234 | 2487 |
| 12th | 142 | 1835 | 20.1 | -0.3 | -0.4 | 1.71 | 1.05 | 0.16 | 20.13 | 2.09 | 1.72 | **1.28%** | 2326 | -1256 | -257 | 2775 |
| 11th | 128.333 | 1835 | 18.3 | -1.2 | -0.4 | 1.79 | 0.91 | 0.02 | 18.40 | 2.02 | 1.73 | **1.23%** | 2694 | -1021 | -736 | 3216 |
| 10th | 114.667 | 1874 | 16.5 | -1.9 | -0.3 | 1.84 | 0.70 | -0.10 | 16.64 | 2.00 | 1.76 | **1.22%** | 3021 | -665 | -1089 | 3494 |
| 9th | 101 | 1874 | 14.6 | -2.4 | -0.1 | 1.94 | 0.48 | -0.18 | 14.76 | 2.05 | 1.88 | **1.25%** | 3306 | -227 | -1239 | 3617 |
| 8th | 87.333 | 1874 | 12.5 | -2.6 | 0.1 | 2.06 | 0.24 | -0.21 | 12.79 | 2.11 | 1.97 | **1.29%** | 3549 | 251 | -1170 | 3822 |
| 7th | 73.667 | 1815 | 10.3 | -2.6 | 0.3 | 2.22 | 0.02 | -0.19 | 10.69 | 2.23 | 2.10 | **1.36%** | 3749 | 728 | -906 | 4090 |
| 6th | 60 | 1938 | 8.0 | -2.4 | 0.4 | 2.29 | -0.20 | -0.13 | 8.49 | 2.31 | 2.20 | **1.41%** | 3906 | 1167 | -507 | 4250 |
| 5th | 49.333 | 1998 | 6.3 | -2.2 | 0.5 | 1.66 | -0.27 | -0.03 | 6.85 | 1.69 | 1.64 | **1.32%** | 4034 | 1558 | -70 | 4350 |
| 4th | 39.333 | 1948 | 4.9 | -1.8 | 0.4 | 1.44 | -0.35 | 0.03 | 5.38 | 1.49 | 1.47 | **1.24%** | 4136 | 1890 | 349 | 4704 |
| 3rd | 29.333 | 1952 | 3.5 | -1.4 | 0.4 | 1.39 | -0.43 | 0.08 | 3.91 | 1.48 | 1.47 | **1.23%** | 4212 | 2150 | 702 | 5086 |
| 2nd | 18 | 1963 | 2.0 | -0.8 | 0.2 | 1.51 | -0.57 | 0.13 | 2.25 | 1.66 | 1.66 | **1.22%** | 4257 | 2311 | 933 | 5352 |
| Ground | 0 | 0 | 0.0 | 0.0 | 0.0 | 1.95 | -0.82 | 0.23 | 0.04 | 2.21 | 2.21 | **1.03%** | 4258 | 2314 | 937 | 5358 |
| BSMT | -16 | 0 | 0.0 | 0.0 | 0.0 | 0.04 | -0.01 | 0.00 | 0.00 | 0.04 | 0.04 | **0.02%** | 4258 | 2314 | 937 | 5358 |
| Base | -16 | 0 | 0.0 | 0.0 | 0.0 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | | | | | |
| Sum | | 28871 | | | | | | | | | max | **1.41%** | | | | |
| | | | | | | | | | | | average | **1.11%** | | | | |

## Step 8, Compute Damper Velocities and Forces



**In frame…**

**In dampers…**

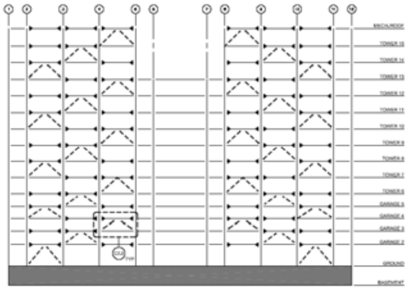
**..and combine demands on columns, etc.**

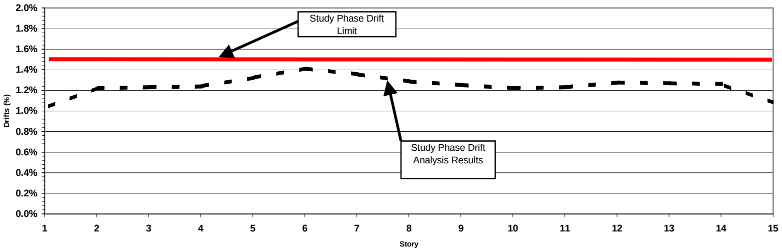
## Issues and Observations

- SAP Pushover analyses had difficulty converging
  - High degree of non-linearity due to existing connections
  - Higher mode pushovers mostly linear, but yield suddenly
- Accidental mass eccentricity difficult to implement
- No account of change in mode shape, or period (less important)
- Difficult to account for interaction of frame and dampers
  - e.g. loss of damper efficiency in upper levels due to column shortening
  - tendency to over estimate damping in higher modes, especially with low-exponent dampers (due to fluid compressibility)
- Very tedious and complex
- Educational

## Study Phase Final Scheme

- Uniform drifts below 1.5%
- Dampers:
  - Checkerboard damper pattern
  - 450kip dampers
  - 0.4 velocity exponent

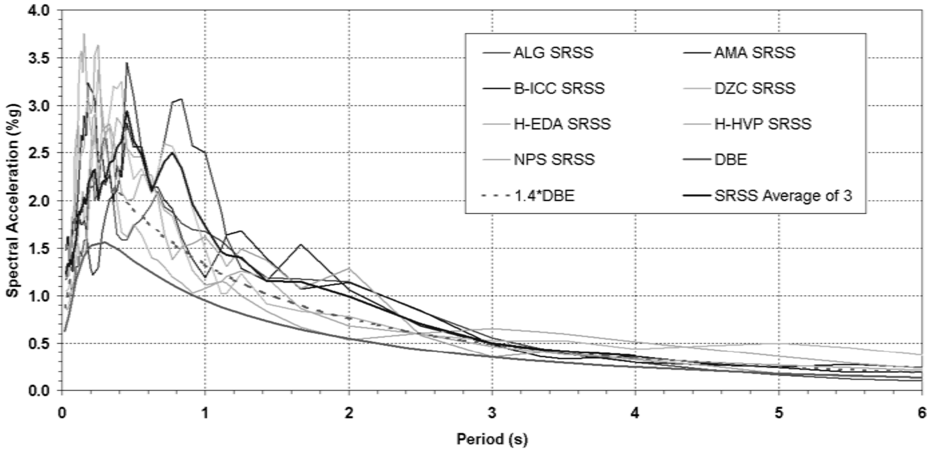


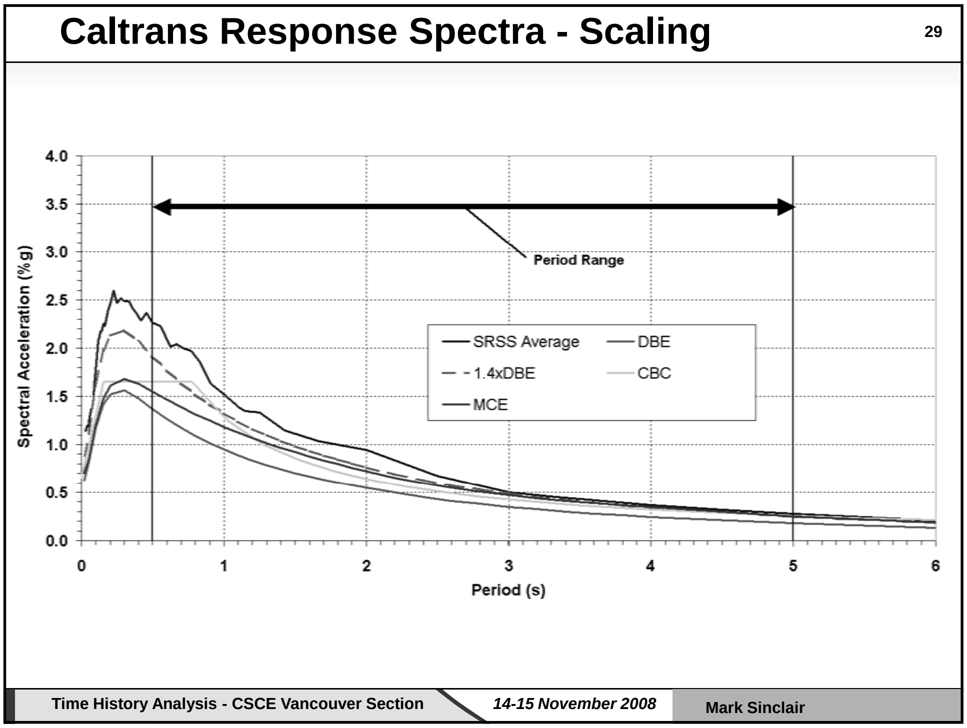


## Working Drawings Phase Analysis and Design

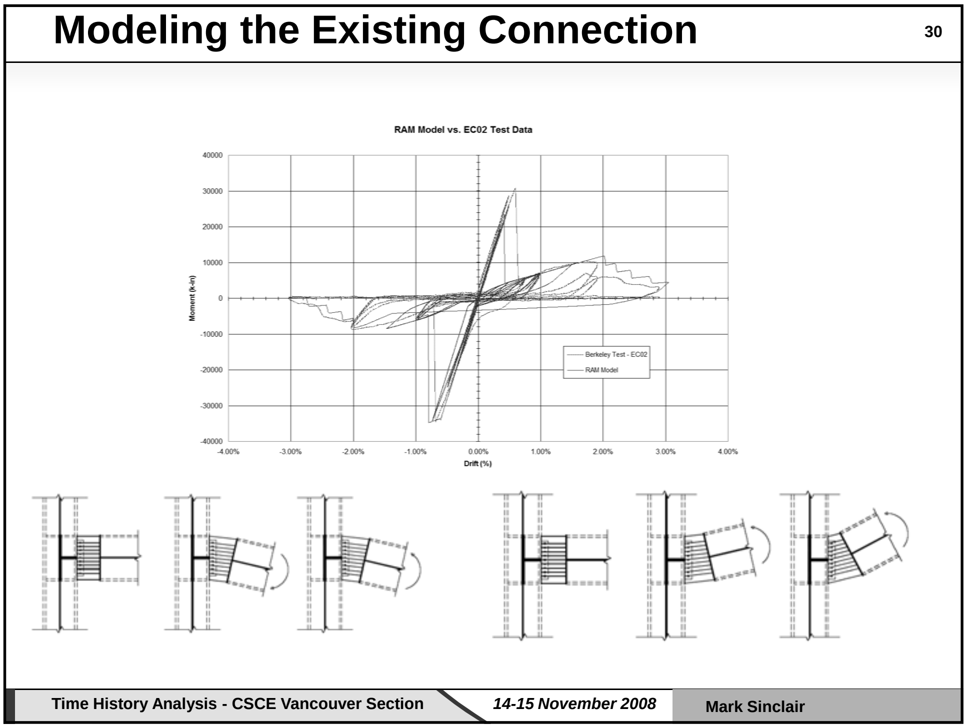
- Time history records now available
- Switch to RAM Perform model
  - Capture existing connection fracture directly
  - Model frame yielding, foundation uplift, diaphragm flexibility
  - Size and complexity of model limits to 2D, factor up demands to account for torsion
  - No (less) iteration required compared to spreadsheet
  - Check performance of existing building, all-connection scheme (1), and 60% connections + dampers scheme (2)
- Connection test program
  - Update model to reflect selected connection type
- Final design
  - Check existing components – columns, beams,
  - Design retrofit components - dampers and connections, collectors, chords

## Caltrans Response Spectra - All Records

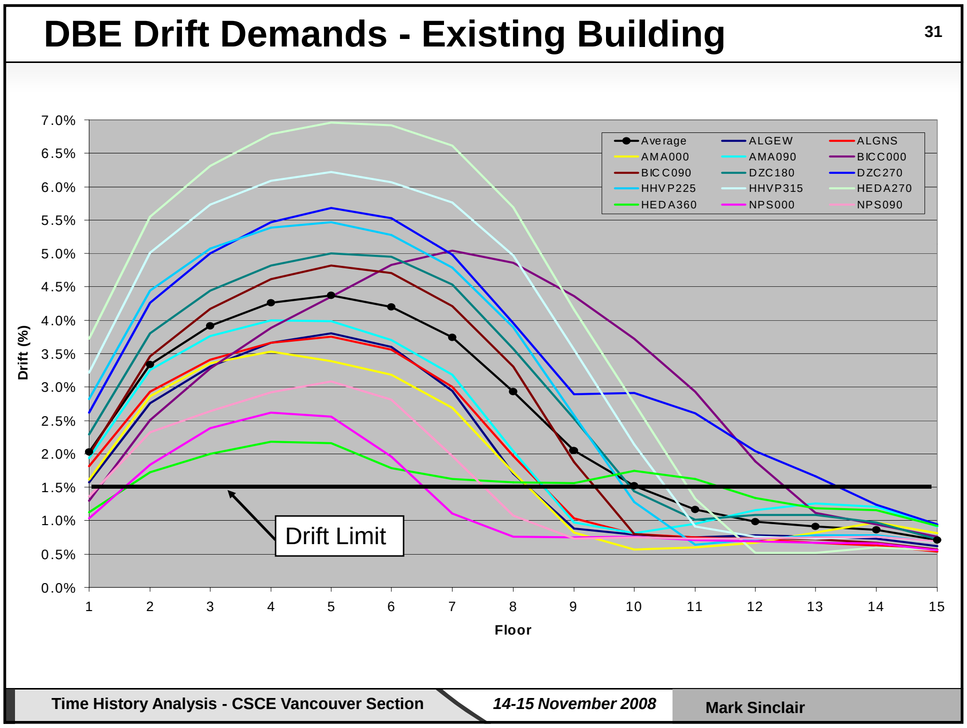




## Caltrans Response Spectra - Scaling



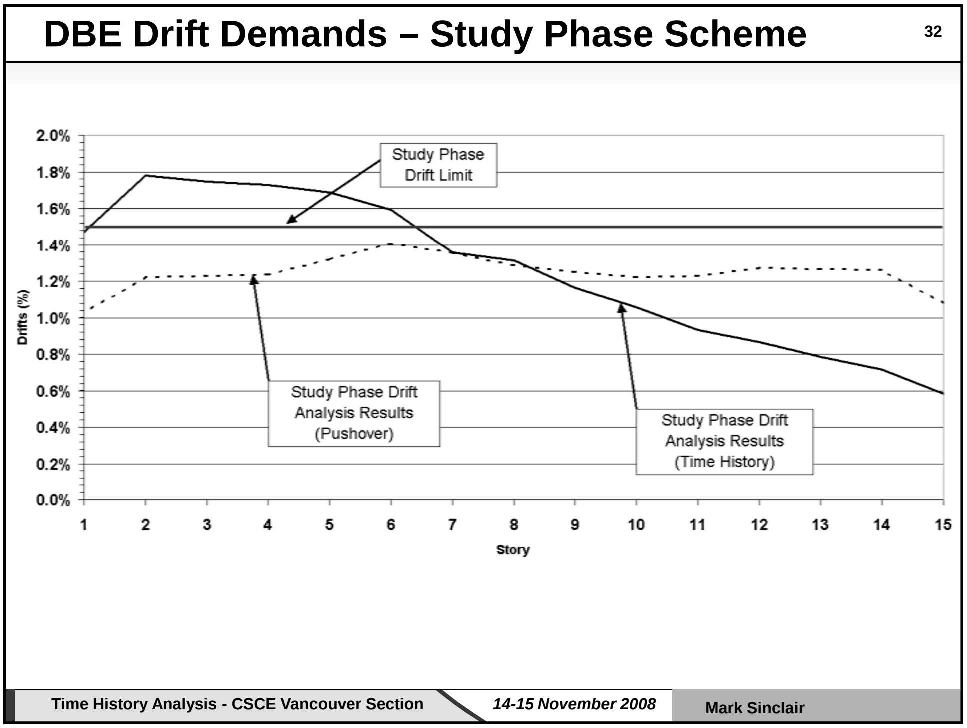
## Modeling the Existing Connection



## DBE Drift Demands - Existing Building

Drift Limit

Time History Analysis - CSCE Vancouver Section
*14-15 November 2008*
Mark Sinclair



## DBE Drift Demands – Study Phase Scheme

Study Phase Drift Limit

Study Phase Drift Analysis Results (Pushover)

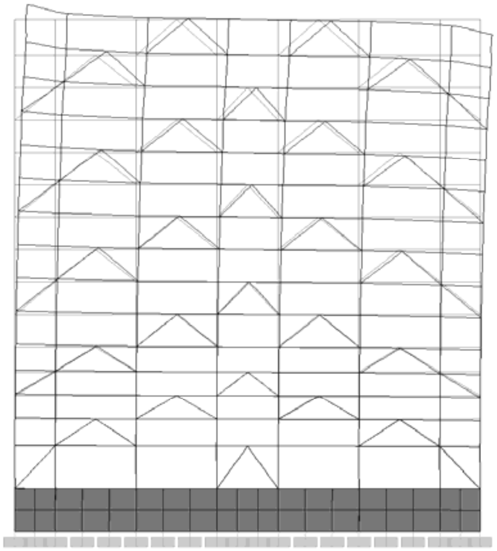
Study Phase Drift Analysis Results (Time History)

Time History Analysis - CSCE Vancouver Section
*14-15 November 2008*
Mark Sinclair

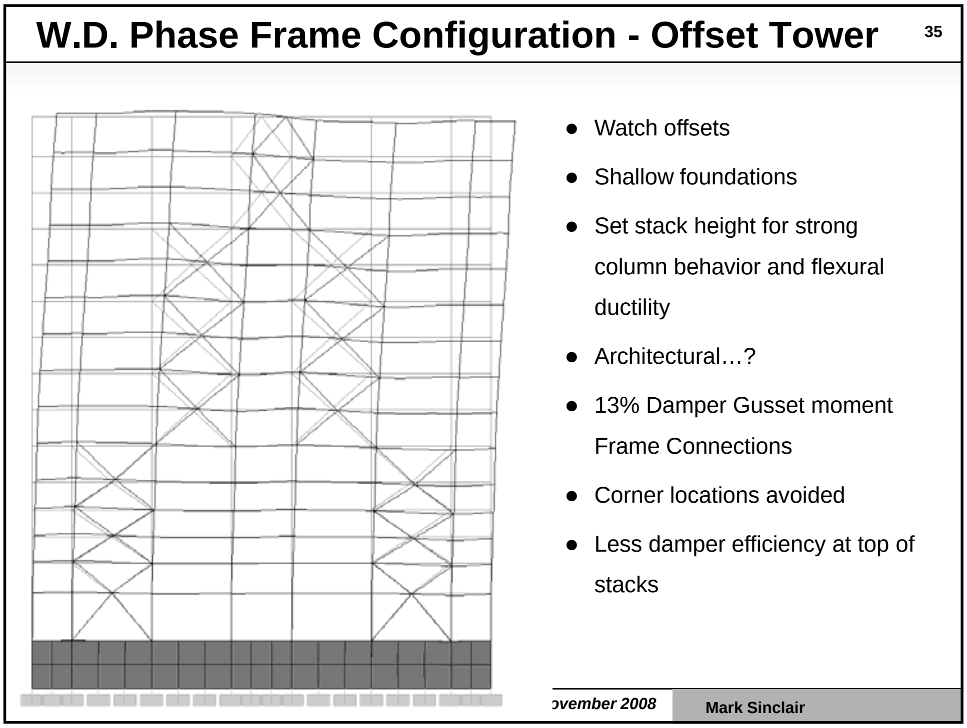
## Revisions to Study Phase Scheme

- Drift distribution by time-history analysis poorly predicted by pushover
  - Highly non-linear problem
- Revised damper sizes and layout
  - Shift more dampers to lower floors
  - Increase sizes
- Propose clarification to drift criteria as follows:
  - 1.5% drift limit for two-dimensional analyses (center of mass)
  - Factor drifts up to account for inherent and accidental mass torsion
  - 1.8% drift limit for maximum corner displacement
  - 1.5% average (roof) drift limit for maximum corner displacement
  - 2.0% (=1.5% x 1 / 0.75) drift limit for two-dimensional (COM) MCE runs
  - Used FEMA351 Appendix A to justify increase (benefit was conservatively estimated during the Study Phase)

## Study Phase Frame Configuration - Checkerboard



- Distributed dampers spread overturning loads
- Shallow foundations
- Reduced column axial loads
- Architectural “Feature”….?
- 31% Damper Gusset Moment Connections
- Requires access to all locations with consequent disruption



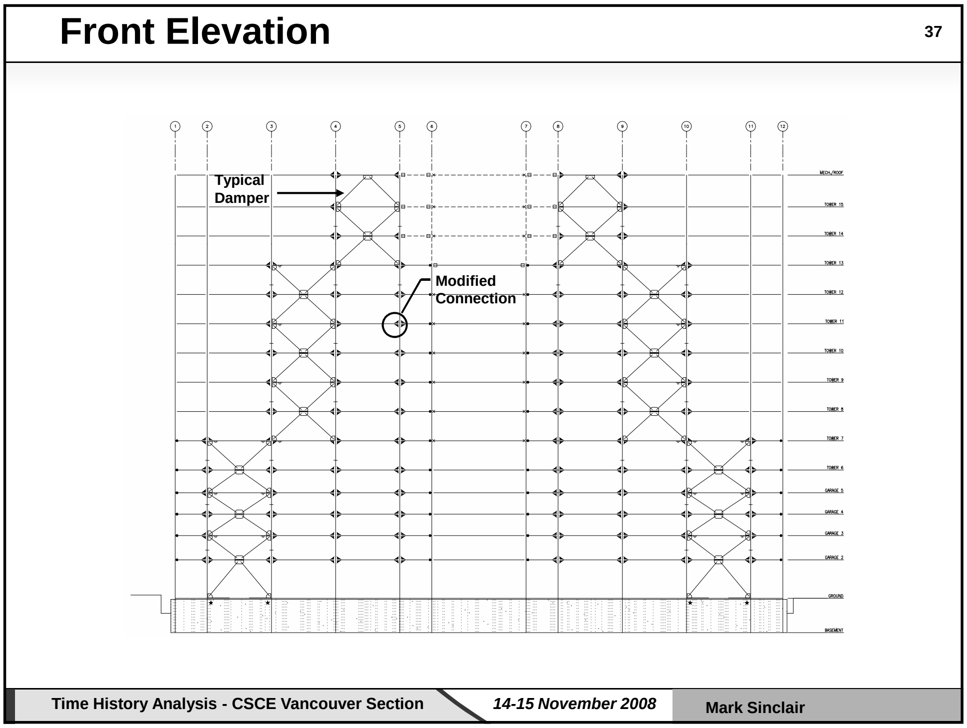
## W.D. Phase Frame Configuration - Offset Tower

- Watch offsets
- Shallow foundations
- Set stack height for strong column behavior and flexural ductility
- Architectural…?
- 13% Damper Gusset moment Frame Connections
- Corner locations avoided
- Less damper efficiency at top of stacks

## Final Scheme Statistics

- 1218 existing connections in building, 746 connections will be strengthened (61% of total connections)
- 612 connections will have a double haunch or similar strengthening
- 60 connections will have both a haunch and a gusset plate
- 76 connections will have double gusset plates

- 228 dampers will be added
  - 56 – 650k Dampers (C = 240 k $(\text{sec/in})^{0.4}$, $\alpha = 0.4$)
  - 148 – 450k Dampers (C = 160 k $(\text{sec/in})^{0.4}$, $\alpha = 0.4$)
  - 24 – 225k Dampers (C = 80 k $(\text{sec/in})^{0.4}$, $\alpha = 0.4$)



## Front Elevation

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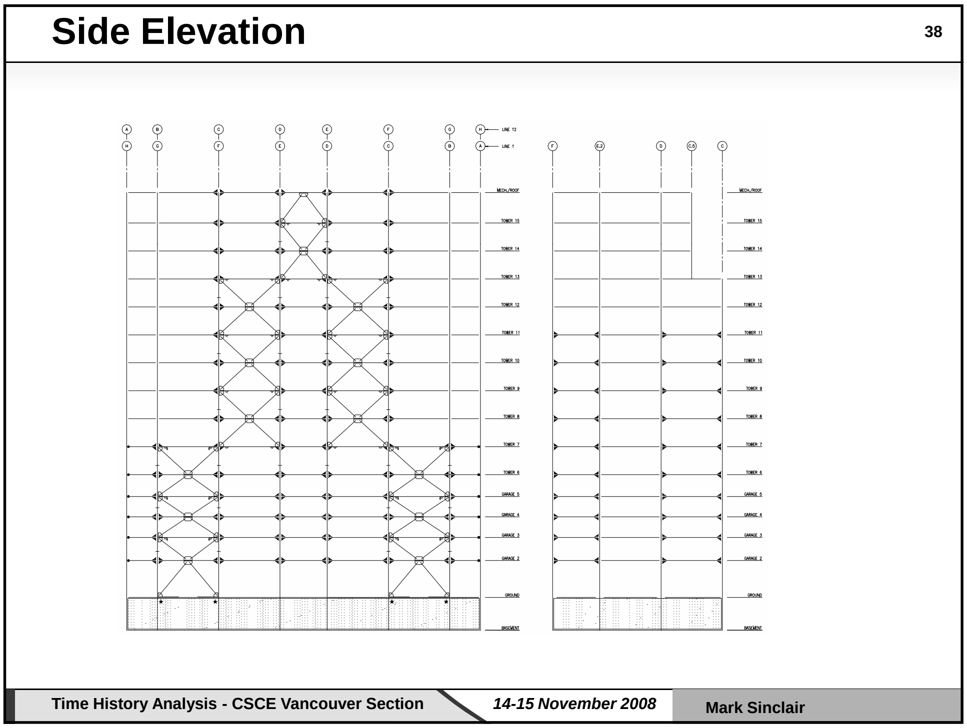

## Side Elevation

Time History Analysis - CSCE Vancouver Section

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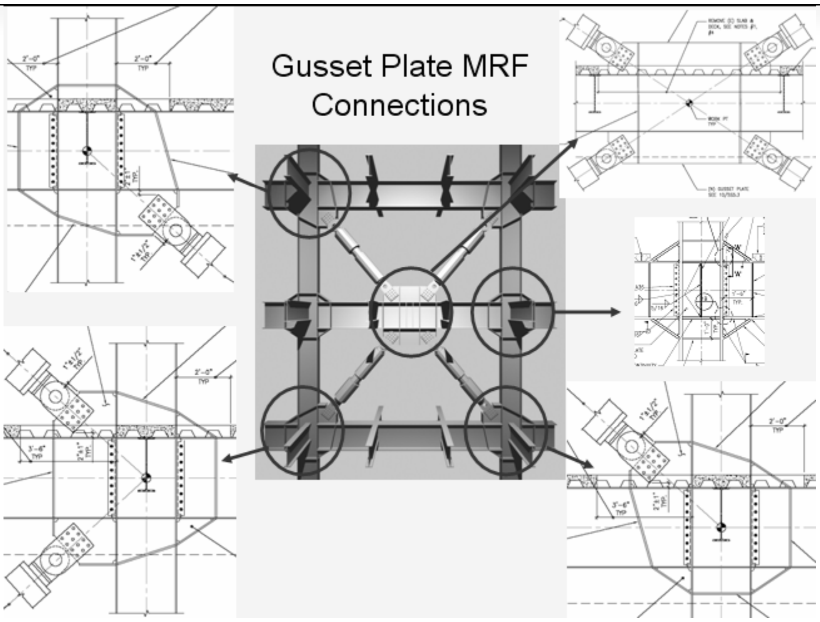
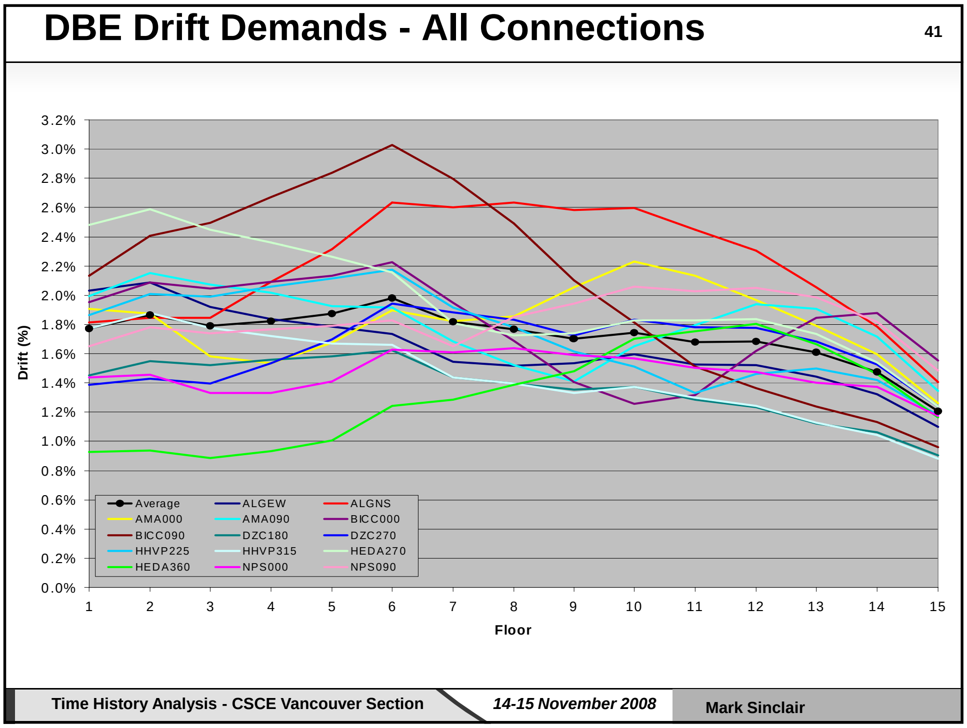
## Retrofit Connections

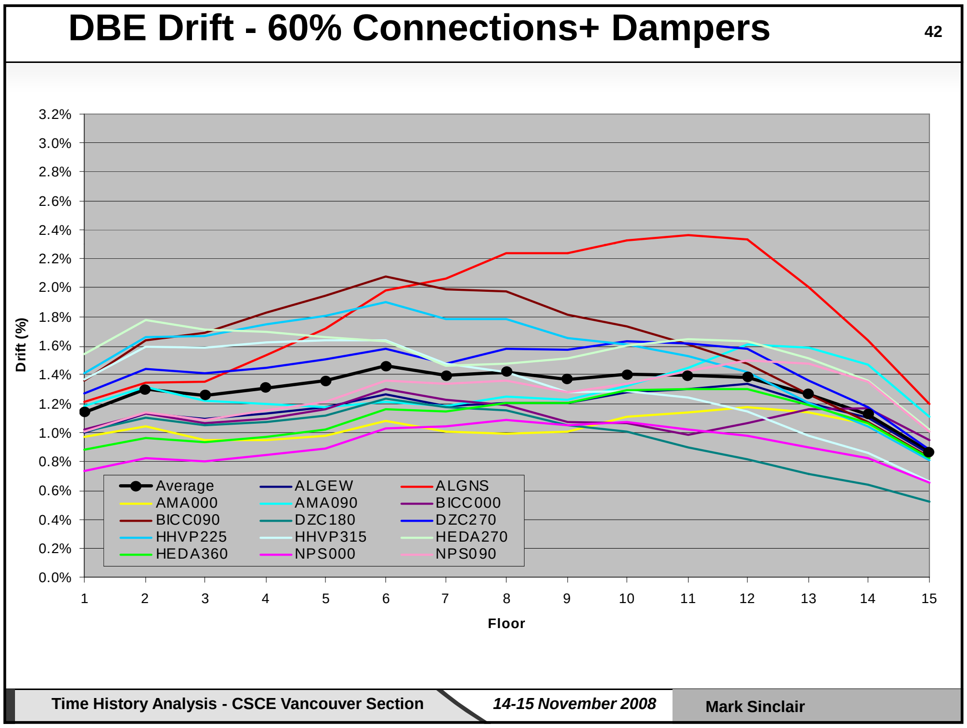


Gusset Plate MRF Connections

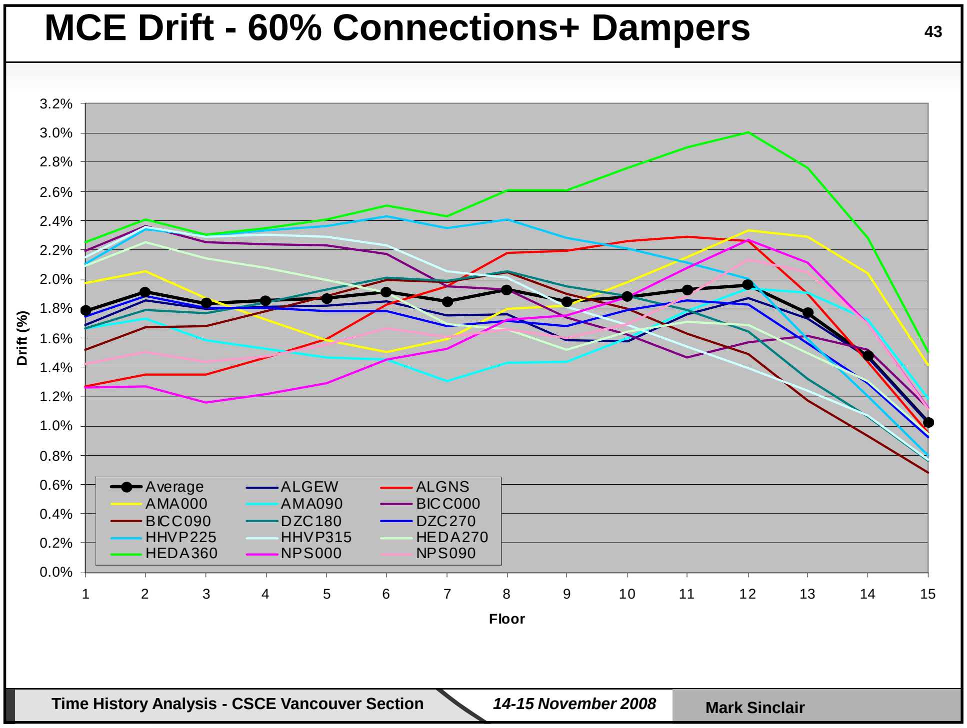
## Rendering of Retrofit



## DBE Drift Demands - All Connections



## DBE Drift - 60% Connections+ Dampers



## MCE Drift - 60% Connections+ Dampers

43

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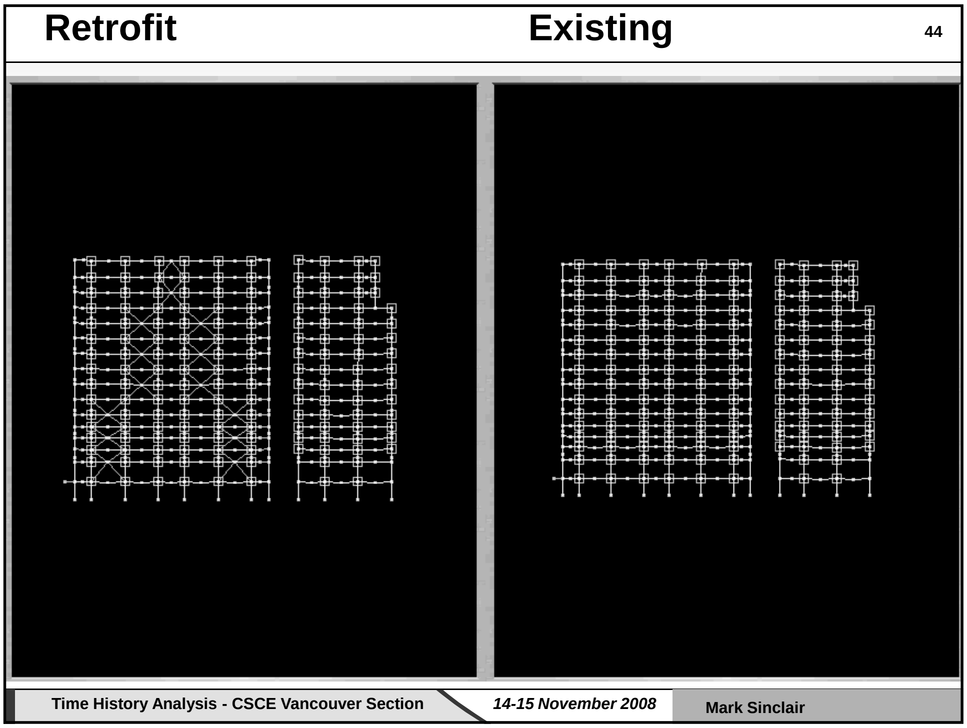

## Retrofit Existing

44

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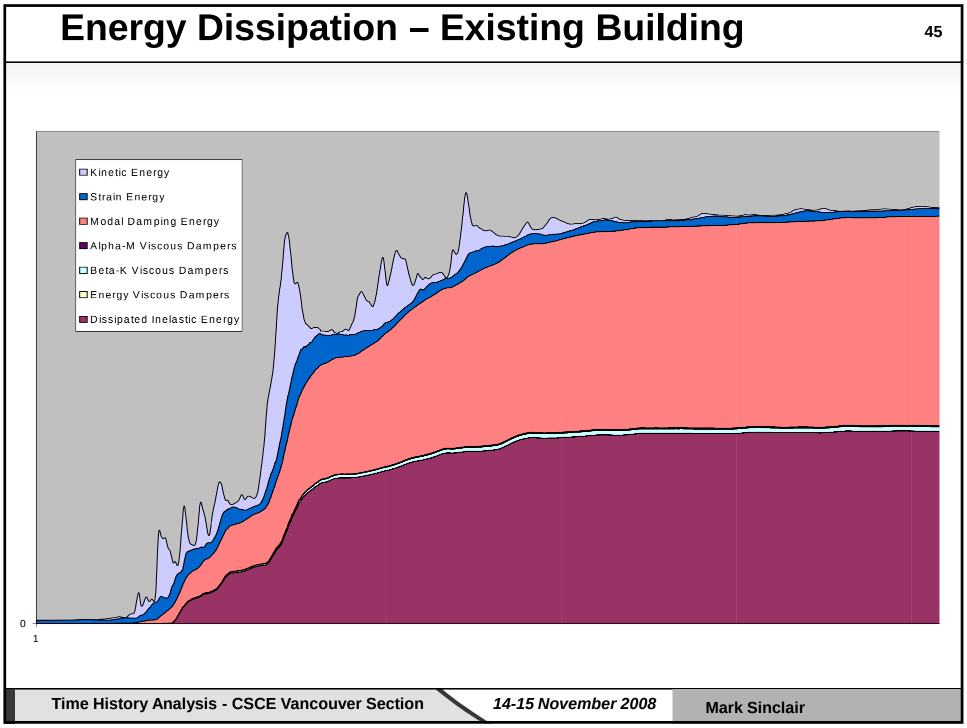
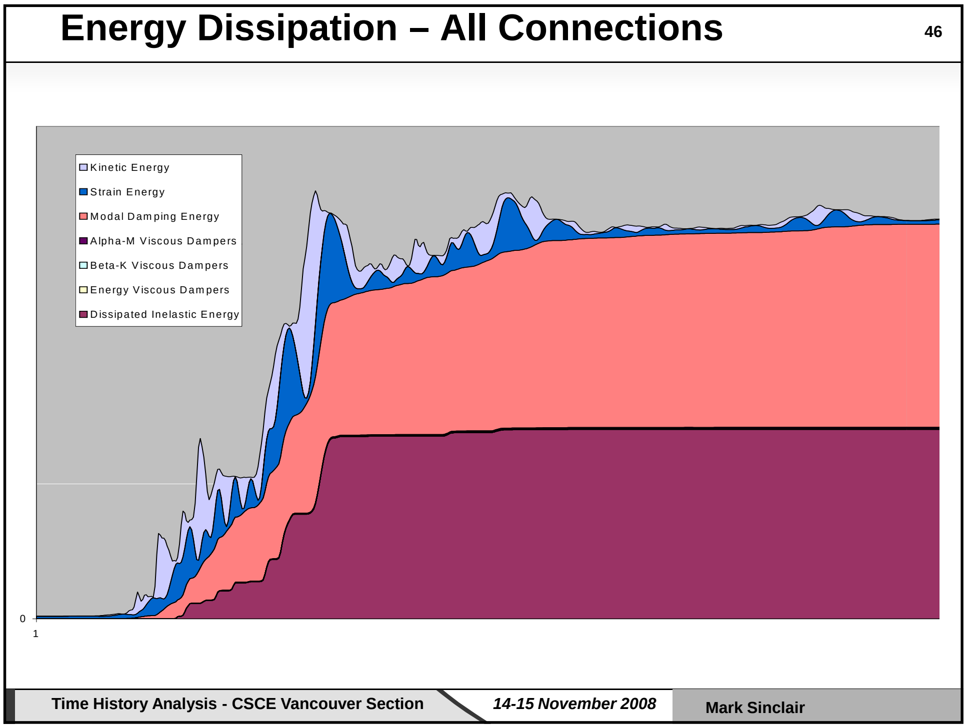


# Energy Dissipation – Existing Building

Time History Analysis - CSCE Vancouver Section

*14-15 November 2008*

Mark Sinclair

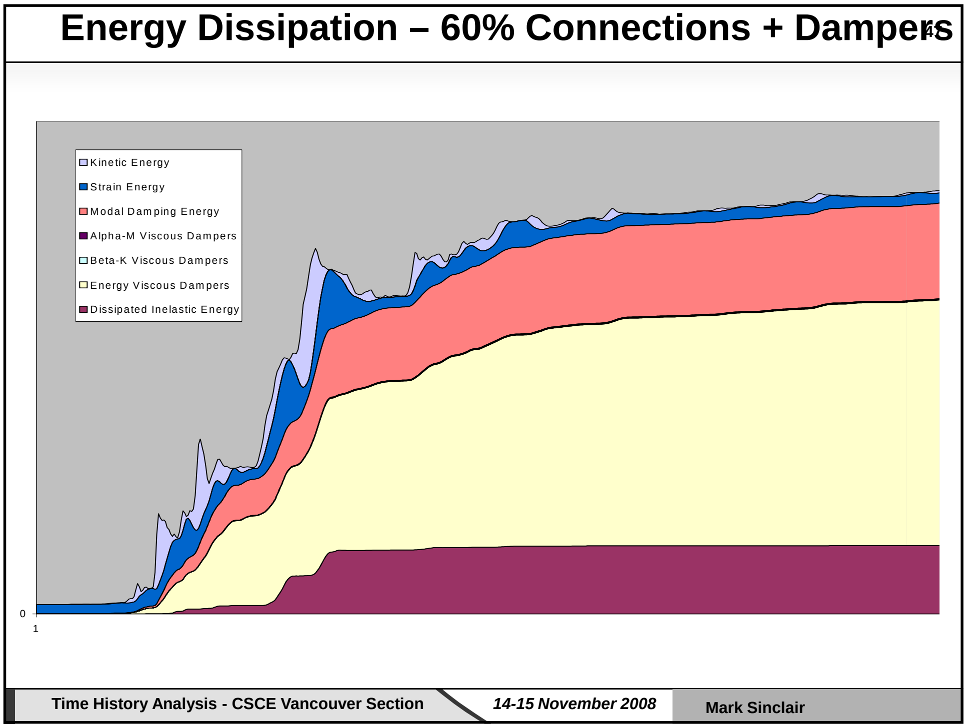


# Energy Dissipation – All Connections

Time History Analysis - CSCE Vancouver Section

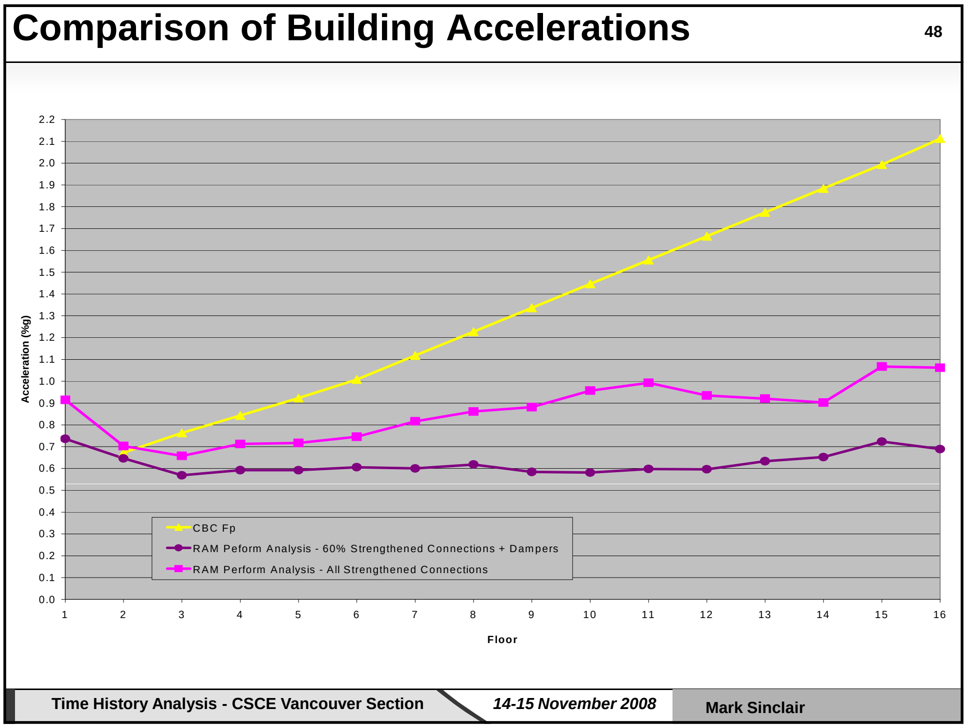
*14-15 November 2008*

Mark Sinclair



## Energy Dissipation – 60% Connections + Dampers

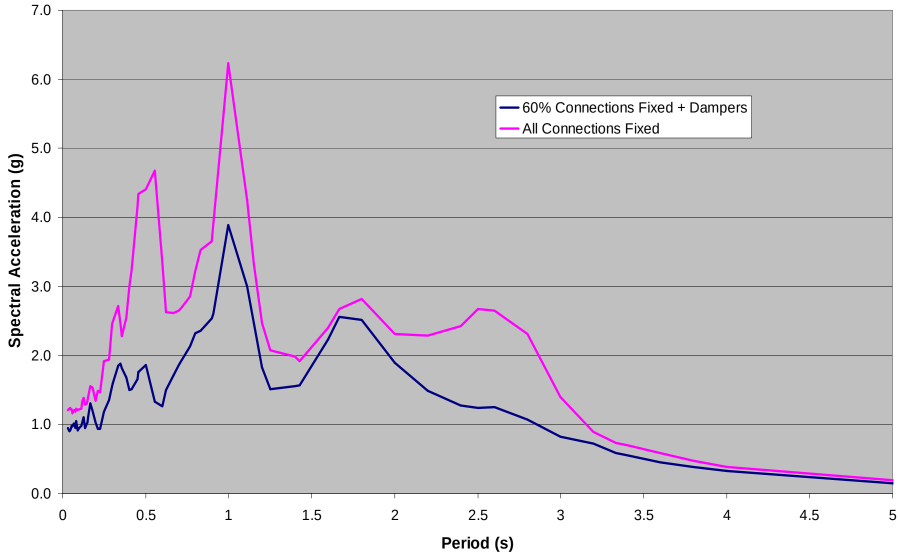
**Time History Analysis - CSCE Vancouver Section** *14-15 November 2008* **Mark Sinclair**



## Comparison of Building Accelerations

**Time History Analysis - CSCE Vancouver Section** *14-15 November 2008* **Mark Sinclair**

## Comparison of Building Roof Spectra



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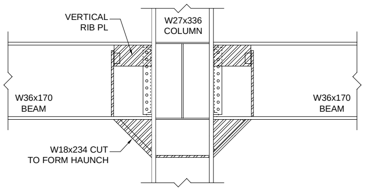
## Project Specific Connection Testing

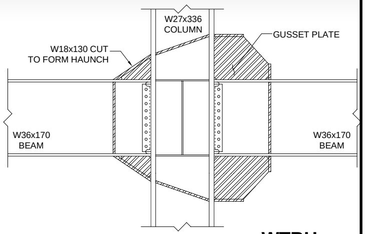
**Specimens 1&4**

**W27x336- Column**

**W36x170- Beams**



Specimen 1 **WBH**

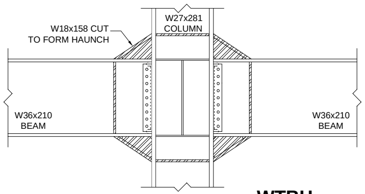


Specimen 4 **WTBH**

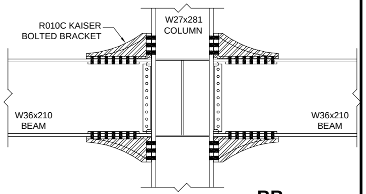
**Specimens 2&3**

**W27x281- Column**

**W36x210- Beams**

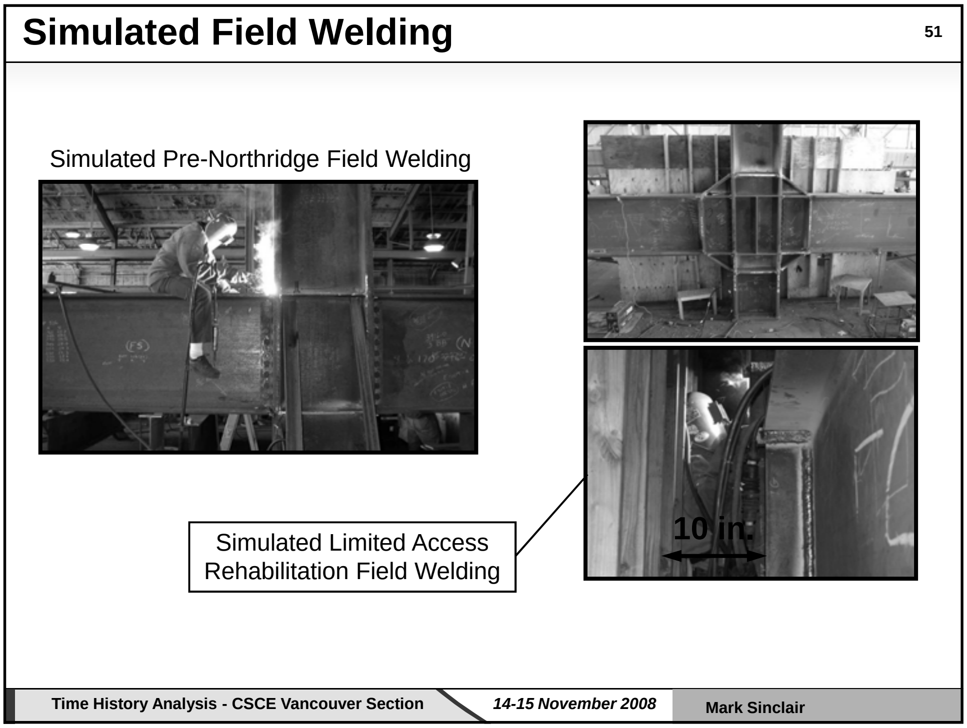


Specimen 2 **WTBH**



Specimen 3 **BB**

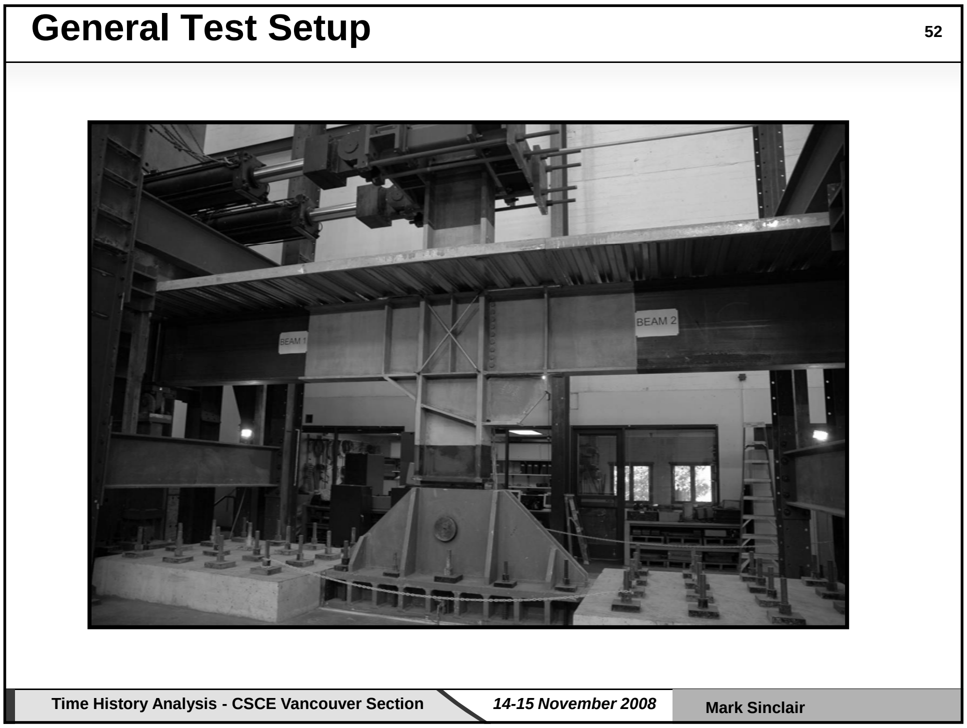
- FEMA 351 pre-qualified connections limited to W14 columns and W36x140 and smaller beam sections
- Previous project-specific tests at UCB performed poorly.



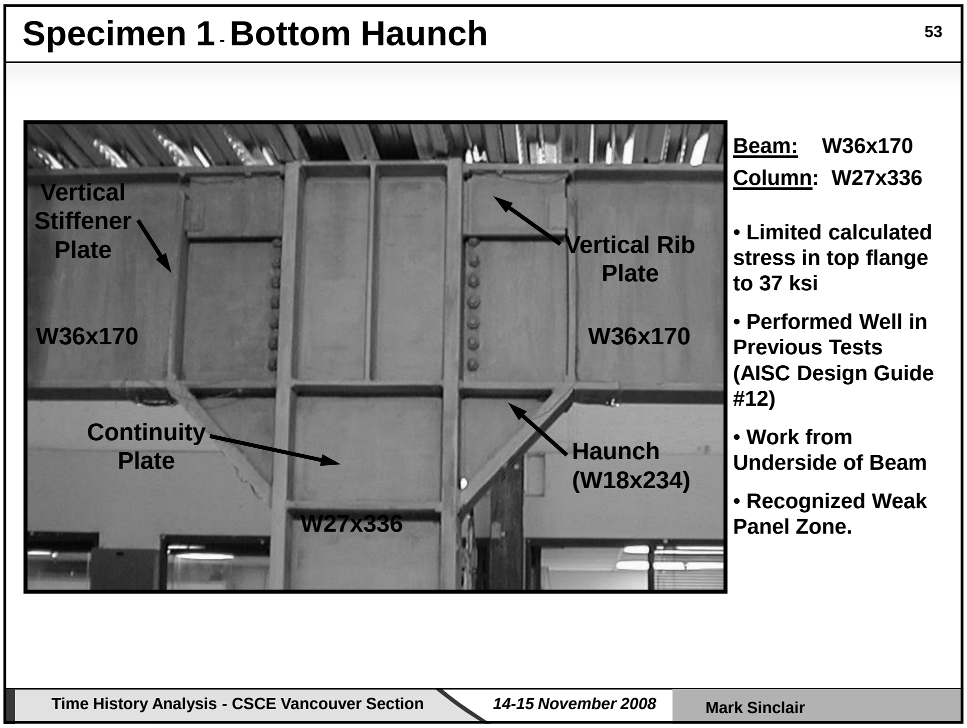
## Simulated Field Welding

Simulated Pre-Northridge Field Welding

Simulated Limited Access Rehabilitation Field Welding



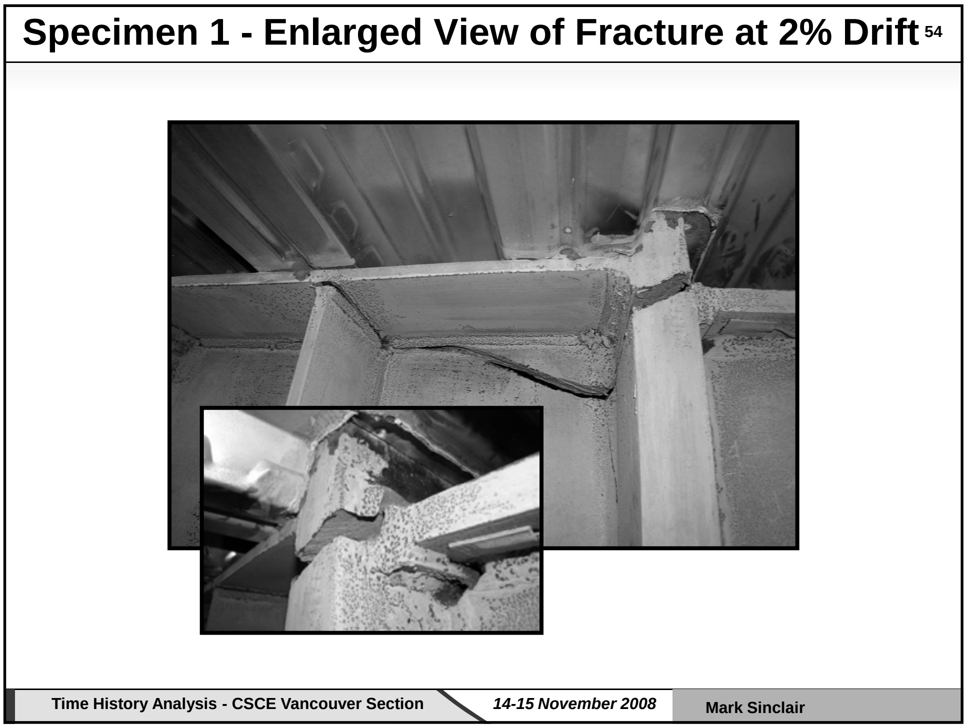
## General Test Setup



## Specimen 1 - Bottom Haunch

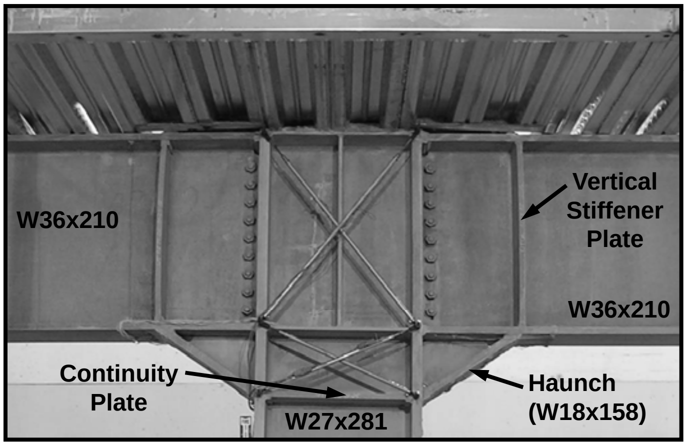
**Beam:** W36x170
**Column:** W27x336

- **Limited calculated stress in top flange to 37 ksi**
- **Performed Well in Previous Tests (AISC Design Guide #12)**
- **Work from Underside of Beam**
- **Recognized Weak Panel Zone.**



## Specimen 1 - Enlarged View of Fracture at 2% Drift

## Specimen 2 - Double Haunch



**Beam:** W36x210
**Column**: W27x281
**Doubler PL**: 3/8 in.

## Specimen 2

**Observations**

**Minor yielding outside haunch @ 1% drift.**

**Doubler Plate buckled on 2nd cycle at 4% Drift.**

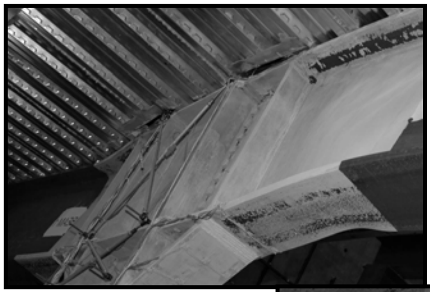
**Flange buckling, Lateral torsional buckling, web buckling noticed at 4% Drift.**

**Fracture**

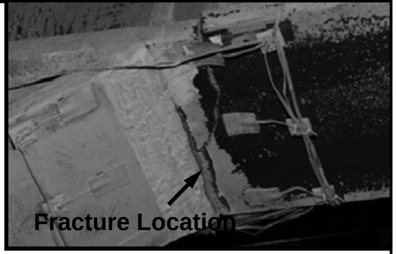
**Second negative excursion at 5% drift**

**Other Observations**

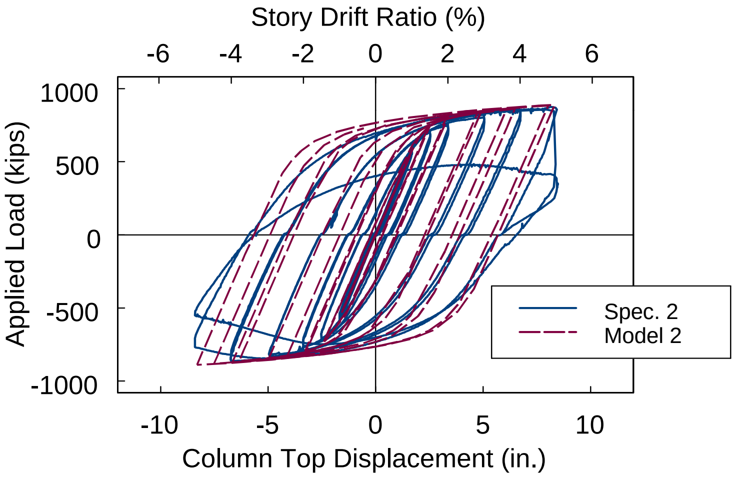
**Significant portion of inelastic drift attributed to panel zone.**



**Beam 2 Yielding and Buckling at 5% Drift**



## Specimen and Model 2



## Specimen 4 - Double Haunch and Gusset Plate

**Beam:** W36x170

**Column:** W27x336

**Observations at 3% Drift**

Local flange buckling, Web buckling, Lateral torsional buckling

**Fracture**

Second negative excursion at 5% drift

**Other Observations**

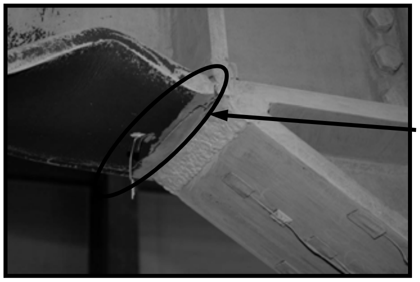
Gusset appeared to protect welds similar to haunch

No noticeable damage to gusset plate.

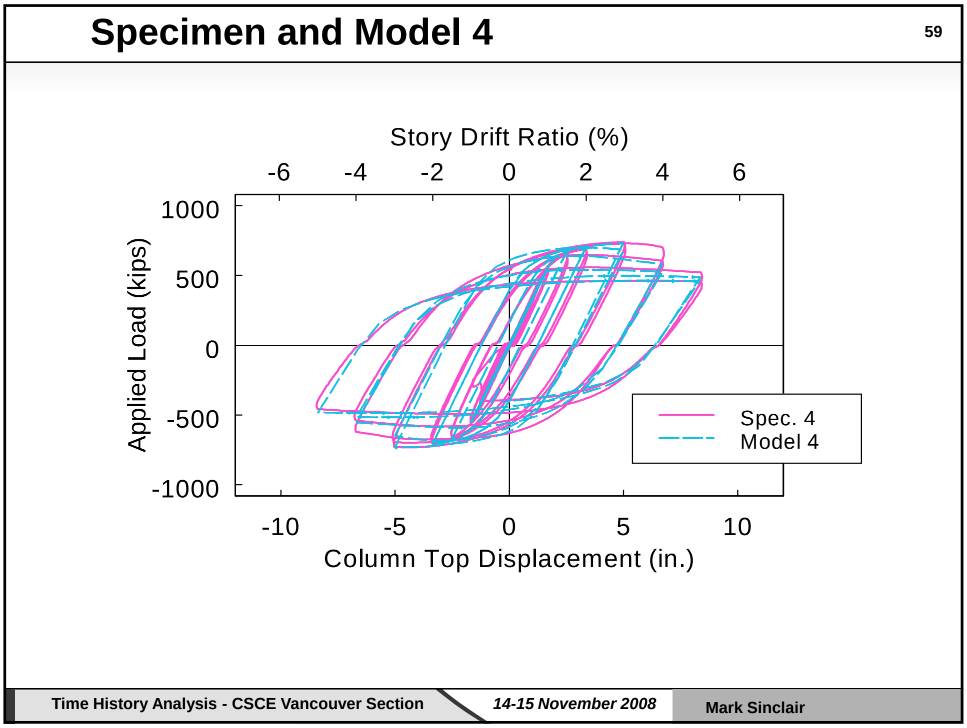
**Beam:** W36x170

**Column:** W27x336

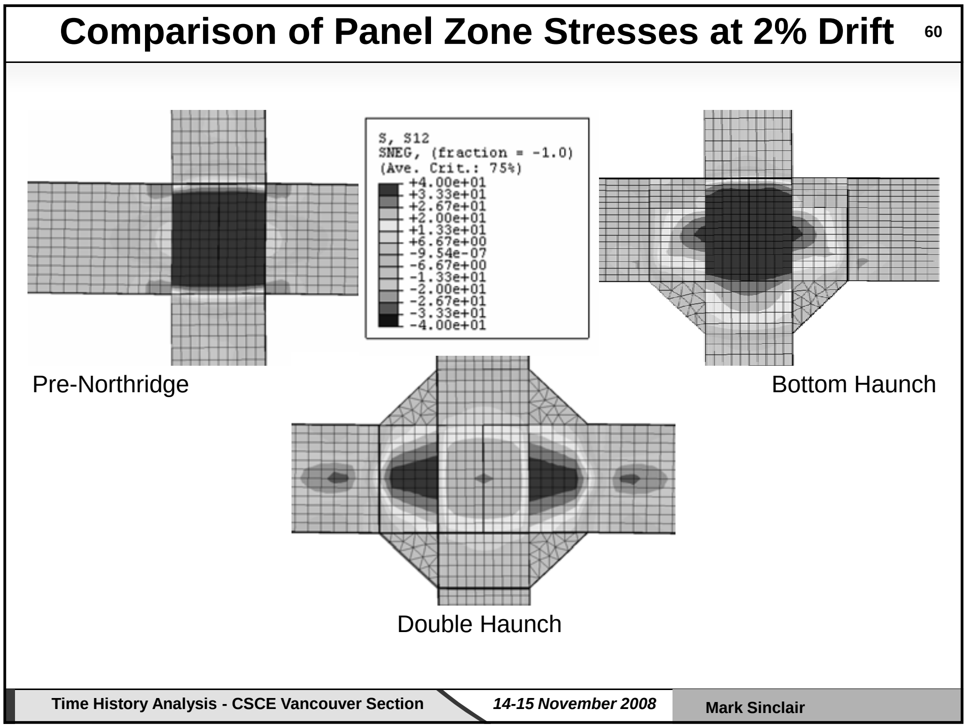
Specimen deformation at 5% drift



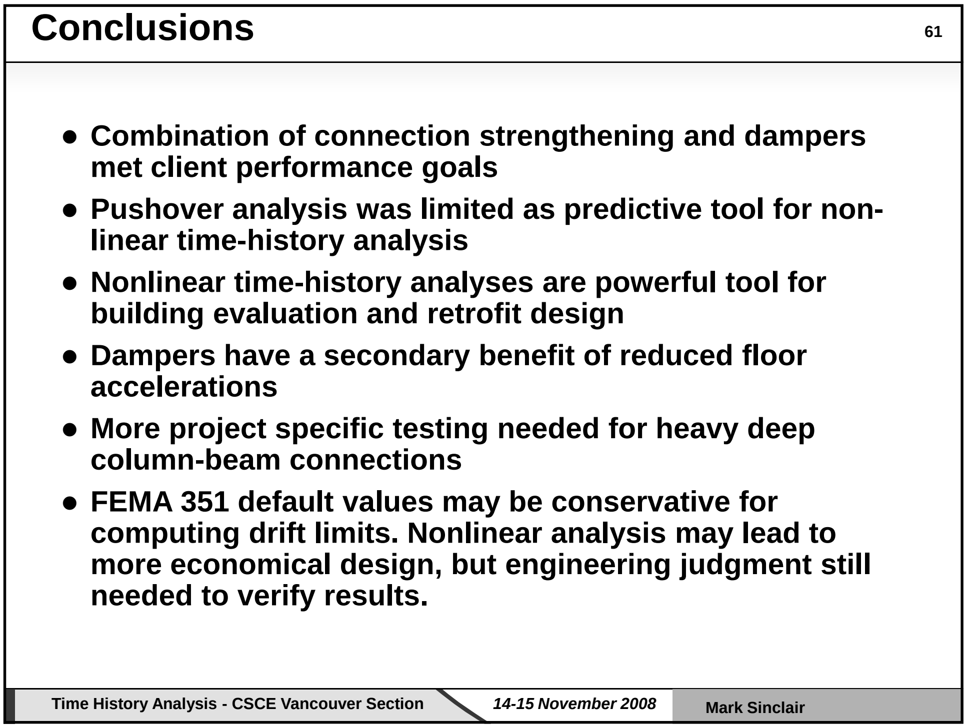
Fracture at 5% Drift



## Specimen and Model 4



## Comparison of Panel Zone Stresses at 2% Drift



## Conclusions

- Combination of connection strengthening and dampers met client performance goals
- Pushover analysis was limited as predictive tool for non-linear time-history analysis
- Nonlinear time-history analyses are powerful tool for building evaluation and retrofit design
- Dampers have a secondary benefit of reduced floor accelerations
- More project specific testing needed for heavy deep column-beam connections
- FEMA 351 default values may be conservative for computing drift limits. Nonlinear analysis may lead to more economical design, but engineering judgment still needed to verify results.

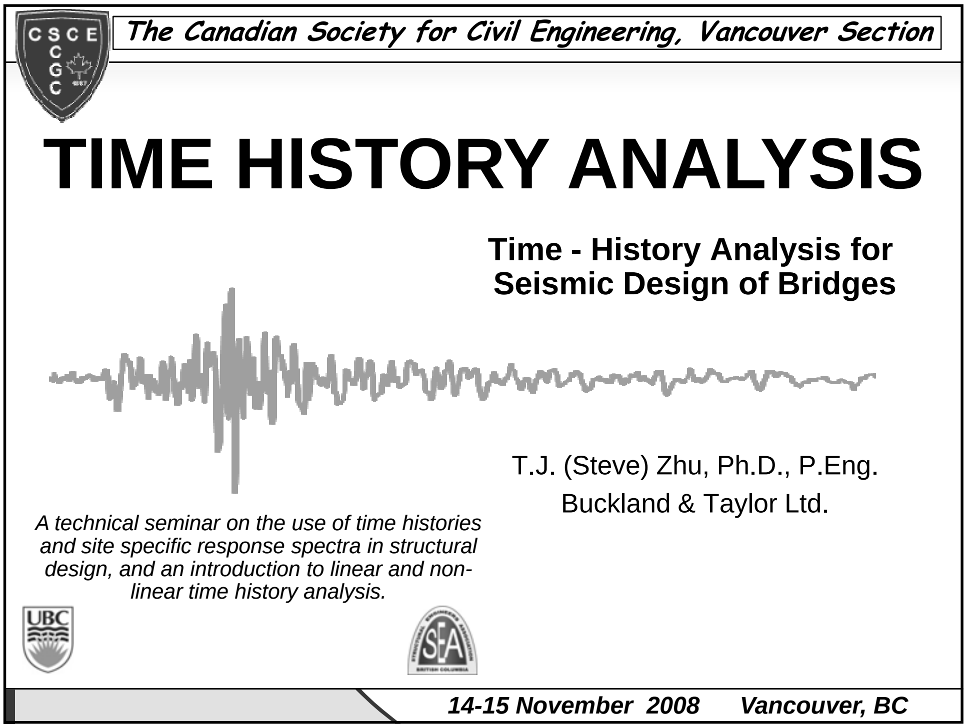
# *TIME HISTORY ANALYSIS*

## LECTURE # 10

# Time-History Analysis for Seismic Design of Bridges

**T.J. (Steve) Zhu, Ph.D., P.Eng.**
**Buckland & Taylor Ltd.**

Steve Zhu is a senior bridge engineer and seismic specialist with Buckland & Taylor Ltd. He obtained his Ph.D. degree in structural and earthquake engineering from McMaster University in 1990. Dr. Zhu joined Buckland & Taylor Ltd. in 1991 and has worked on most of Company's major seismic projects since, including both design of new bridges and retrofit of existing bridges. His bridge project experience includes the Golden Ears Bridge and the Canadian Line North Arm Bridge in Vancouver, the Confederation Bridge in PEI, the Arthur Ravenel Jr. Bridge in the US, the Messina Strait Crossing in Italy, the Chacao Channel Bridge in Chile, the Rion Antirion Bridge in Greece, and seismic retrofit design of the Golden Gate Bridge in the US, the Second Narrows, Port Mann, Lions Gate, Granville and Burrard Bridges in Vancouver. He serves on the seismic subcommittee of the Canadian Highway Bridge Design Code and has co-authored several papers on seismic analysis and design of bridge structures.



*The Canadian Society for Civil Engineering, Vancouver Section*

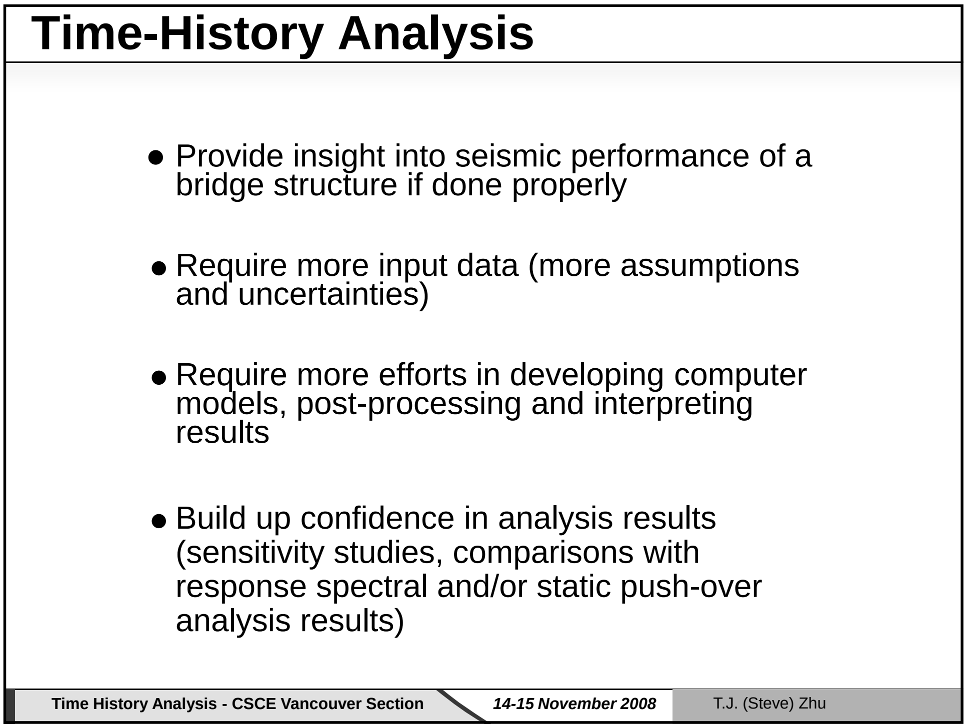
# TIME HISTORY ANALYSIS

## Time - History Analysis for Seismic Design of Bridges

T.J. (Steve) Zhu, Ph.D., P.Eng.
Buckland & Taylor Ltd.

*A technical seminar on the use of time histories and site specific response spectra in structural design, and an introduction to linear and non-linear time history analysis.*

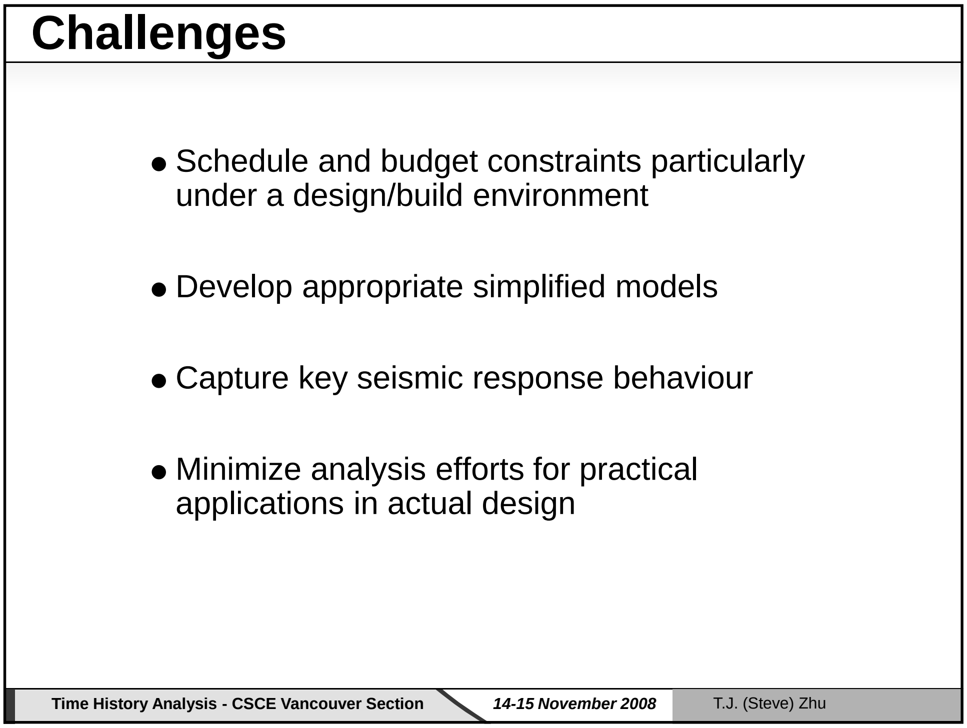
***14-15 November 2008 Vancouver, BC***



## Time-History Analysis

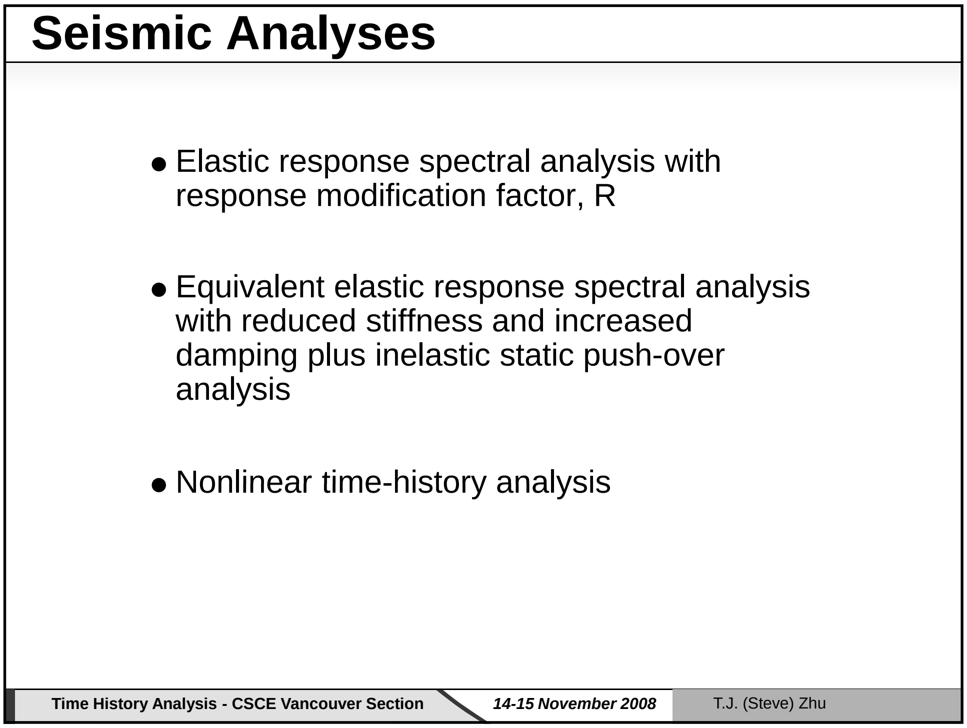
- Provide insight into seismic performance of a bridge structure if done properly
- Require more input data (more assumptions and uncertainties)
- Require more efforts in developing computer models, post-processing and interpreting results
- Build up confidence in analysis results (sensitivity studies, comparisons with response spectral and/or static push-over analysis results)

**Time History Analysis - CSCE Vancouver Section** ***14-15 November 2008*** T.J. (Steve) Zhu



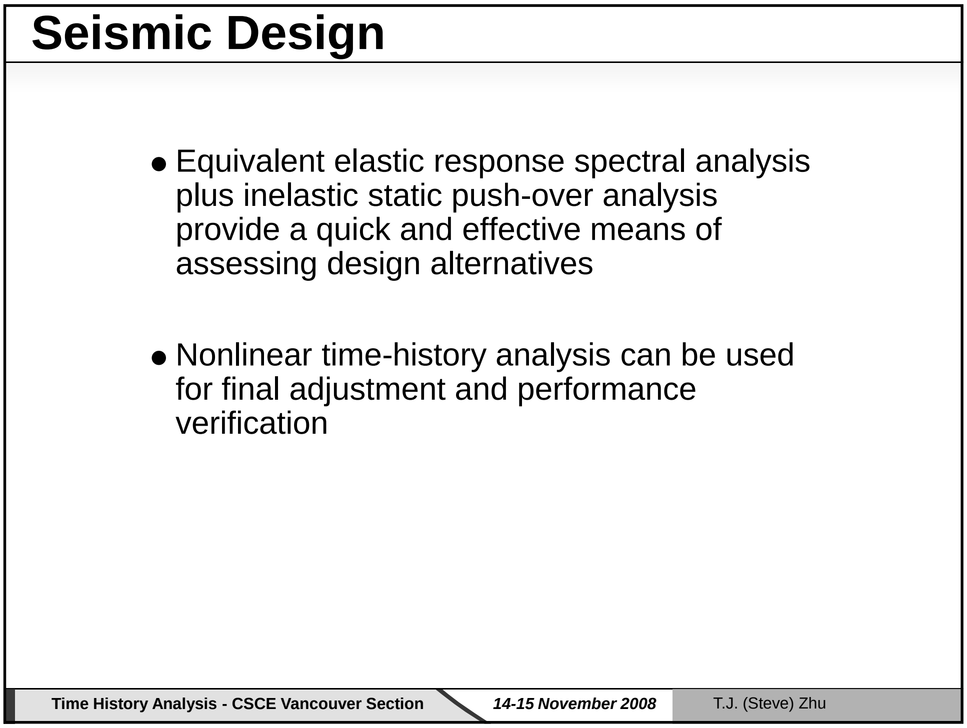
## Challenges

- Schedule and budget constraints particularly under a design/build environment
- Develop appropriate simplified models
- Capture key seismic response behaviour
- Minimize analysis efforts for practical applications in actual design



## Seismic Analyses

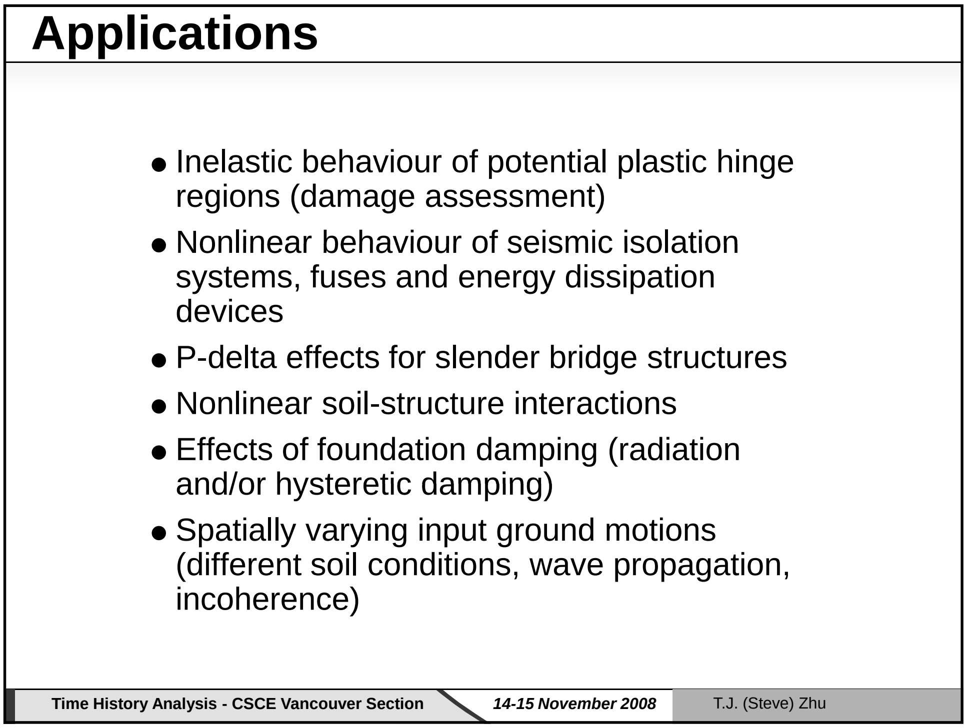
- Elastic response spectral analysis with response modification factor, R
- Equivalent elastic response spectral analysis with reduced stiffness and increased damping plus inelastic static push-over analysis
- Nonlinear time-history analysis



## Seismic Design

- Equivalent elastic response spectral analysis plus inelastic static push-over analysis provide a quick and effective means of assessing design alternatives
- Nonlinear time-history analysis can be used for final adjustment and performance verification

Time History Analysis - CSCE Vancouver Section *14-15 November 2008* T.J. (Steve) Zhu



## Applications

- Inelastic behaviour of potential plastic hinge regions (damage assessment)
- Nonlinear behaviour of seismic isolation systems, fuses and energy dissipation devices
- P-delta effects for slender bridge structures
- Nonlinear soil-structure interactions
- Effects of foundation damping (radiation and/or hysteretic damping)
- Spatially varying input ground motions (different soil conditions, wave propagation, incoherence)

Time History Analysis - CSCE Vancouver Section *14-15 November 2008* T.J. (Steve) Zhu

## Assumptions and Uncertainties

- Input ground motions
  - Select earthquake records resulting from appropriate seismic environment
- Soil-structure interactions
  - Based on best estimate values
  - Consider lower and upper bound values
- Material properties & hysteretic models
  - Expected material strengths
  - Calibration with available experimental data
- Nonlinear properties of seismic isolation bearings, fuses and energy dissipation devices
  - Calibration with testing data

## Strategies

- What are the seismic design strategies?
- What are the key seismic responses to capture?
- What are the purposes of time-history analysis?
- How to capture key seismic response behaviour but keep analysis model relatively simple?
- How to deal with interface between structural and geotechnical modelling?
- Progressive analyses to build up confidence on the results
- What are the main assumptions? How to address uncertainties associated with these assumptions?
- How to apply analysis results in design?

## Progressive Analyses

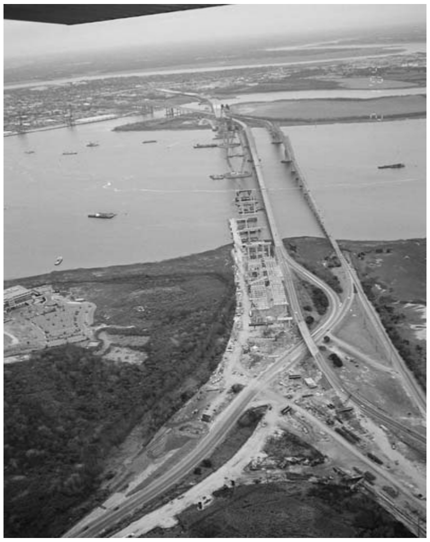
- Modal periods and mode shapes
- Elastic response spectral analysis
- Equivalent elastic response spectral analysis with reduced stiffness and increased damping plus inelastic static push-over analysis
- Elastic time-history analysis
- Nonlinear time-history analysis
- Sensitivity studies

## Projects

### The Arthur Ravenel Jr. Bridge

Charleston, South Carolina, USA

# Construction



# Construction

## Construction

## The Arthur Ravenel Jr. Bridge

## The Arthur Ravenel Jr. Bridge

## Seismic Performance Criteria

**Function Evaluation Earthquake (FEE) - 500 Year**

- Remain in the elastic range

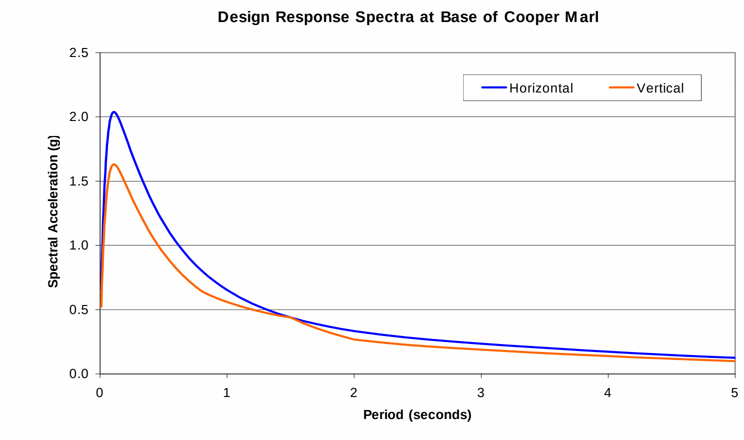
**Safety Evaluation Earthquake (SEE) - 2500 Year**

- Provide access for emergency traffic immediately following SEE
- Repaired and returned to service shortly after SEE

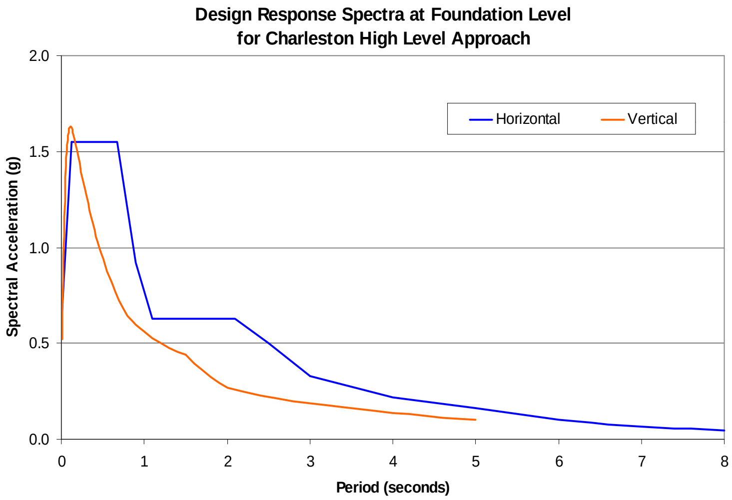
# Soil Condition

- A top layer of soft alluvial deposits for riverbed and soft surficial soils for land portions - 15 to 18 m (50 to 60 ft) deep
- A deep layer of stiff clay known as Cooper Marl
- Outcropping firm ground (soft rock) estimated at a depth of 90 m (300 ft) below ground surface
- Bearing stratum throughout bridge site - Cooper Marl

# Soft Rock Design Response Spectra



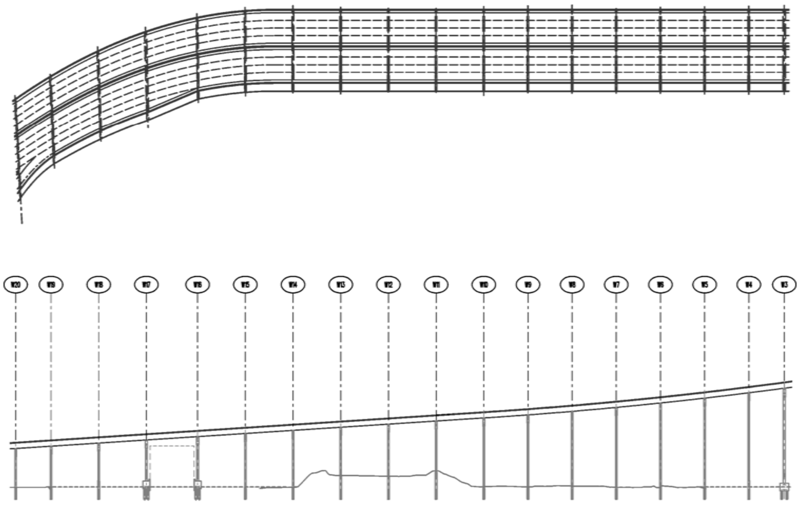
# Soil Design Response Spectra



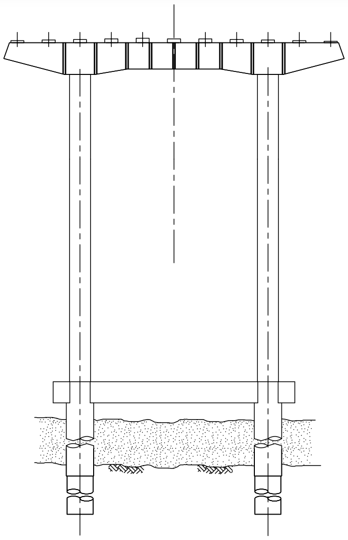
# Design Strategy

- Minimize weight of superstructure (steel plate girders composite with a concrete deck)
- Introduce sufficient flexibility in substructure (tall slender double column bents founded on drilled shafts)
- Make each high-level approach continuous over a significant length

## Charleston High Level Approach



## Charleston High Level Approach



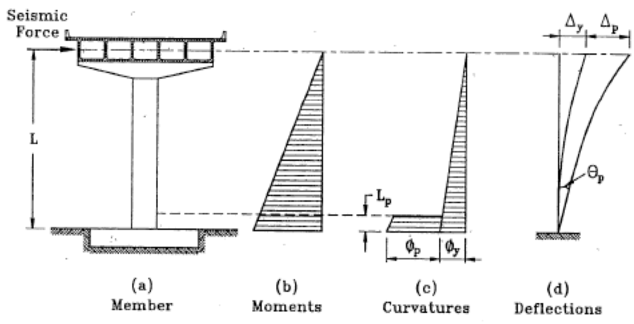
## Key Seismic Behaviour

- Geometric nonlinearities (P-delta effect, slenderness effect, large deformation)
- Inelastic behaviour of column potential plastic hinge regions

## Modelling Strategies

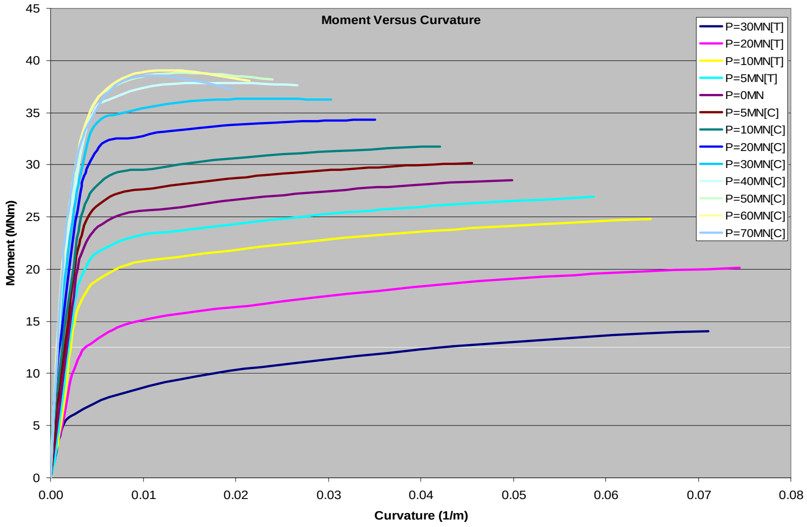
- Inelastic elements for column potential plastic hinge regions only
- Large deformation formulation to capture geometric nonlinearities
- Simplified equivalent elastic elements (secant stiffness) for overall stiffness of drilled shaft foundations – iterations with L-Pile analysis

# Modelling of Plastic Hinge Regions

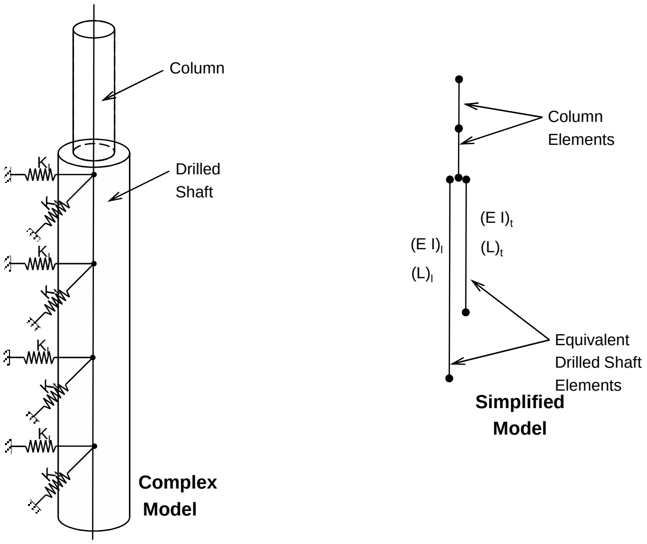


$$L_p = \begin{cases} 0.08L + 0.022 f_{ye} d_{bl} \geq 0.044 f_{ye} d_{bl} & (f_{ye} \text{ in MPa}) \\ 0.08L + 0.15 f_{ye} d_{bl} \geq 0.3 f_{ye} d_{bl} & (f_{ye} \text{ in ksi}) \end{cases}$$

# Modelling of Plastic Hinge Regions



# Modelling of Drilled Shaft Foundations



# Modelling of Drilled Shaft Foundations

- Detailed model of drilled shaft using L-Pile analysis
- Simplified model of drilled shaft in bridge global model
- Apply seismic loads from global model to top of drilled shaft in L-Pile model
- Select values of EI and L in simplified model to match top deflections and rotations from L-Pile analysis
- Iterations required to arrive at appropriate overall effective stiffness

# Performance Criteria

- Limit peak strains in concrete and reinforcing steel to allowable values in column plastic hinge regions

| | |
|---|---|
| Concrete | $\varepsilon_{cmax} \le 0.67\varepsilon_{cu}$ |
| Reinforcing steel | $\varepsilon_{smax} \le 0.67\varepsilon_{su}$ |

$\varepsilon_{cmax}$ = peak compressive strain demand in confined concrete

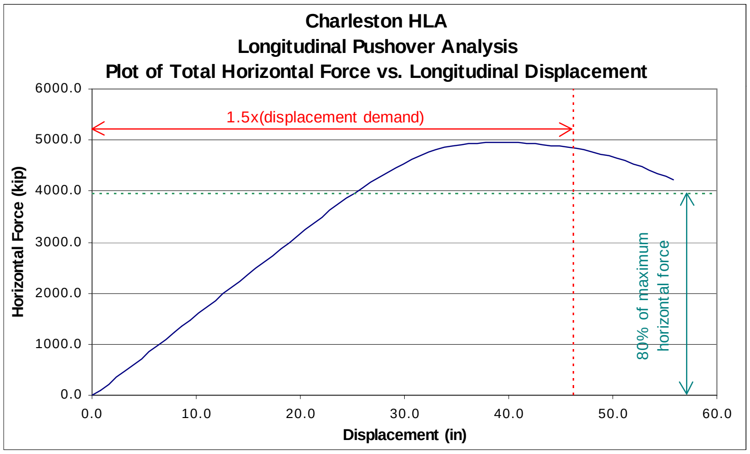
$\varepsilon_{cu}$ = ultimate compressive strain of confined concrete

$\varepsilon_{smax}$ = peak tensile strain demand in reinforcing steel

$\varepsilon_{su}$ = ultimate tensile strain of reinforcing steel

- Drop in horizontal force due to P-delta effects shall not exceed 20% at a horizontal deck displacement of 1.5 x maximum displacement

# Inelastic Static Pushover Analysis

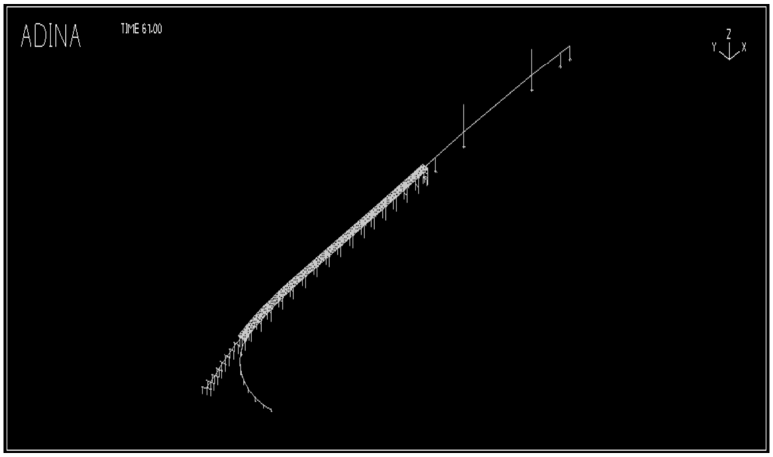


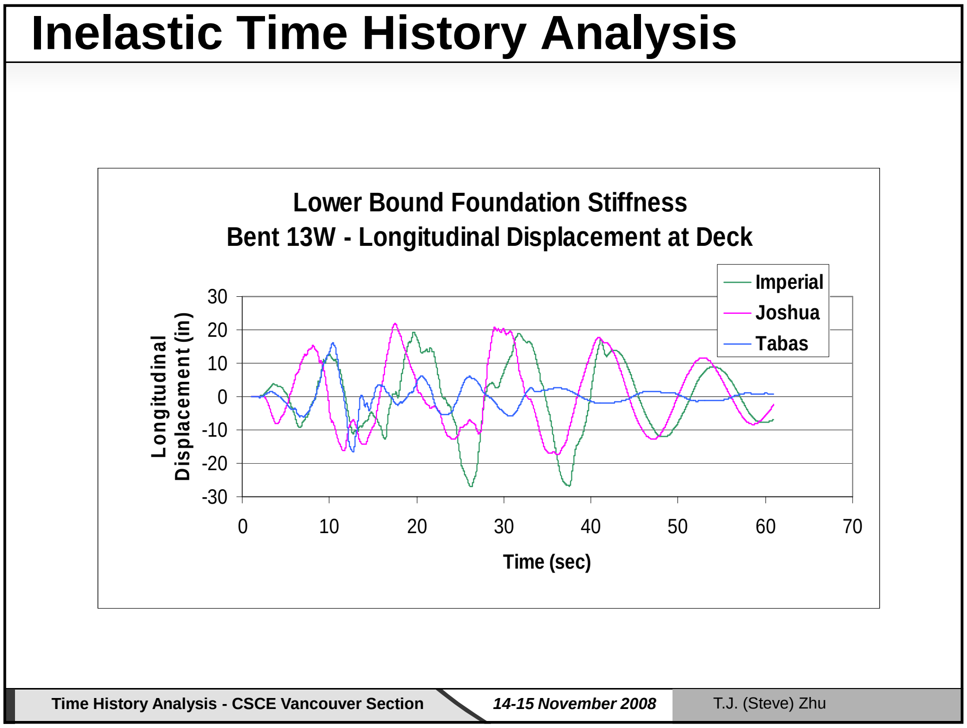
## Performance Verification

- Inelastic time-history analysis
- Both geometric and material nonlinearities considered
- Soil-structure interactions considered
- Three sets of input ground motion time histories
- Spatially varying displacement time history inputs
- Compare peak curvature demand with allowable curvature in all column plastic hinge regions
- Check dynamic stability during seismic response

## 3D Computer Model

**Charleston High Level Approach**





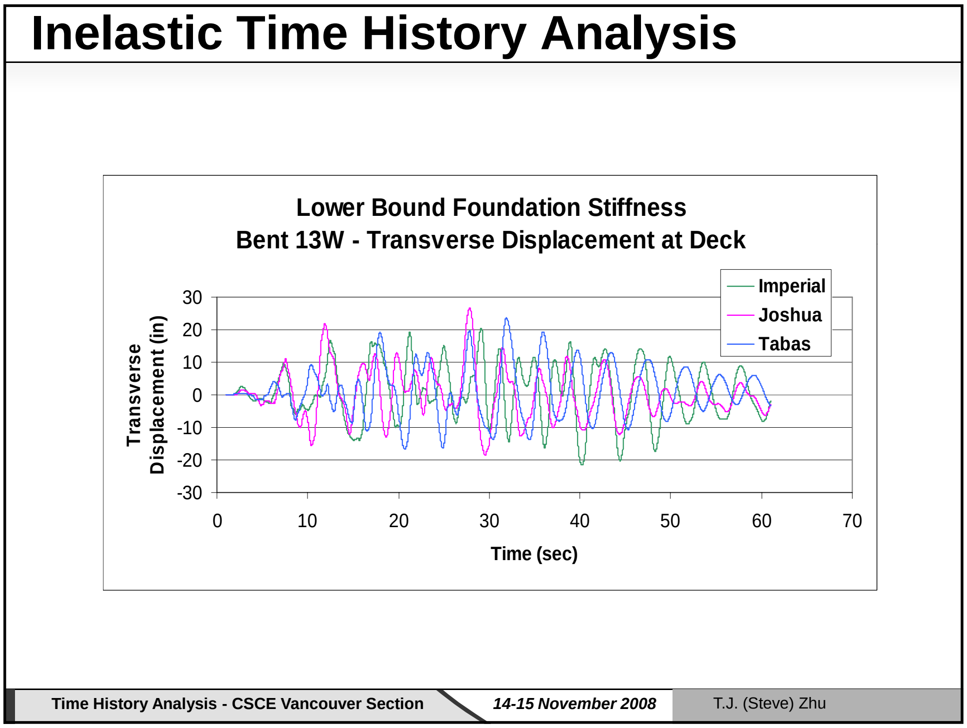
# Inelastic Time History Analysis

Lower Bound Foundation Stiffness
Bent 13W - Longitudinal Displacement at Deck

Time History Analysis - CSCE Vancouver Section

*14-15 November 2008*

T.J. (Steve) Zhu



# Inelastic Time History Analysis

Lower Bound Foundation Stiffness
Bent 13W - Transverse Displacement at Deck

Time History Analysis - CSCE Vancouver Section

*14-15 November 2008*

T.J. (Steve) Zhu

## Projects

# The Golden Ears Bridge

Vancouver, British Columbia, Canada

## Construction

# Construction

# Construction

## Construction

## Main Spans

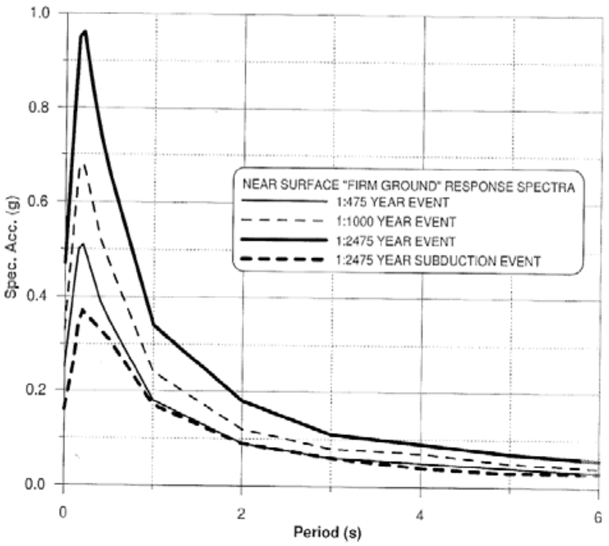
## Seismic Performance Criteria

| Ground Motion Return Period | Service Performance Level | Damage Performance Level |
|---|---|---|
| 475 years<br>(10% in 50 years) | Immediate Access | Minimal Damage |
| 1000 years<br>(5% in 50 years) | Limited Access | Repairable Damage |
| 2500 years<br>(2% in 50 years) | - | Significant Damage |

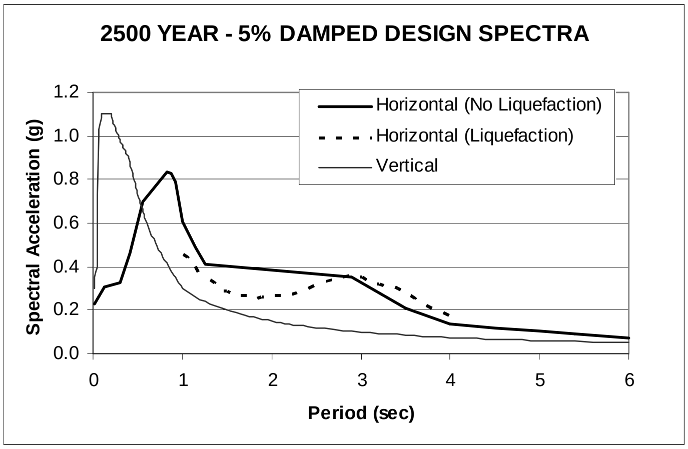
## Soil Condition

- Zone 1 - a top layer of varied deposits of sand, gravel, firm to soft silt, organic silt ranging from 10 m to 40 m deep
- Zone 2 - A deep layer of stiff clay with sections of clayey silt and silt (no bottom found at depth of about 120 m)
- Bearing stratum throughout bridge site – Zone 2

# Firm Ground Response Spectra



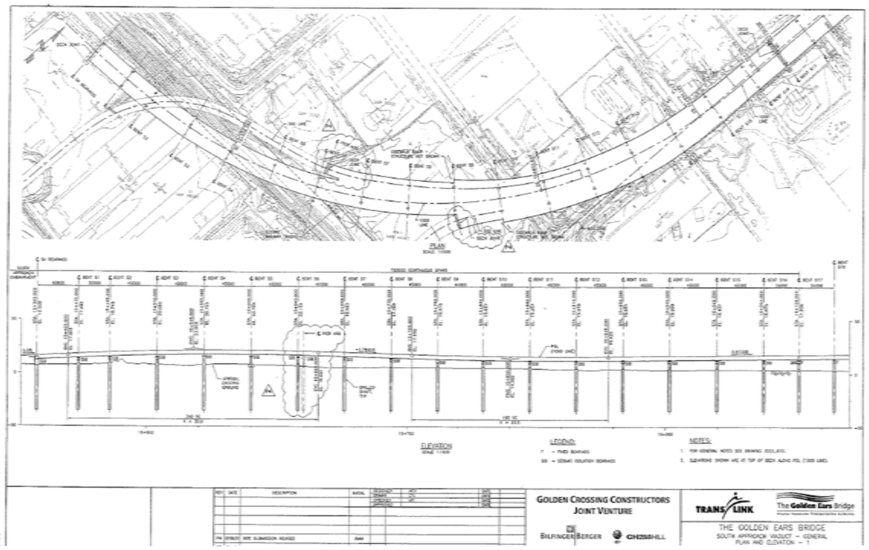
# Soil Design Response Spectra



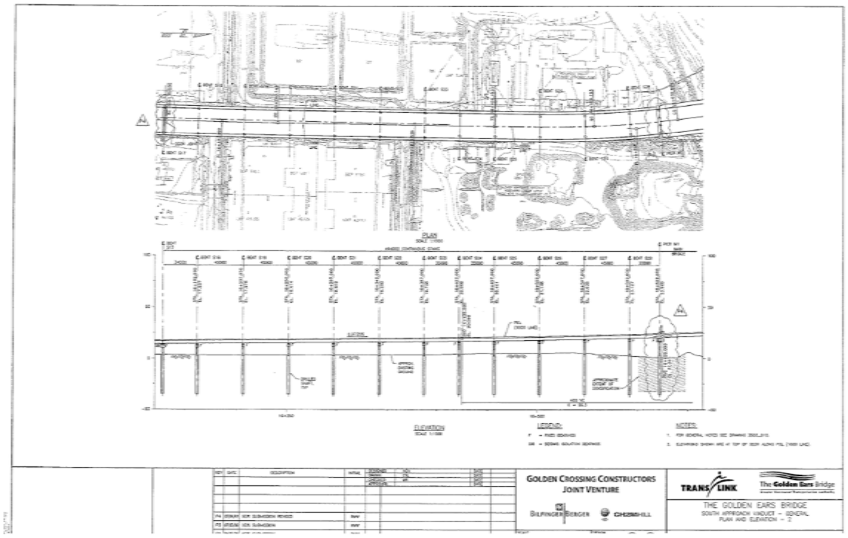
# Design Strategies for Approaches

- Seismic isolation of a continuous south approach section with short piers (733 m from south abutment to Pier S17)
- Use of a continuous deck to tie tall piers together (494 m from Piers S17 to M1) - inelastic behaviour in potential column plastic hinges to dissipate seismic energy
- Design of drilled shaft foundations for soil liquefaction at Piers S22 to S28

# South Approach



# South Approach



# Key Seismic Behaviour for Approaches

- Nonlinear behaviour of seismic isolation bearings
- Inelastic behaviour of column potential plastic hinge regions
- Geometric nonlinearities for tall piers with liquefied soil conditions

## Performance Criteria

- Limit peak strains in concrete and reinforcing steel to allowable values in column plastic hinge regions

| | |
|---|---|
| Concrete | $\varepsilon_{cmax} \leq 0.75\varepsilon_{cu}$ |
| Reinforcing steel | $\varepsilon_{smax} \leq 0.75\varepsilon_{su}$ |

$\varepsilon_{cmax}$ = peak compressive strain demand in confined concrete
$\varepsilon_{cu}$ = ultimate compressive strain of confined concrete
$\varepsilon_{smax}$ = peak tensile strain demand in reinforcing steel
$\varepsilon_{su}$ = ultimate tensile strain of reinforcing steel

- Stability of tall piers with liquefied soil conditions
- Stability of seismic isolation bearings

## Modelling Strategies for Approaches

- Nonlinear elements for seismic isolation bearings
- Inelastic elements for column potential plastic hinge regions only
- Large deformation formulation to capture geometric nonlinearities for tall piers with liquefied soil conditions
- Iterations with L-Pile analysis to capture overall effective stiffness of drilled shaft foundations
- Use of p-multipliers to simulate post-liquefaction soil properties in L-Pile analysis – calibrated with FLAC analysis
- Pre- and post-liquefaction input time histories from FLAC analysis

# Testing of Seismic Isolation Bearings

*EUCENTRE TREES LAB*

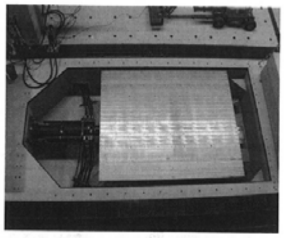


*Figure 1.* The TREES Lab Shaking Table

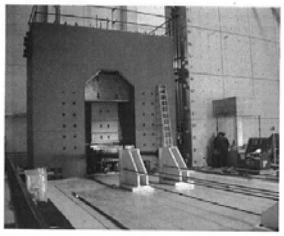


*Figure 2.* The TREES Lab Bearing Tester

# Testing of Seismic Isolation Bearings

*Prototype Tests of 2 ALGAPEND APS 9100 1200-5* 17

**6.2 ALGAPEND APS 9100/1200-5 #2**

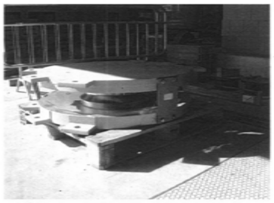


*Figure 11.* ALGAPEND APS 9100/1200-5 # 2 prior testing

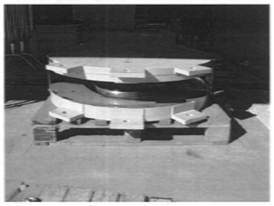


*Figure 12.* ALGAPEND APS 9100/1200-5 # 2 prior testing

18 *EUCENTRE TREES LAB*

*Figure 13.* ALGAPEND APS 9100/1200-5 # 2 testing

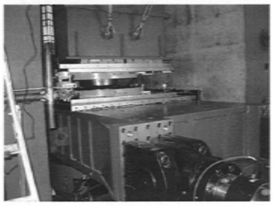
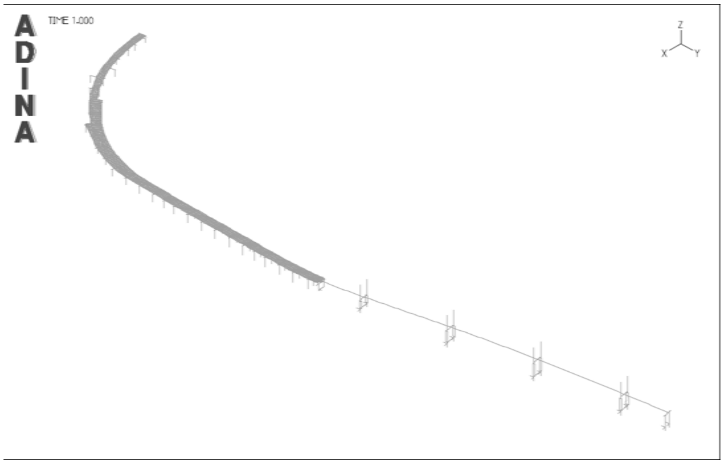


*Figure 14.* ALGAPEND APS 9100/1200-5 # 2 testing

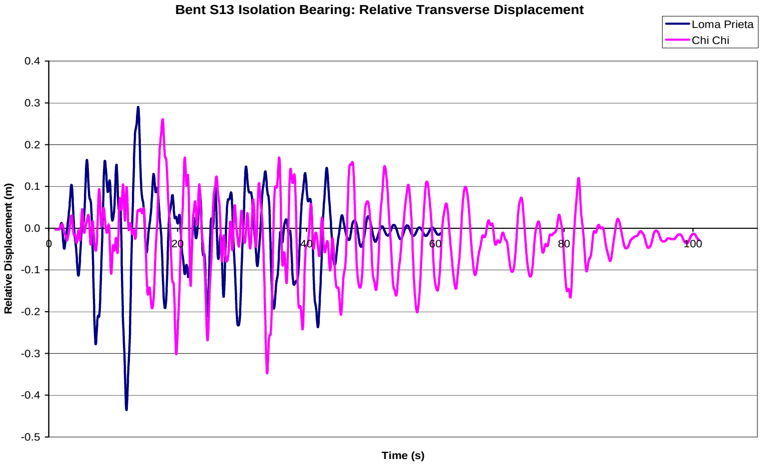
# 3D Computer Model

South Approach



Time History Analysis - CSCE Vancouver Section | *14-15 November 2008* | T.J. (Steve) Zhu

# Nonlinear Time History Analysis



Time History Analysis - CSCE Vancouver Section | *14-15 November 2008* | T.J. (Steve) Zhu

## Design of Drilled Shaft Foundations

- L-Pile analysis used for design of drilled shafts
- Both inertia and kinematic effects considered
- Inertia effects from bridge global model applied in L-Pile analysis
- Ground displacements from FLAC analysis used to evaluate kinematic effects in L-Pile analysis
- Combination of inertia and kinematic effects
- Calibrations with simplified FLAC analysis
- Deeper and heavier rebar cages for drilled shafts in liquefied soil conditions

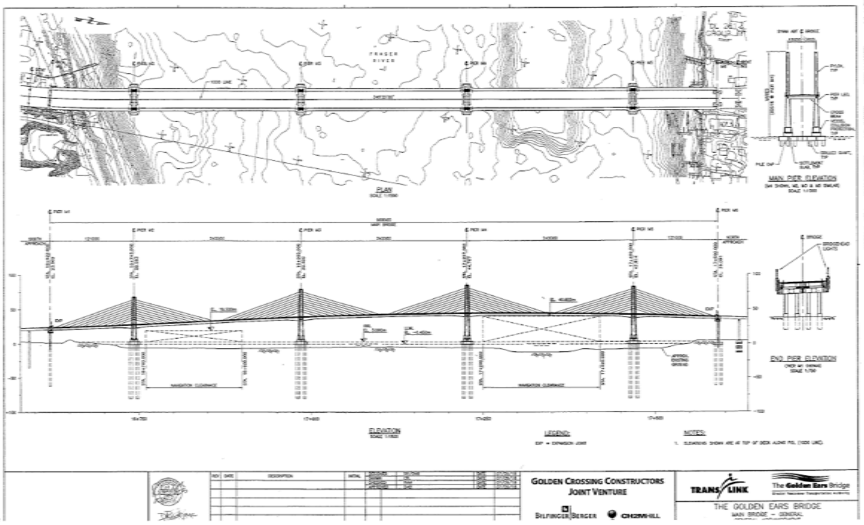
## Drilled Shaft Construction

# Drilled Shaft Construction

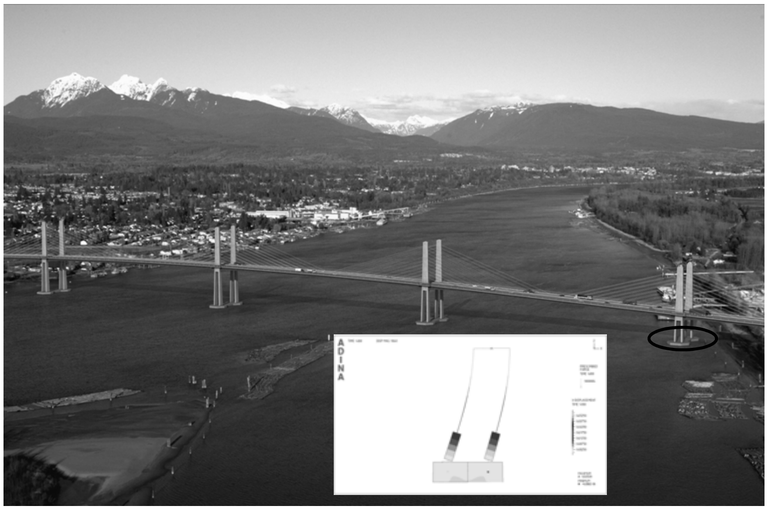
# Design Strategies for Main Spans

- Introduction of a physical pin (flexural yielding of steel plate about transverse axis) at base of pier legs for Pier M2 to prevent significant damage to this short pier
- Inelastic behaviour in potential plastic hinges of lower pier legs (above settlement slab & below deck) to dissipate seismic energy

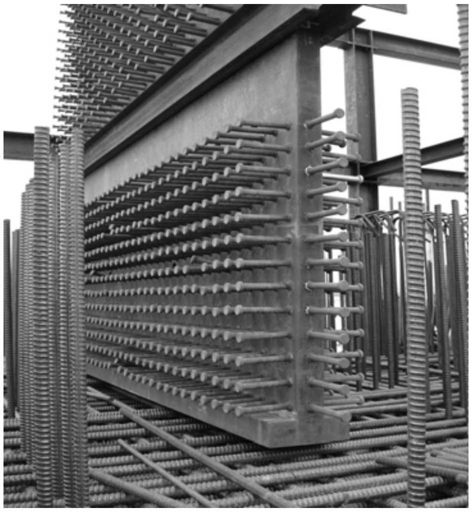
# Main Spans



# Main Spans



## Physical Hinge at Pier M2

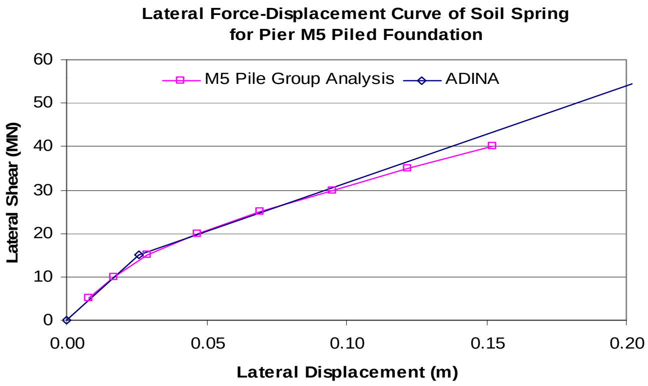


## Modelling Strategies for Main Spans

- Introduction of physical pins (flexural yielding of steel plate about transverse axis) at base of Pier M2
- Inelastic elements for potential plastic hinge regions of lower pier legs – interaction of high axial compression and flexure in plastic hinge regions
- Nonlinear soil springs for piled foundations of the main piers

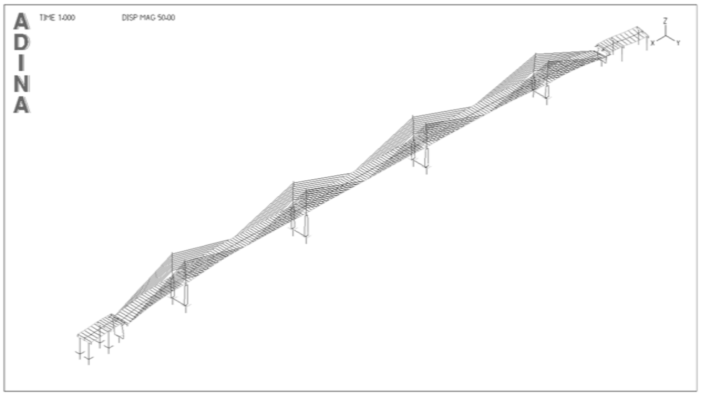
# Piled Foundations of Main Piers

# Nonlinear Soil Springs for Piled Foundations



# 3D Computer Model

**Main Spans**



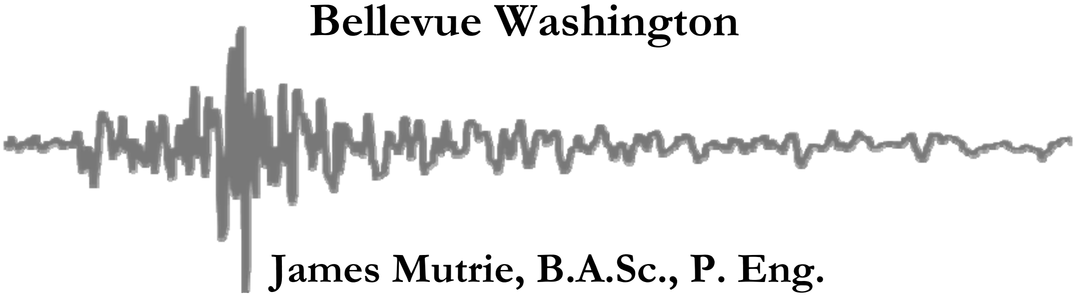
# Summary

- Develop seismic design strategies
- Identify key seismic responses
- Capture key seismic response behaviour while simplifying computer model
- Equivalent elastic response spectral analysis plus inelastic static push-over analysis to assess design alternatives
- Nonlinear time-history analysis to fine tune and verify final design

# TIME HISTORY ANALYSIS

## LECTURE # 11

### Lincoln Square
### Bellevue Washington



**James Mutrie, B.A.Sc., P. Eng.**

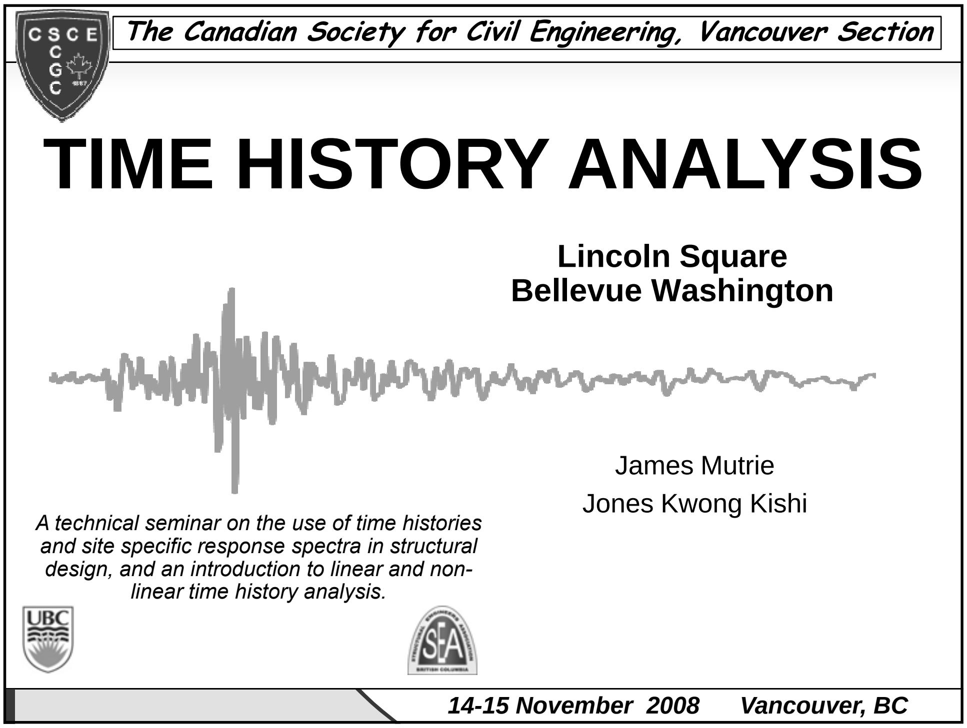
**Jones Kwong Kishi**

James G. Mutrie graduated from the University of British Columbia in 1966, Jim began his engineering career as a Design Engineer, then later Shareholder and Director of Read Jones Christoffersen Ltd. where, for 18 years, he was project engineer on many significant architectural projects in Vancouver. In 1984 he accepted the invitation to become a Partner of Jones Kwong Kishi Consulting Engineers and helped establish the firm as a leader among Vancouver's engineering firms. Over the past 22 years, Jim has been the Principal Engineer on such high-profile projects as, Waterfront Centre, Surrey "Central City" complex, Shaw Tower, and Living Shangri-La which is currently the tallest building in Vancouver. He has a career total of over 25 high rise towers.

His theoretical interest is reflected in the numerous committees he has served on, including active involvement in the development of the Concrete Code as a member since 1980 of the Canadian Standards Association Committee A23.3 "Design of Concrete Structures". He was one of the principal authors of the 1984 edition of A23.3 Clause 21 "Special Provision for Seismic Design" and served as Chairman of the A23.3 Seismic Sub-Committee from 1986 to 2007.

Jim is a member and former councilor of the Association of Professional Engineers of British Columbia, a Director of the Structural Engineers Association of British Columbia and a Fellow of the Canadian Society for Civil Engineering.

His considerable expertise with all areas of the seismic design of high-rise concrete buildings is the result of over 40 years experience in building design and code development.



The Canadian Society for Civil Engineering, Vancouver Section

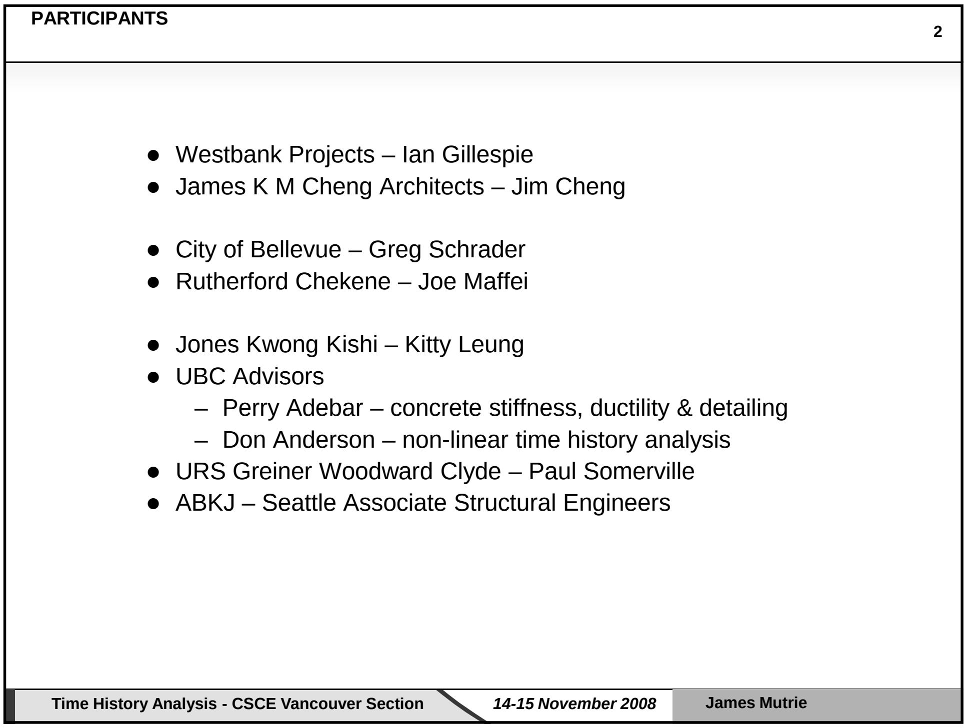
# TIME HISTORY ANALYSIS

## Lincoln Square
## Bellevue Washington

James Mutrie
Jones Kwong Kishi

*A technical seminar on the use of time histories and site specific response spectra in structural design, and an introduction to linear and non-linear time history analysis.*

***14-15 November 2008 Vancouver, BC***



**PARTICIPANTS**

- Westbank Projects – Ian Gillespie
- James K M Cheng Architects – Jim Cheng

- City of Bellevue – Greg Schrader
- Rutherford Chekene – Joe Maffei

- Jones Kwong Kishi – Kitty Leung
- UBC Advisors
  - Perry Adebar – concrete stiffness, ductility & detailing
  - Don Anderson – non-linear time history analysis
- URS Greiner Woodward Clyde – Paul Somerville
- ABKJ – Seattle Associate Structural Engineers

## The Project

## The Project

## Lincoln Square - Bellevue, Washington

- Two Towers
  - 28-floor office tower, steel floor with a connected double concrete core at the center
  - 42-floor hotel / residential tower, concrete flat-plate with 2 concrete cores, one at each end of the floor (discussion will focus on hotel tower)
  - 3-storey structural steel podium consists of 2 levels of retail, the 16-screen Lincoln Square Cinemas, a 20,000 square feet sports club
  - all on top of 6 levels of underground parking

## Uniform Building Code

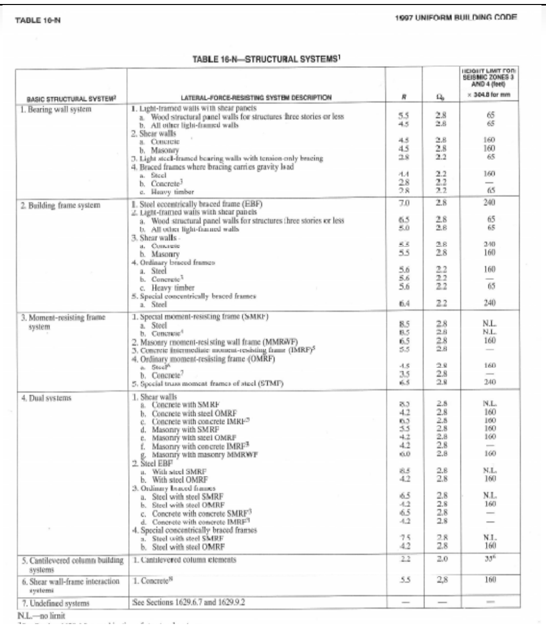


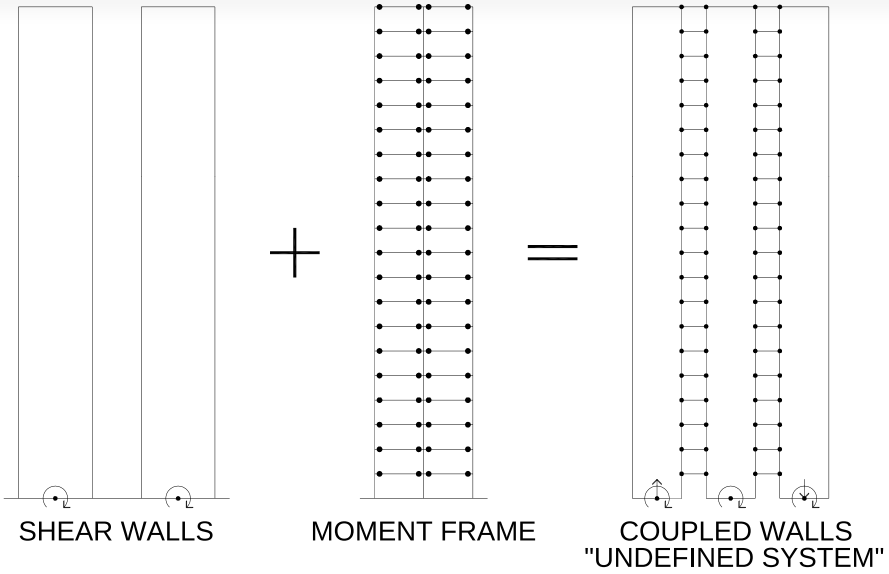
TABLE 16-N 1997 UNIFORM BUILDING CODE

TABLE 16-N—STRUCTURAL SYSTEMS[1]

| BASIC STRUCTURAL SYSTEM[2] | LATERAL-FORCE-RESISTING SYSTEM DESCRIPTION | $R$ | $\Omega_o$ | HEIGHT LIMIT FOR SEISMIC ZONES 3 AND 4 (feet) × 304.8 for mm |
|---|---|---|---|---|
| 1. Bearing wall system | 1. Light-framed walls with shear panels | | | |
| | a. Wood structural panel walls for structures three stories or less | 5.5 | 2.8 | 65 |
| | b. All other light-framed walls | 4.5 | 2.8 | 65 |
| | 2. Shear walls | | | |
| | a. Concrete | 4.5 | 2.8 | 160 |
| | b. Masonry | 4.5 | 2.8 | 160 |
| | 3. Light steel-framed bearing walls with tension-only bracing | 2.8 | 2.2 | 65 |
| | 4. Braced frames where bracing carries gravity load | | | |
| | a. Steel | 4.4 | 2.2 | 160 |
| | b. Concrete[3] | 2.8 | 2.2 | — |
| | c. Heavy timber | 2.8 | 2.2 | 65 |
| 2. Building frame system | 1. Steel eccentrically braced frame (EBF) | 7.0 | 2.8 | 240 |
| | 2. Light-framed walls with shear panels | | | |
| | a. Wood structural panel walls for structures three stories or less | 6.5 | 2.8 | 65 |
| | b. All other light-framed walls | 5.0 | 2.8 | 65 |
| | 3. Shear walls | | | |
| | a. Concrete | 5.5 | 2.8 | 240 |
| | b. Masonry | 5.5 | 2.8 | 160 |
| | 4. Ordinary braced frames | | | |
| | a. Steel | 5.6 | 2.2 | 160 |
| | b. Concrete[3] | 5.6 | 2.2 | — |
| | c. Heavy timber | 5.6 | 2.2 | 65 |
| | 5. Special concentrically braced frames | | | |
| | a. Steel | 6.4 | 2.2 | 240 |
| 3. Moment-resisting frame system | 1. Special moment-resisting frame (SMRF) | | | |
| | a. Steel | 8.5 | 2.8 | N.L. |
| | b. Concrete[4] | 8.5 | 2.8 | N.L. |
| | 2. Masonry moment-resisting wall frame (MMRWF) | 6.5 | 2.8 | 160 |
| | 3. Concrete intermediate moment-resisting frame (IMRF)[5] | 5.5 | 2.8 | — |
| | 4. Ordinary moment-resisting frame (OMRF) | | | |
| | a. Steel[6] | 4.5 | 2.8 | 160 |
| | b. Concrete[7] | 3.5 | 2.8 | — |
| | 5. Special truss moment frames of steel (STMF) | 6.5 | 2.8 | 240 |
| 4. Dual systems | 1. Shear walls | | | |
| | a. Concrete with SMRF | 8.5 | 2.8 | N.L. |
| | b. Concrete with steel OMRF | 4.2 | 2.8 | 160 |
| | c. Concrete with concrete IMRF[5] | 6.5 | 2.8 | 160 |
| | d. Masonry with SMRF | 5.5 | 2.8 | 160 |
| | e. Masonry with steel OMRF | 4.2 | 2.8 | 160 |
| | f. Masonry with concrete IMRF[5] | 4.2 | 2.8 | — |
| | g. Masonry with masonry MMRWF | 6.0 | 2.8 | 160 |
| | 2. Steel EBF | | | |
| | a. With steel SMRF | 8.5 | 2.8 | N.L. |
| | b. With steel OMRF | 4.2 | 2.8 | 160 |
| | 3. Ordinary braced frames | | | |
| | a. Steel with steel SMRF | 6.5 | 2.8 | N.L. |
| | b. Steel with steel OMRF | 4.2 | 2.8 | 160 |
| | c. Concrete with concrete SMRF[3] | 6.5 | 2.8 | — |
| | d. Concrete with concrete IMRF[3] | 4.2 | 2.8 | — |
| | 4. Special concentrically braced frames | | | |
| | a. Steel with steel SMRF | 7.5 | 2.8 | N.L. |
| | b. Steel with steel OMRF | 4.2 | 2.8 | 160 |
| 5. Cantilevered column building systems | 1. Cantilevered column elements | 2.2 | 2.0 | 35[6] |
| 6. Shear wall-frame interaction systems | 1. Concrete[8] | 5.5 | 2.8 | 160 |
| 7. Undefined systems | See Sections 1629.6.7 and 1629.9.2 | — | — | — |

N.L.—no limit

## Conceptual Framework



## Uniform Building Code

**1629.9.2 UNDEFINED STRUCTURAL SYSTEM**

The value of R substantiated by approved cyclic testing and analyses with the following items addressed for an Undefined System

1. Dynamic response characteristics
2. Lateral force resistance
3. Overstrength and strain hardening or softening
4. Strength and stiffness degradation
5. Energy dissipation characteristics
6. System ductility
7. Redundancy

## Uniform Building Code

- Design the complete building lateral system
- Non-linear Time History Analysis
  - Appropriate earthquake records, choice of:
    - 3 events and take to maximum effect or
    - 7+ events and take the average value
- Design Review Process
  - By independent engineering team (working for Bellevue)
  - Review the development of site-specific spectra and ground-motion time histories
  - Review the preliminary design of the lateral force resisting systems
  - Review the final design of the lateral force resisting systems and all supporting analyses

## "Undefined System" Investigation and Review Process

- Guidelines and standards
  - There were no standards or guidelines in existence in 1998 to guide the process
  - We, along with the peer reviewer, developed the Bellevue process as we went along
  - Guidelines now exist such as the one published by the Los Angles Tall Building Structural Design Council and apparently there is also a good one published by SEAONC
  - The next speaker may add to the discussion on how the required process has developed

## Modeling and Detailing Issues

- Understand basic structure behaviour
  - Static “push-over” analyses
- Stiffness assumptions
  - Tri-linear stiffness model
  - Results formed the basis of a paper presented at the 8th CCEE
- Limits on element rotational capacity
  - Taken from ATC 40-1996 and FEMA 273-1998
  - Known to be conservative
- Element detailing
  - UBC 97, CSA 94, NZS 3101-95 , ACI 99

## Acceptance Criteria

- Chord rotations for diagonally reinforced coupling beams

| **Hazard Level** | **Performance Level** | **Acceptable Chord Rotation** |
|---|---|---|
| DBE | Life Safety | 0.018 |
| MCE | Collapse Prevention or Structural Stability | 0.030 |

- Interstorey Drifts

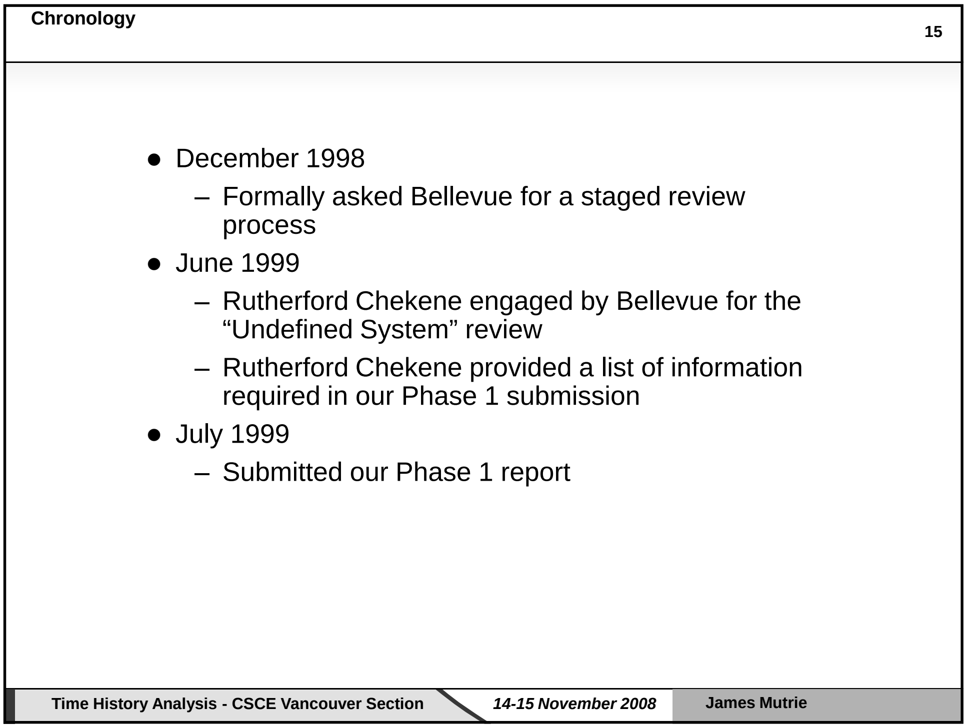
| **Hazard Level** | **Performance Level** | **Acceptable Interstorey Drift** |
|---|---|---|
| DBE | Life Safety | 1.5% |
| MCE | Collapse Prevention or Structural Stability | 2.5% |

## Independent Peer Review

- The depth of thought that goes into each and every step of the design and detailing process is increased by orders of magnitude when you know that the results of your work will be reviewed in detail by a knowledgeable reviewer
- It is so easy for any firm to become insular and believe that what they are doing is absolutely correct, it likely never is
- Knowledgeable and independent peer review must be a mandatory part of the process

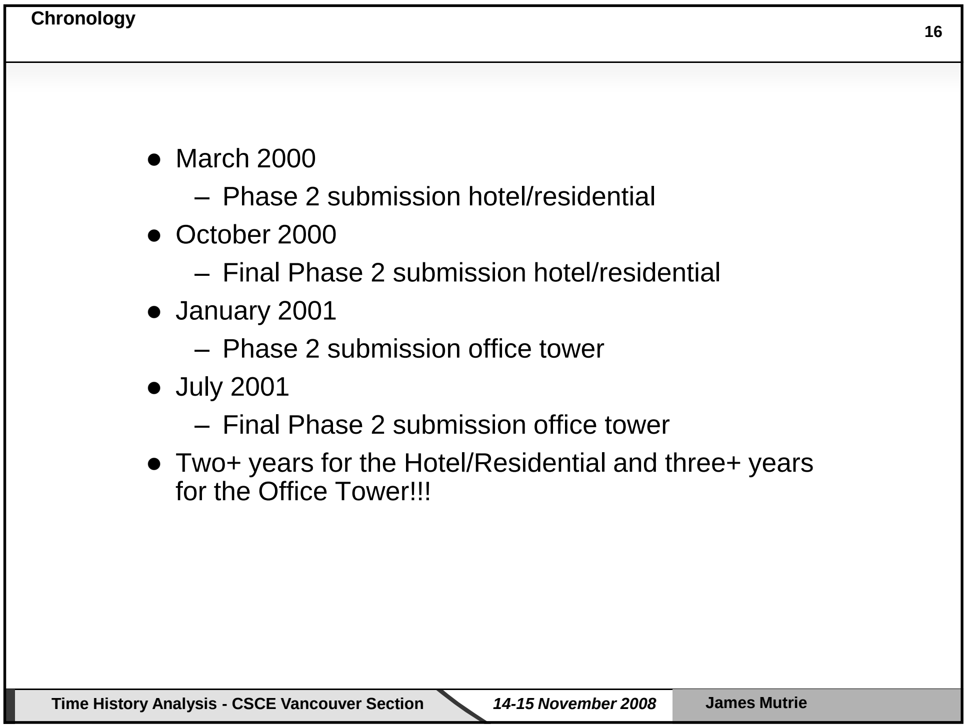
## Chronology

- March 1998
  - First approach to Bellevue
  - Discussed the idea of “Undefined System”
  - Almost never used to that point in time, one project in Seattle around 300 ft. high, Lincoln Square was 450 ft
- August 1998
  - Second approach to Bellevue
  - Provided a detailed outline of our proposed procedure
  - Discussed the behaviour of coupled shear walls and our approach to their design
  - Presented conceptual drawings of both towers
  - Bellevue undertook to develop a process for both the basic review and undefined system review



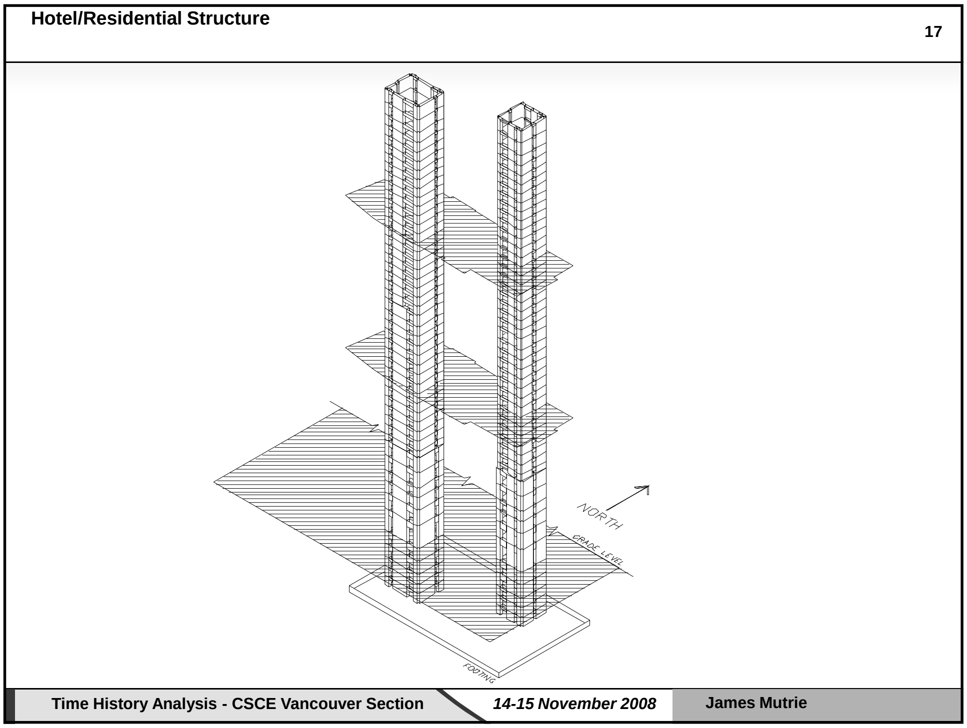
## Chronology

- December 1998
  - Formally asked Bellevue for a staged review process
- June 1999
  - Rutherford Chekene engaged by Bellevue for the “Undefined System” review
  - Rutherford Chekene provided a list of information required in our Phase 1 submission
- July 1999
  - Submitted our Phase 1 report

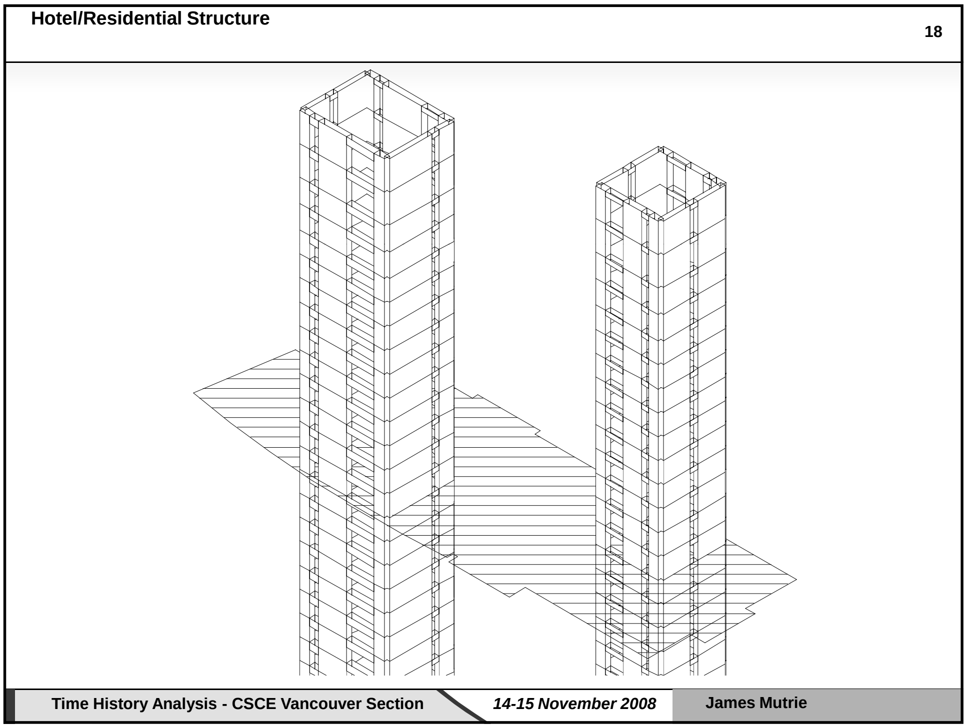


## Chronology

- March 2000
  - Phase 2 submission hotel/residential
- October 2000
  - Final Phase 2 submission hotel/residential
- January 2001
  - Phase 2 submission office tower
- July 2001
  - Final Phase 2 submission office tower
- Two+ years for the Hotel/Residential and three+ years for the Office Tower!!!



## Hotel/Residential Structure

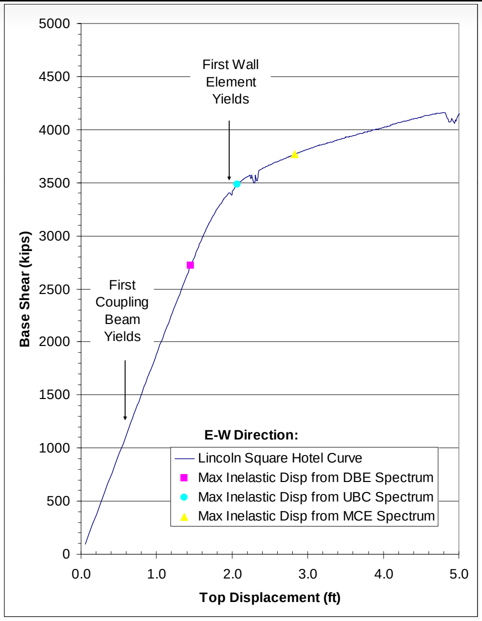


## Hotel/Residential Structure

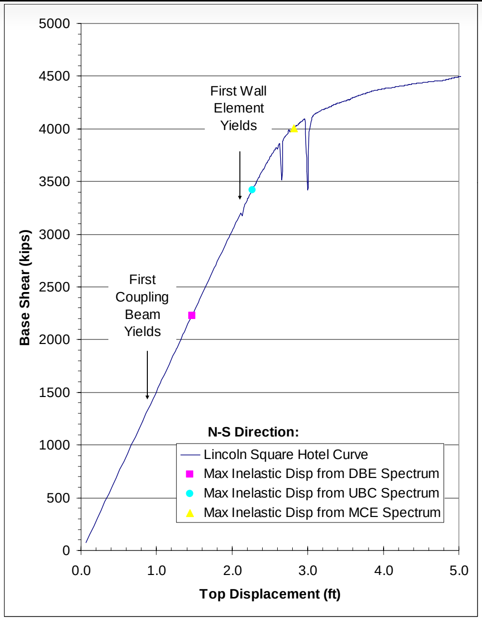
## Analysis Programs

- Linear
  - 3D building finite element program for
    - Static analysis
    - Dynamic modal analysis
- Non-linear static (push-over)
  - Modified 2D linear frame program
  - Drain 2DX
- Non-linear dynamic time history
  - Drain 2DX
- Data reduction outside Drain – **Very** large task – Many hours

## Push-Over Analysis



## Push-Over Analysis

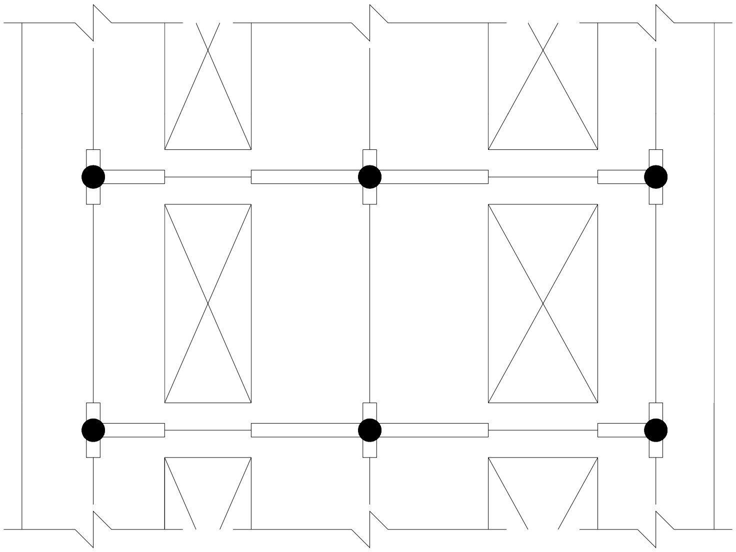


## Drain Modeling Elements

- Drain 2DX Elements
  - Plastic Hinge Beam-Column Element (Type 2)
  - Not too good for P-M interaction but since the primary non-linear building deformation being studied was coupling beam rotation we used the same element for the walls as well.
  - Three yield surface shapes available for this element
    - Used the P-M interaction concrete section yield surface for the walls
    - Simple beam hinge surface for the coupling beams.

## Drain Modeling Elements

- Diagram of model



## Drain2DX Model Assumptions

- Effective stiffness as used in linear models
- 2D model with horizontal nodes slaved together
- Spring elements used to simulate below grade diaphragms
- Additional column element with vertical masses to act as P-Delta driver
- Assumed 3% viscous damping

## Uniform Building Code Base Shear Equations

$$T(\sec) = C_t (h_n)^{3/4} = 1.93\sec$$

$$C_t = 0.02 \quad h_n(ft) = 444$$

$$Dynamic\ Periods = 5.60\sec NS\ and\ 5.15\sec EW$$

$$Maximum\ Allowed = 1.4\times 1.93 = 2.70\sec$$

$$V = \frac{C_v I}{RT} W = 3054$$

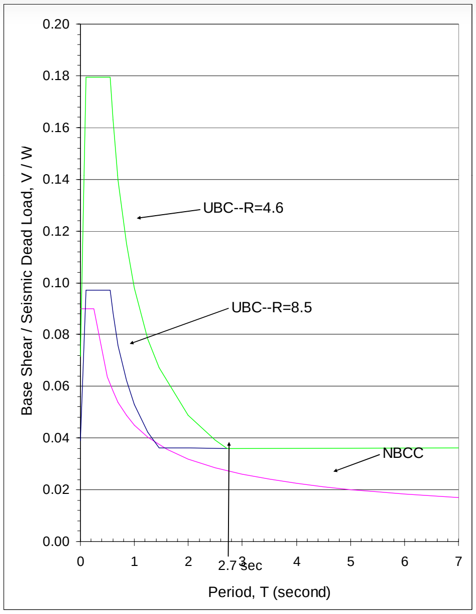
$$C_v = 0.45 \quad I = 1 \quad R = 5.5(assumed) \quad W = 100{,}768 kips$$

$$V = 0.11 C_a I W = 3658 (Minimum\ Base\ Shear)$$

$$C_a = 0.33$$

$$"Effective"\ R = \frac{5.5\times 3054}{3658} = 4.59$$

$$Note: Dynamic\ Analysis\ Allows\ Scaling\ To\ 80\%\ Of\ Static$$



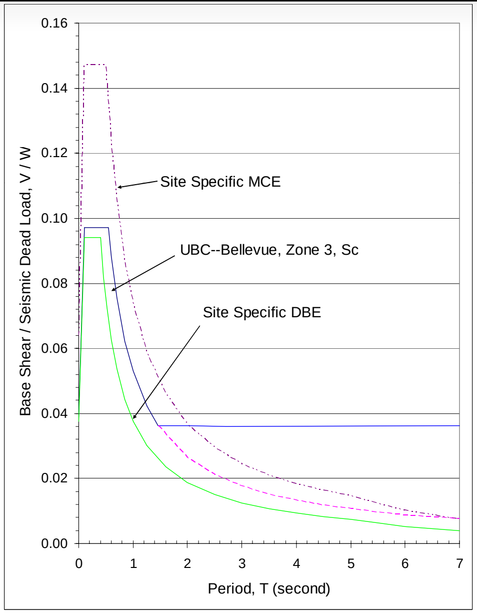
## Site Specific Spectrum and Time Histories

- After discussion with seismologist Paul Somerville we opted for the 3 event approach.
- There are three types of seismic sources for the site, interplate and intraplate subduction sources and shallow crustal sources
- Probabilistic seismic hazard analysis was performed to estimate the levels of ground motion corresponding to DBE and MCE hazard levels
- Site specific design spectra were developed from the uniform probability determined by the PSHA

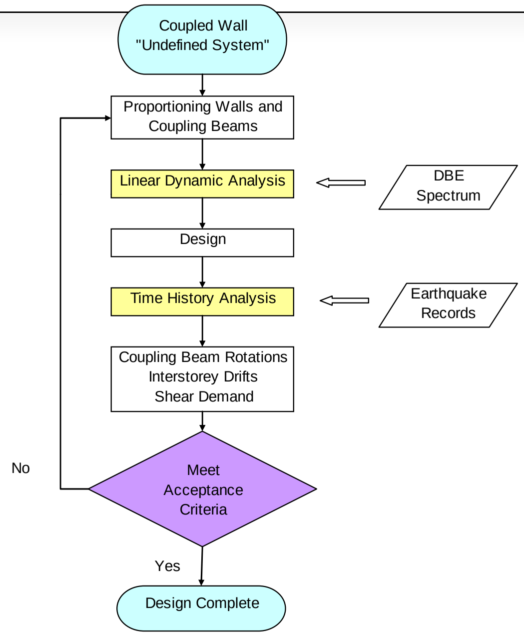
## Site Specific and Time Histories

- Analysis of basin response of the Puget Trough
- Time histories were taken from:
  - the M 7.1 Olympia 1949
  - Hachinohe recording of the M 7.9 Tokachi-oki 1968
  - Llollelo recording of the M 8.0 Valparaiso 1985
- Time histories were spectral matched to both DBE and MCE levels
- The following are graphs of the site specific spectrum and selected graphs of building response taken from the undefined system submissions

## Spectrum Comparisons



## Design – Non-Linear Process



Coupled Wall "Undefined System"

Proportioning Walls and Coupling Beams

Linear Dynamic Analysis

DBE Spectrum

Design

Time History Analysis

Earthquake Records

Coupling Beam Rotations
Interstorey Drifts
Shear Demand

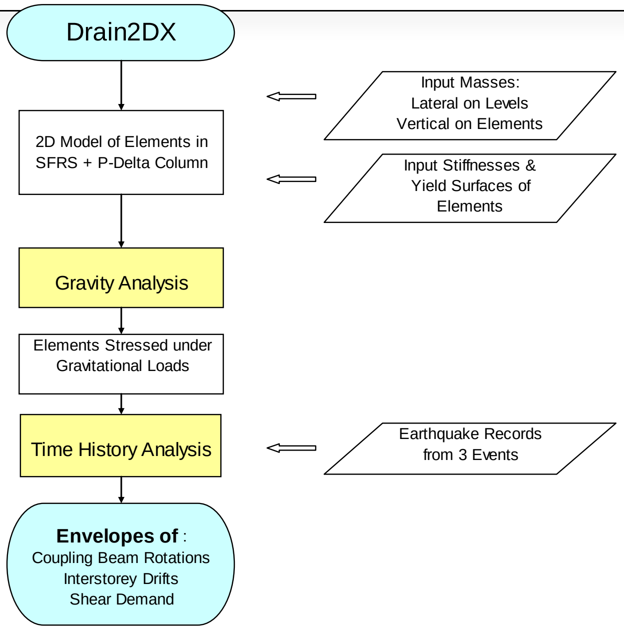
Meet Acceptance Criteria

No

Yes

Design Complete

## Design – Non-Linear Process



Drain2DX

2D Model of Elements in SFRS + P-Delta Column

Input Masses:
Lateral on Levels
Vertical on Elements

Input Stiffnesses & Yield Surfaces of Elements

Gravity Analysis

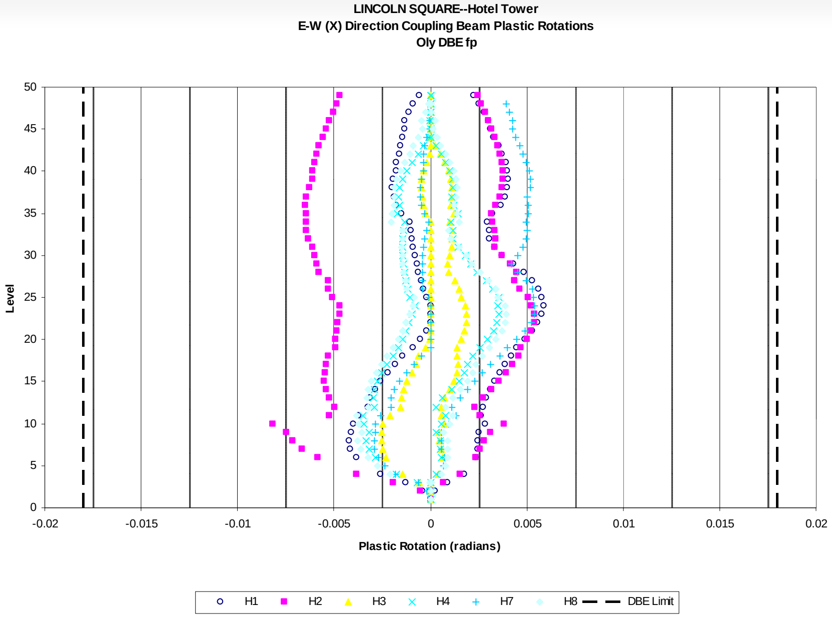
Elements Stressed under Gravitational Loads

Time History Analysis

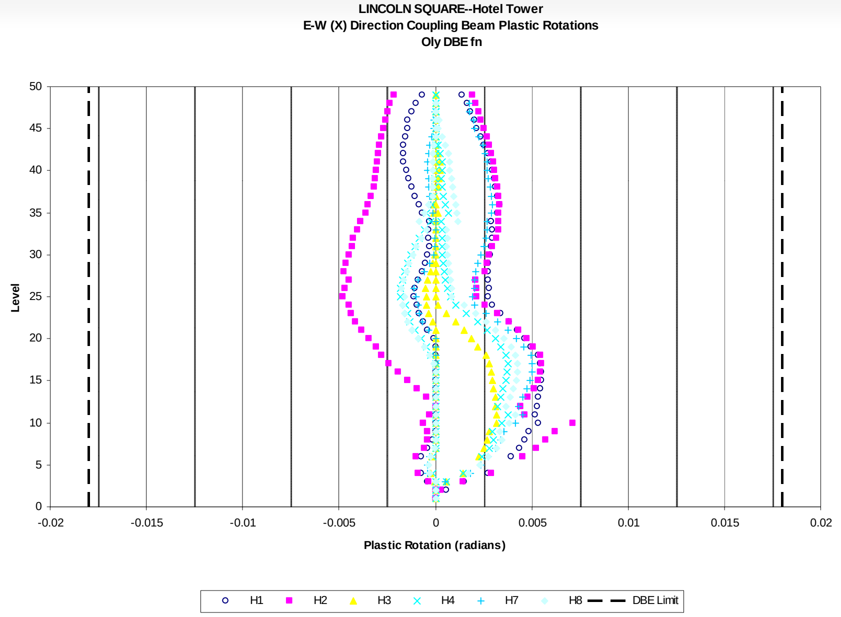
Earthquake Records from 3 Events

**Envelopes of** :
Coupling Beam Rotations
Interstorey Drifts
Shear Demand

## Coupling Beam Rotations



## Coupling Beam Rotations



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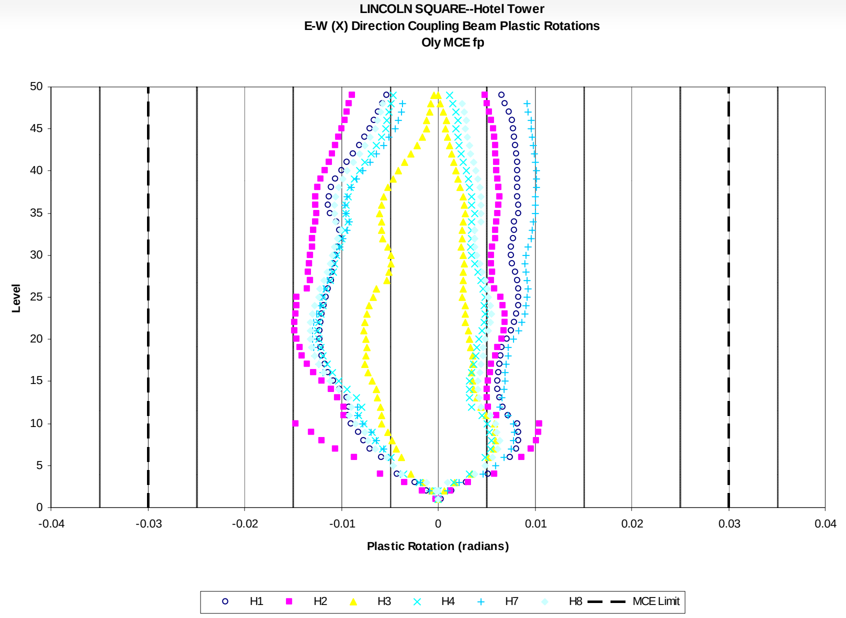
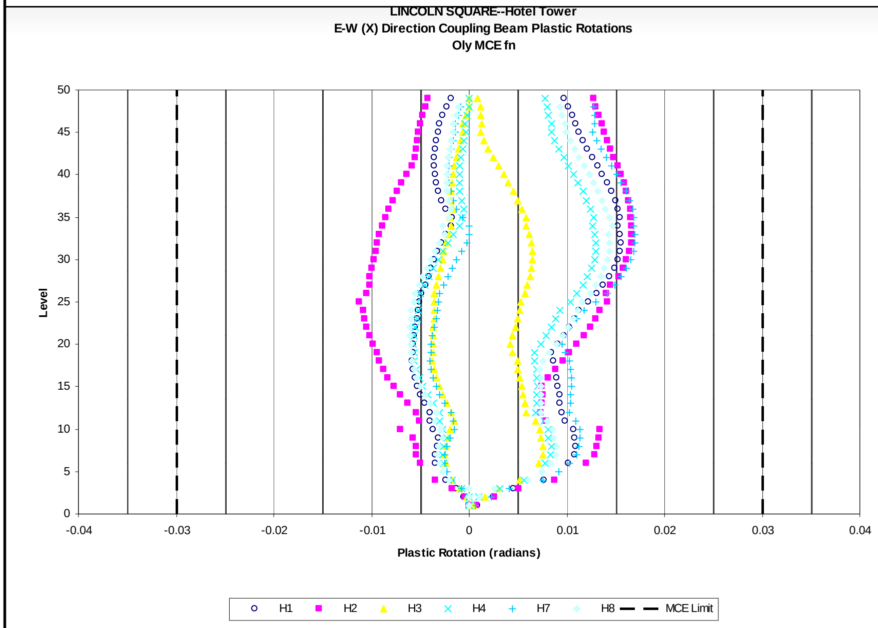
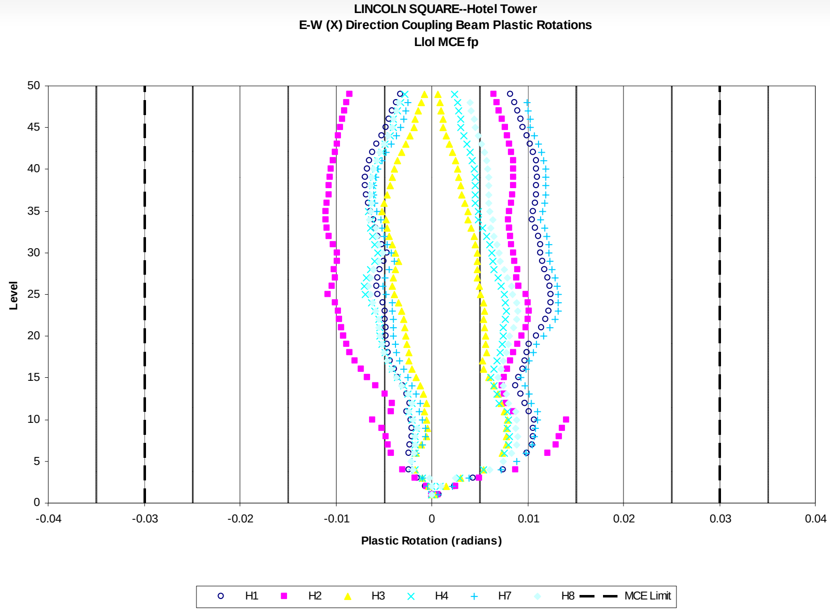
## Coupling Beam Rotations



## Coupling Beam Rotations

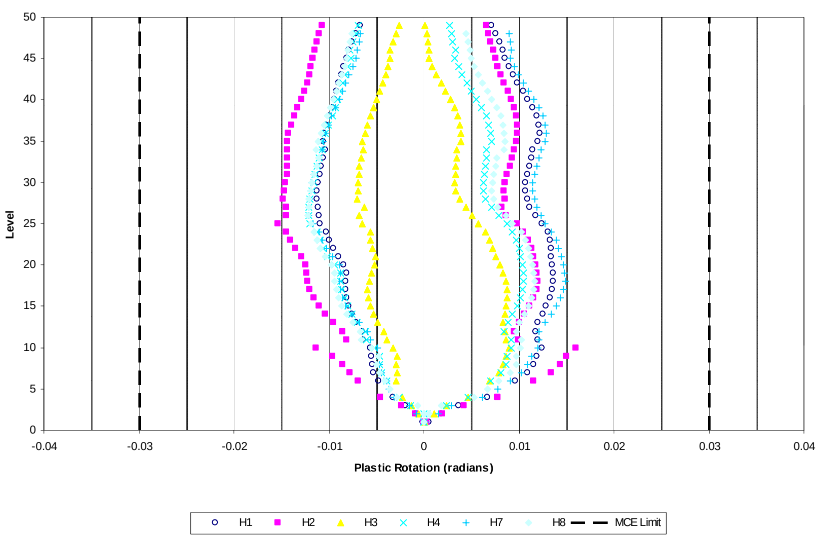


## Coupling Beam Rotations



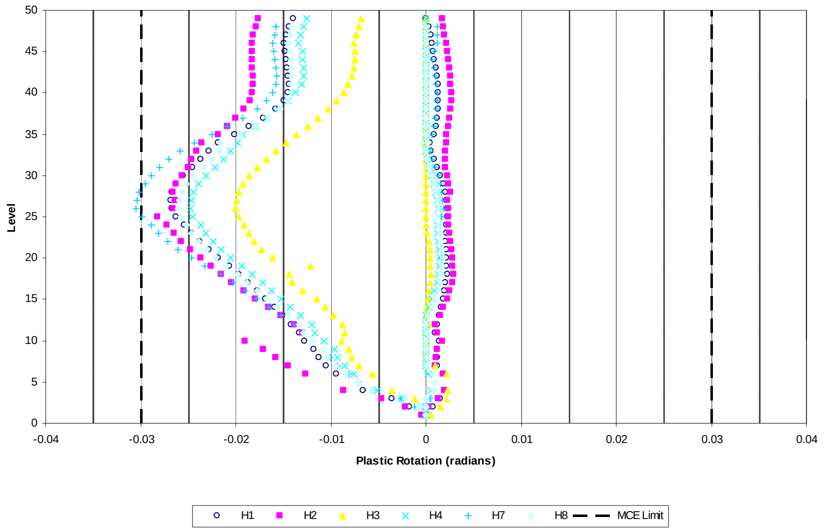
## Coupling Beam Rotations

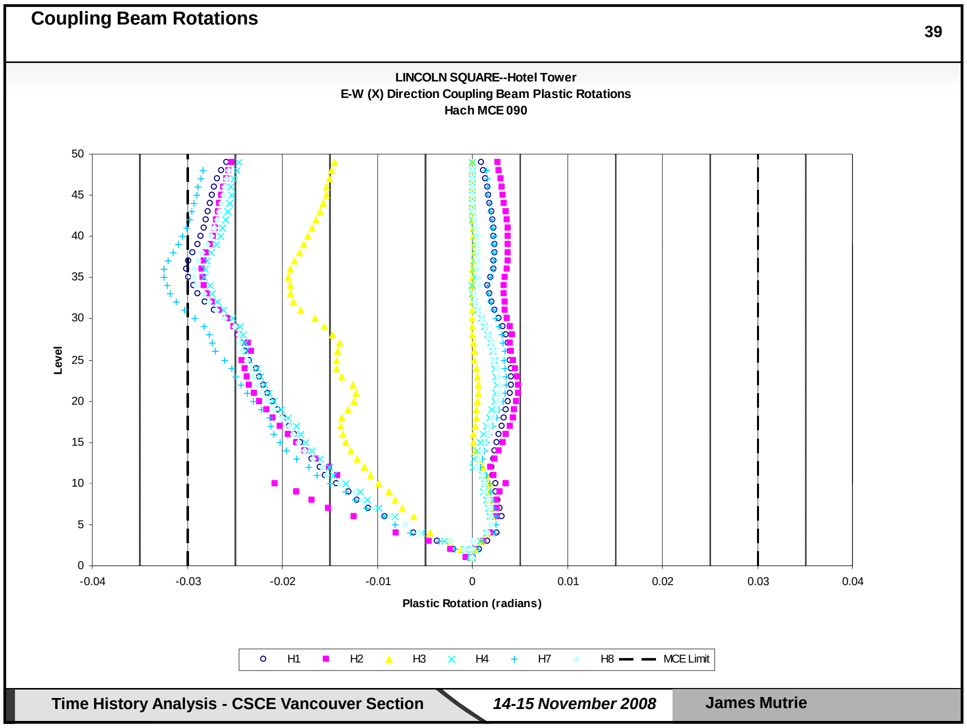
LINCOLN SQUARE--Hotel Tower
E-W (X) Direction Coupling Beam Plastic Rotations
Llol MCE fn



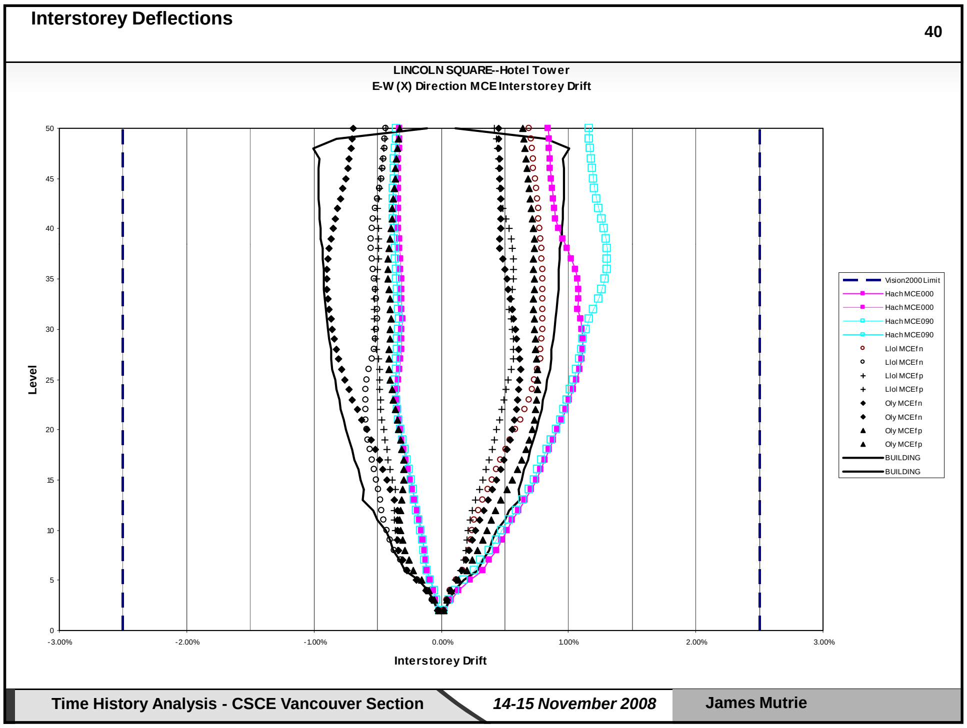
## Coupling Beam Rotations

LINCOLN SQUARE--Hotel Tower
E-W (X) Direction Coupling Beam Plastic Rotations
Hach MCE 000



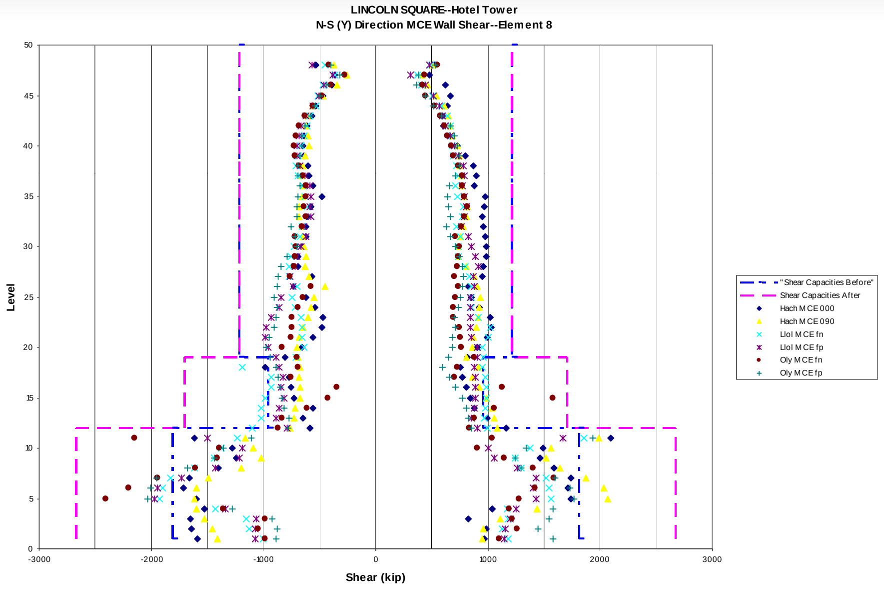


## Coupling Beam Rotations

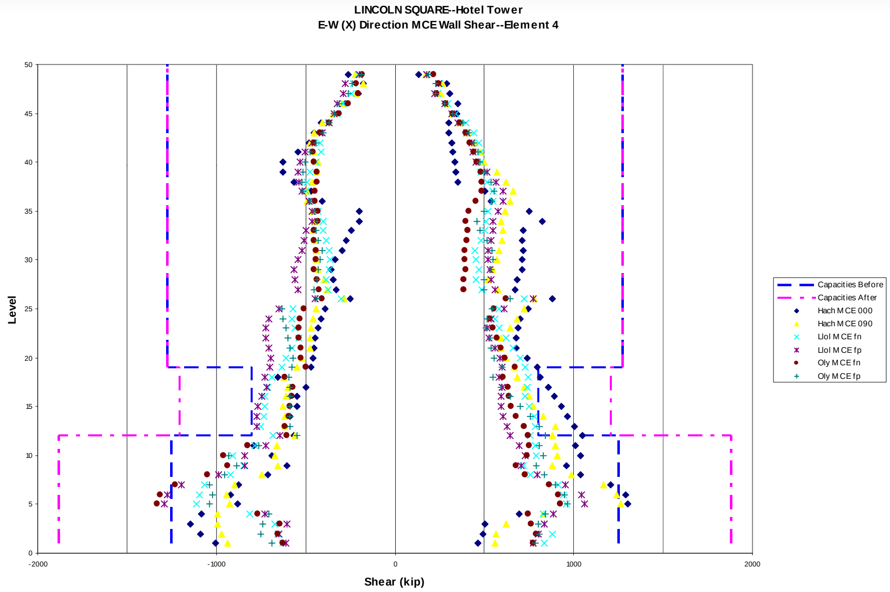


## Interstorey Deflections

## Building Shears



## Building Shears

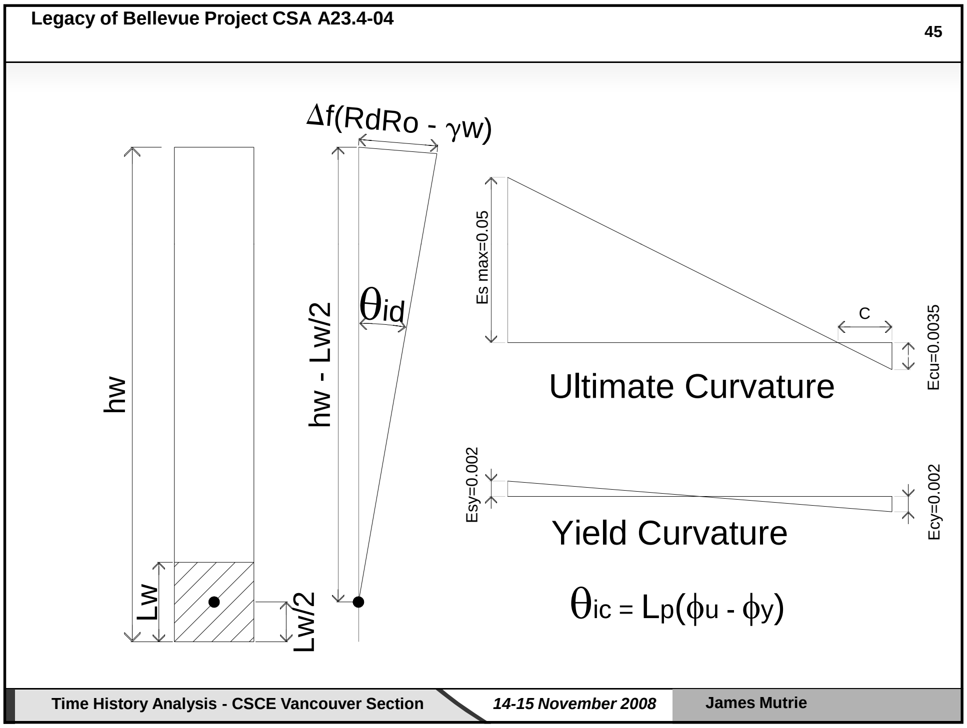


## Summary of Approach

- Building Designed to the UBC 97 code
  - Used results of linear dynamic modal analysis
  - Force levels based on code static base shear minimum force level
- Building system checked by non-linear time history analysis
  - Some modifications to building configuration required to conform to predetermined conservative deformation limits
- Non-linear analysis **not** required or perhaps even desirable for a conforming building

## Legacy of Bellevue Project

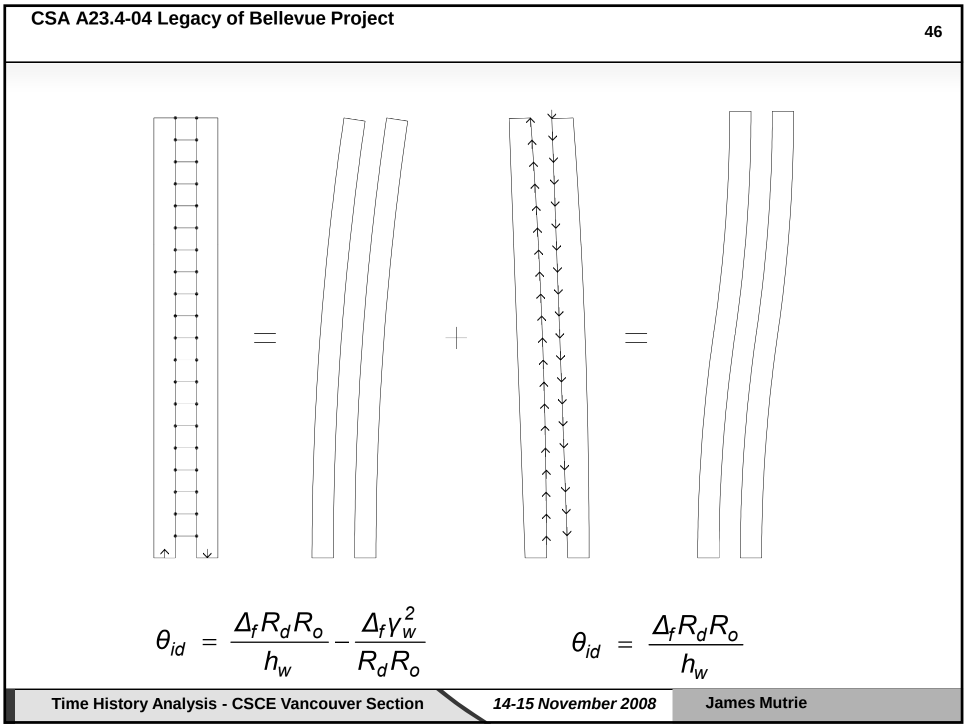
- CSA A23.3-04 Clause 21
  - Much of the new material introduced in 2004 was the result of lessons learned during the Bellevue design plus the research motivated by the questions raised
- Ductility Limit States
  - Inelastic rotational capacity > Inelastic rotational demand



**Legacy of Bellevue Project CSA A23.4-04**

$$\theta_{ic} = L_p(\phi_u - \phi_y)$$

Time History Analysis - CSCE Vancouver Section | *14-15 November 2008* | James Mutrie

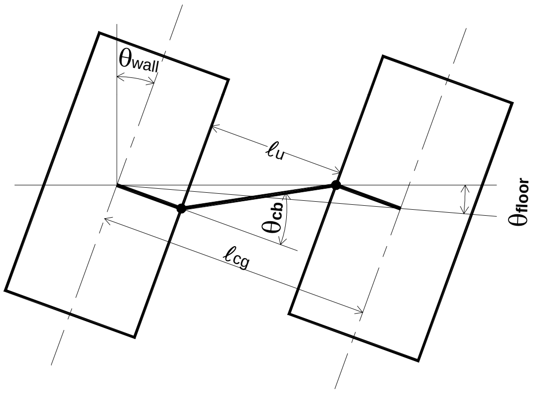


**CSA A23.4-04 Legacy of Bellevue Project**

$$\theta_{id} = \frac{\Delta_f R_d R_o}{h_w} - \frac{\Delta_f \gamma_w^2}{R_d R_o} \qquad \theta_{id} = \frac{\Delta_f R_d R_o}{h_w}$$

Time History Analysis - CSCE Vancouver Section | *14-15 November 2008* | James Mutrie

## CSA A23.4-04 Legacy of Bellevue Project



$$\theta_{id} = \left(\frac{\Delta_f R_0 R_d}{h_w}\right)\frac{\ell_{cg}}{\ell_u}$$

## Independent Peer Review

- The positive role of peer review
  - In my opinion non-linear time history analysis should only be used either for research or in cases where the standard code allowed solution is not practical
  - Independent peer review should be mandatory, there is nothing better than having to answer tough questions asked by a knowledgeable peer reviewer
  - Preparing for the review and trying to anticipate the questions is as valuable as the review itself
- **Bellevue selected reviewer was most important**

# TIME HISTORY ANALYSIS

## LECTURE # 11

## Performance Based Design of a 39 Story Concrete High Rise

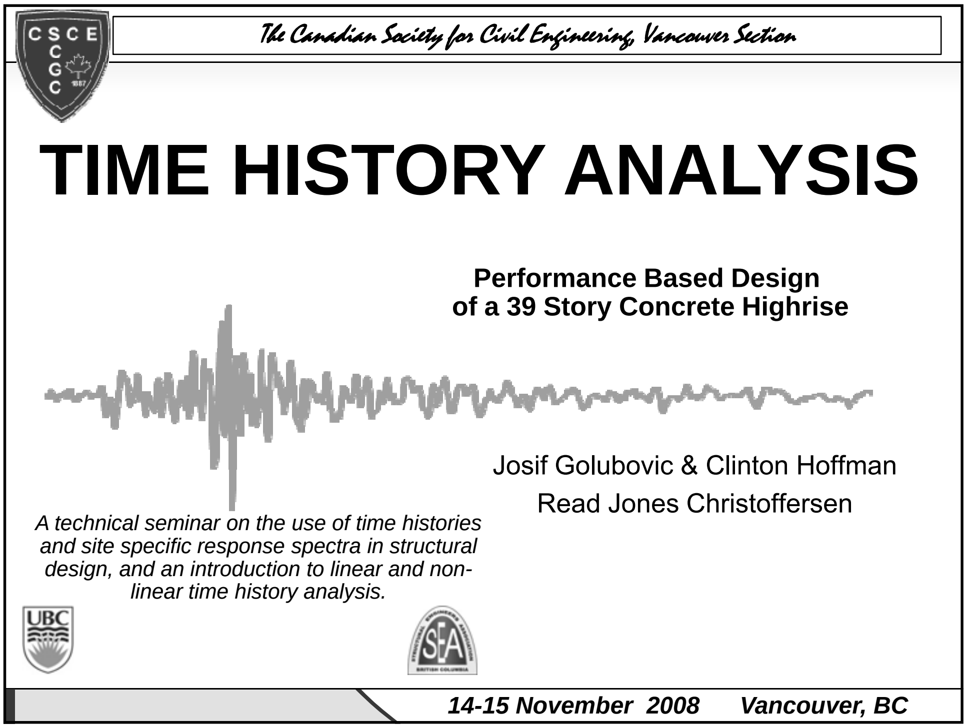
**Josif Golubovic and Clinton Hoffman**
**Read Jones Christoffersen Ltd**

Golubovic  Dipl.Ing, P.Eng

Obtained Engineering degree in civil–structural engineering in 1984 at University of Belgrade Serbia (Yugoslavia.).  He is with RJC since moving to Canada in 1995.

Clinton  on Hoffman

Obtained a Bachelors of Applied Science Degree in Civil Engineering at UBC in 2003 and have just recently a Masters of Engineering Degree in Structural Engineer at UBC. I have been working for Read Jones Christoffersen here in Vancouver for almost 3 years and have lived in BC for most of my life. My hometown is Campbell River on Vancouver Island.



The Canadian Society for Civil Engineering, Vancouver Section

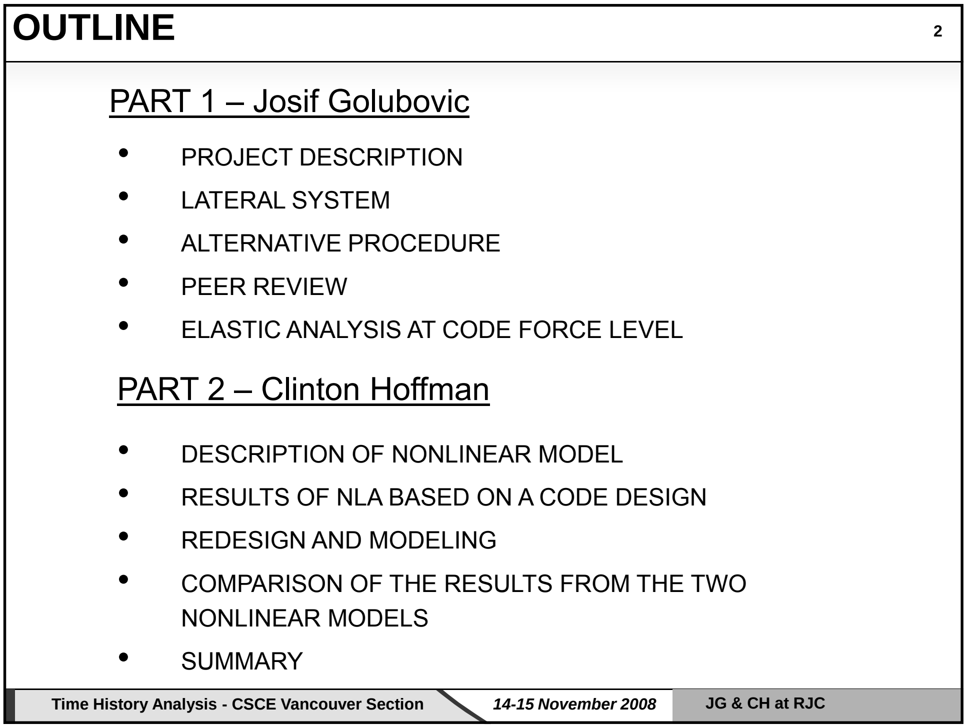
# TIME HISTORY ANALYSIS

## Performance Based Design of a 39 Story Concrete Highrise

Josif Golubovic & Clinton Hoffman
Read Jones Christoffersen

*A technical seminar on the use of time histories and site specific response spectra in structural design, and an introduction to linear and non-linear time history analysis.*

***14-15 November 2008 Vancouver, BC***



# OUTLINE

2

## PART 1 – Josif Golubovic

- PROJECT DESCRIPTION
- LATERAL SYSTEM
- ALTERNATIVE PROCEDURE
- PEER REVIEW
- ELASTIC ANALYSIS AT CODE FORCE LEVEL

## PART 2 – Clinton Hoffman

- DESCRIPTION OF NONLINEAR MODEL
- RESULTS OF NLA BASED ON A CODE DESIGN
- REDESIGN AND MODELING
- COMPARISON OF THE RESULTS FROM THE TWO NONLINEAR MODELS
- SUMMARY

**Time History Analysis - CSCE Vancouver Section** ***14-15 November 2008*** **JG & CH at RJC**

## PROJECT DESCRIPTION

- Mixed use building - Hotel & Residential
- Location - 8th/ Pine, Seattle, Washington
- Owner - Executive Hotels from Vancouver
- Architect - Weber Thompson from Seattle
- Peer Review - Rutherford Chekene, Joe Maffei (appointed by city of Seattle)
- UBC advisors - Professors: Don Anderson, Ken Elwood, Perry Adebar
- Structural Design - RJC
- UBC Computer lab - Data processing

---

## BUILDING CONSISTS OF:

- 2 Penthouse levels
- 22 Typical residential levels
- 9 Typical Hotel levels
- 6 Podium levels
- 4 Underground parking levels

Total of 39 over the ground floors

Total height 418 ft

## SITE AND SUBSURFACE CONDITION

- Glacial till – like soils
- Glaciolacustrine silts and clays
- Lower sand

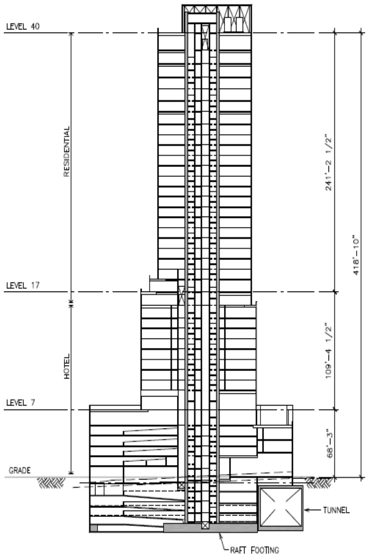
Allowable bearing pressure

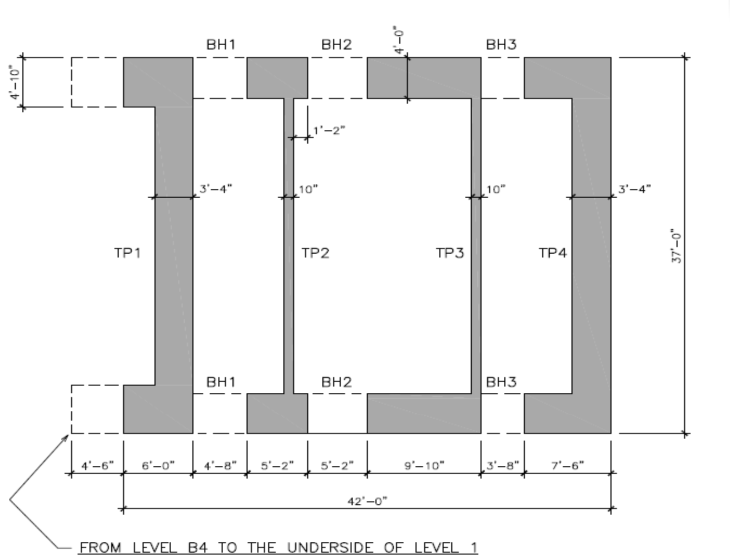
- 14ksf for spread footings
- 12ksf for Mat foundations

## BUILDING STRUCTURE

- Parking levels ( P4 to P1) - 8 or 9 inch flat plates
- Ground floor -Transfers over the tunnel
- Podium levels ( L2 to L7 ) - 8 inch flat plates
- Level 8 - 6 feet deep transfer slab
- Typical floors ( level 9 to 39 ) -7.5 inch P/T

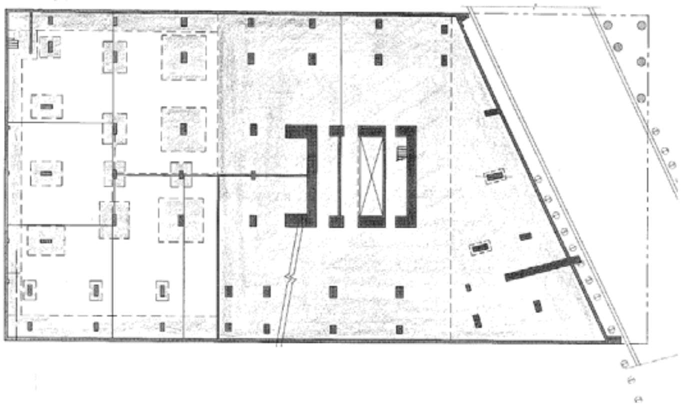
- Podium shear walls
- Basement shear – retaining walls
- Central core





## FOUNDATIONS

- Spread footings
- Strip footings
- Raft slab
- Drilled piers



## LATERAL SYSTEM

For concrete building of this height (418 ft ) in the Seattle area prescribed (ASCE7-02 table 9.5.2.2.) lateral systems are:

Moment frame or Dual system.

**Chosen Lateral system: ( Why ? )**

Special reinforced concrete shear walls. (Bearing wall system)

Code height limit ------------160 ( 240 ) ft

**Exception to height is based on :**

**ASCE 7-02** § 9.5.2.2 and § 9.5.2.5.1.

Prove that overall seismic performance of the proposed system is at least equivalent to that prescribed by code. ( How ? )

**Alternative design procedure-Peer review**

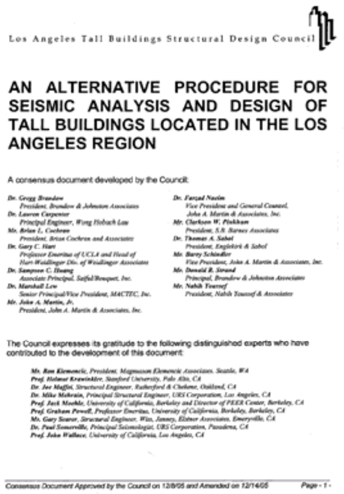
## ALTERNATIVE PROCEDURE

- Performance Base Design

  Performance level
  Ground motion or hazard level
  Objectives
  Acceptability criteria - Developed from resources documents and with Peer Reviewer

- No clear consensus yet

## RESOURCE MATERIAL



Los Angeles Tall Buildings Structural Design Council

**AN ALTERNATIVE PROCEDURE FOR SEISMIC ANALYSIS AND DESIGN OF TALL BUILDINGS LOCATED IN THE LOS ANGELES REGION**

A consensus document developed by the Council:

***Dr. Gregg Brandow***
*President, Brandow & Johnston Associates*
***Dr. Lauren Carpenter***
*Principal Engineer, Wong Hobach Lau*
***Mr. Brian L. Cochran***
*President, Brian Cochran and Associates*
***Dr. Gary C. Hart***
*Professor Emeritus of UCLA and Head of Hart-Weidlinger Div. of Weidlinger Associates*
***Dr. Sampson C. Huang***
*Associate Principal, Saiful/Bouquet, Inc.*
***Dr. Marshall Lew***
*Senior Principal/Vice President, MACTEC, Inc.*
***Mr. John A. Martin, Jr.***
*President, John A. Martin & Associates, Inc.*
***Dr. Farzad Naeim***
*Vice President and General Counsel, John A. Martin & Associates, Inc.*
***Mr. Clarkson W. Pinkham***
*President, S.B. Barnes Associates*
***Dr. Thomas A. Sabol***
*President, Englekirk & Sabol*
***Mr. Barry Schindler***
*Vice President, John A. Martin & Associates, Inc.*
***Mr. Donald R. Strand***
*Principal, Brandow & Johnston Associates*
***Mr. Nabih Youssef***
*President, Nabih Youssef & Associates*

The Council expresses its gratitude to the following distinguished experts who have contributed to the development of this document:

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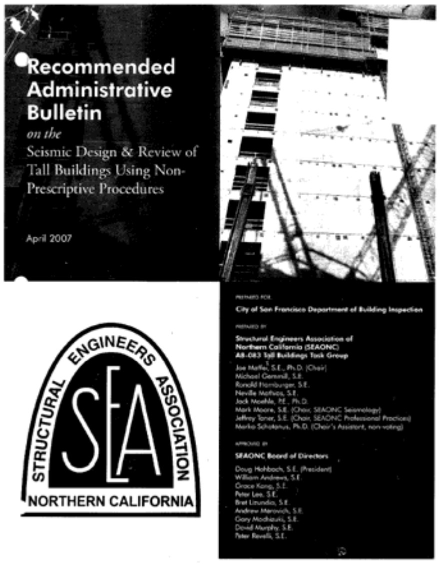
***Prof. Helmut Krawinkler**, Stanford University, Palo Alto, CA*
***Dr. Joe Maffei**, Structural Engineer, Rutherford & Chekene, Oakland, CA*
***Dr. Mike Mehrain**, Principal Structural Engineer, URS Corporation, Los Angeles, CA*
***Prof. Jack Moehle**, University of California, Berkeley and Director of PEER Center, Berkeley, CA*
***Prof. Graham Powell**, Professor Emeritus, University of California, Berkeley, Berkeley, CA*
***Mr. Gary Searer**, Structural Engineer, Wiss, Janney, Elstner Associates, Emeryville, CA*
***Dr. Paul Somerville**, Principal Seismologist, URS Corporation, Pasadena, CA*
***Prof. John Wallace**, University of California, Los Angeles, CA*

Consensus Document Approved by the Council on 12/8/05 and Amended on 12/14/05 Page - 1 -



**Recommended Administrative Bulletin**
*on the*
Seismic Design & Review of Tall Buildings Using Non-Prescriptive Procedures

April 2007

STRUCTURAL ENGINEERS ASSOCIATION
SEA
NORTHERN CALIFORNIA

PREPARED FOR
**City of San Francisco Department of Building Inspection**

PREPARED BY
**Structural Engineers Association of Northern California (SEAONC)**
**AB-083 Tall Buildings Task Group**
Joe Maffei, S.E., Ph.D. (Chair)
Michael Gemmill, S.E.
Ronald Hamburger, S.E.
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Mark Moore, S.E. (Chair, SEAONC Seismology)
Jeffrey Tarver, S.E. (Chair, SEAONC Professional Practices)
Marko Schotanus, Ph.D. (Chair's Assistant, non-voting)

APPROVED BY
**SEAONC Board of Directors**
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Bret Lizundia, S.E.
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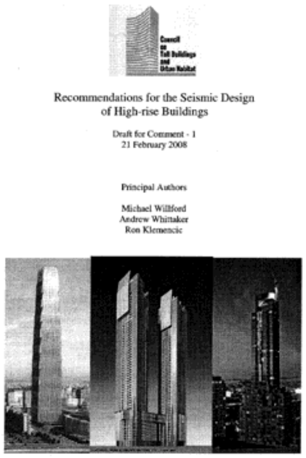
David Murphy, S.E.
Peter Revelli, S.E.

## RESOURCE MATERIAL



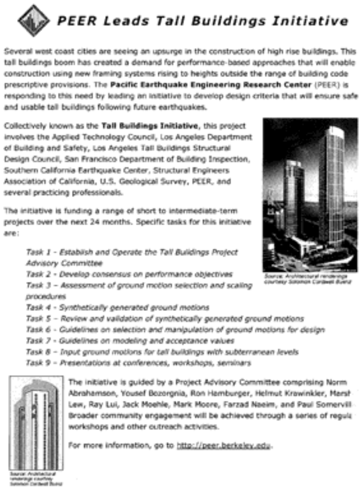
Council on Tall Buildings and Urban Habitat

Recommendations for the Seismic Design
of High-rise Buildings

Draft for Comment - 1
21 February 2008

Principal Authors

Michael Willford
Andrew Whittaker
Ron Klemencic



***PEER Leads Tall Buildings Initiative***

Several west coast cities are seeing an upsurge in the construction of high rise buildings. This tall buildings boom has created a demand for performance-based approaches that will enable construction using new framing systems rising to heights outside the range of building code prescriptive provisions. The **Pacific Earthquake Engineering Research Center** (PEER) is responding to this need by leading an initiative to develop design criteria that will ensure safe and usable tall buildings following future earthquakes.

Collectively known as the **Tall Buildings Initiative**, this project involves the Applied Technology Council, Los Angeles Department of Building and Safety, Los Angeles Tall Buildings Structural Design Council, San Francisco Department of Building Inspection, Southern California Earthquake Center, Structural Engineers Association of California, U.S. Geological Survey, PEER, and several practicing professionals.

Source: Architectural renderings courtesy Solomon Cordwell Buenz

The initiative is funding a range of short to intermediate-term projects over the next 24 months. Specific tasks for this initiative are:

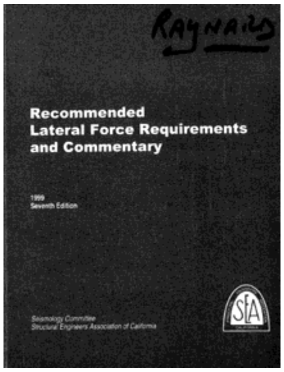
*Task 1 - Establish and Operate the Tall Buildings Project Advisory Committee*
*Task 2 - Develop consensus on performance objectives*
*Task 3 – Assessment of ground motion selection and scaling procedures*
*Task 4 - Synthetically generated ground motions*
*Task 5 – Review and validation of synthetically generated ground motions*
*Task 6 - Guidelines on selection and manipulation of ground motions for design*
*Task 7 - Guidelines on modeling and acceptance values*
*Task 8 – Input ground motions for tall buildings with subterranean levels*
*Task 9 – Presentations at conferences, workshops, seminars*

The initiative is guided by a Project Advisory Committee comprising Norm Abrahamson, Yousef Bozorgnia, Ron Hamburger, Helmut Krawinkler, Marsh Lew, Ray Lui, Jack Moehle, Mark Moore, Farzad Naeim, and Paul Somervill Broader community engagement will be achieved through a series of regul workshops and other outreach activities.

For more information, go to http://peer.berkeley.edu.

Source: Architectural renderings courtesy Solomon Cordwell Buenz

# RESOURCE MATERIAL



Raynard

Recommended
Lateral Force Requirements
and Commentary

1999
Seventh Edition

Seismology Committee
Structural Engineers Association of California

FEDERAL EMERGENCY MANAGEMENT AGENCY

Evaluation of
Earthquake Damaged
Concrete and Masonry Wall
Buildings

*Basic Procedures Manual*

FEDERAL EMERGENCY MANAGEMENT AGENCY

PRESTANDARD AND COMMENTARY FOR THE
SEISMIC REHABILITATION OF BUILDINGS

Performance level and objectives

| Performance level | Probability of exceedance % per year | Reacurrance interval | Objectives |
|---|---|---|---|
| **Serviceability** | 50/30 | 43 years ( Frequent ) | Building to remain servicable |
| **Life Safety** | 10/50 | 475 years ( Rare ) | Provide life safety |
| **Collapse prevention** | 2/50 | 2475 years ( Extremely rare) | Does not experience collapse |

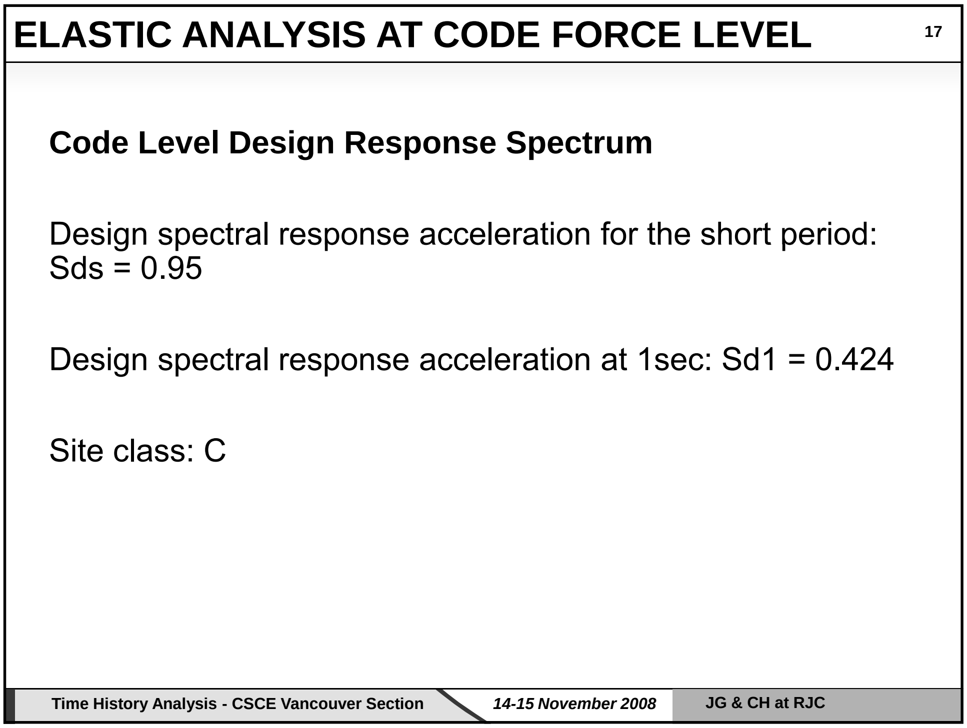
Method of analysis and acceptabilility criteria

| Performance level | Type of analysies | Seismic reduction factor R | Accidental Torsion Considered | Strength Reduction factor ( ∅ ) | Material strength | Acceptability Criteria |
|---|---|---|---|---|---|---|
| **Serviceability** | Linear dynamic | 1 | No | 1 | Expected | None of the members to exceed USD limit |
| **Life Safety** | Linear dynamic | per code | Yes | per code | Specified | Per code with following exceptions: ( Height limit, Cs=0.045, ρ=1, Coupling Beam rotation <= 0.05, Vn depend on ductility demends as per FEMA 306 ) |
| **Collapse prevention** | Nonlinear dynamc | N/A | No | 1 | Expected | Deformational capacities:<br>Interstory drift 1.5x 0.02 = 0.03<br>Coupling Beams rotation <= 0.05<br>Max compression strain <= 0.004 |

# PEER REVIEW

The main purpose of Peer Review is to provide an independent and objective technical review of those aspects of the structural design of the building that relate to seismic performance.

- STEP BY STEP PROCESS
- LOG-MILE STONES
- DESIGN CRITERIA – Developed with Peer Reviewers
- GREAT LEARNING EXPERIENCE



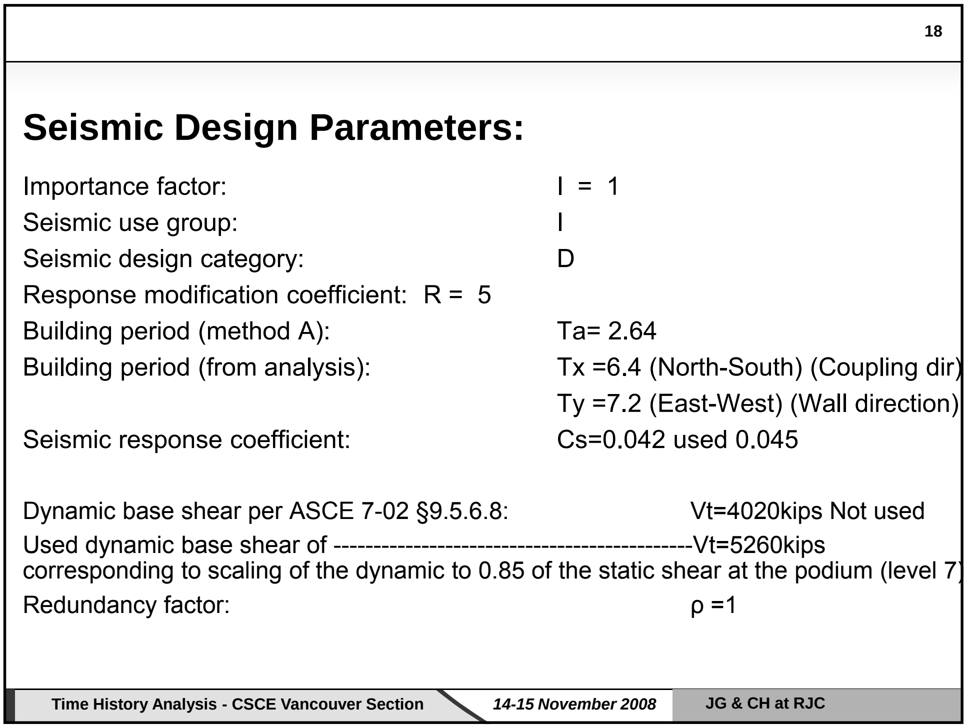
## ELASTIC ANALYSIS AT CODE FORCE LEVEL

### Code Level Design Response Spectrum

Design spectral response acceleration for the short period: Sds = 0.95

Design spectral response acceleration at 1sec: Sd1 = 0.424

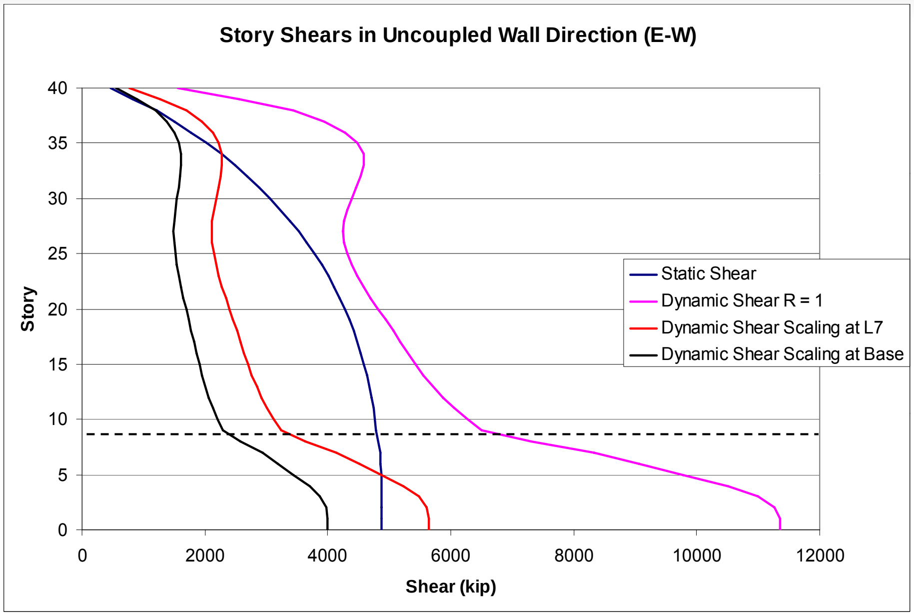
Site class: C



## Seismic Design Parameters:

Importance factor: I = 1
Seismic use group: I
Seismic design category: D
Response modification coefficient: R = 5
Building period (method A): Ta= 2.64
Building period (from analysis): Tx =6.4 (North-South) (Coupling dir)
Ty =7.2 (East-West) (Wall direction)
Seismic response coefficient: Cs=0.042 used 0.045

Dynamic base shear per ASCE 7-02 §9.5.6.8: Vt=4020kips Not used
Used dynamic base shear of ---------------------------------------------Vt=5260kips corresponding to scaling of the dynamic to 0.85 of the static shear at the podium (level 7)
Redundancy factor: ρ =1



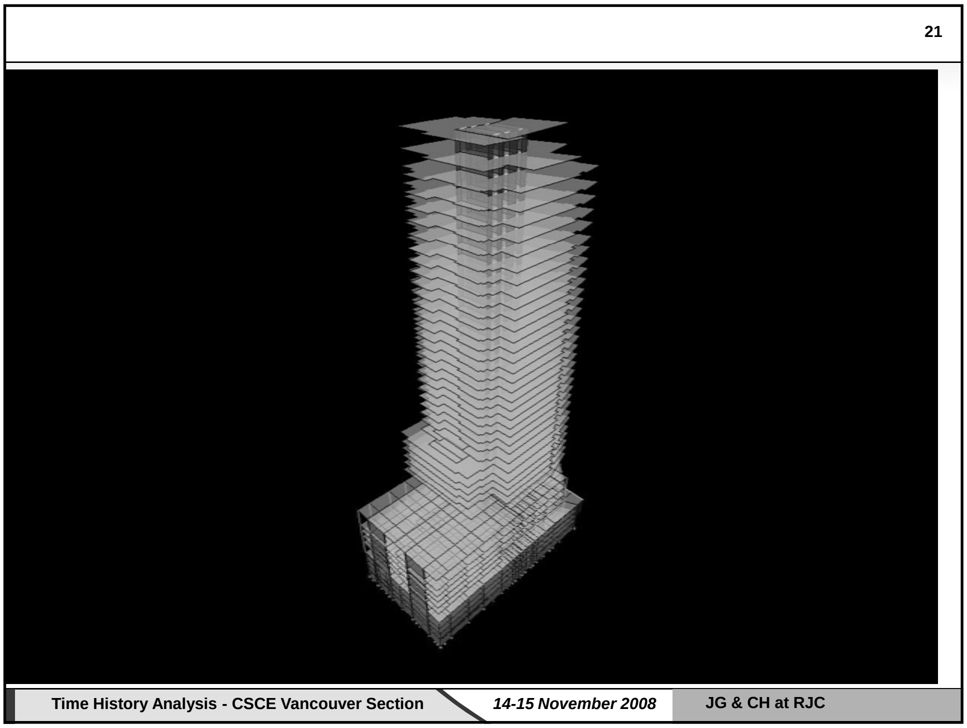
Story Shears in Uncoupled Wall Direction (E-W)

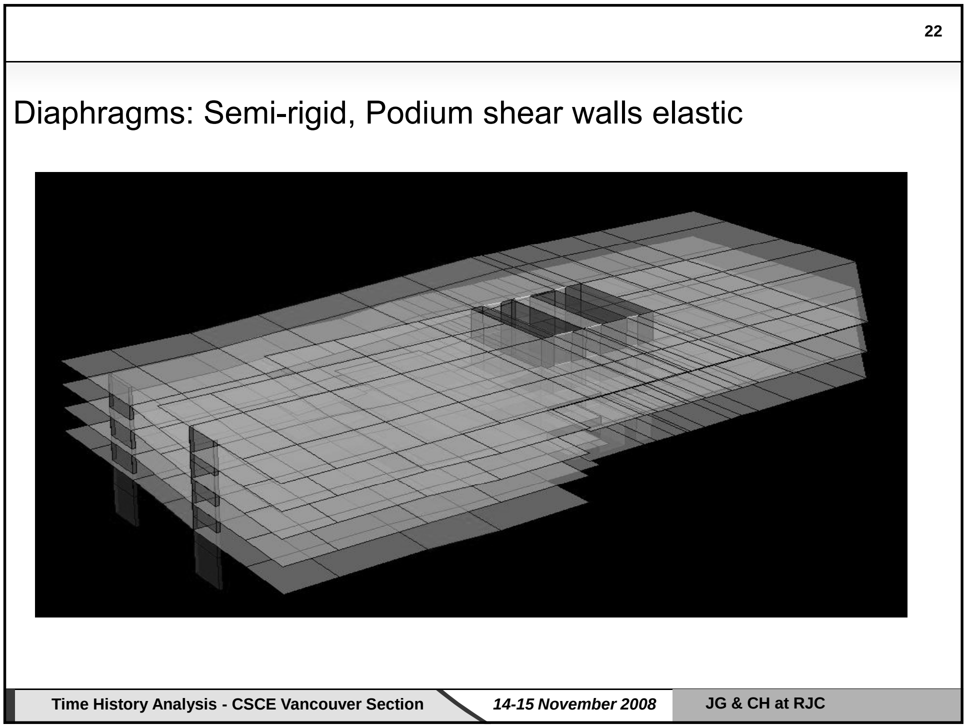
## MODELING ASSUMPTIONS

Element stiffness properties were modeled as follows:

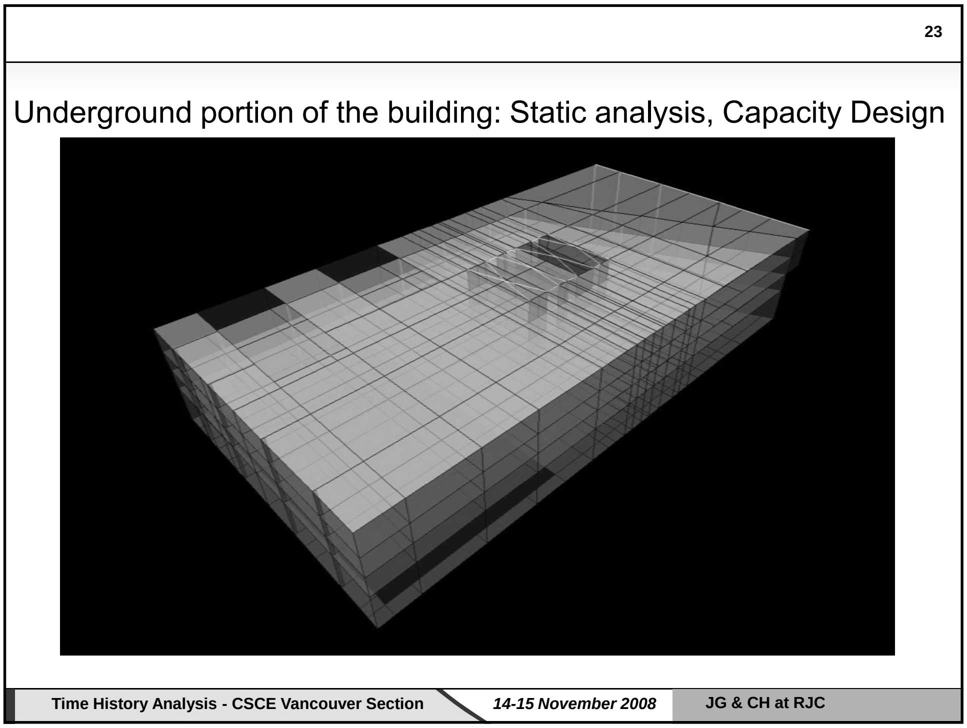
| | |
|---|---|
| Concrete core walls: | Ie= 0.33Ig; Av=0.3Ag |
| Basement walls: | Ie= 0.33Ig; Av=0.3Ag |
| Coupling beams: | Ie= 0.25Ig; Av=0.45Ag |
| Diaphragms: | In plane bending Ie=0.3Ig<br>In plane shear stiffness Av=0.3Ag<br>Out of plane banding stiffness Ie=0.35Ig |
| Footing: | Out of plane raft slab bending stiffness Ie=0.6Ig |

Elastic soil spring stiffness k=46pci (provided by “Geo Engineers”)

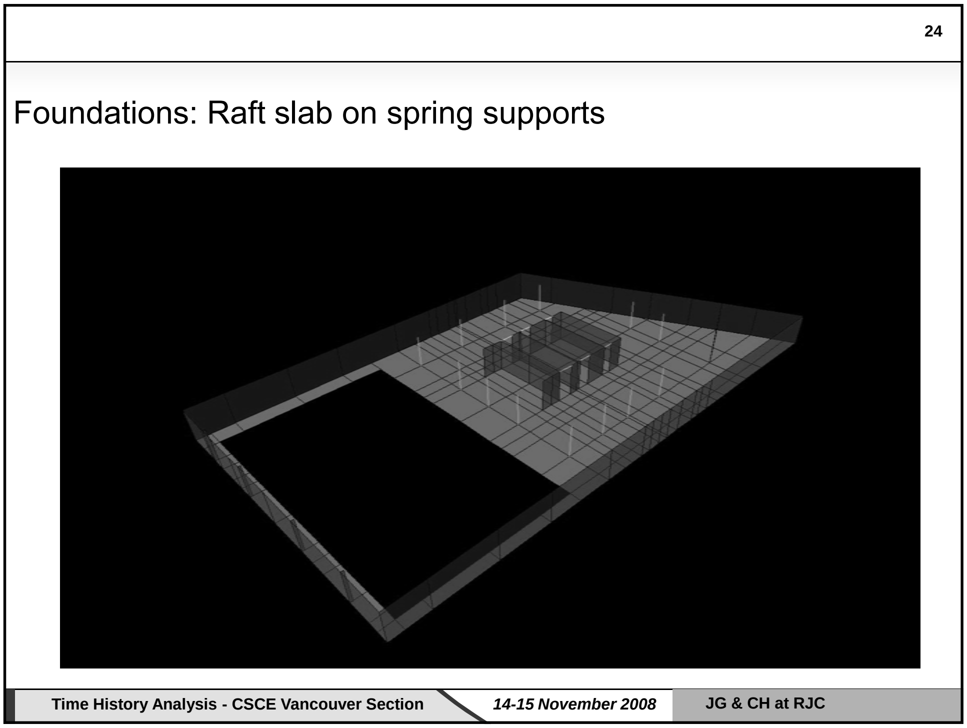




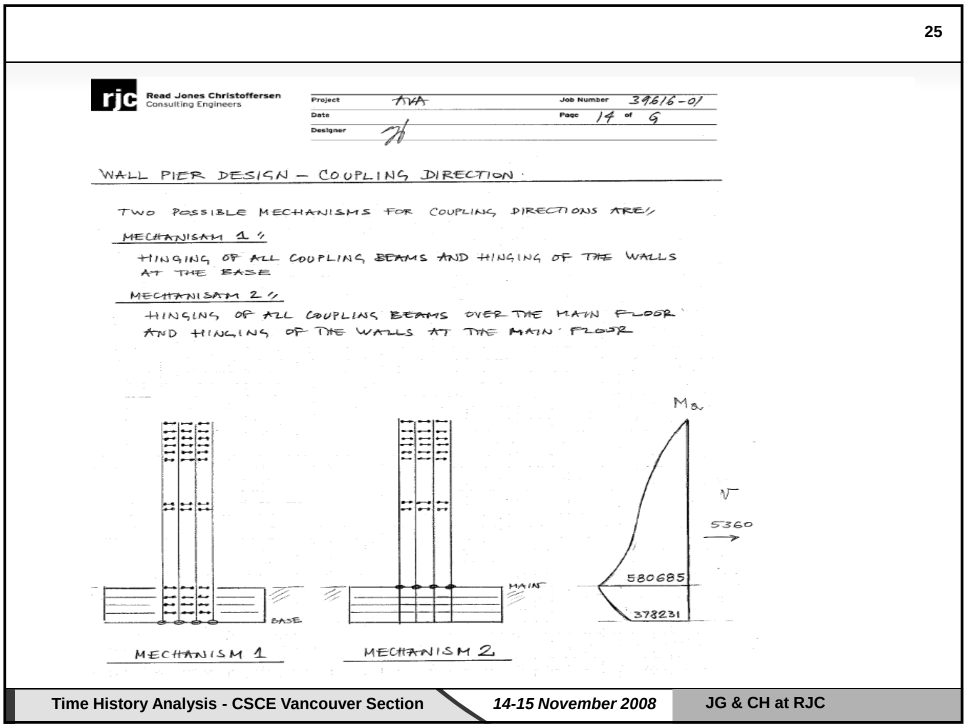
Diaphragms: Semi-rigid, Podium shear walls elastic



## Underground portion of the building: Static analysis, Capacity Design



## Foundations: Raft slab on spring supports



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Consulting Engineers

Project AVA
Job Number 39616-01
Date
Page 14 of 9
Designer JJ

WALL PIER DESIGN – COUPLING DIRECTION

TWO POSSIBLE MECHANISMS FOR COUPLING DIRECTIONS ARE//

MECHANISAM 1//

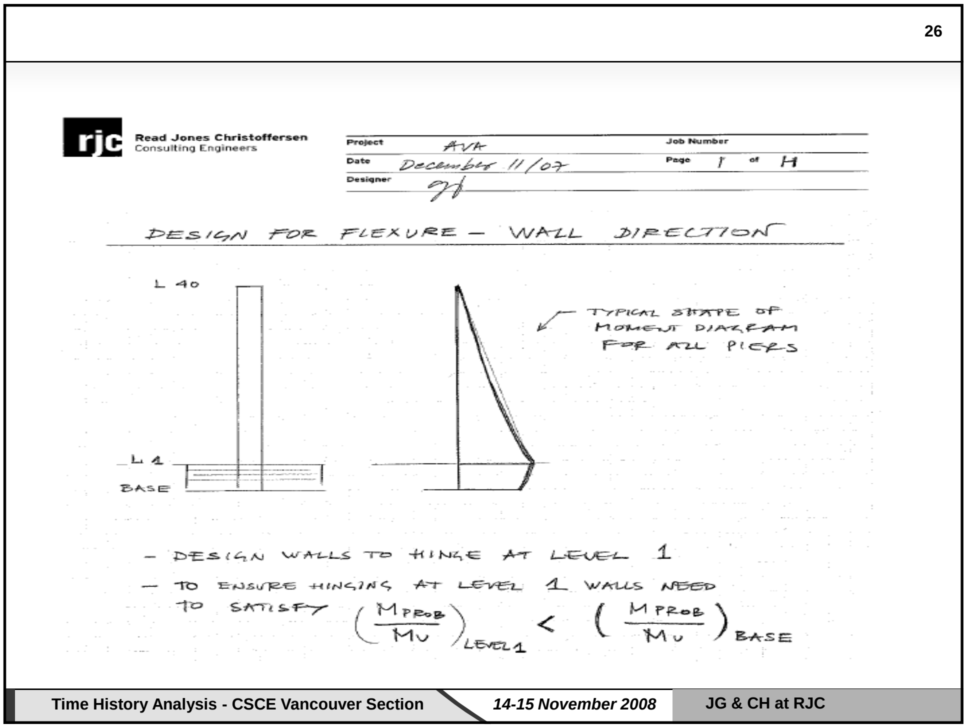
HINGING OF ALL COUPLING BEAMS AND HINGING OF THE WALLS AT THE BASE

MECHANISAM 2//

HINGING OF ALL COUPLING BEAMS OVER THE MAIN FLOOR AND HINGING OF THE WALLS AT THE MAIN FLOOR

MECHANISM 1

MECHANISM 2



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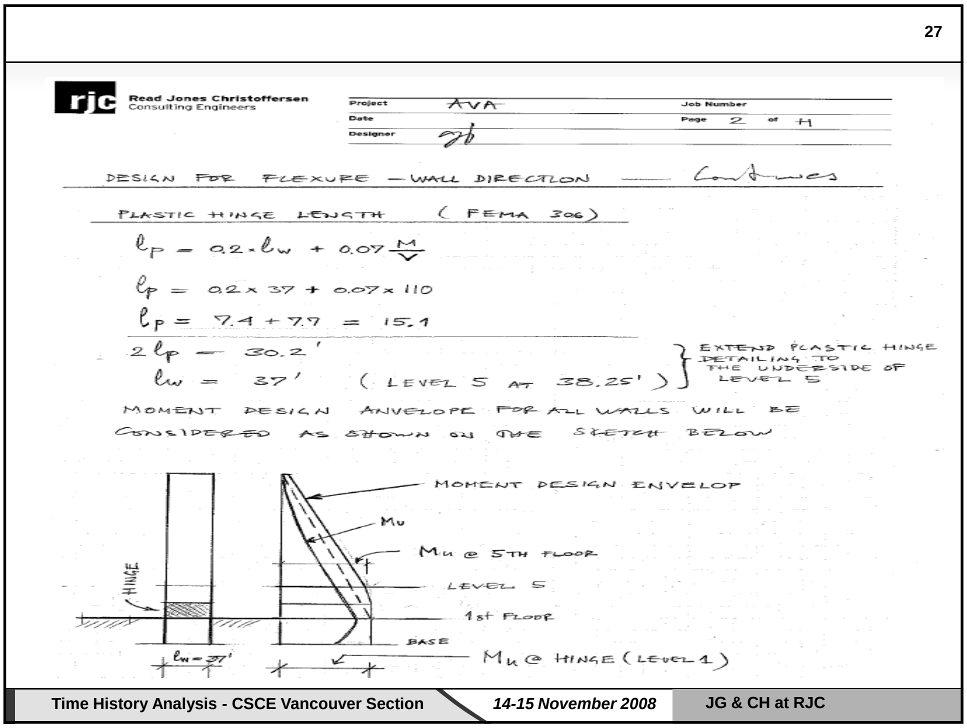
Consulting Engineers

Project AVA
Job Number
Date December 11/07
Page 1 of 4
Designer JJ

DESIGN FOR FLEXURE – WALL DIRECTION

- DESIGN WALLS TO HINGE AT LEVEL 1
- TO ENSURE HINGING AT LEVEL 1 WALLS NEED TO SATISFY $\left(\frac{M_{PROB}}{M_U}\right)_{LEVEL 1} < \left(\frac{M_{PROB}}{M_U}\right)_{BASE}$



**Read Jones Christoffersen** Consulting Engineers

Project AVA
Job Number
Date
Page 2 of H
Designer gb

DESIGN FOR FLEXURE — WALL DIRECTION —— Continues

PLASTIC HINGE LENGTH (FEMA 306)

$$l_p = 0.2 \cdot l_w + 0.07 \frac{M}{V}$$

$$l_p = 0.2 \times 37 + 0.07 \times 110$$

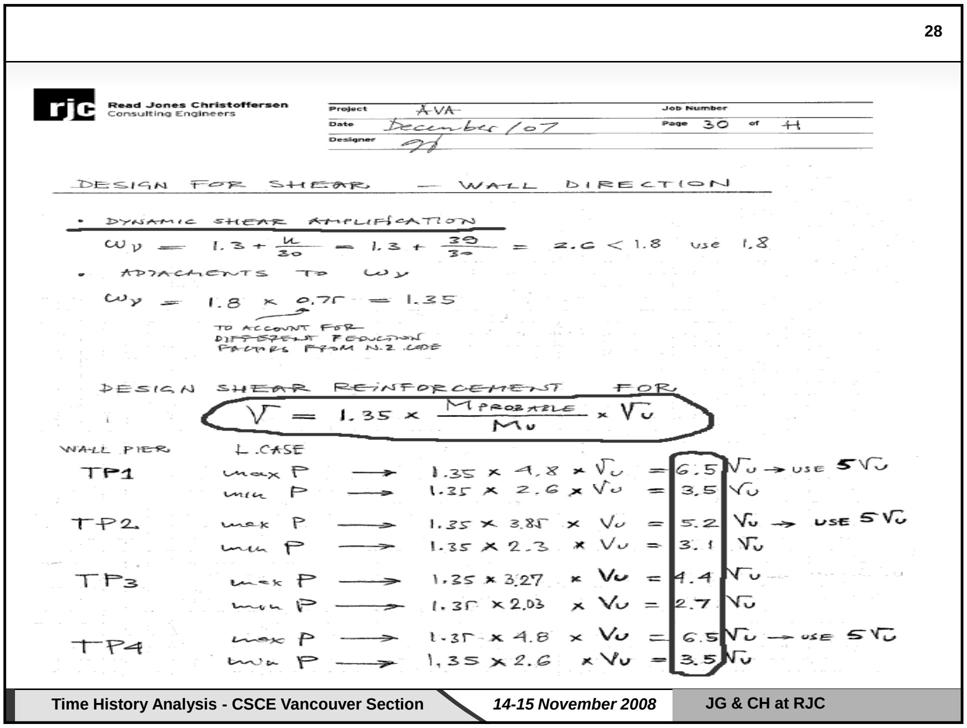
$$l_p = 7.4 + 7.7 = 15.1$$

$$2 l_p = 30.2'$$

$$l_w = 37' \quad (\text{LEVEL 5 AT } 38.25')$$

} EXTEND PLASTIC HINGE DETAILING TO THE UNDERSIDE OF LEVEL 5

MOMENT DESIGN ANVELOPE FOR ALL WALLS WILL BE CONSIDERED AS SHOWN ON THE SKETCH BELOW



**Read Jones Christoffersen** Consulting Engineers

Project AVA
Job Number
Date December /07
Page 30 of H
Designer gb

DESIGN FOR SHEAR — WALL DIRECTION

- DYNAMIC SHEAR AMPLIFICATION

$$\omega_v = 1.3 + \frac{n}{30} = 1.3 + \frac{39}{30} = 2.6 < 1.8 \quad \text{use } 1.8$$

- ADJACMENTS TO $\omega_v$

$$\omega_v = 1.8 \times 0.75 = 1.35$$

TO ACCOUNT FOR DIFFERENT REDUCTION FACTORS FROM N.Z. CODE

DESIGN SHEAR REINFORCEMENT FOR

$$\boxed{V = 1.35 \times \frac{M_{PROBABLE}}{M_U} \times V_U}$$

| WALL PIER | L.CASE | | |
|---|---|---|---|
| TP1 | max P | $\rightarrow 1.35 \times 4.8 \times V_U = 6.5\, V_U$ | → USE $5V_U$ |
| | min P | $\rightarrow 1.35 \times 2.6 \times V_U = 3.5\, V_U$ | |
| TP2 | max P | $\rightarrow 1.35 \times 3.85 \times V_U = 5.2\, V_U$ | → USE $5V_U$ |
| | min P | $\rightarrow 1.35 \times 2.3 \times V_U = 3.1\, V_U$ | |
| TP3 | max P | $\rightarrow 1.35 \times 3.27 \times V_U = 4.4\, V_U$ | |
| | min P | $\rightarrow 1.35 \times 2.03 \times V_U = 2.7\, V_U$ | |
| TP4 | max P | $\rightarrow 1.35 \times 4.8 \times V_U = 6.5\, V_U$ | → USE $5V_U$ |
| | min P | $\rightarrow 1.35 \times 2.6 \times V_U = 3.5\, V_U$ | |

## Reminder
## Basic seismic design philosophy.

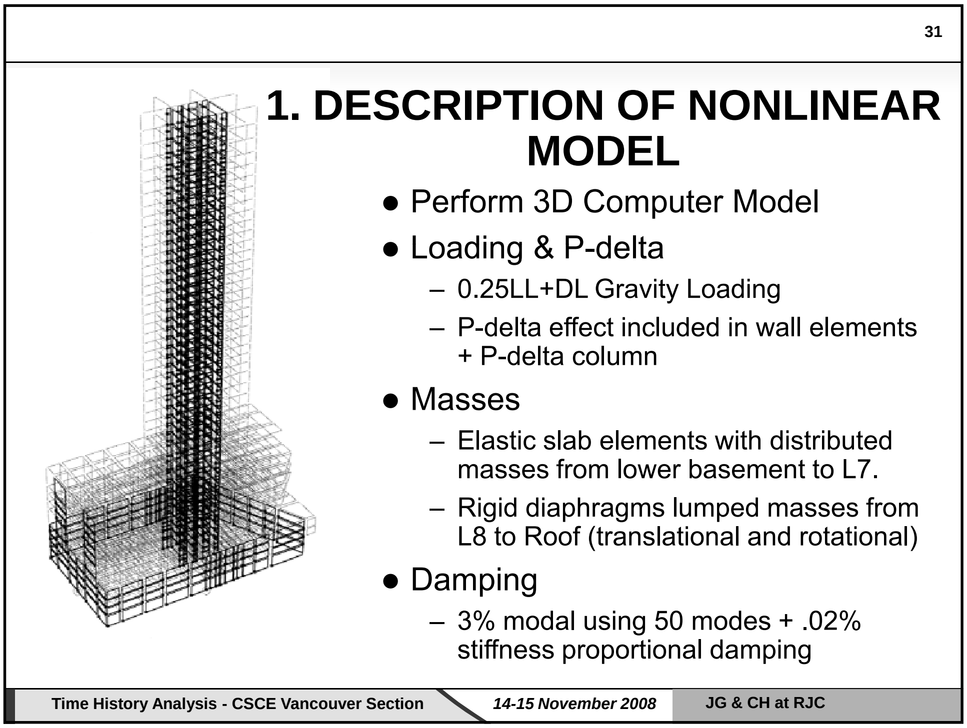
Building  dissipate seismic energy through flexural yielding at the wall base and through yielding of the coupling beams. All other elements of the building should remain elastic.

Did we achieve this ?

Answer by NLTH analysis.  !!!!!   ?????

# NONLINEAR ANALYSIS PRESENTATION OUTLINE

1. DESCRIPTION OF NONLINEAR MODEL
2. RESULTS OF NLA BASED ON A CODE DESIGN
3. REDESIGN AND MODELING
4. COMPARISON OF THE RESULTS FROM THE TWO NONLINEAR MODELS
5. SUMMARY



## 1. DESCRIPTION OF NONLINEAR MODEL

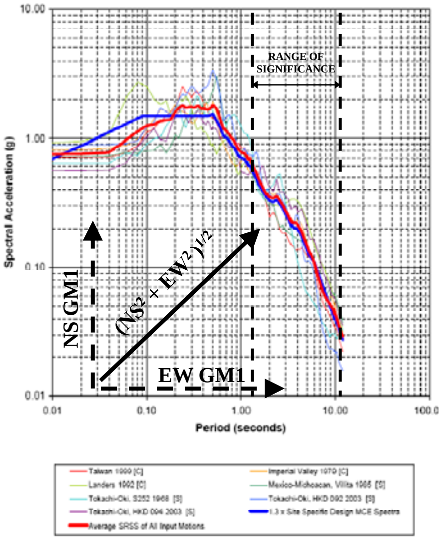
- Perform 3D Computer Model
- Loading & P-delta
  - 0.25LL+DL Gravity Loading
  - P-delta effect included in wall elements + P-delta column
- Masses
  - Elastic slab elements with distributed masses from lower basement to L7.
  - Rigid diaphragms lumped masses from L8 to Roof (translational and rotational)
- Damping
  - 3% modal using 50 modes + .02% stiffness proportional damping

---

## 1. DESCRIPTION OF NONLINEAR MODEL

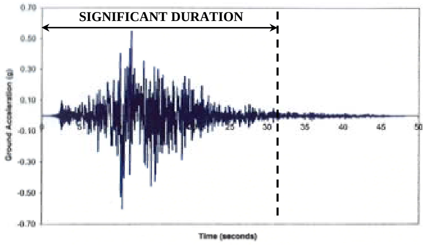
- EARTHQUAKE RECORDS
- CONCRETE CORE WALL ELEMENTS
- COUPLING BEAM ELEMENTS
- FOOTING SPRING ELEMENTS
- SLAB & BASEMENT WALL ELEMENTS
- SLAB/COLUMN ELEMENTS

## EARTHQUAKE RECORDS



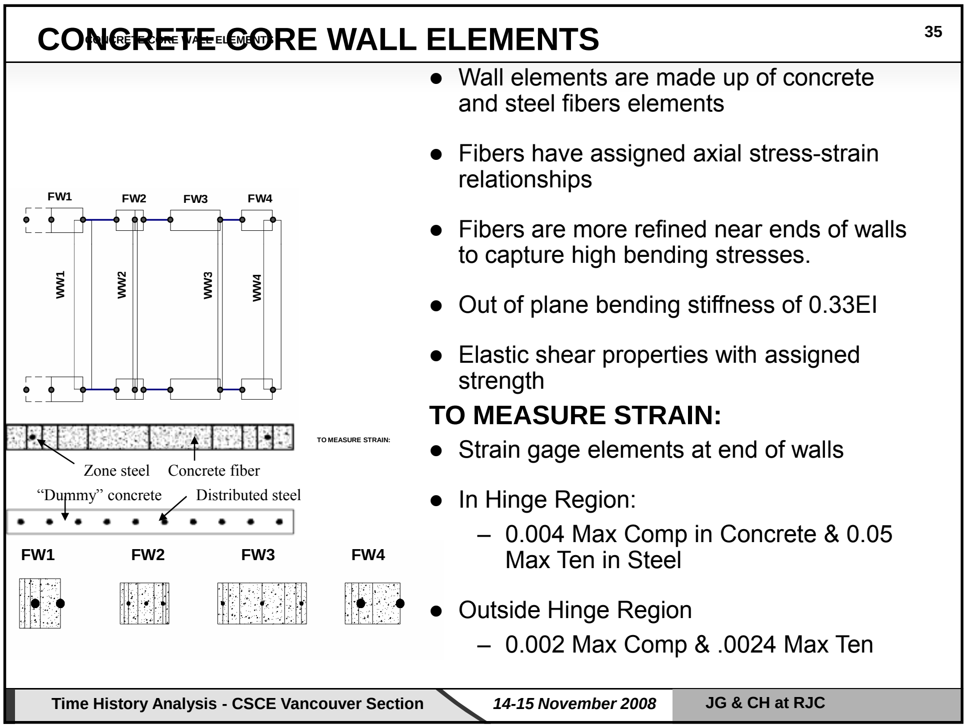
- 7 pairs of records (horizontal only)
- Chosen and scaled to the MCE Response Spectrum (RS) by the geotechnical engineer
- The average SRSS of all ground motions pairs scaled between 0.2T and 1.5T to 1.3 x the MCE RS
- The average of all ground motion components can not be fall below 80% of the MCE Spectrum

## EARTHQUAKE RECORDS



- Average computation time of 28hrs using standard 2.8 GHz processor.
- The significant duration of the record was used to save on computation time.
- Defined as a Arias intensity of 95%

| Record Pair | Type | Peak Acceleration | Duration Used | Time Step | Comp. Time |
|---|---|---|---|---|---|
| Taiwan 1999 | Crustal EQ | 0.55g | 41 sec | 0.005 sec | 31.7 hrs |
| Imperial Valley | Crustal EQ | 0.71g | 35 sec | 0.005 sec | 24.9 hrs |
| Landers 1992 | Crustal EQ | 0.71g | 31 sec | 0.005 sec | 25.7 hrs |
| Mexico 1985 | Subduction EQ | 0.51g | 62 sec | 0.005 sec | 47.5 hrs |
| Tokachi-Oki 1968 | Subduction EQ | 0.50g | 100 sec | 0.02 sec | 18.0 hrs |
| Tokachi-Oki 2003 092 | Subduction EQ | 0.72g | 76 sec | 0.02 sec | 15.6 hrs |
| Tokachi-Oki 2003 094 | Subduction EQ | 0.48g | 120 sec | 0.02 sec | 30.3 hrs |



## CONCRETE CORE WALL ELEMENTS

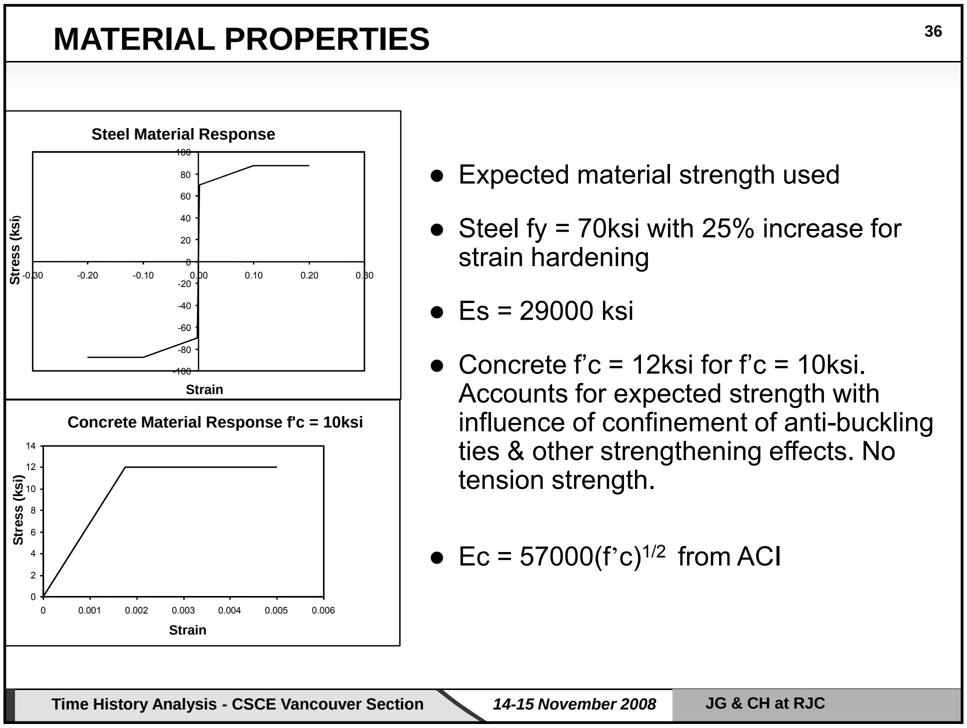
CONCRETE CORE WALL ELEMENTS

- Wall elements are made up of concrete and steel fibers elements
- Fibers have assigned axial stress-strain relationships
- Fibers are more refined near ends of walls to capture high bending stresses.
- Out of plane bending stiffness of 0.33EI
- Elastic shear properties with assigned strength

**TO MEASURE STRAIN:**

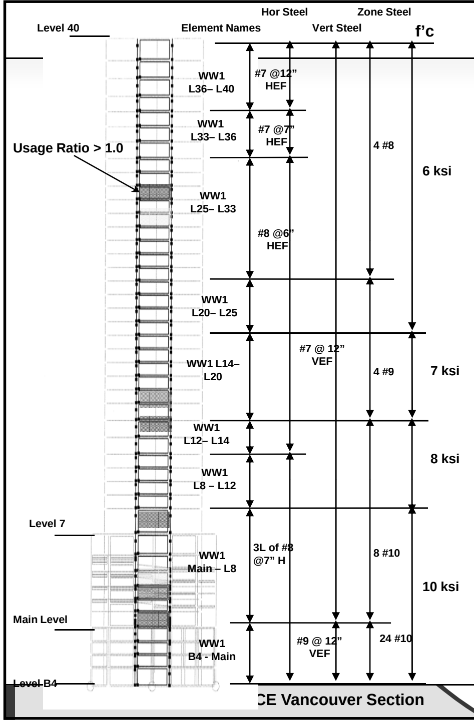
TO MEASURE STRAIN:

- Strain gage elements at end of walls
- In Hinge Region:
  - 0.004 Max Comp in Concrete & 0.05 Max Ten in Steel
- Outside Hinge Region
  - 0.002 Max Comp & .0024 Max Ten



## MATERIAL PROPERTIES

- Expected material strength used
- Steel fy = 70ksi with 25% increase for strain hardening
- Es = 29000 ksi
- Concrete f'c = 12ksi for f'c = 10ksi. Accounts for expected strength with influence of confinement of anti-buckling ties & other strengthening effects. No tension strength.
- Ec = 57000(f'c)$^{1/2}$ from ACI



## DISTRIBUTION OF WALL ELEMENT PROPERTIES

- Shear wall elements properties vary up the height of the core.
- This occurs where there is a change in f’c, zone steel, vertical steel, and horizontal steel.
- A large number of elements with different properties.
- Stiffness of each element depends on cracking of fiber elements.
- Program outputs strength usage (demand/capacity)ratios for each element based on user input.

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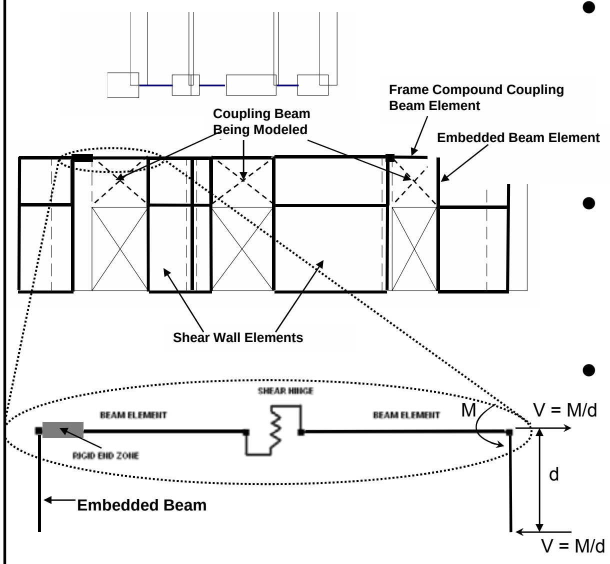
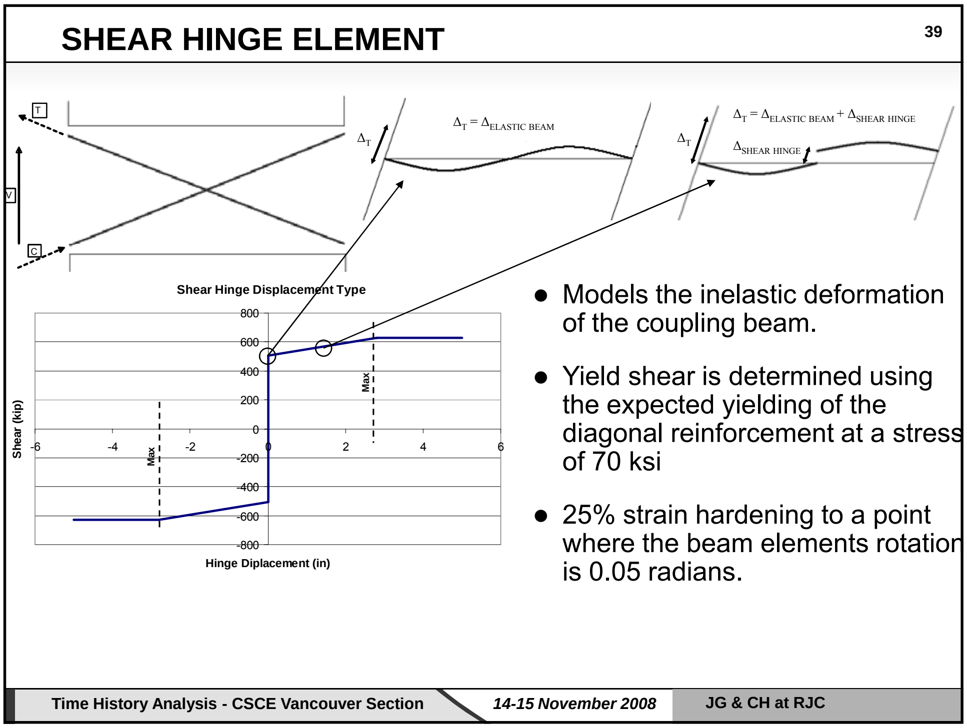
## COUPLING BEAM ELEMENTS

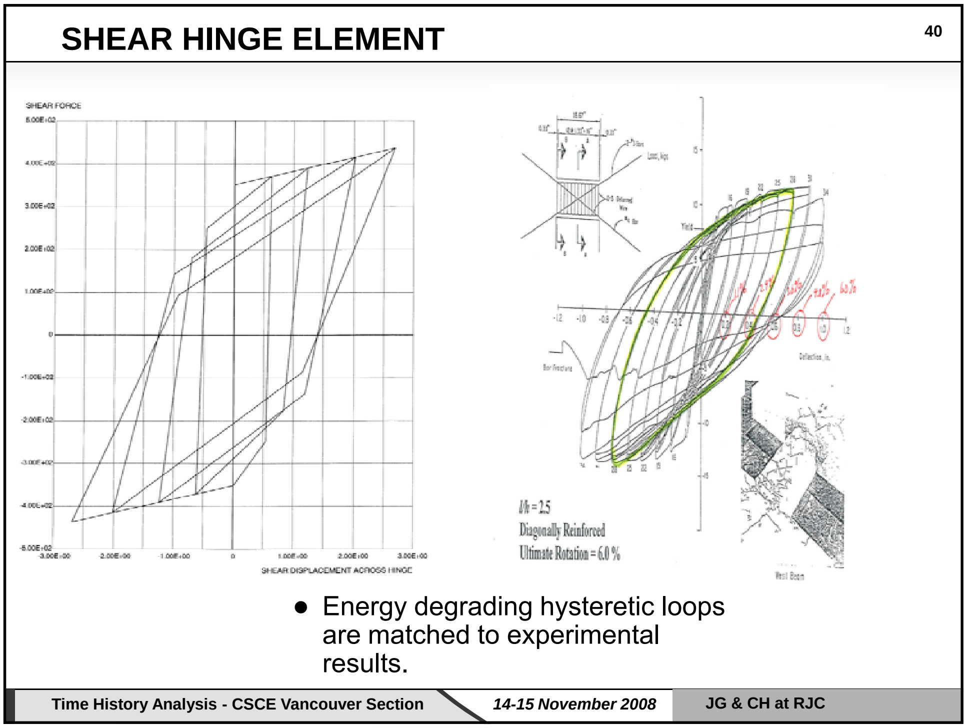


- Frame compound component made up of a rigid end zone, two beam elements, and a shear hinge displacement type element.
- Beam element properties vary up the building with a bending stiffness of 0.25EIg and shear stiffness of 0.25GAg.
- Vertical embedded beam element to transfer coupling beam moment to wall.



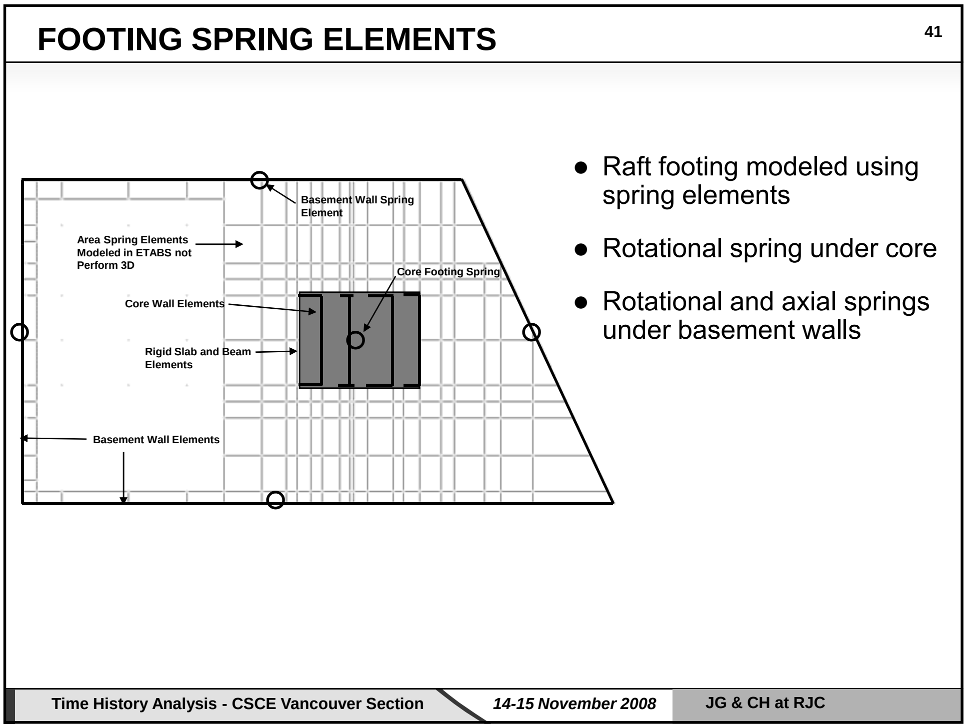
## SHEAR HINGE ELEMENT

- Models the inelastic deformation of the coupling beam.
- Yield shear is determined using the expected yielding of the diagonal reinforcement at a stress of 70 ksi
- 25% strain hardening to a point where the beam elements rotation is 0.05 radians.



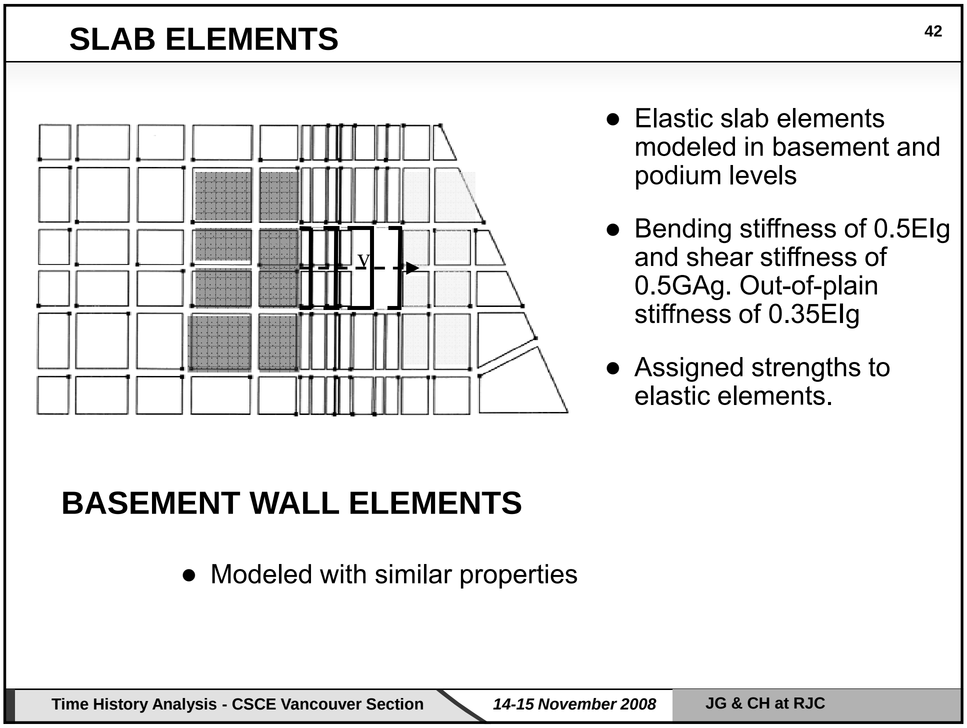
## SHEAR HINGE ELEMENT

- Energy degrading hysteretic loops are matched to experimental results.



# FOOTING SPRING ELEMENTS

- Raft footing modeled using spring elements
- Rotational spring under core
- Rotational and axial springs under basement walls



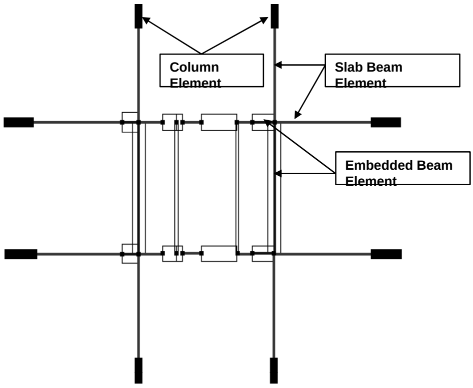
# SLAB ELEMENTS

- Elastic slab elements modeled in basement and podium levels
- Bending stiffness of 0.5EIg and shear stiffness of 0.5GAg. Out-of-plain stiffness of 0.35EIg
- Assigned strengths to elastic elements.

# BASEMENT WALL ELEMENTS

- Modeled with similar properties

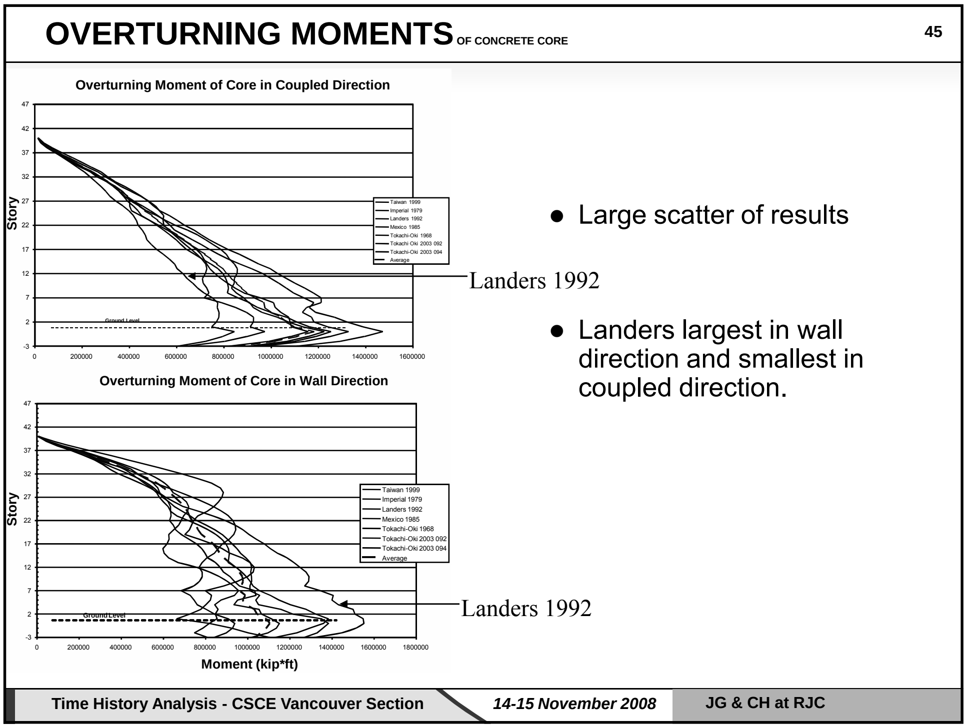
## SLAB-COLUMN ELEMENTS



- To model the slab to column and slab to core interaction up the tower , check for punching.
- Column elements with a bending stiffness of 0.7EIg.
- Beam elements with a bending stiffness of 0.35EIg and an EPP hinge modeled at its ends.
- Embedded beam elements to transfer beam moments.

## 2. RESULTS OF NLA BASED ON A CODE DESIGN

- Overturning Moments of Concrete Core
- Moment – Curvatures
- Story Shears of the Concrete Core
- Core Displacements
- Coupling Beam Rotations

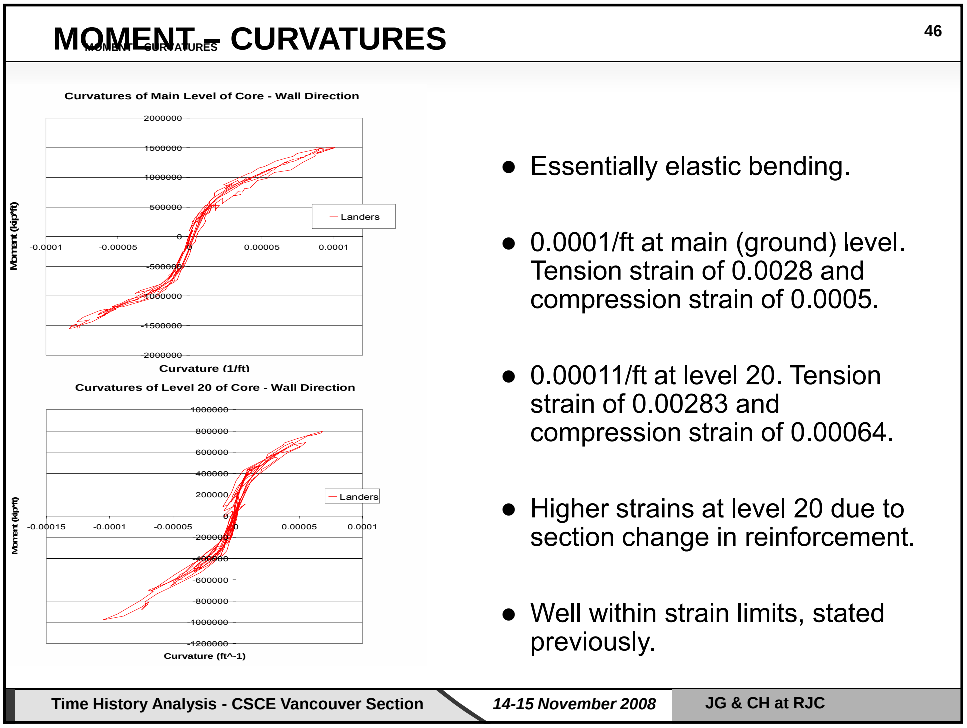


## OVERTURNING MOMENTS OF CONCRETE CORE

Landers 1992

Landers 1992

- Large scatter of results
- Landers largest in wall direction and smallest in coupled direction.

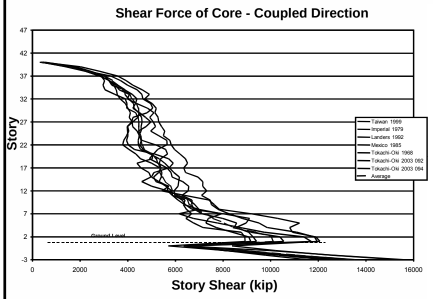


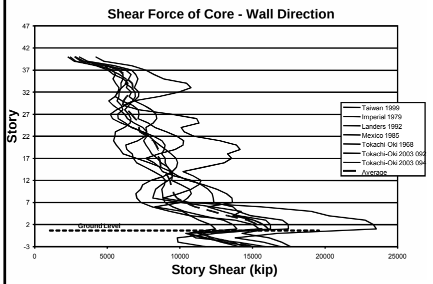
## MOMENT – CURVATURES

MOMENT - CURVATURES

- Essentially elastic bending.
- 0.0001/ft at main (ground) level. Tension strain of 0.0028 and compression strain of 0.0005.
- 0.00011/ft at level 20. Tension strain of 0.00283 and compression strain of 0.00064.
- Higher strains at level 20 due to section change in reinforcement.
- Well within strain limits, stated previously.

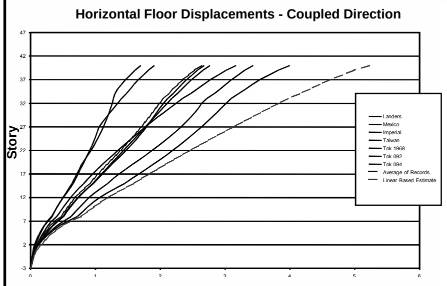
## STORY SHEARS OF THE CONCRETE CORE

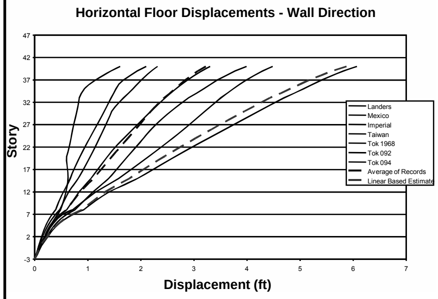

Shear Force of Core - Coupled Direction


Shear Force of Core - Wall Direction

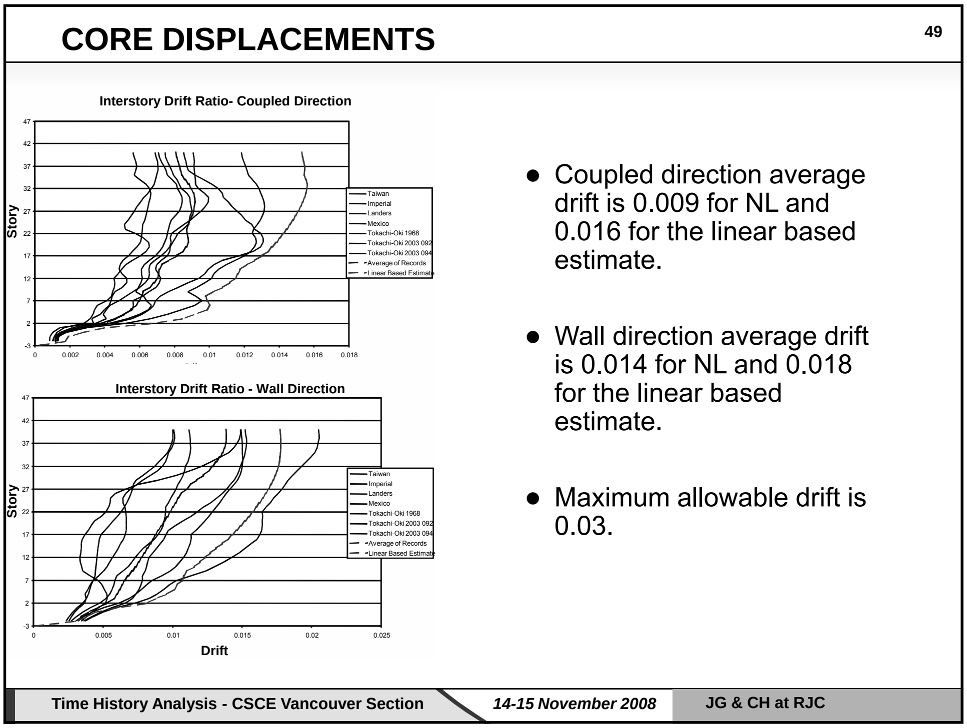
- Larger scatter in wall direction.
- Probable shear capacity of 24,000 kips in coupled direction and 39,000 kips in wall direction
- Large over strength resulting from a capacity design.
- Room for cost savings on shear reinforcing
- Code allows to detail shear reinforcing from average result of NLA.

## CORE DISPLACEMENTS


Horizontal Floor Displacements - Coupled Direction

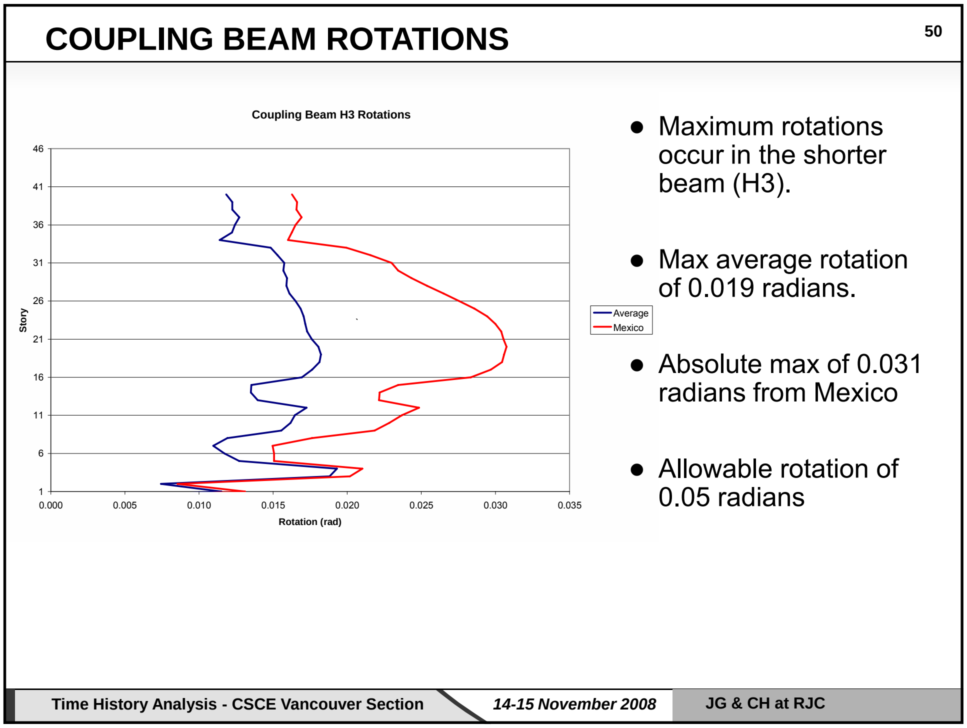

Horizontal Floor Displacements - Wall Direction

- Large scatter of results
- Linear based estimate using modal analysis in ETABS
- Average NL roof displacement of 2.6 ft in coupled direction and 5.2 ft for linear based estimate.
- Average NL roof displacement of 3.2 ft in wall direction and 5.9 ft for linear based estimate.



## CORE DISPLACEMENTS

- Coupled direction average drift is 0.009 for NL and 0.016 for the linear based estimate.
- Wall direction average drift is 0.014 for NL and 0.018 for the linear based estimate.
- Maximum allowable drift is 0.03.



## COUPLING BEAM ROTATIONS

- Maximum rotations occur in the shorter beam (H3).
- Max average rotation of 0.019 radians.
- Absolute max of 0.031 radians from Mexico
- Allowable rotation of 0.05 radians

## 3. REDESIGN BASED ON RESULTS FROM THE NLA

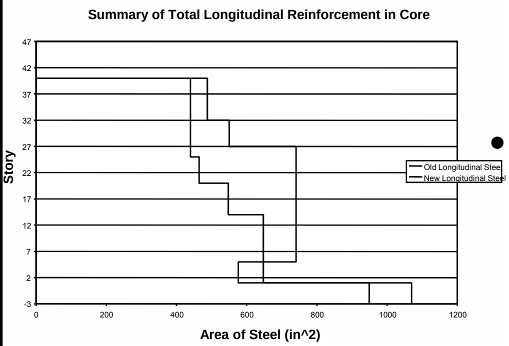
- The coupling beams showed to be too strong. Capacity designing the coupled direction created large uplift demands on adjacent walls. This governed wall direction steel, creating a high overstrength and small strains.
- Need more energy dissipation from the Nonlinear model.
- Need elastic strains above hinge region
- Required by Peer Reviewers.

---

## CHANGES TO COUPLING BEAMS

- Strength of coupling beams weakened by reducing amount of diagonal reinforcing
  - Reducing uplift demand on adjacent walls
  - Allows more energy dissipation through higher inelastic deformation of coupling beams.

## REDISTRIBUTION OF VERTICAL CORE REINFORCEMENT

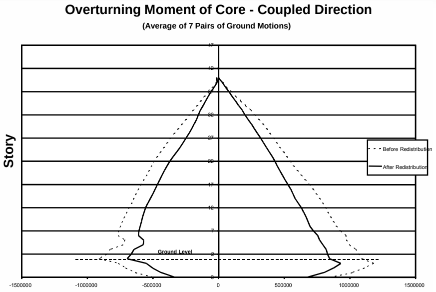


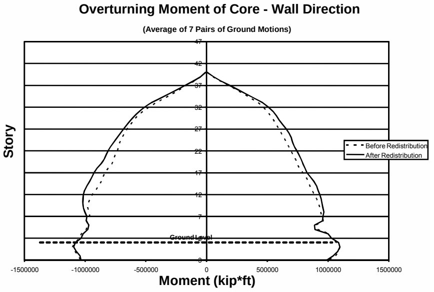
- Vertical reinforcement reduced from reduction of uplift demand from coupling beams
  - Allows more inelastic deformations in intended hinge region
  - Allows more energy dissipation.
- Vertical reinforcement increased above the hinge region (L5 – Roof)
  - Creates a notch effect to force a hinging mechanism in the intended hinge region
  - Strengthens the core above the hinge to reduce flexural strains to an elastic range

## 4. COMPARISON OF THE TWO NONLINEAR MODELS

- Overturning Moments
- Moment – Curvatures
- Story Shears
- Displacements
- Coupling Beam Rotations

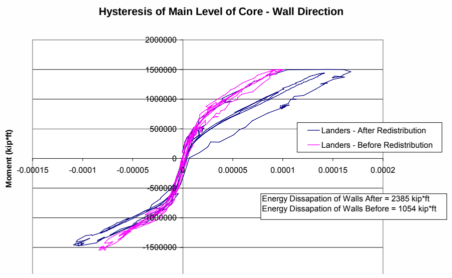
## OVERTURNING MOMENTS


Overturning Moment of Core - Coupled Direction
(Average of 7 Pairs of Ground Motions)

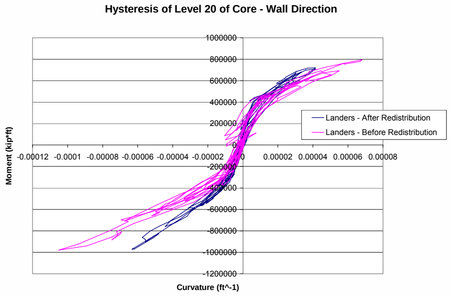

Overturning Moment of Core - Wall Direction
(Average of 7 Pairs of Ground Motions)

- Coupled direction decreased
  - From reduction of coupling beam strength
- Slight increase above hinge region in wall direction
  - Could be from an increase of flexural stiffness from increase of reinforcing

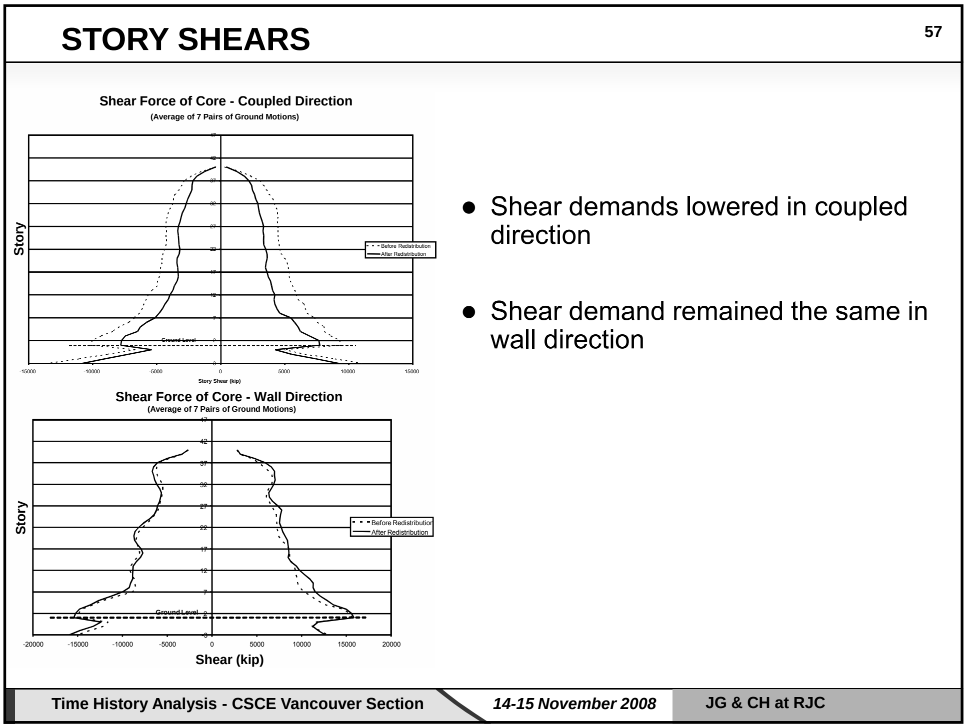
## MOMENT – CURVATURES


Hysteresis of Main Level of Core - Wall Direction

Energy Dissapation of Walls After = 2385 kip*ft
Energy Dissapation of Walls Before = 1054 kip*ft

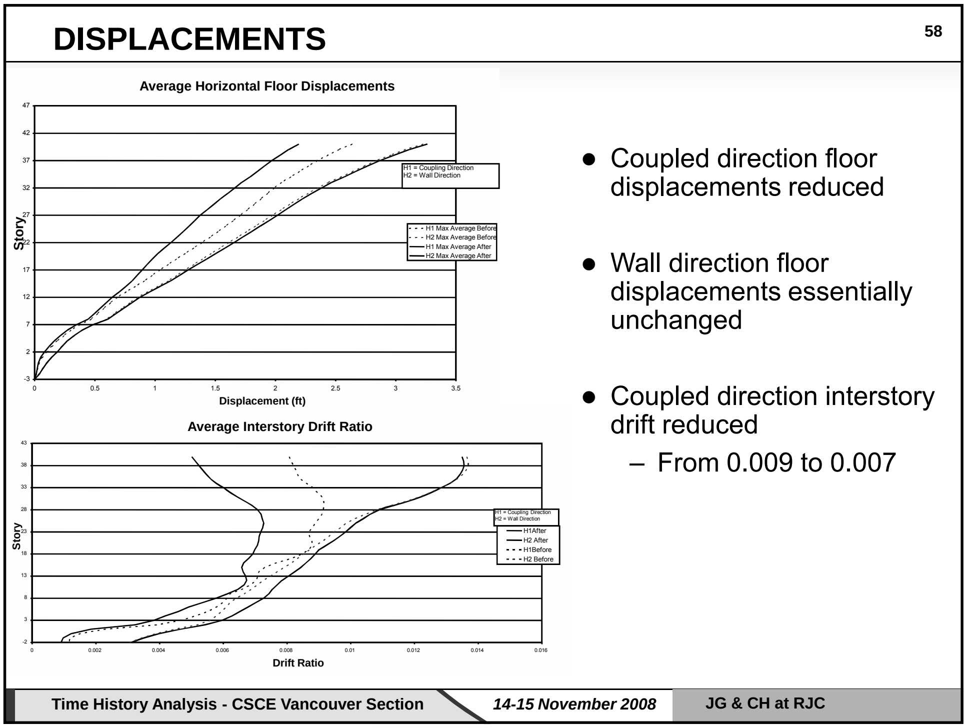

Hysteresis of Level 20 of Core - Wall Direction

- Ground Level (Hinge region)
  - Max strains of 0.0047 in tension and .0006 in comp.
  - Double the energy dissipation
- Level 20 (Elastic region)
  - Strain reduced to elastic range
  - Max strain of 0.0016 in tension and 0.0005 in comp.



## STORY SHEARS

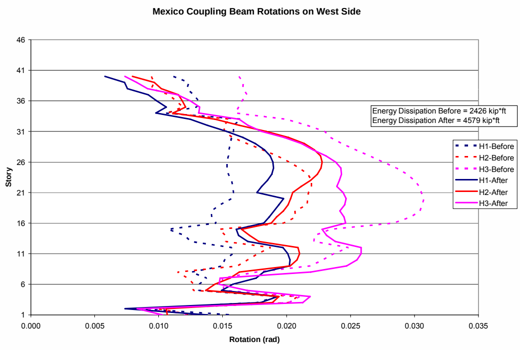
- Shear demands lowered in coupled direction
- Shear demand remained the same in wall direction



## DISPLACEMENTS

- Coupled direction floor displacements reduced
- Wall direction floor displacements essentially unchanged
- Coupled direction interstory drift reduced
  - From 0.009 to 0.007

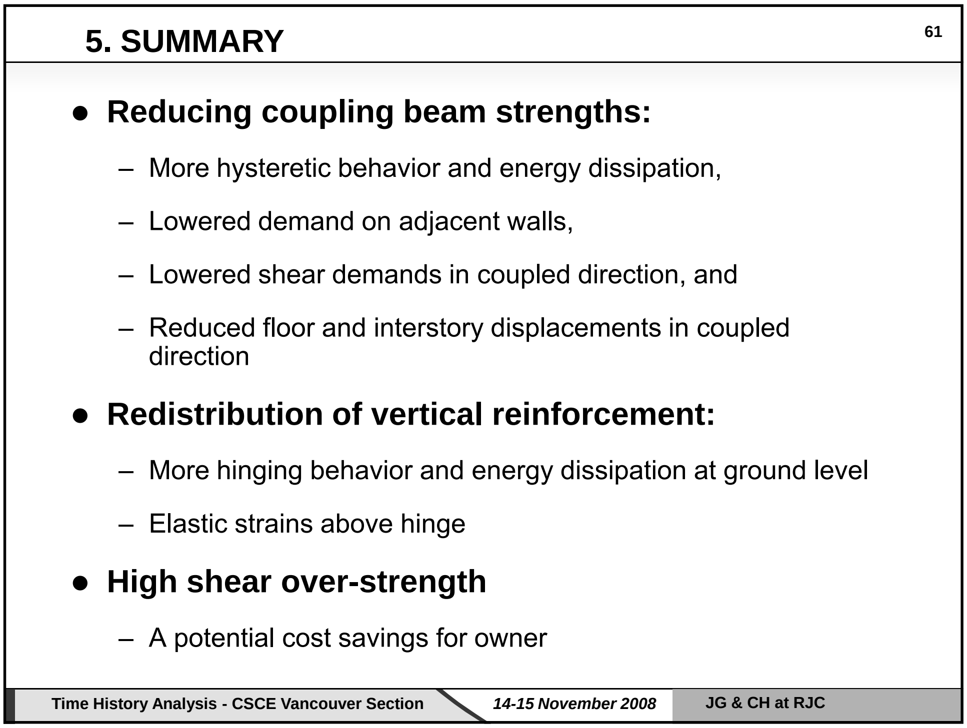
## COUPLING BEAM ROTATIONS


Mexico Coupling Beam Rotations on West Side

- Max rotation reduced
- Double the energy dissipation
  - More use out of coupling beam rotational capacity

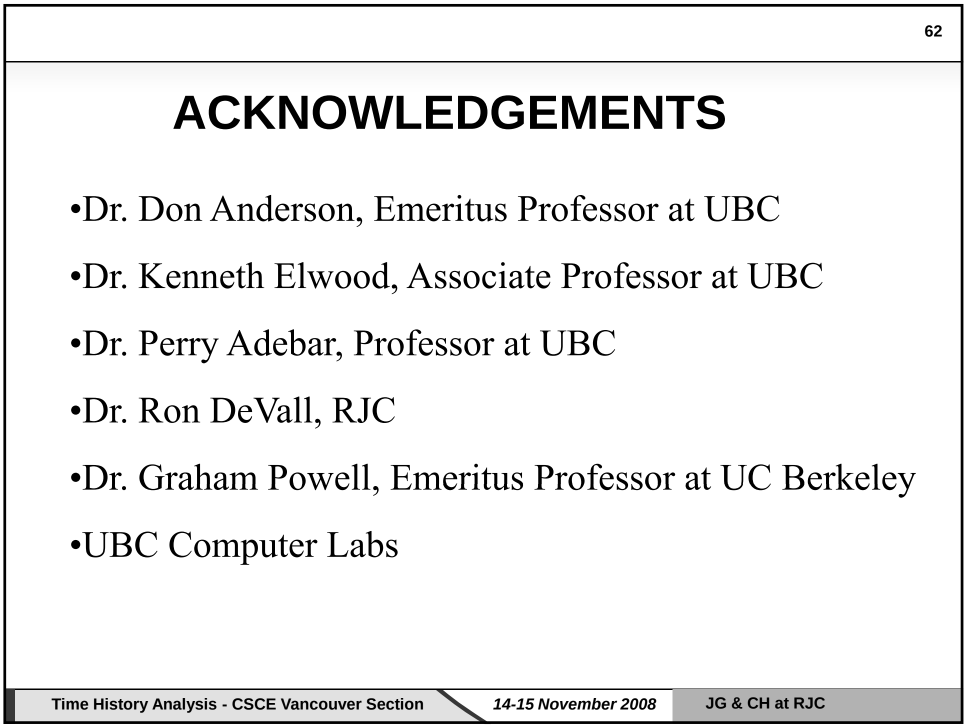
## 5. SUMMARY

- **A redesign and NLA was required in order to achieve a desired performance of:**
  - hinging at the ground level,
  - an elastic flexural response above the hinge, and
  - more energy dissipation (particularly in coupling direction).



## 5. SUMMARY

- **Reducing coupling beam strengths:**
  - More hysteretic behavior and energy dissipation,
  - Lowered demand on adjacent walls,
  - Lowered shear demands in coupled direction, and
  - Reduced floor and interstory displacements in coupled direction
- **Redistribution of vertical reinforcement:**
  - More hinging behavior and energy dissipation at ground level
  - Elastic strains above hinge
- **High shear over-strength**
  - A potential cost savings for owner



## ACKNOWLEDGEMENTS

•Dr. Don Anderson, Emeritus Professor at UBC

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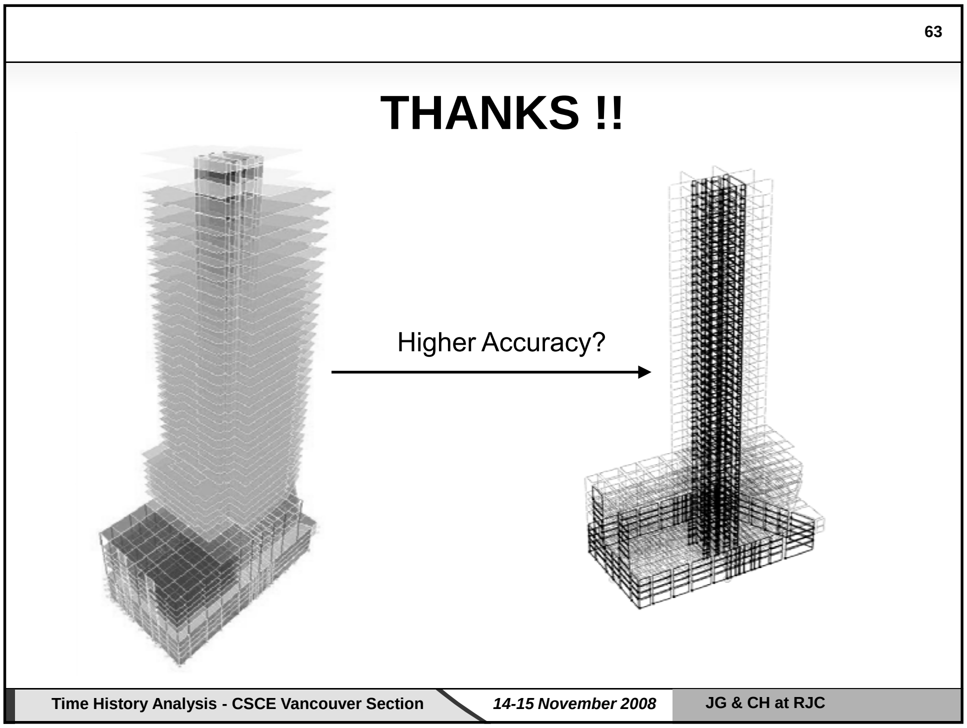
•Dr. Perry Adebar, Professor at UBC

•Dr. Ron DeVall, RJC

•Dr. Graham Powell, Emeritus Professor at UC Berkeley

•UBC Computer Labs



# THANKS !!

Higher Accuracy?

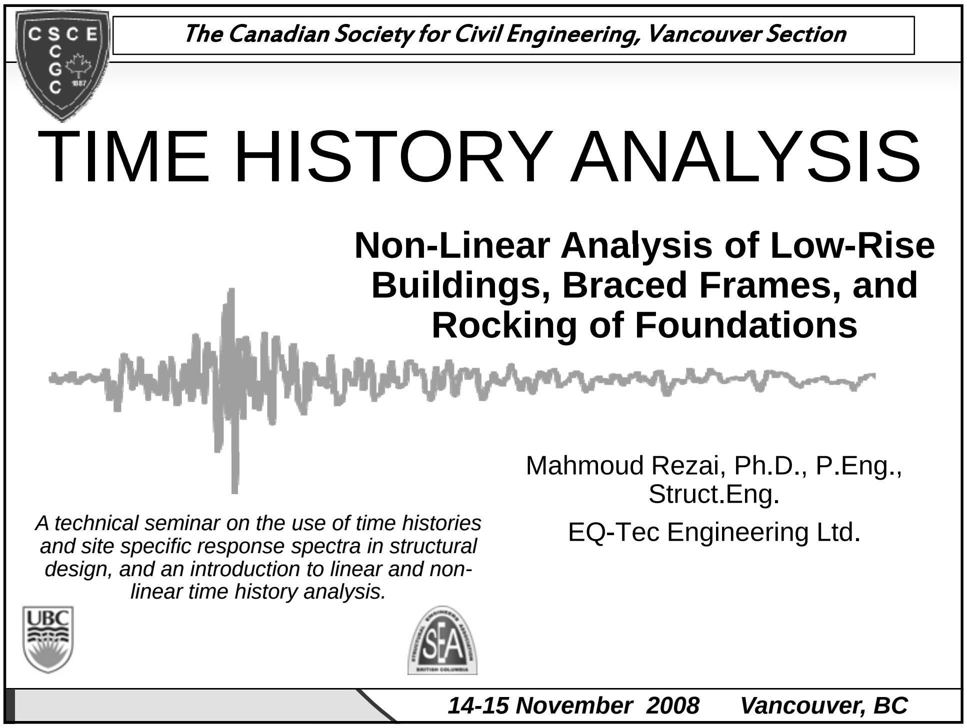
# TIME HISTORY
# ANALYSIS

## LECTURE # 12

## Non-linear Analysis of Low Rise Buildings, Braced Frames, and Rocking of Foundations

**Mahmoud Rezai**
**EQ-Tec Engineering Ltd.**

Dr. Rezai specializes in the analysis/design and understanding of non-linear behaviour of structures and their components. He has successfully incorporated ”innovative technologies” in various projects including using Ballast Water Tanks to increase the overall damping and thus minimizing the effect of wave motions, Fibre Reinforced Polymers (FRP), passive energy dissipation devices such as viscous dampers as well as base isolation system. He has carried out seismic assessment and design of a number of buildings and bridges in the past decade. He has provided peer reviews and design checks of numerous upgrade projects including analysis/design and construction field services for a number of concrete high-rise buildings, the Pattullo Bridge, Lions Gate Bridge and upgrade and assessment services for many different structures including Vancouver schools and hospitals. He has authored more than 50 papers and reports on structural analysis/design and behaviour/response of structural systems. Over the past ten years he has taught courses related to seismic analysis and design and retrofit of existing structures as a lecturer for UBC’s Certificate Program to the practicing engineers.



*The Canadian Society for Civil Engineering, Vancouver Section*

# TIME HISTORY ANALYSIS

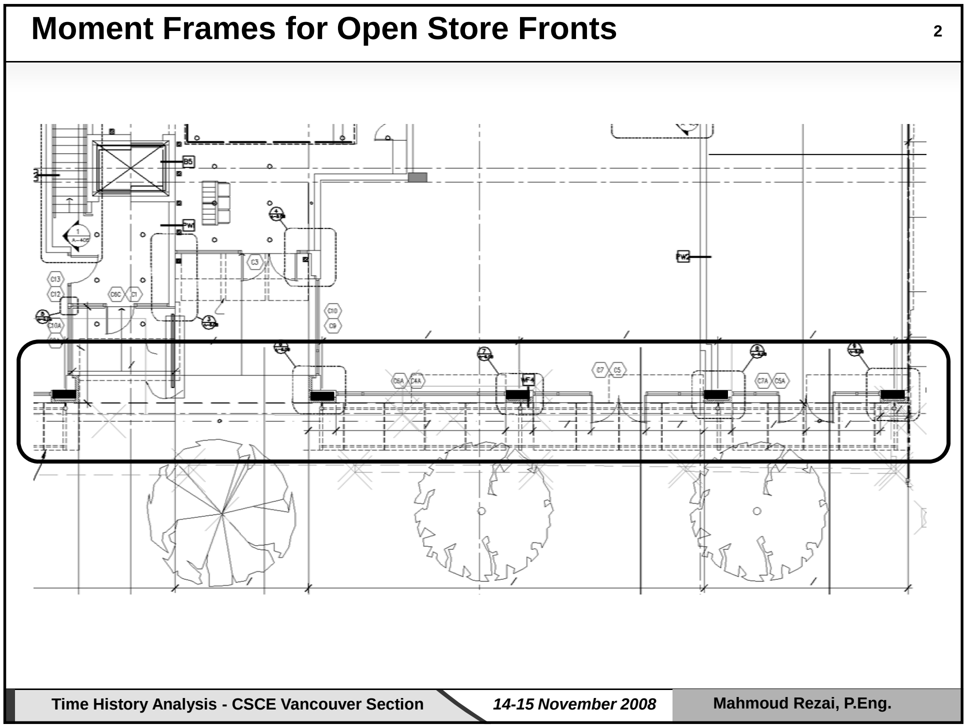
## Non-Linear Analysis of Low-Rise Buildings, Braced Frames, and Rocking of Foundations

Mahmoud Rezai, Ph.D., P.Eng.,
Struct.Eng.

EQ-Tec Engineering Ltd.

*A technical seminar on the use of time histories and site specific response spectra in structural design, and an introduction to linear and non-linear time history analysis.*

***14-15 November 2008 Vancouver, BC***

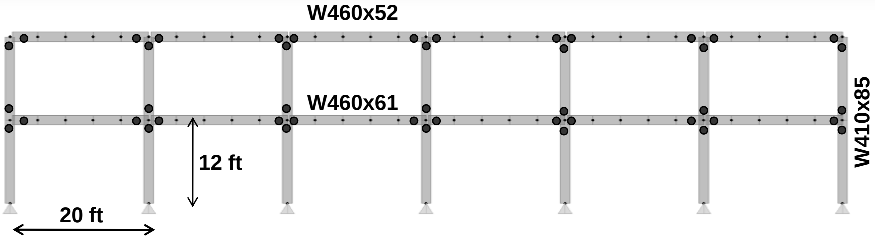


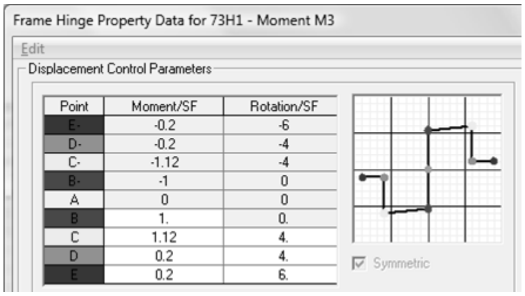
## Moment Frames for Open Store Fronts

2

**Time History Analysis - CSCE Vancouver Section** ***14-15 November 2008*** **Mahmoud Rezai, P.Eng.**

## Two-Storey Moment Frame



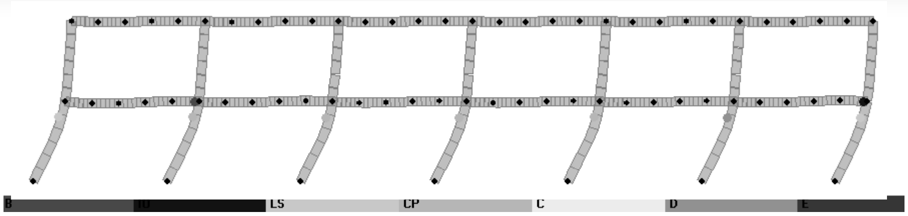

Frame Hinge Property Data for 73H1 - Moment M3

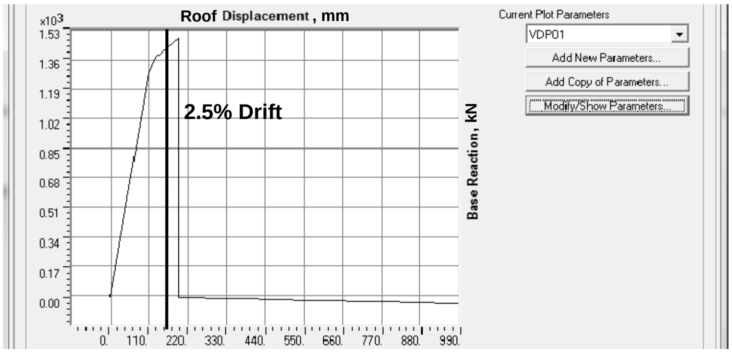
Displacement Control Parameters

| Point | Moment/SF | Rotation/SF |
|---|---|---|
| E- | -0.2 | -6 |
| D- | -0.2 | -4 |
| C- | -1.12 | -4 |
| B- | -1 | 0 |
| A | 0 | 0 |
| B | 1. | 0. |
| C | 1.12 | 4. |
| D | 0.2 | 4. |
| E | 0.2 | 6. |

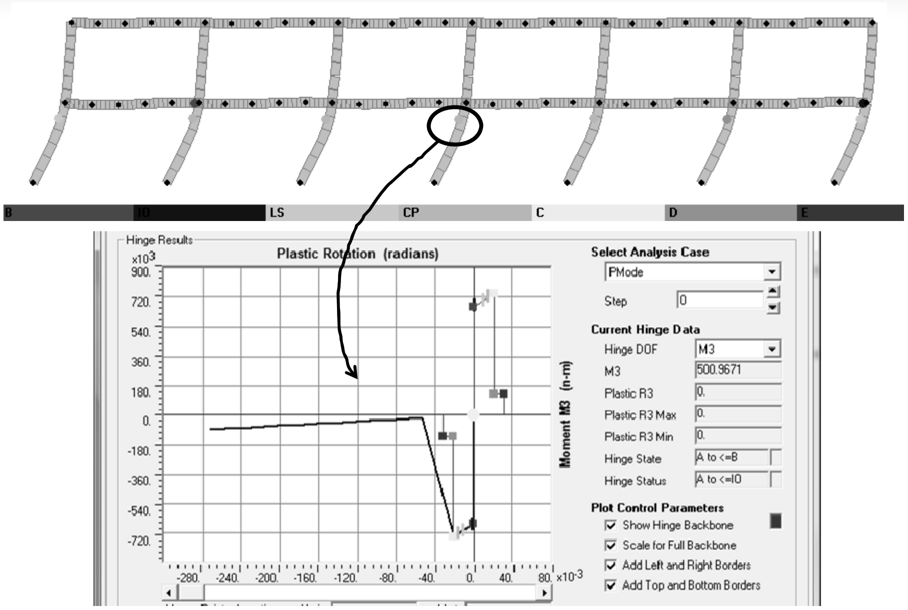
☑ Symmetric

## Two-Storey Moment Frame





## Two-Storey Moment Frame

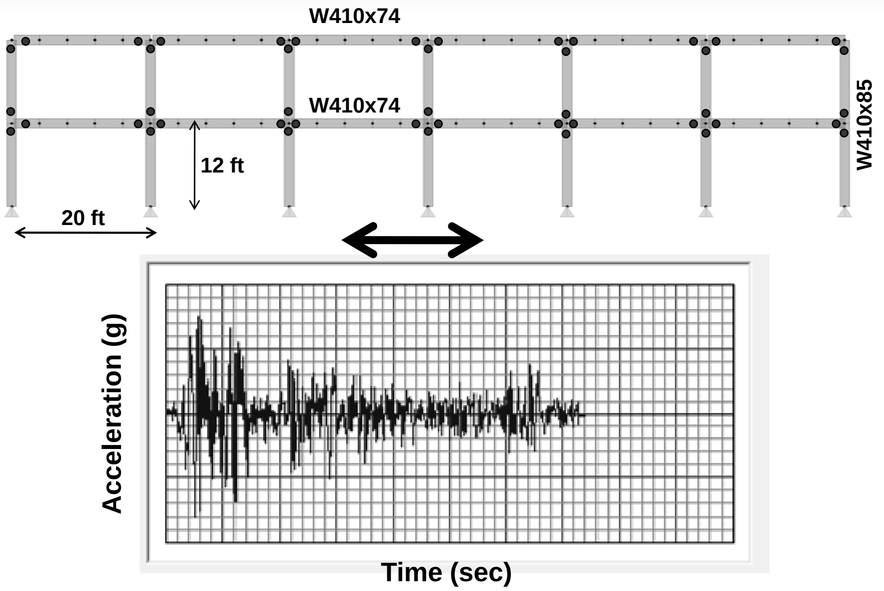


**Select Analysis Case**
PMode
Step: 0

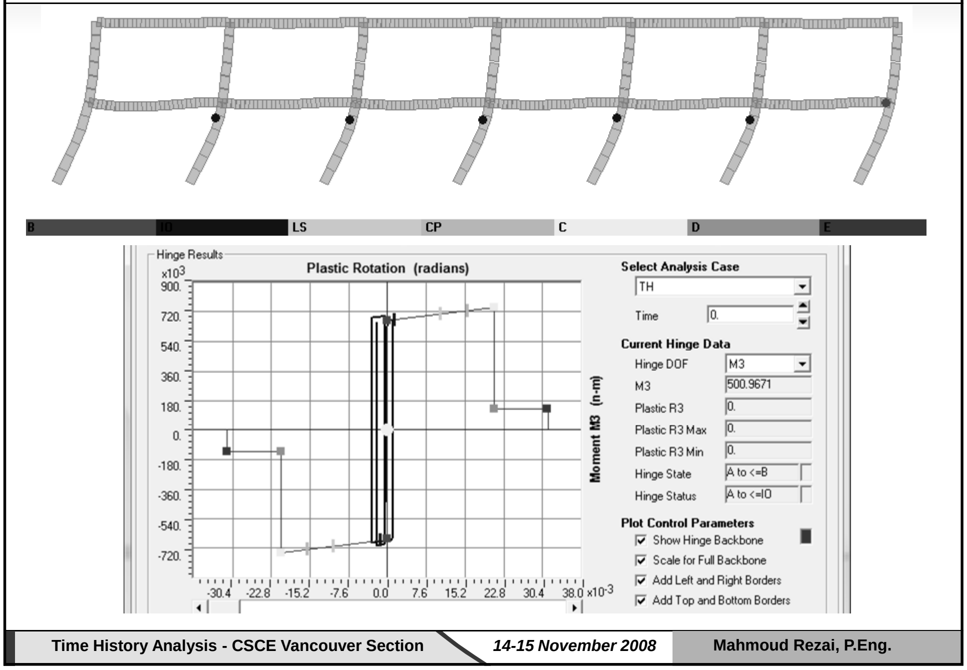
**Current Hinge Data**
Hinge DOF: M3
M3: 500.9671
Plastic R3: 0.
Plastic R3 Max: 0.
Plastic R3 Min: 0.
Hinge State: A to <=B
Hinge Status: A to <=IO

**Plot Control Parameters**
☑ Show Hinge Backbone
☑ Scale for Full Backbone
☑ Add Left and Right Borders
☑ Add Top and Bottom Borders

## Two-Storey Moment Frame



## Two-Storey Moment Frame



Select Analysis Case: TH

Time: 0.

**Current Hinge Data**

Hinge DOF: M3

M3: 500.9671

Plastic R3: 0.

Plastic R3 Max: 0.

Plastic R3 Min: 0.

Hinge State: A to <=B

Hinge Status: A to <=IO

**Plot Control Parameters**

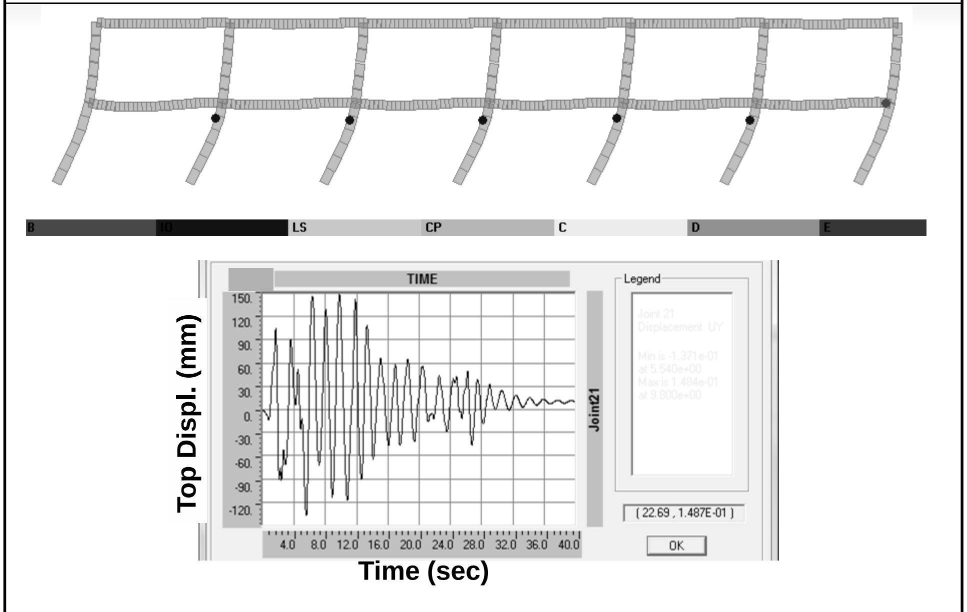
☑ Show Hinge Backbone

☑ Scale for Full Backbone

☑ Add Left and Right Borders

☑ Add Top and Bottom Borders

## Two-Storey Moment Frame

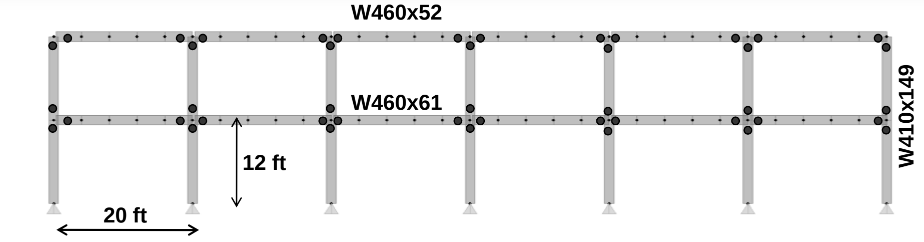


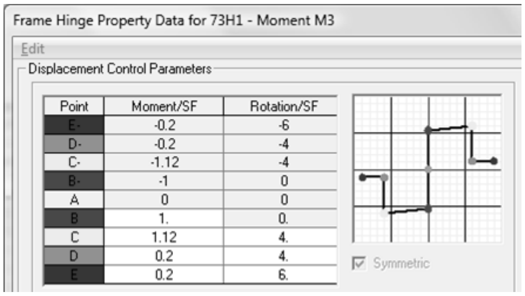
Joint 21
Displacement UY

Min is -1.371e-01
at 5.540e+00
Max is 1.484e-01
at 9.800e+00

( 22.69 , 1.487E-01 )

## Two-Storey Moment Frame – Stronger Column



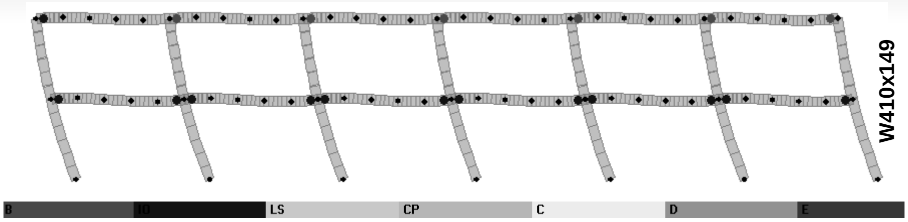

Frame Hinge Property Data for 73H1 - Moment M3

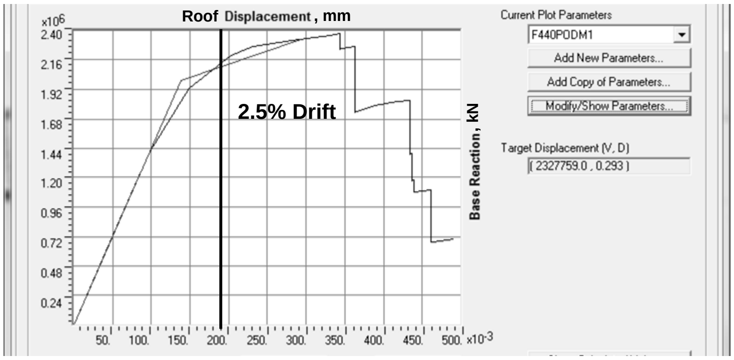
Displacement Control Parameters

| Point | Moment/SF | Rotation/SF |
|---|---|---|
| E- | -0.2 | -6 |
| D- | -0.2 | -4 |
| C- | -1.12 | -4 |
| B- | -1 | 0 |
| A | 0 | 0 |
| B | 1. | 0. |
| C | 1.12 | 4. |
| D | 0.2 | 4. |
| E | 0.2 | 6. |

☑ Symmetric

## Two-Storey Moment Frame – Stronger Column

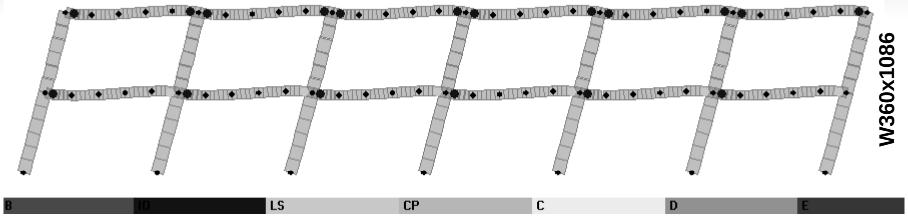


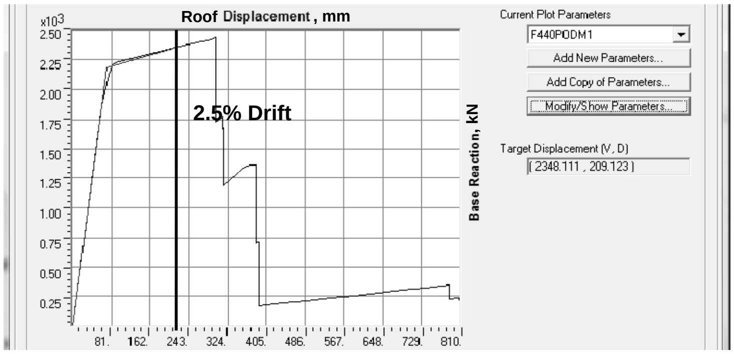


Current Plot Parameters
F440PODM1

Target Displacement (V, D)
( 2327759.0 , 0.293 )

## Two-Storey Moment Frame – Stronger Column





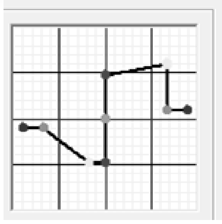
Current Plot Parameters: F440PODM1

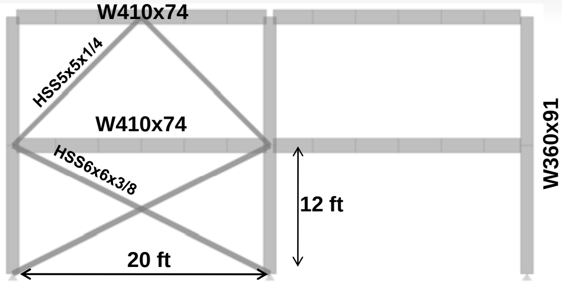
Target Displacement (V, D): ( 2348.111 , 209.123 )

Time History Analysis - CSCE Vancouver Section | 14-15 November 2008 | Mahmoud Rezai, P.Eng.

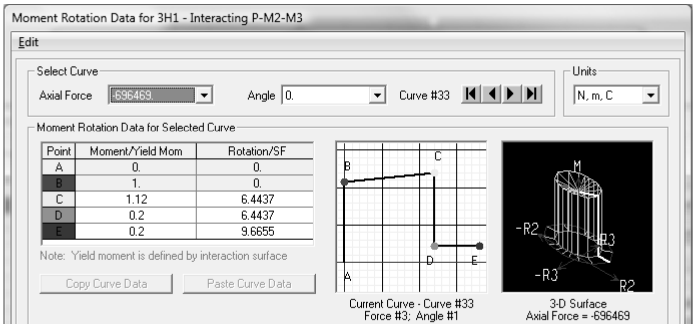
## Two-Storey Braced Frame

**Brace**





**Columns**



Moment Rotation Data for 3H1 - Interacting P-M2-M3

Select Curve

Axial Force: -696469.    Angle: 0.    Curve #33

Units: N, m, C

Moment Rotation Data for Selected Curve

| Point | Moment/Yield Mom | Rotation/SF |
|---|---|---|
| A | 0. | 0. |
| B | 1. | 0. |
| C | 1.12 | 6.4437 |
| D | 0.2 | 6.4437 |
| E | 0.2 | 9.6655 |

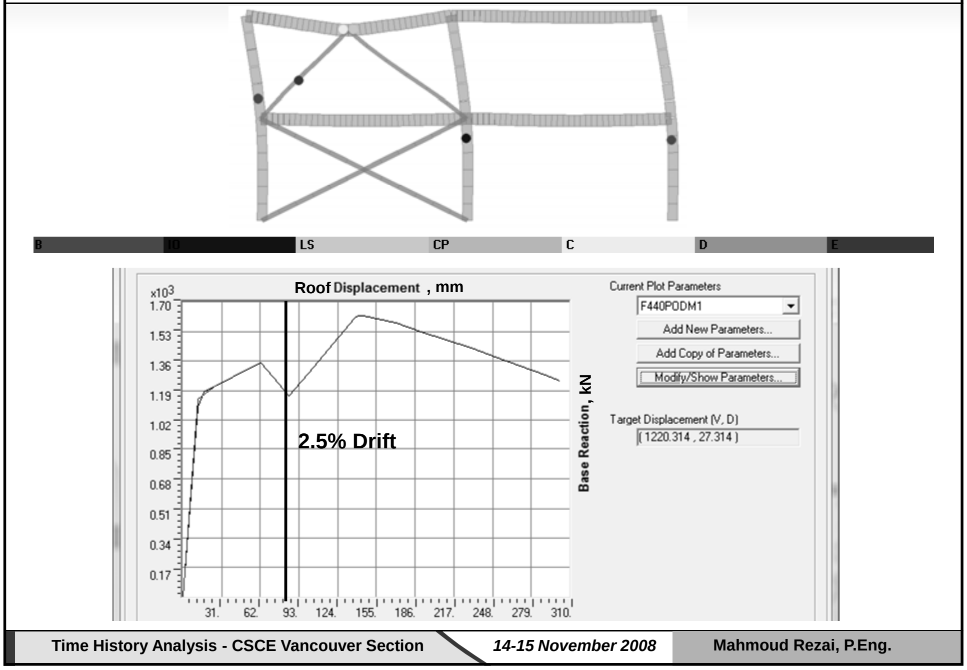
Note: Yield moment is defined by interaction surface

Current Curve - Curve #33
Force #3; Angle #1

3-D Surface
Axial Force = -696469

Time History Analysis - CSCE Vancouver Section | 14-15 November 2008 | Mahmoud Rezai, P.Eng.

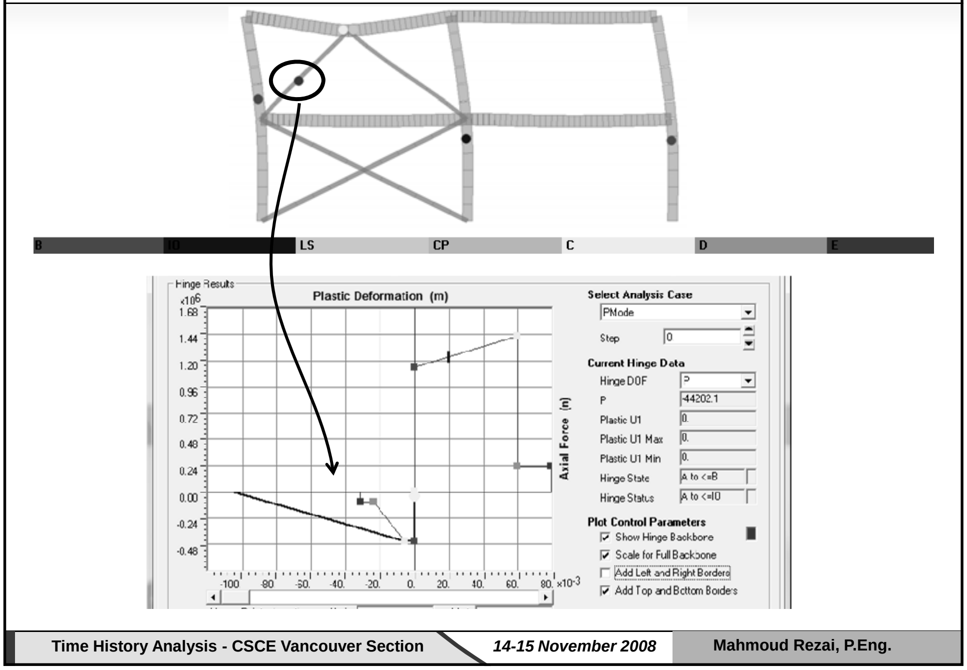
## Two-Storey Braced Frame – Load Displ. Plot



Current Plot Parameters: F440PODM1

Target Displacement (V, D): ( 1220.314 , 27.314 )

## Two-Storey Braced Frame

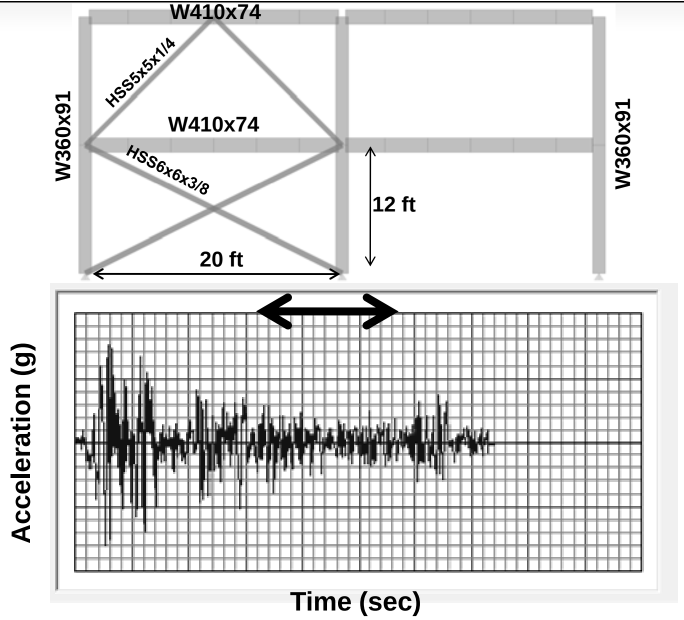


**Select Analysis Case**
PMode
Step: 0

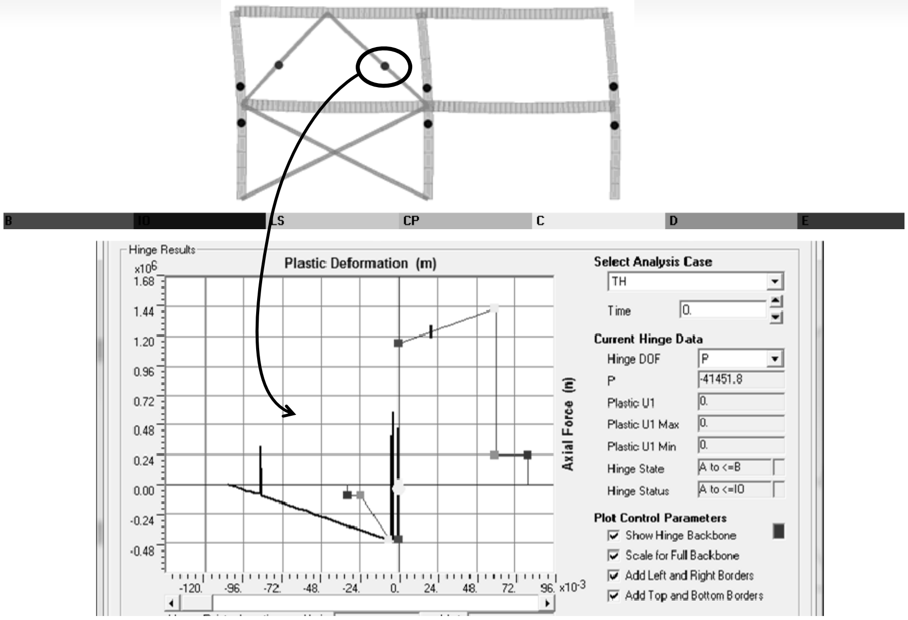
**Current Hinge Data**
Hinge DOF: P
P: -44202.1
Plastic U1: 0.
Plastic U1 Max: 0.
Plastic U1 Min: 0.
Hinge State: A to <=B
Hinge Status: A to <=IO

**Plot Control Parameters**
☑ Show Hinge Backbore
☑ Scale for Full Backbone
☐ Add Left and Right Borders
☑ Add Top and Bottom Borders

## Two-Storey Braced Frame



## Two-Storey Braced Frame



Select Analysis Case: TH

Time: 0.

Current Hinge Data

Hinge DOF: P

P: -41451.8

Plastic U1: 0.

Plastic U1 Max: 0.

Plastic U1 Min: 0.

Hinge State: A to <=B

Hinge Status: A to <=IO

Plot Control Parameters

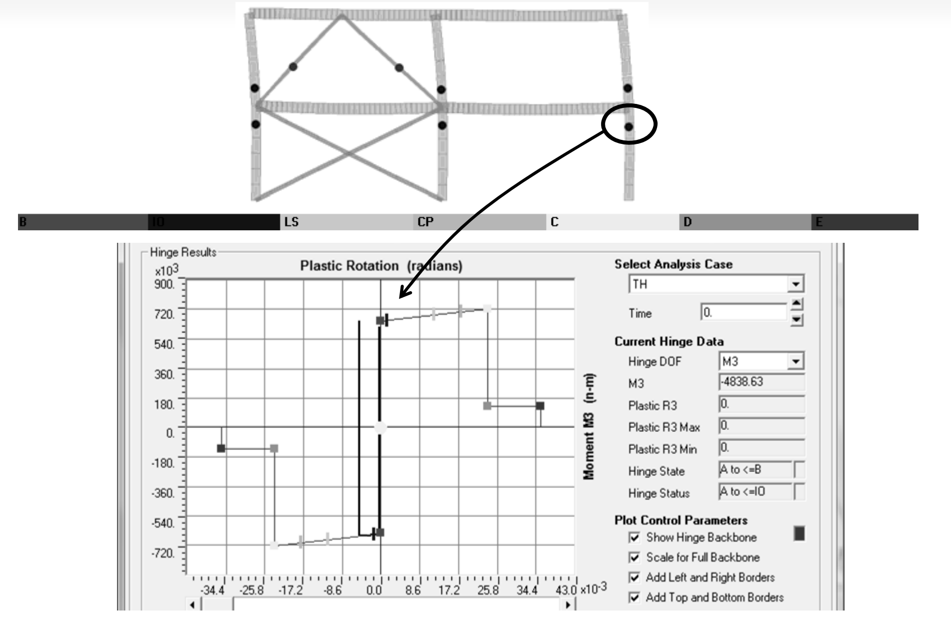
☑ Show Hinge Backbone

☑ Scale for Full Backbone

☑ Add Left and Right Borders

☑ Add Top and Bottom Borders

## Two-Storey Braced Frame



**Select Analysis Case**

TH

Time 0.

**Current Hinge Data**

Hinge DOF M3

M3 -4838.63

Plastic R3 0.

Plastic R3 Max 0.

Plastic R3 Min 0.

Hinge State A to <=B

Hinge Status A to <=IO

**Plot Control Parameters**

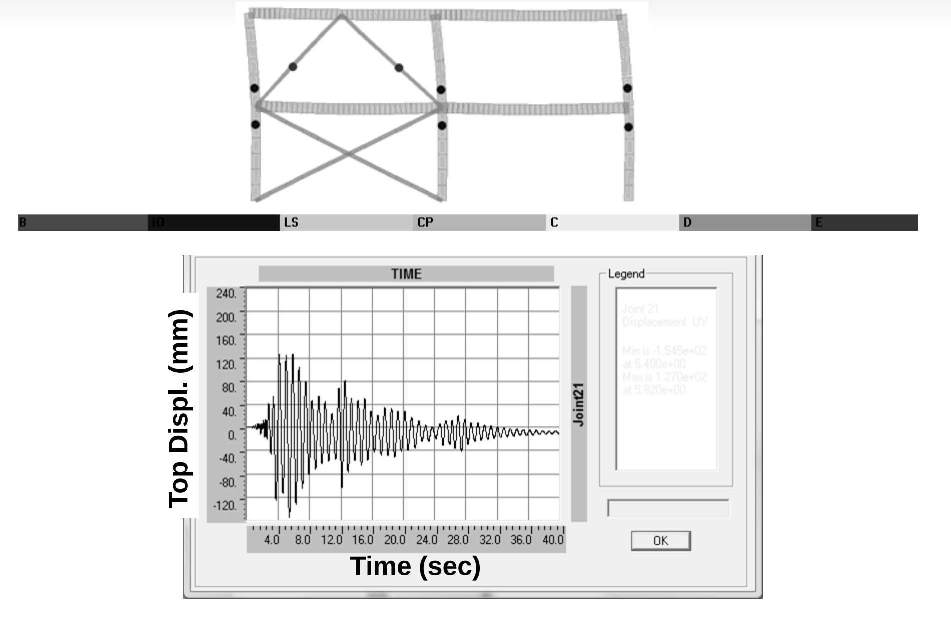
☑ Show Hinge Backbone

☑ Scale for Full Backbone

☑ Add Left and Right Borders

☑ Add Top and Bottom Borders

## Two-Storey Braced Frame



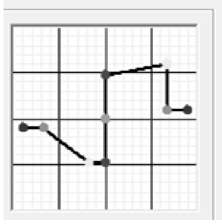
Legend

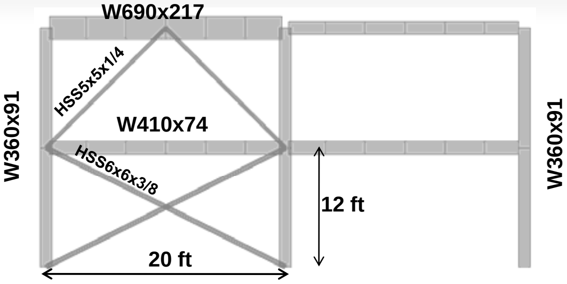
Joint 21
Displacement UY

Min is -1.545e+02
at 5.400e+00
Max is 1.270e+02
at 5.820e+00

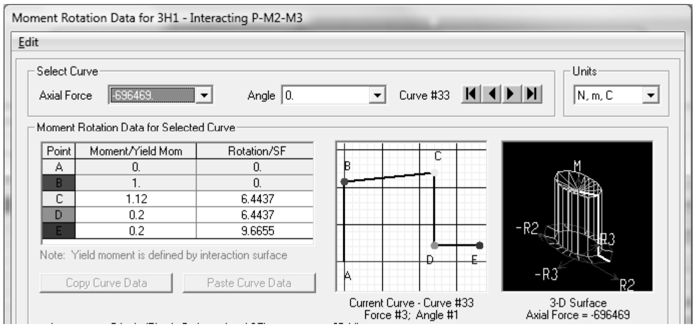
## Two-Storey Braced Frame – Stronger Chevron Beam

**Brace**





**Columns**



Moment Rotation Data for 3H1 - Interacting P-M2-M3

Select Curve
Axial Force: -696469.
Angle: 0.
Curve #33
Units: N, m, C

Moment Rotation Data for Selected Curve

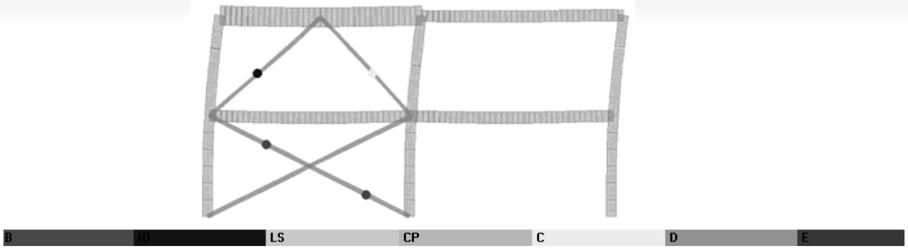
| Point | Moment/Yield Mom | Rotation/SF |
|---|---|---|
| A | 0. | 0. |
| B | 1. | 0. |
| C | 1.12 | 6.4437 |
| D | 0.2 | 6.4437 |
| E | 0.2 | 9.6655 |

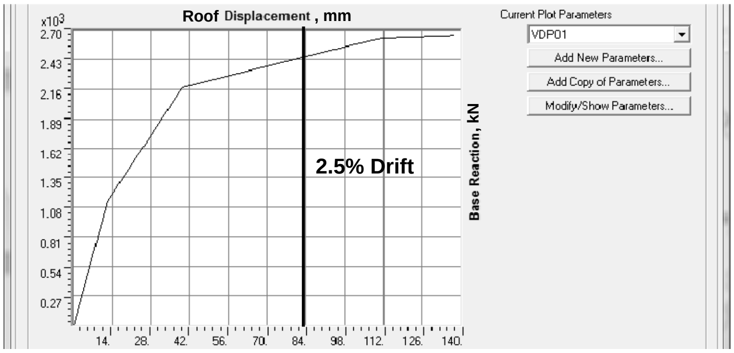
Note: Yield moment is defined by interaction surface

Current Curve - Curve #33
Force #3; Angle #1

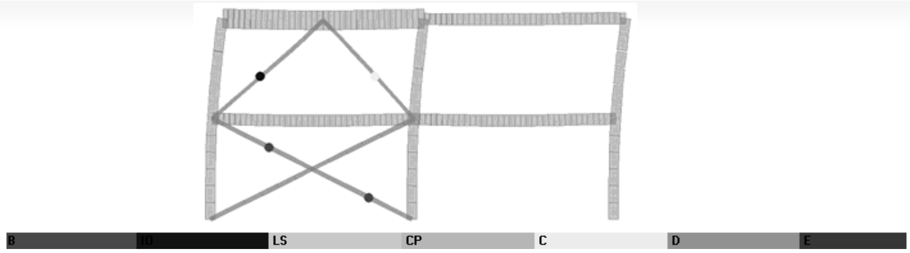
3-D Surface
Axial Force = -696469

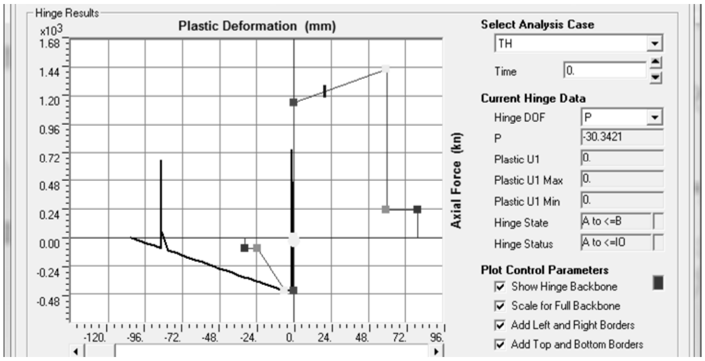
## Two-Storey Braced Frame – Load Displ. Plot





## Two-Storey Braced Frame – Stronger Chevron Beam





**Select Analysis Case**

TH

Time: 0.

**Current Hinge Data**

Hinge DOF: P

P: -30.3421

Plastic U1: 0.

Plastic U1 Max: 0.

Plastic U1 Min: 0.

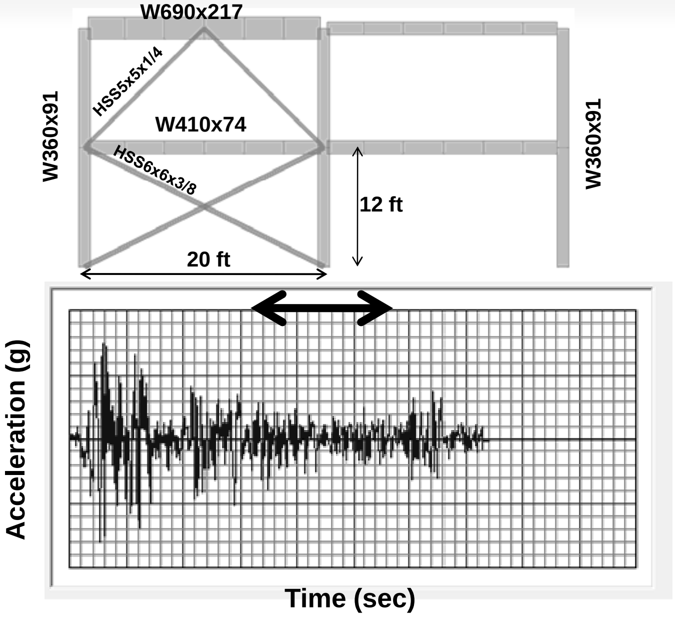
Hinge State: A to <=B

Hinge Status: A to <=IO

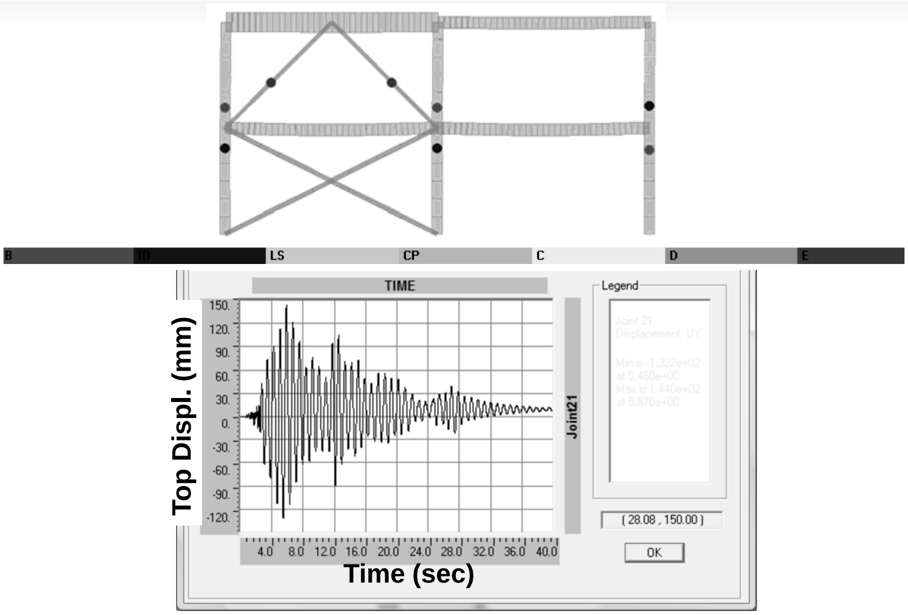
**Plot Control Parameters**

☑ Show Hinge Backbone
☑ Scale for Full Backbone
☑ Add Left and Right Borders
☑ Add Top and Bottom Borders

## Two-Storey Braced Frame – Stronger Chevron Beam



## Two-Storey Braced Frame – Stronger Chevron Beam



Joint 21
Displacement UY

Min is -1.322e+02
at 5.460e+00
Max is 1.440e+02
at 5.870e+00

( 28.08 , 150.00 )

## Silo Tank

| **Loads:** | **Weight (lb)** | **Centroid (ft)** |
|---|---|---|
| **Tank DL (Shell)** | **19173** | **18.12** |
| **Tank DL (Deck)** | **2372** | **36.24** |
| **Tank DL (Deck Equip.)** | **800** | **36.24** |
| **Tank DL (Hopper)** | **6101** | **7.97** |

**Use material bulk density of 30 lb/cu.ft**

| | | |
|---|---|---|
| **Product (Tank)** | **234,488** | **24.43** |
| **Product (Hopper)** | **67,912** | **7.97** |
| **Total Weight:** | **330,846 lb** | |
| | **1470.5 kN** | |

<u>Seismic Data (BCBC 1995):</u>

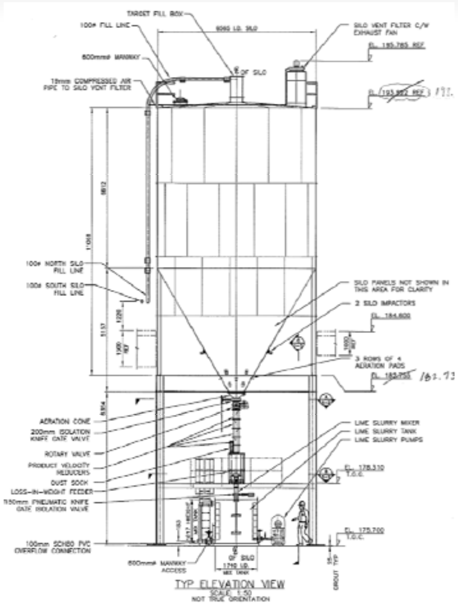
Za = 4

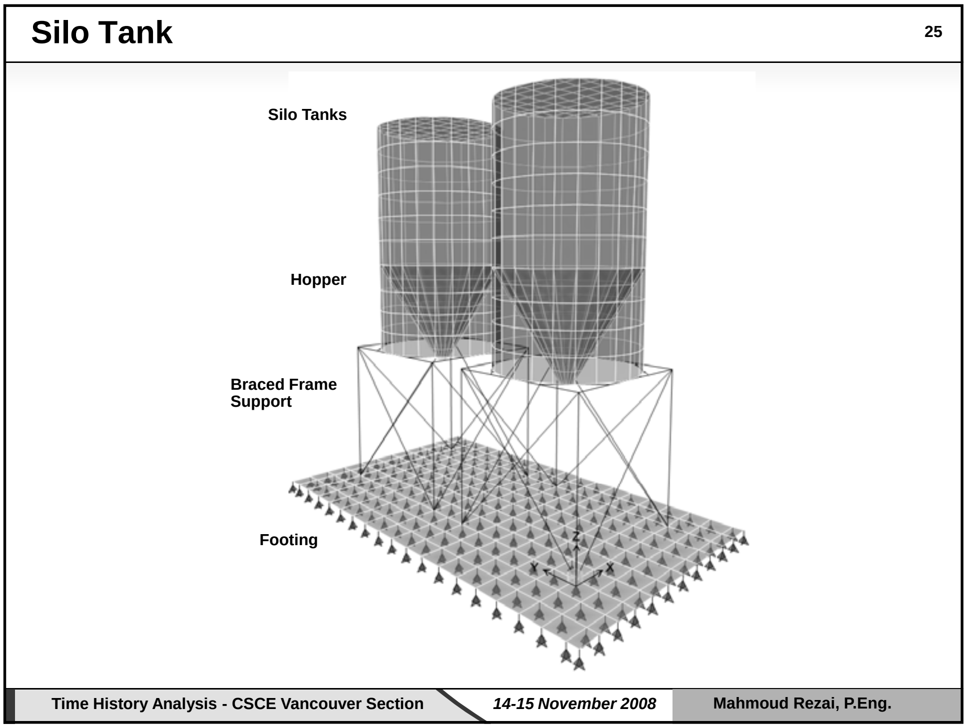
Zv = 4

v = 0.5

Assume:

R = 1





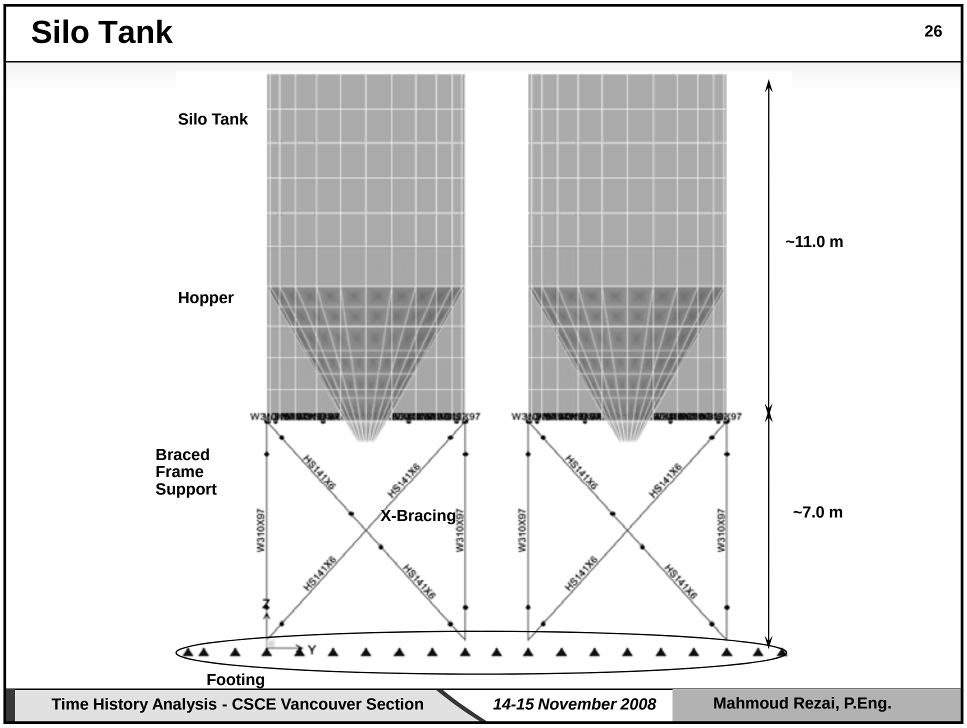
## Silo Tank

Silo Tanks

Hopper

Braced Frame Support

Footing



## Silo Tank

Silo Tank

Hopper

Braced Frame Support

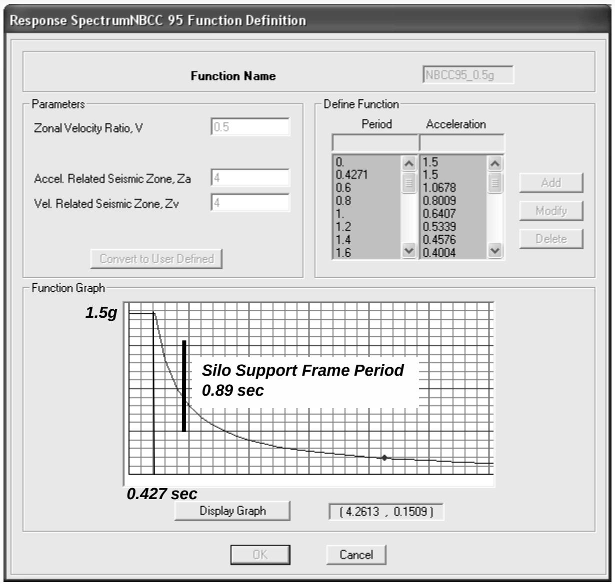
X-Bracing

Footing

~11.0 m

~7.0 m

## NBCC Dynamic Analysis



Response SpectrumNBCC 95 Function Definition

**Function Name** NBCC95_0.5g

Parameters

Zonal Velocity Ratio, V: 0.5

Accel. Related Seismic Zone, Za: 4

Vel. Related Seismic Zone, Zv: 4

Define Function

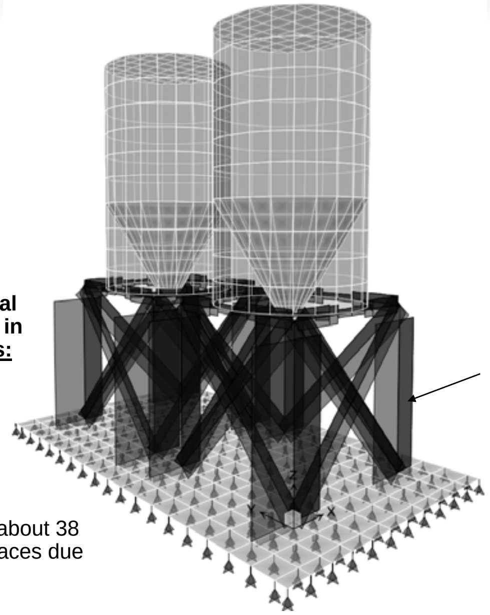
| Period | Acceleration |
|---|---|
| 0. | 1.5 |
| 0.4271 | 1.5 |
| 0.6 | 1.0678 |
| 0.8 | 0.8009 |
| 1. | 0.6407 |
| 1.2 | 0.5339 |
| 1.4 | 0.4576 |
| 1.6 | 0.4004 |

Function Graph

( 4.2613 , 0.1509 )

*Silo Support Frame Period 0.89 sec*

## Steps in Structural Analysis



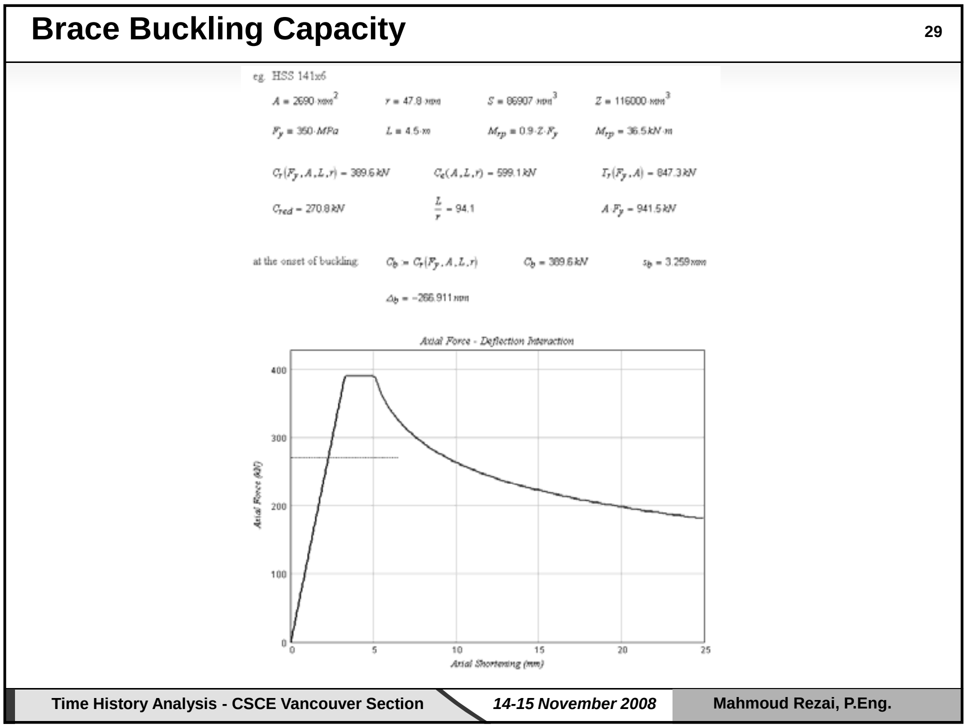
**Max. Axial Demand in <u>Columns:</u>**

**1021 kN**

**Max. Axial Demand in <u>Braces*:</u>**

**288 kN**

* Note that there is about 38 kN axial load in braces due to gravity.



## Brace Buckling Capacity

eg. HSS 141x6

$A = 2690 \cdot mm^2$ $r = 47.8 \cdot mm$ $S = 86907 \cdot mm^3$ $Z = 116000 \cdot mm^3$

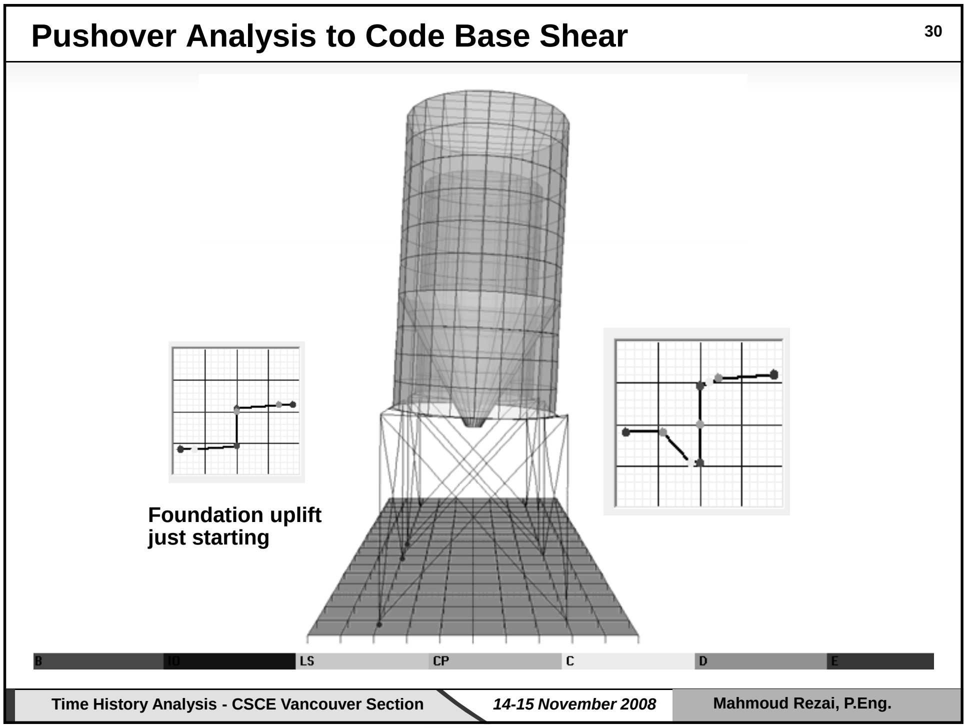
$F_y = 350 \cdot MPa$ $L = 4.5 \cdot m$ $M_{rp} = 0.9 \cdot Z \cdot F_y$ $M_{rp} = 36.5 kN \cdot m$

$C_r(F_y, A, L, r) = 389.6 kN$ $C_e(A, L, r) = 599.1 kN$ $T_r(F_y, A) = 847.3 kN$

$C_{red} = 270.8 kN$ $\frac{L}{r} = 94.1$ $A \cdot F_y = 941.5 kN$

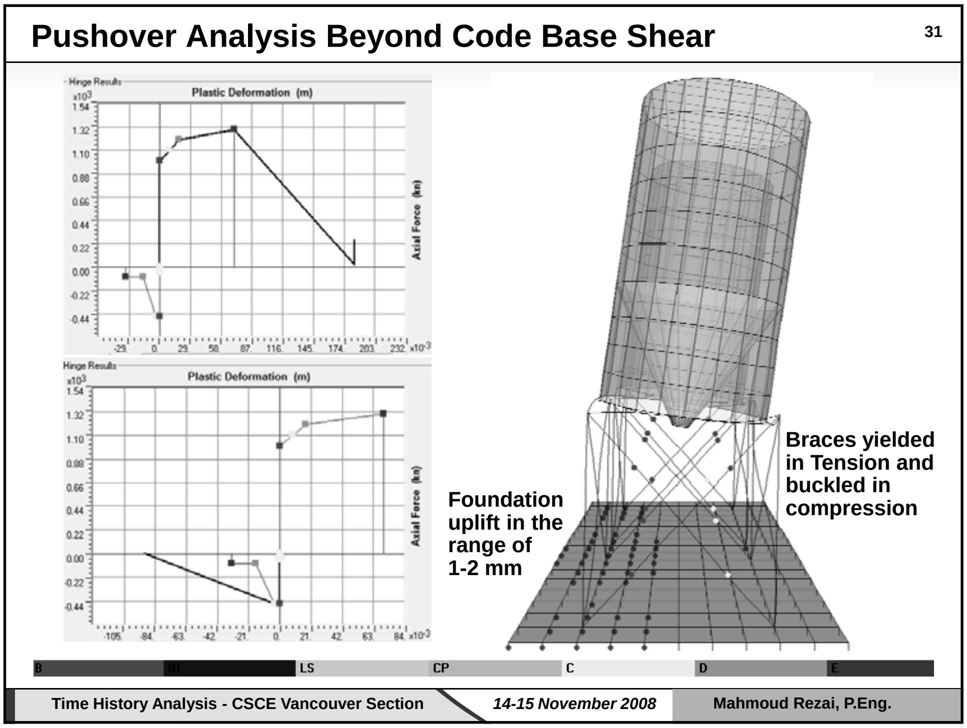
at the onset of buckling $C_b := C_r(F_y, A, L, r)$ $C_b = 389.6 kN$ $s_b = 3.259 mm$

$\Delta_b = -266.911 mm$



## Pushover Analysis to Code Base Shear

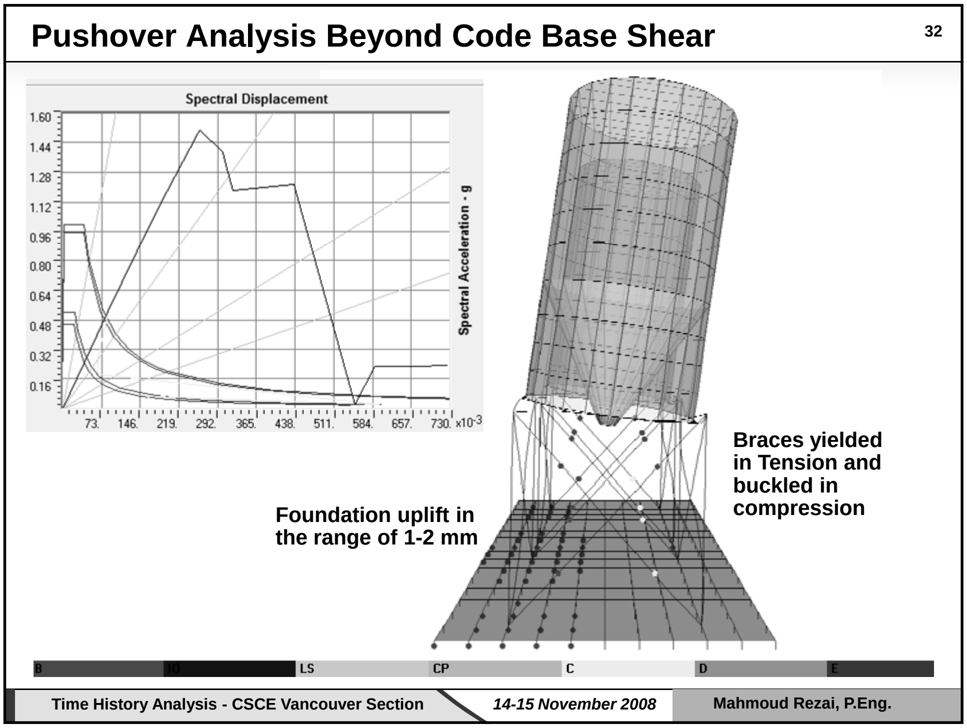
Foundation uplift just starting



## Pushover Analysis Beyond Code Base Shear

Braces yielded in Tension and buckled in compression

Foundation uplift in the range of 1-2 mm

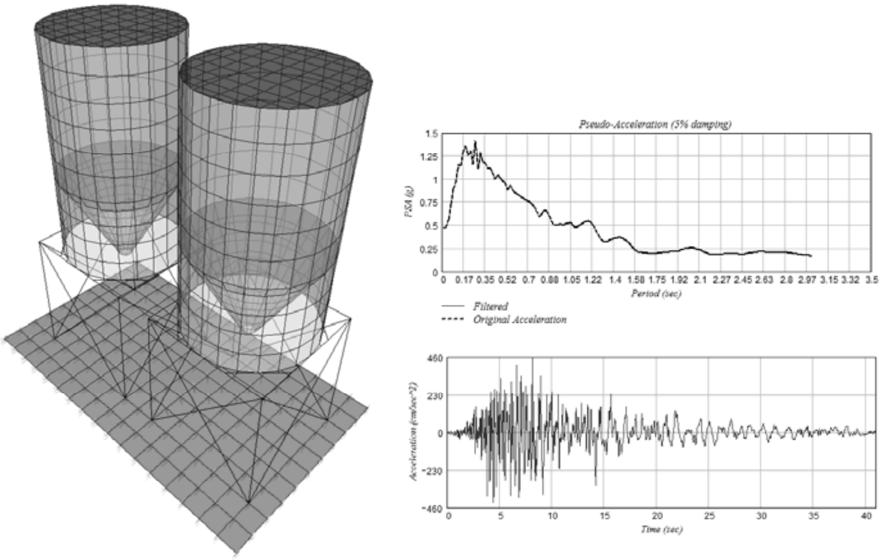


## Pushover Analysis Beyond Code Base Shear

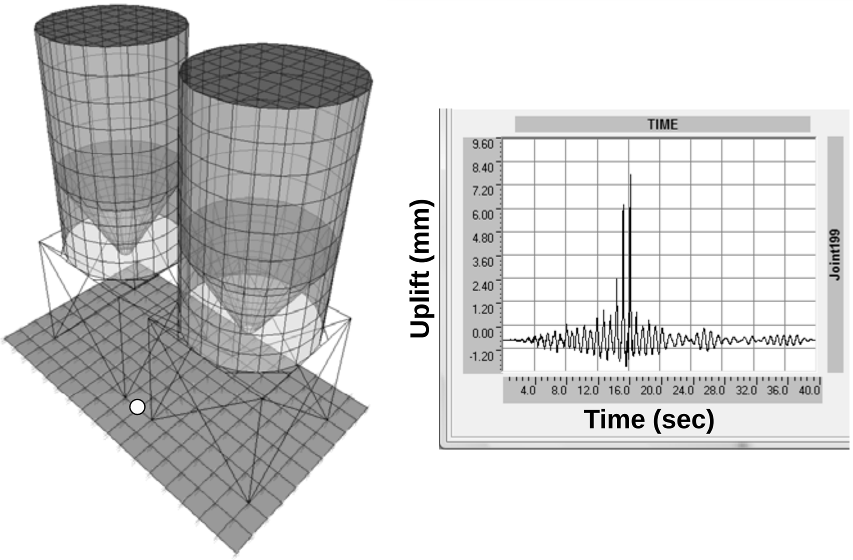
Braces yielded in Tension and buckled in compression

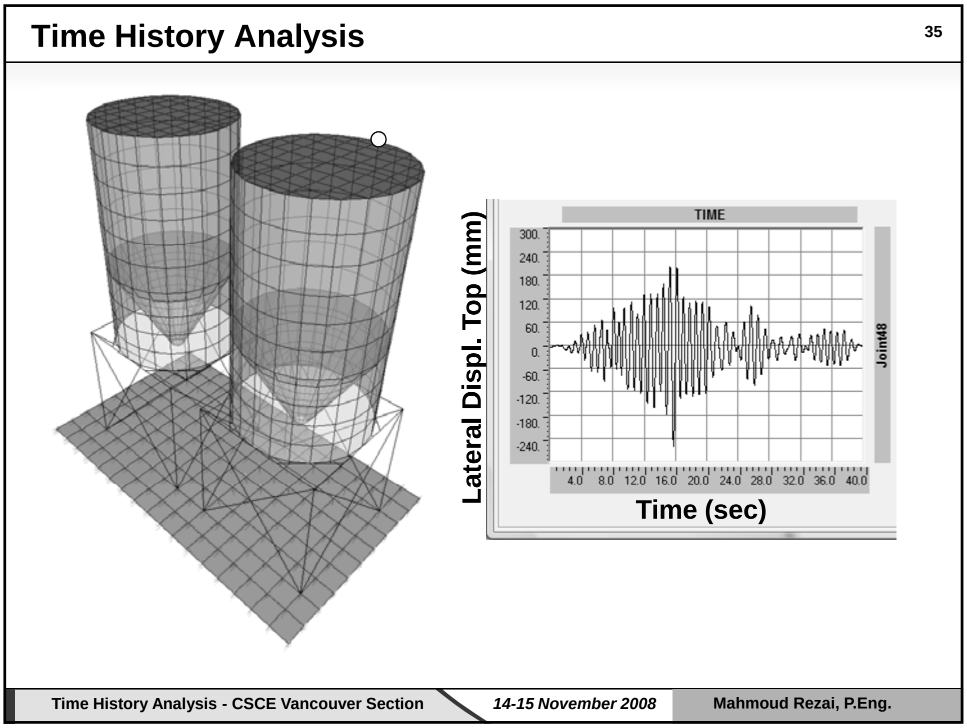
Foundation uplift in the range of 1-2 mm

## Time History Analysis

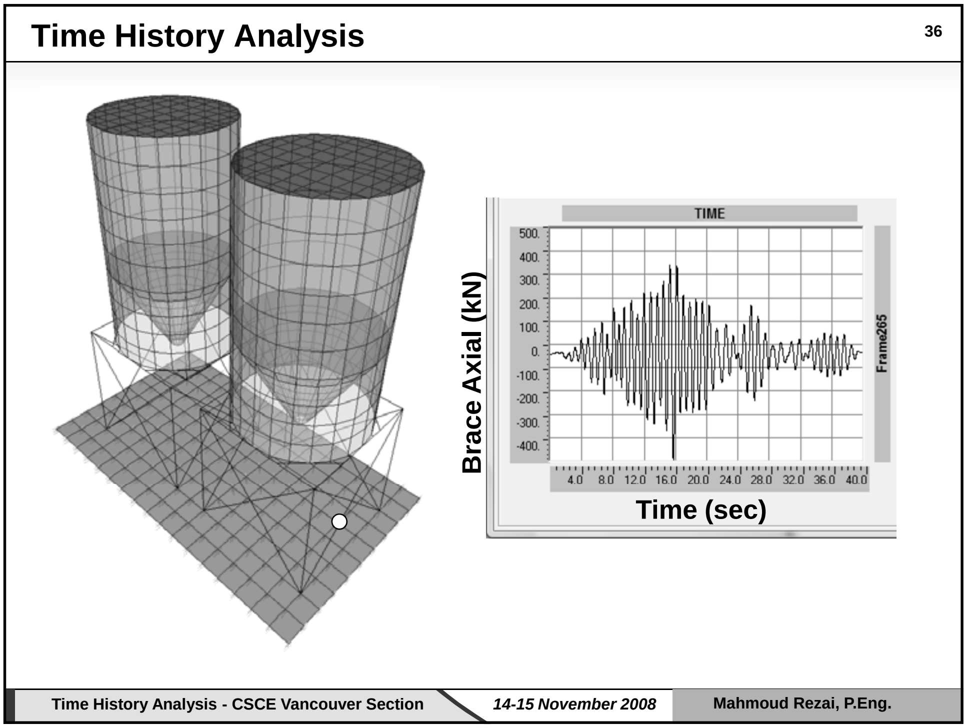


## Time History Analysis



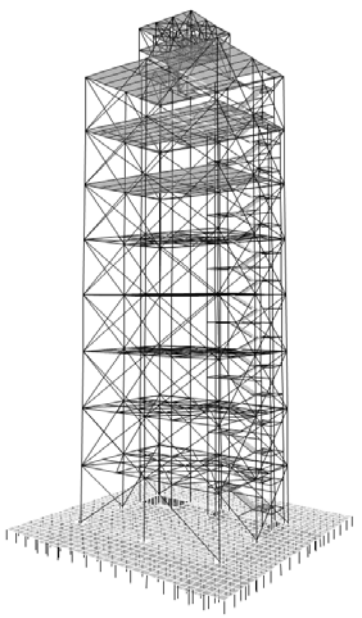


## Time History Analysis

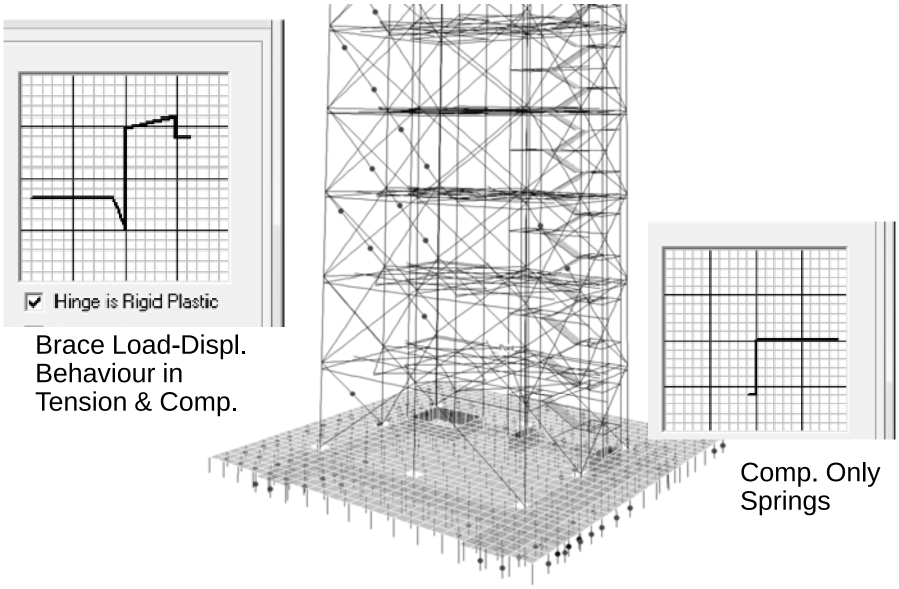


## Time History Analysis

## Steel Braced Tower on Raft Slab

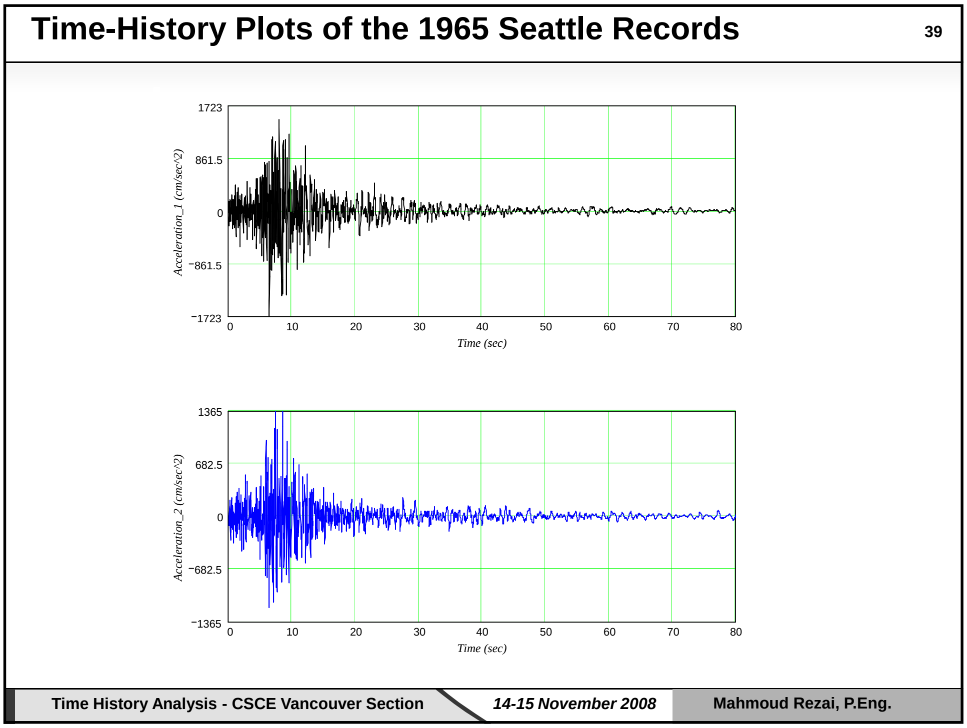


## Steel Braced Tower on Raft Slab

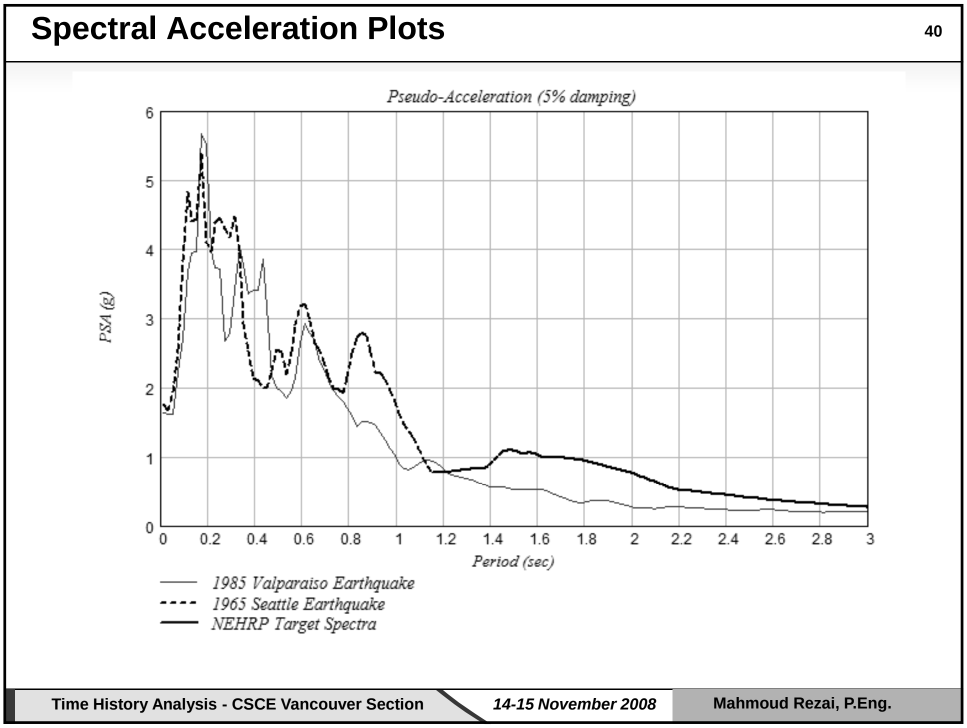


Brace Load-Displ.
Behaviour in
Tension & Comp.

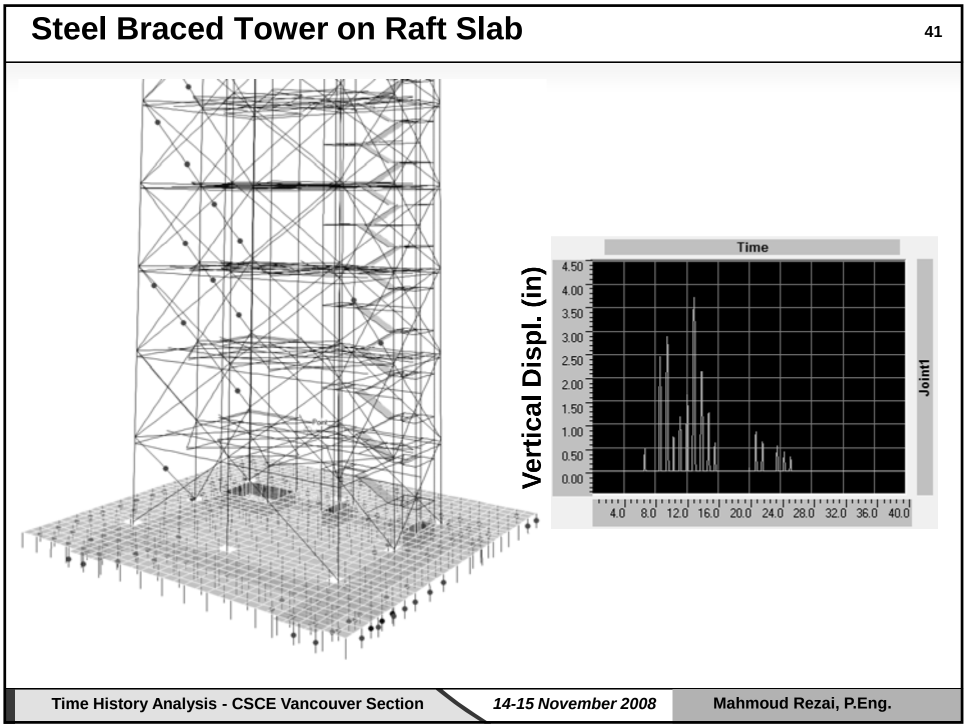
Comp. Only
Springs



## Time-History Plots of the 1965 Seattle Records

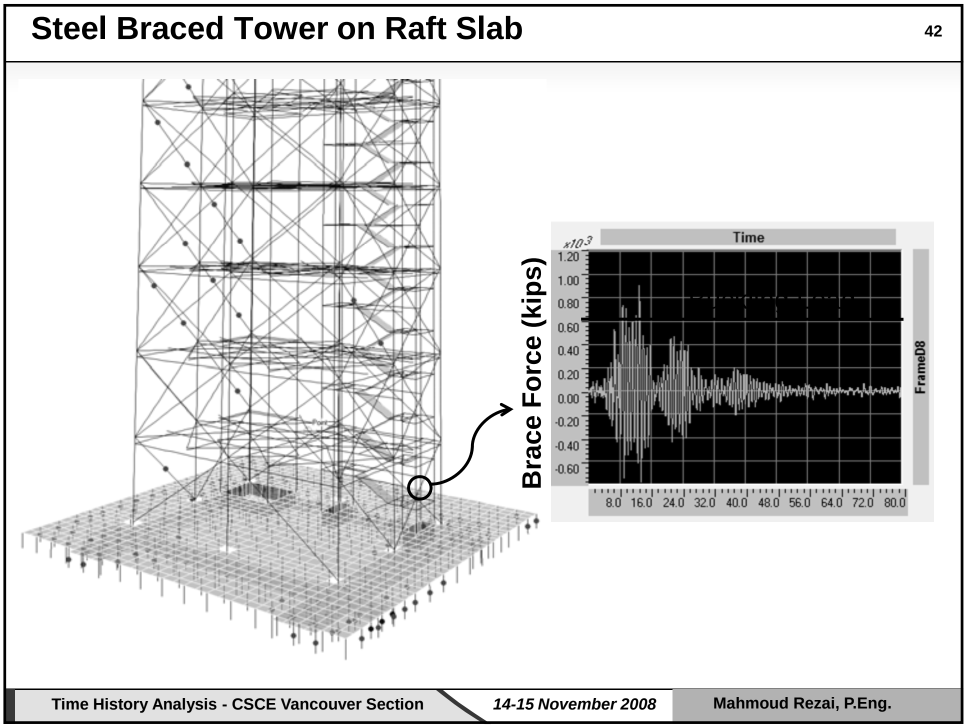


## Spectral Acceleration Plots



## Steel Braced Tower on Raft Slab

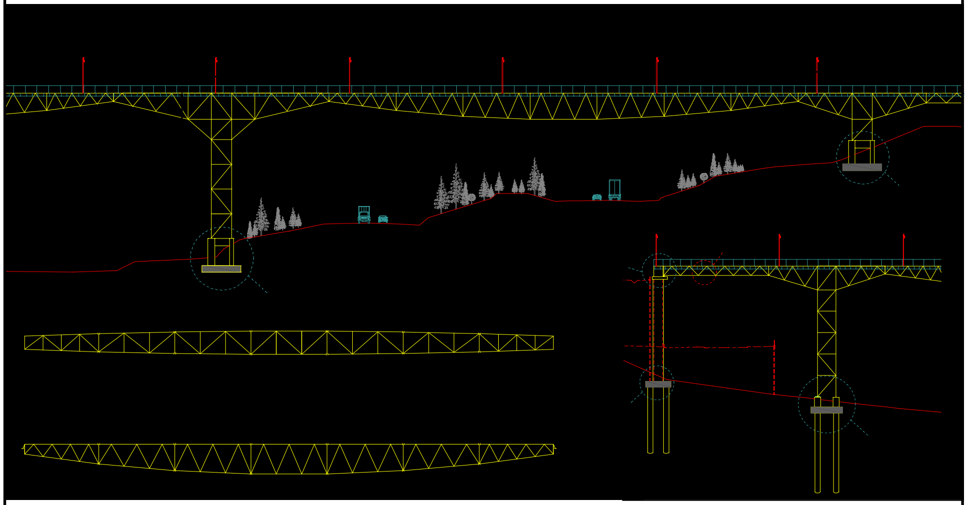
Time History Analysis - CSCE Vancouver Section | 14-15 November 2008 | Mahmoud Rezai, P.Eng.



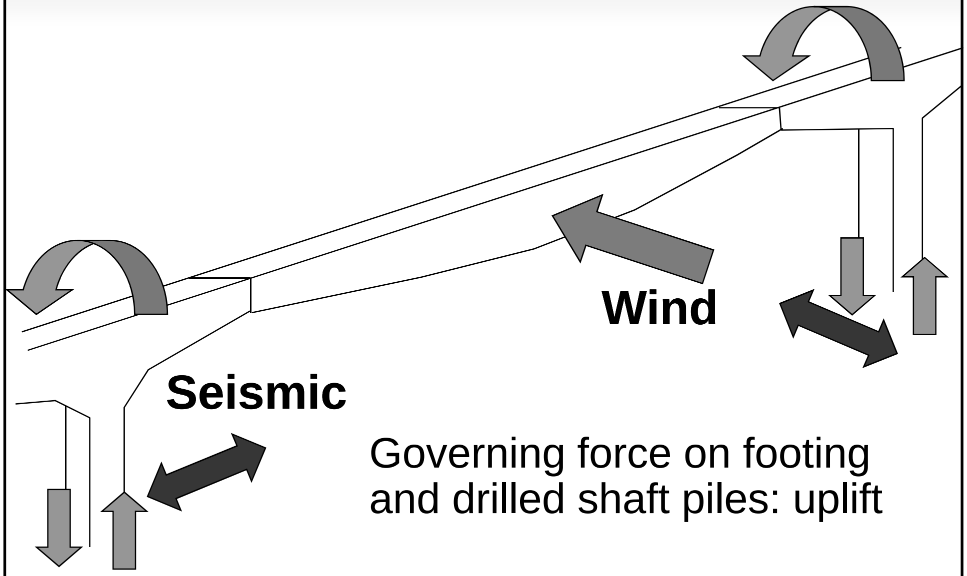
## Steel Braced Tower on Raft Slab

Time History Analysis - CSCE Vancouver Section | 14-15 November 2008 | Mahmoud Rezai, P.Eng.

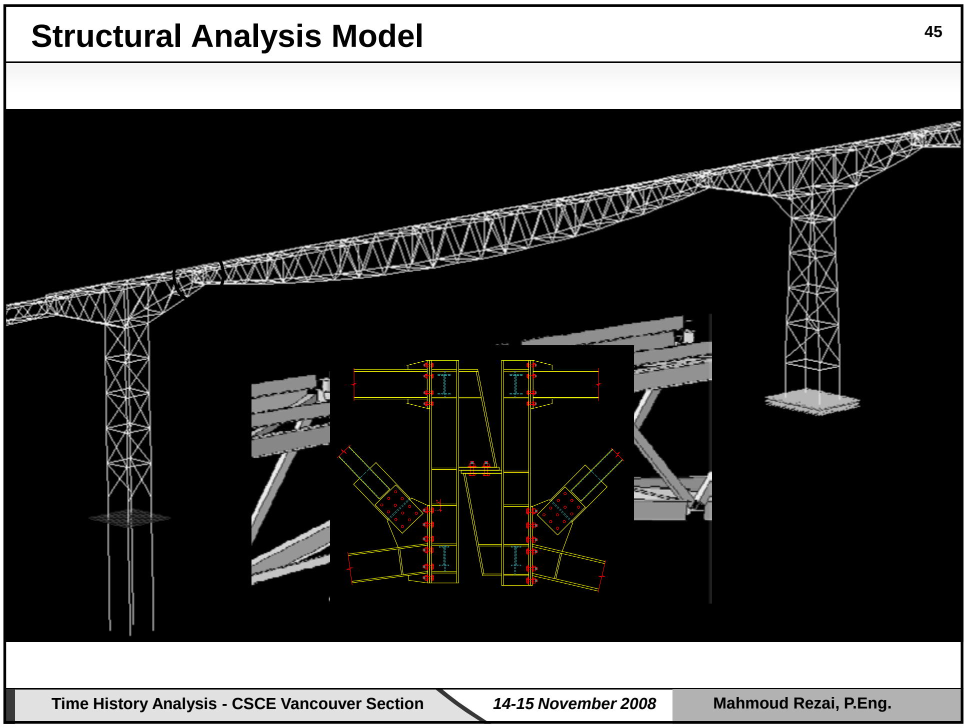
## Steel Truss Deck Bridge



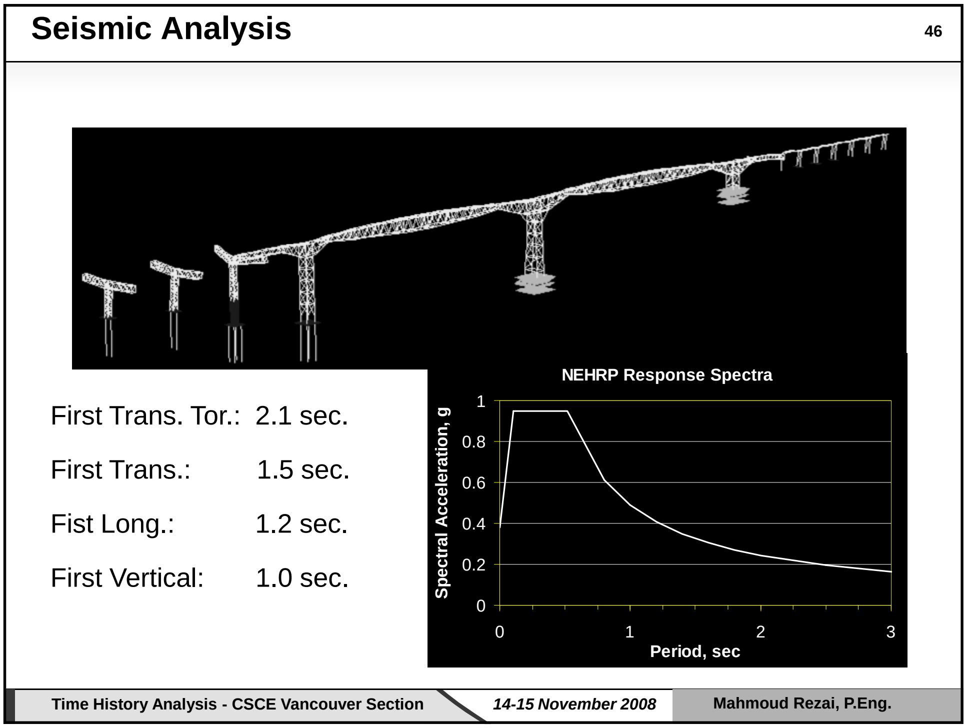
## Steel Truss Deck Bridge



Governing force on footing and drilled shaft piles: uplift



## Structural Analysis Model



## Seismic Analysis

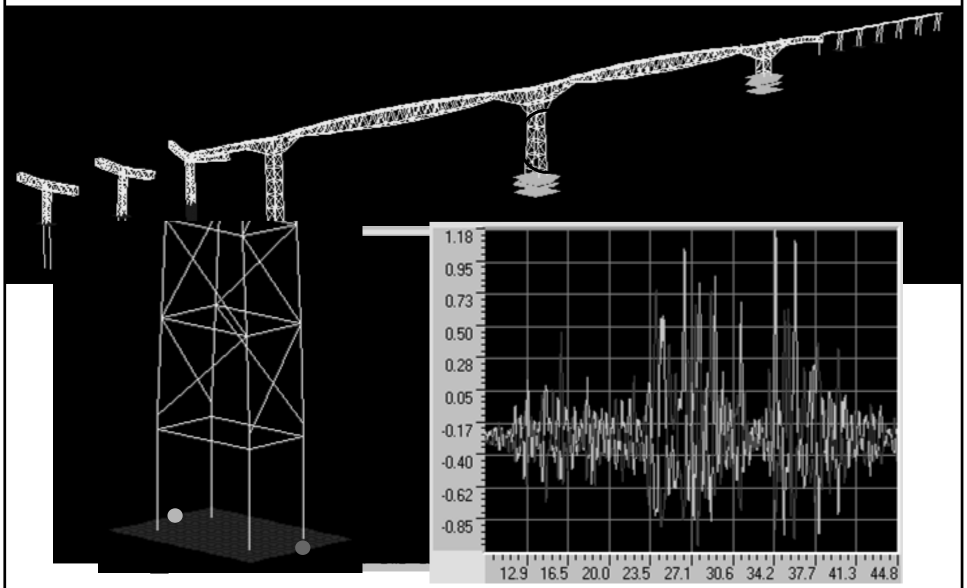
First Trans. Tor.: 2.1 sec.

First Trans.: 1.5 sec.

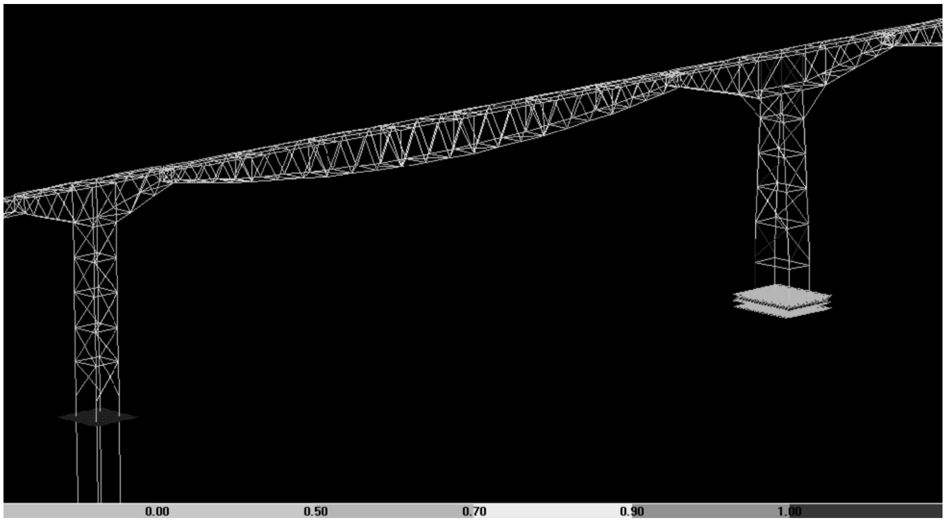
Fist Long.: 1.2 sec.

First Vertical: 1.0 sec.

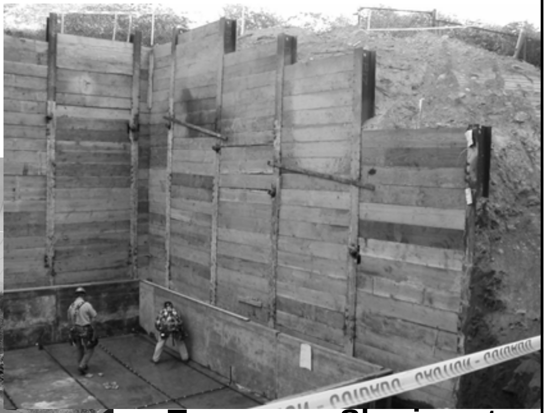
## Time History Analysis



## Seismic Demand/Capacity Ratios



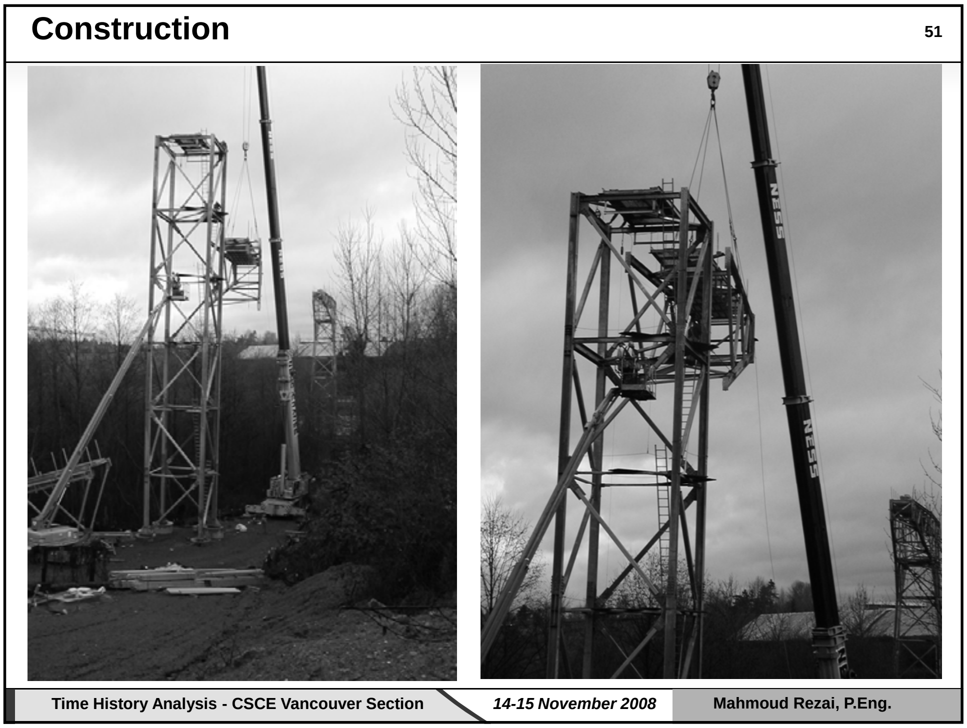
## Construction



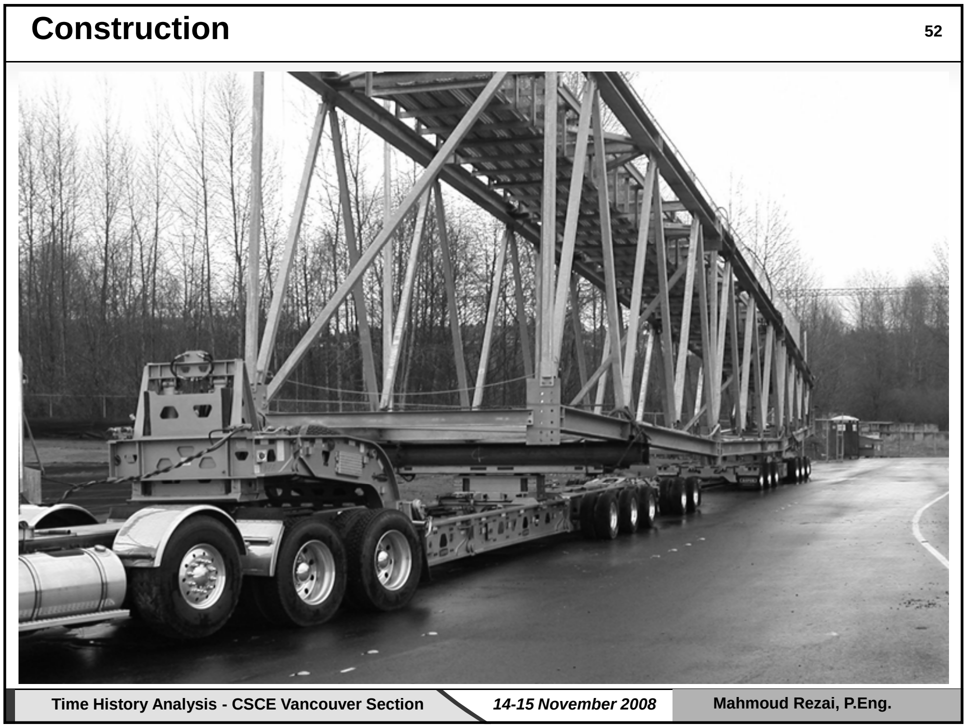
Temporary Shoring at Station 15

Congestion at Station 8

## Construction



## Construction



## Construction

## Construction

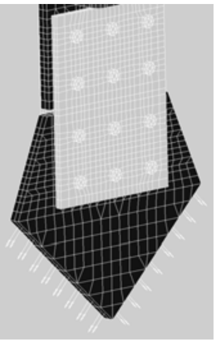
## Construction

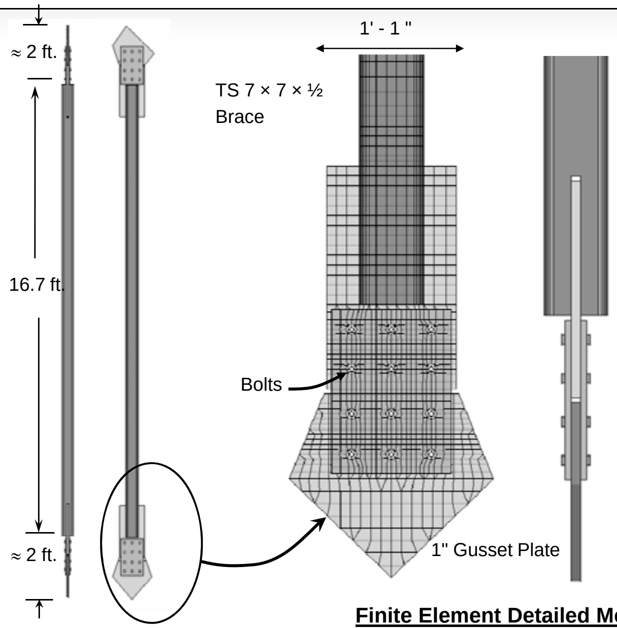
# Tensile and Post Buckling Analyses of a Concentrically Braced Frame Element

## <u>Using LS-DYNA</u>

## Finite Element Model of the Brace:

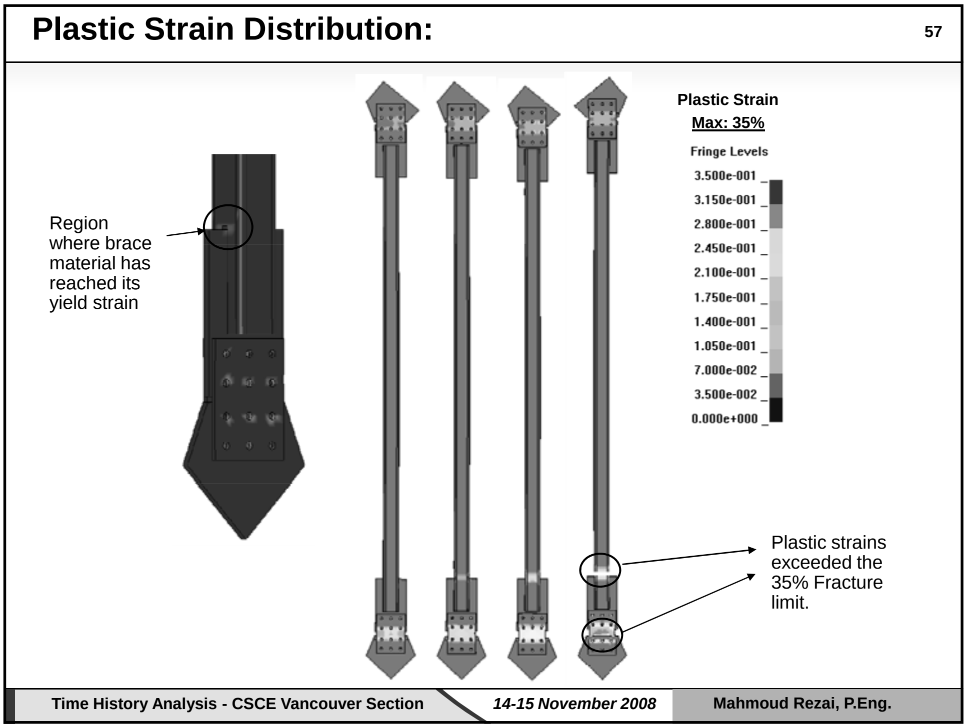
Static Loading, <u>Incremental</u>





<u>**Finite Element Detailed Mesh**</u>

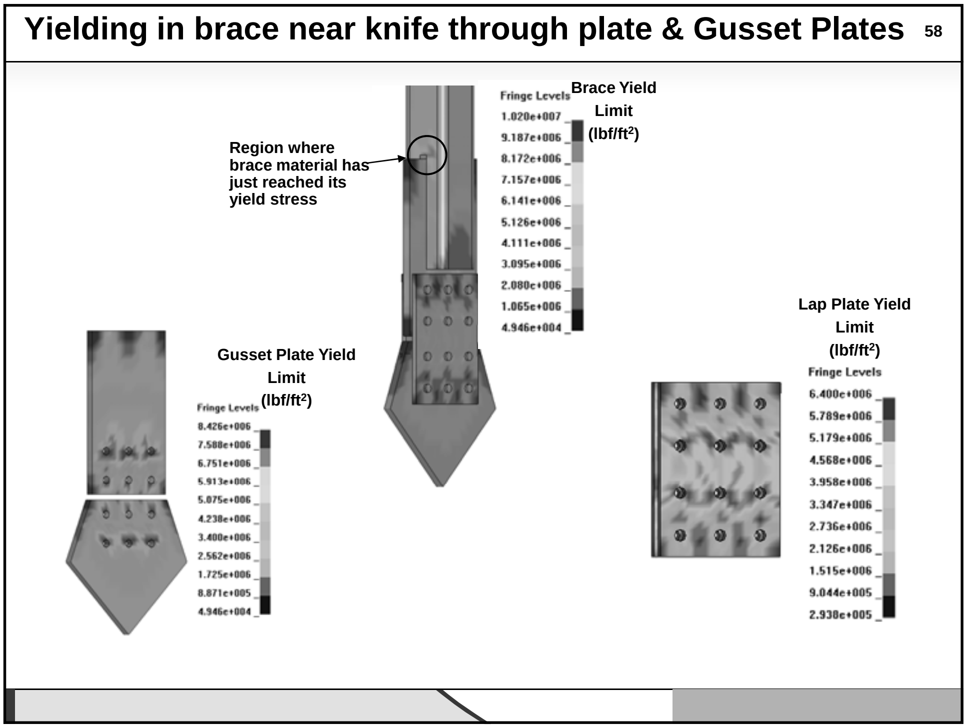
## Plastic Strain Distribution:



Region where brace material has reached its yield strain

Plastic strains exceeded the 35% Fracture limit.

## Yielding in brace near knife through plate & Gusset Plates

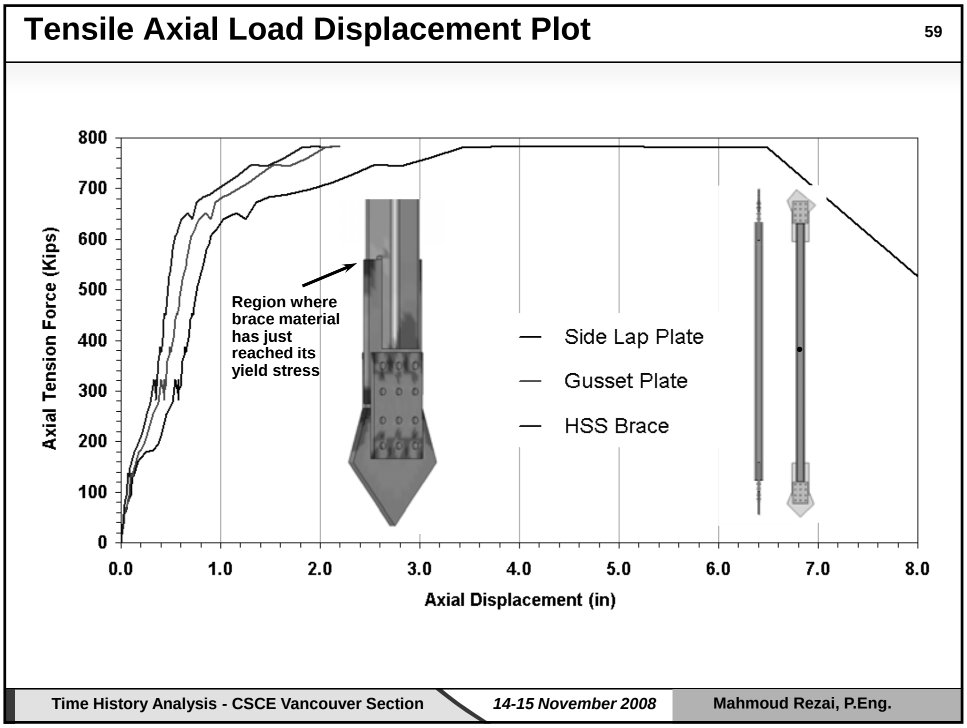


**Brace Yield Limit (lbf/ft$^2$)**

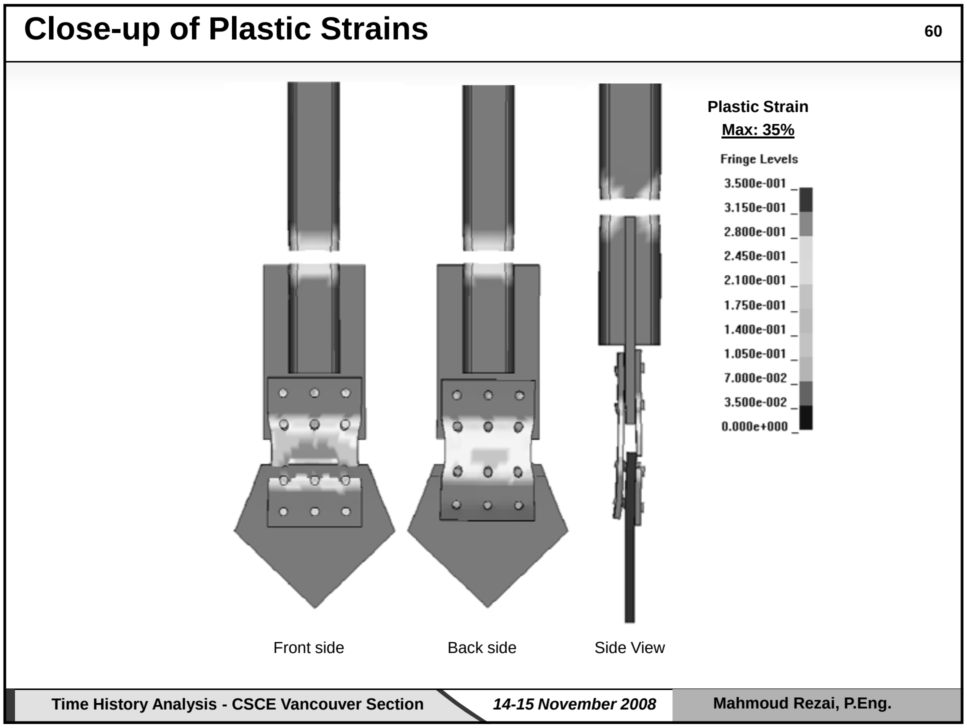
**Region where brace material has just reached its yield stress**

**Gusset Plate Yield Limit (lbf/ft$^2$)**

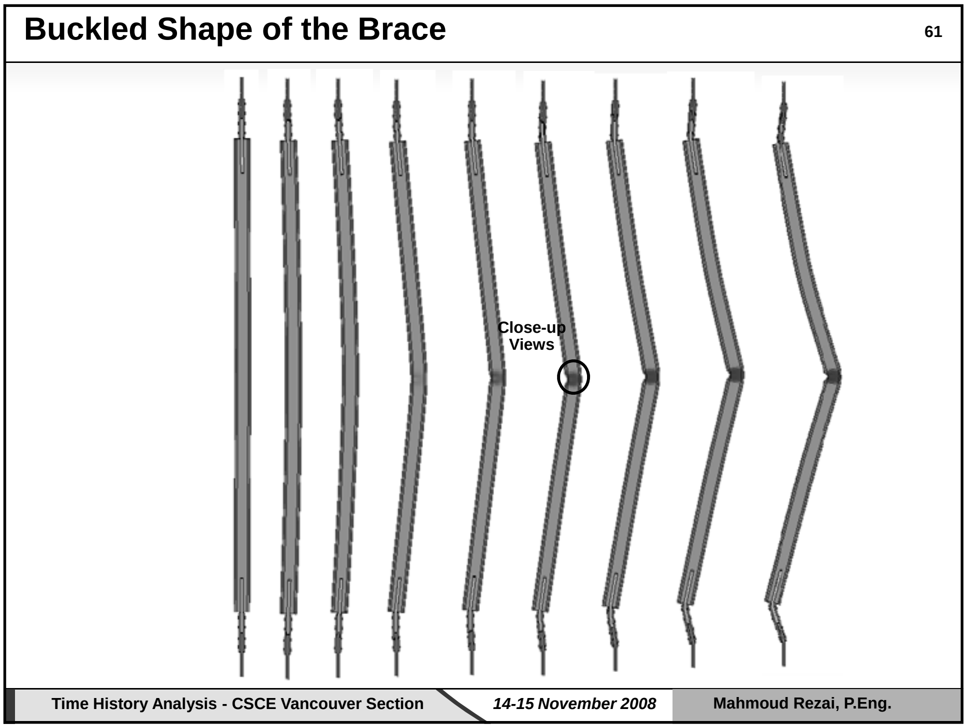
**Lap Plate Yield Limit (lbf/ft$^2$)**



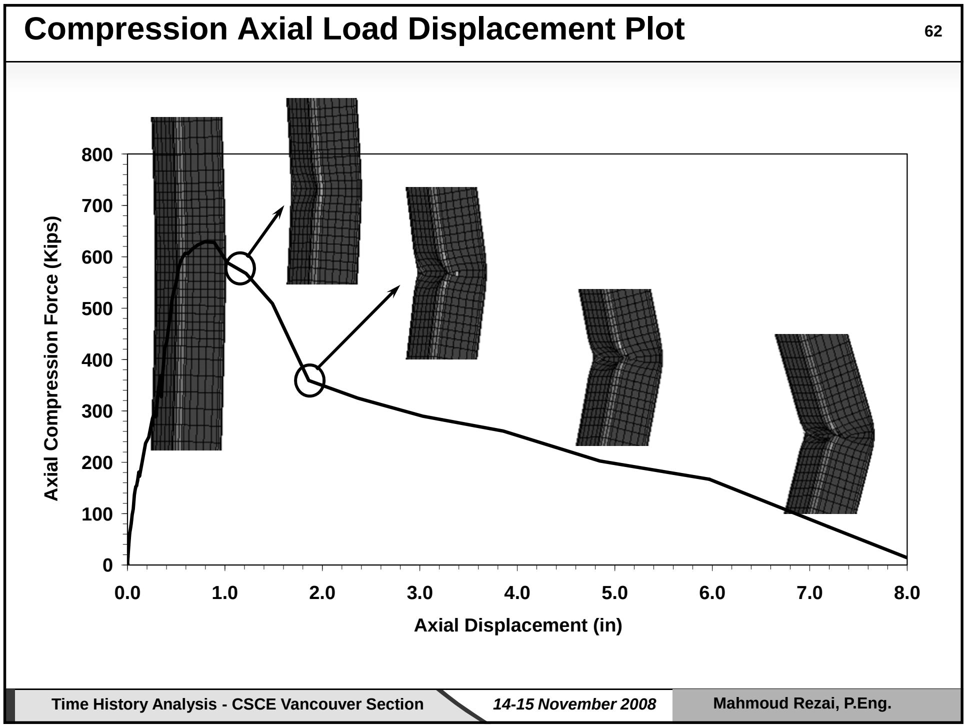
## Tensile Axial Load Displacement Plot



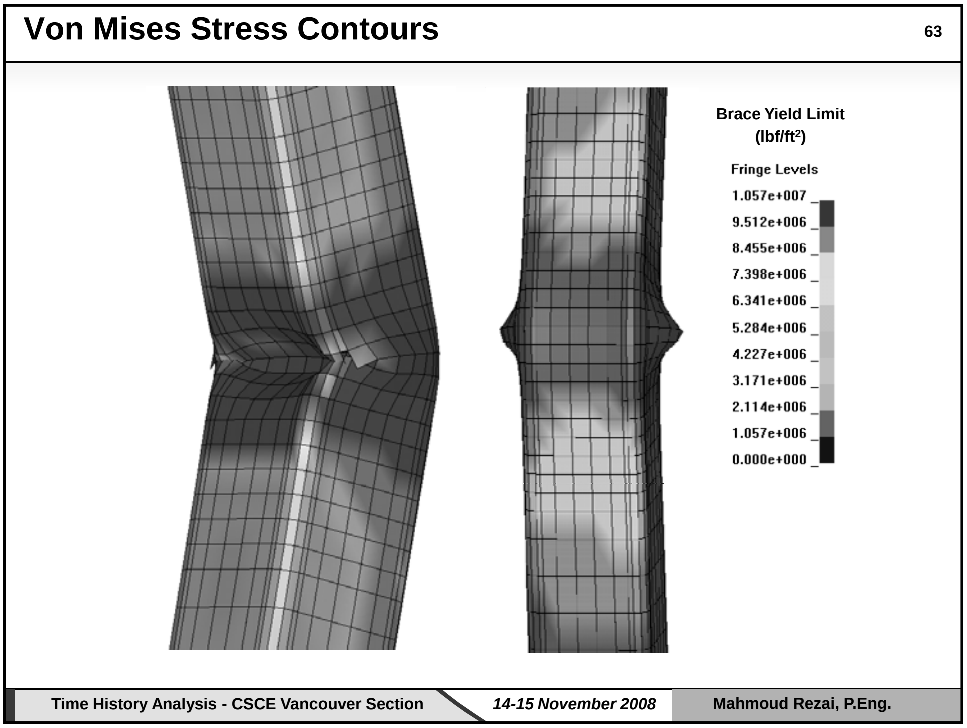
## Close-up of Plastic Strains



## Buckled Shape of the Brace



## Compression Axial Load Displacement Plot



## Von Mises Stress Contours

63

Brace Yield Limit
(lbf/ft$^2$)

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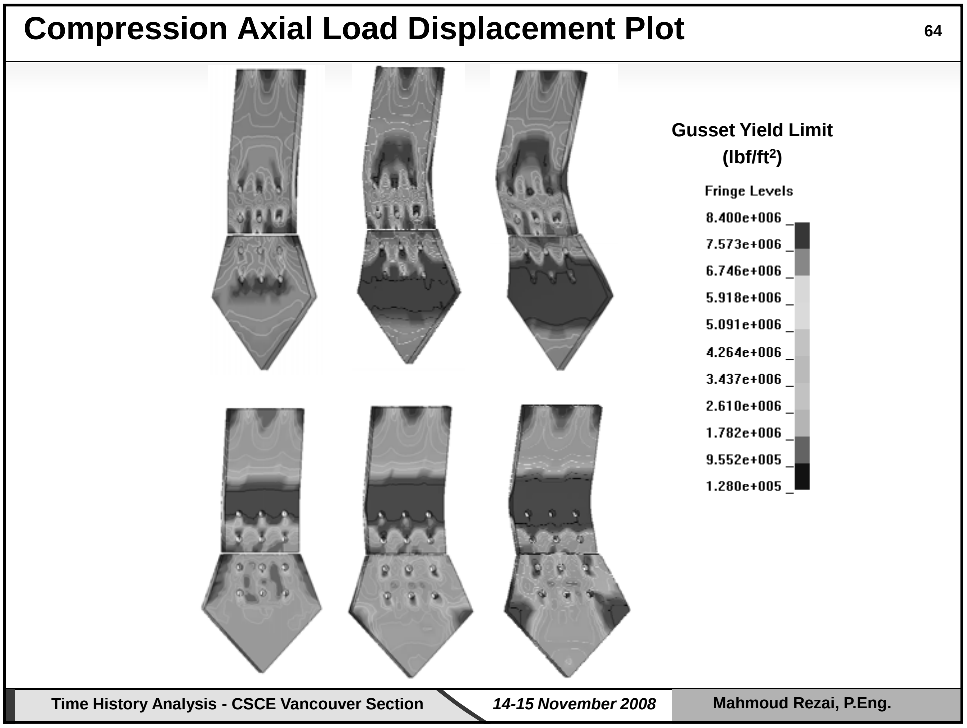

## Compression Axial Load Displacement Plot

64

Gusset Yield Limit
(lbf/ft$^2$)

Time History Analysis - CSCE Vancouver Section | *14-15 November 2008* | Mahmoud Rezai, P.Eng.

# Von Mises Stress Contours on each side of Lap Splice PL