CSRN
RCRP
Canadian Seismic Research Network
Réseau canadien pour la recherche parasismique

Funded by NSERC / Subventionné par le CRSNG

NEES

Joint CSRN –NEES Workshop on the Seismic Isolation and Damping of Bridge Structures

Irving K. Barber Learning Centre (IBLC) Room 182
1961 East Mall, University of British Columbia
Vancouver V6T 1Z1

Monday April 30, 2012

The Canadian Seismic Research Network (CSRN) and the U.S. Network for Earthquake Engineering Simulation (NEES) are pleased to invite you to a one-day co-sponsored Workshop on Seismic Isolation and Damping of Bridge Structures.

This Workshop is free but requires registration by email to rene.tinawi@polymtl.ca, CSRN Manager, before April 20, 2012. Space is limited.

Time	Speaker	Title
8:00		Coffee and welcome
8:30	Denis Mitchell*	Future directions of CSA S6 Code
8:50	Robert Tremblay*	Current and future designs – Base isolation
9:10	Constantin Christopoulos*	Numerical Studies for the Calibration of Design Methodologies for Damped and Isolated Bridges
9:30	Frédéric Légeron*	Aspects of retrofit design and testing of typical bridges
9:50		Break
10:20	Patrick Paultre*	Fragility curves with and without isolation
10:40	Carlos Ventura*	Seismic Instrumentation for bridges in Vancouver
11:00	Luc Chouinard*	Combining seismic and temperature deformations
11:20	Najib Bouaanani*	Performance-based assessment of isolated bridges
11:40		Sandwich Lunch
12:30	Michael Constantinou, SUNY, Buffalo	Unified LRFD-Based analysis and design procedures
13:00	Tim Delis, Caltrans	California applications and Caltrans design philosophy
13:30	Ian Aiken, SIE Inc.	Applications and performance of full-scale devices
14:00		Break
14:15	Steve Zhu, Buckland & Taylor	Examples of bridge isolation design
14:45	Don Kennedy, Associated Engineering	Examples of seismic bridge retrofit design
15:15	Sharlie Huffman, BC Ministry of Transportation	BC Ministry of transportation perspective
15:45		Discussions and wrap-up
16:30		End of Workshop

* CSRN Researcher

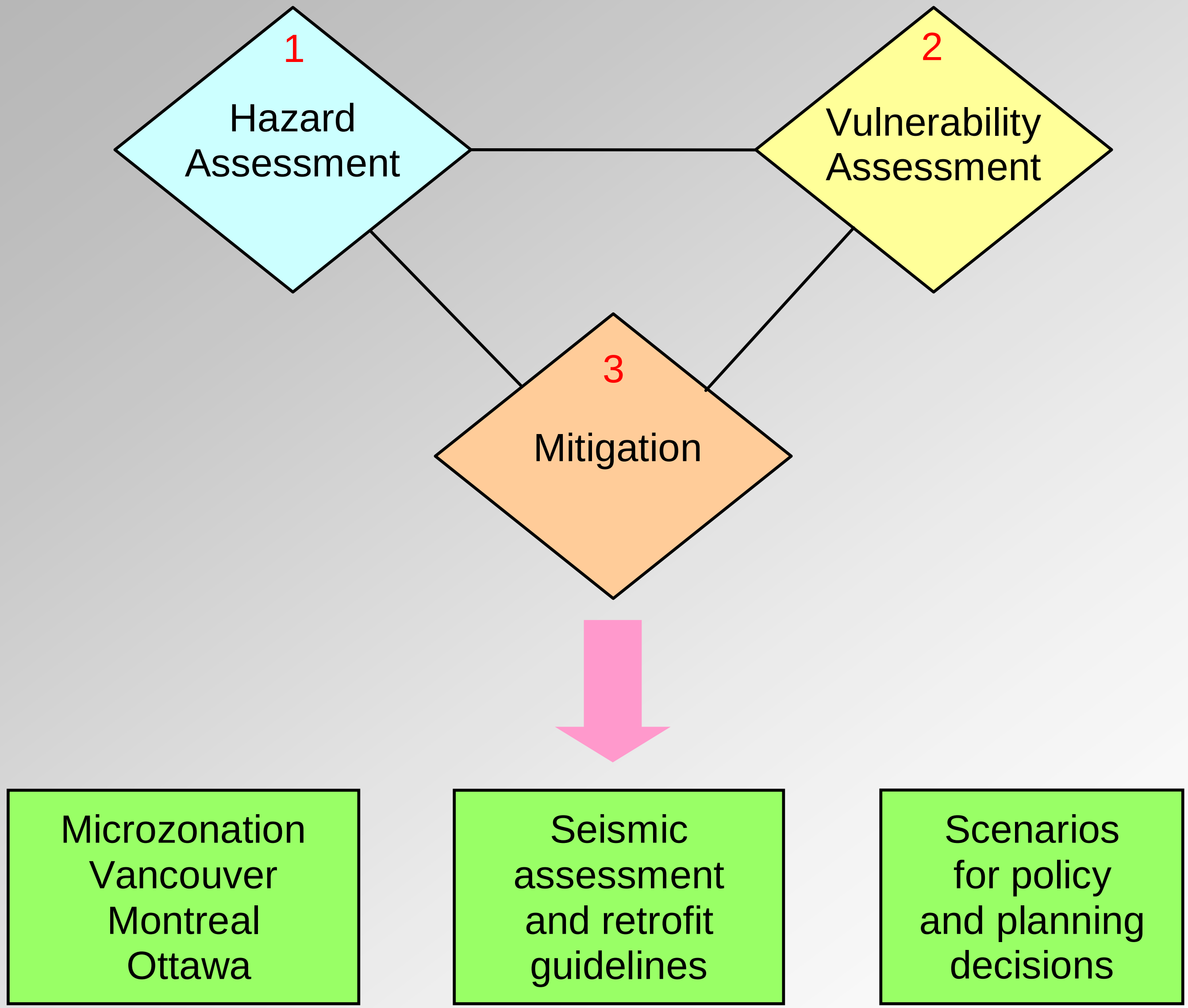
CANADIAN SEISMIC RESEARCH NETWORK (CSRN)

Network Goal:
Reduce Urban Seismic Risk

Five-Year Program

Funded by the Natural Sciences and Engineering Research Council

Research Themes and Deliverables



3 Themes and 16 Projects

- **Theme 1 – Hazard Assessment**
- **Theme 2 – Vulnerability Assessment**

  Project 2.6 Bridge Substructures

- **Theme 3 – Mitigation**

  Project 3.4 Seismic Upgrade with Base Isolators

The Researchers

26 researchers from 8 Universities

The UBC Team:

- Perry Adebar
- Stephanie Chang
- Ken Elwood
- Liam Finn
- Terje Haukaas
- Carlos Ventura

- Network Manager:

René Tinawi

McGill

ÉCOLE POLYTECHNIQUE

UNIVERSITÉ DE SHERBROOKE

Carleton UNIVERSITY

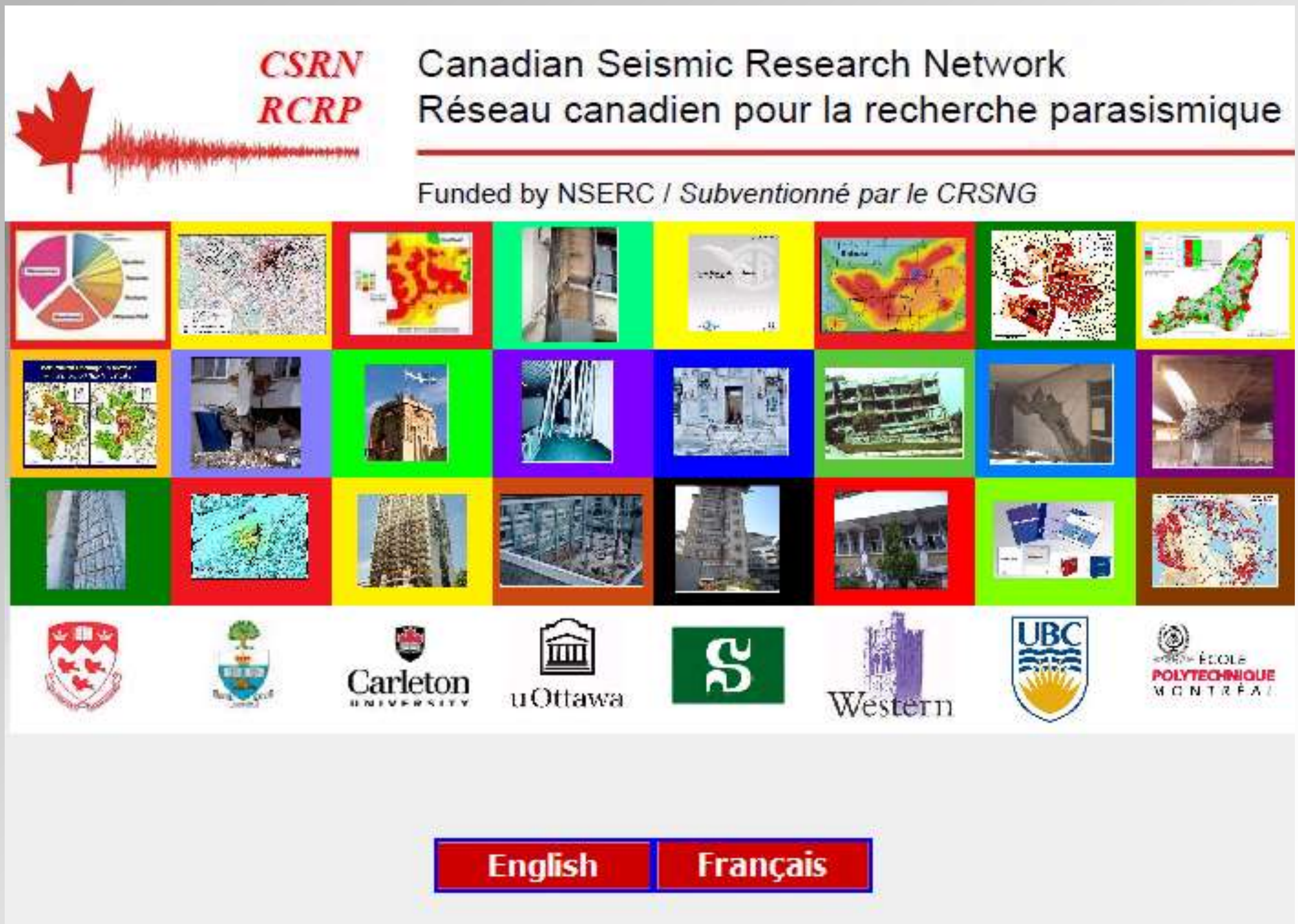
uOttawa

UNIVERSITY OF TORONTO

Western

UBC THE UNIVERSITY OF BRITISH COLUMBIA

The Network Web Site: www.CSRN.mcgill.ca



Future Directions of CSA S6 Code (CHBDC 2014)

Denis Mitchell
McGill University

Vancouver
April 30, 2012

CHBDC Seismic Subcommittee Members

Name		Affiliation
Denis Mitchell, Chair	QC	McGill University
Rafiq Hasan	ON	MTO
Nicolas Theodor	ON	MTO
Michel Bruneau	NY	University of Buffalo
Steve Zhu	BC	Buckland & Taylor
Robert Tremblay	QC	Ecole Polytechnique
Don Kennedy	BC	Associated Engineering
Upul Atukorala	BC	Golder Associates
John Adams	ON	Geological Survey of Can.
Patrick Paultre	QC	Univ. of Sherbrooke
Luc Chouinard	QC	McGill University
Carlos Ventura	BC	Univ. of British Columbia
Sharlie Huffman	BC	Ministry of Transp.

S6-06 Elastic Seismic Response Coefficient

$$C_{sm} = \frac{1.2AIS}{T_m^{2/3}} \leq 2.5AI$$

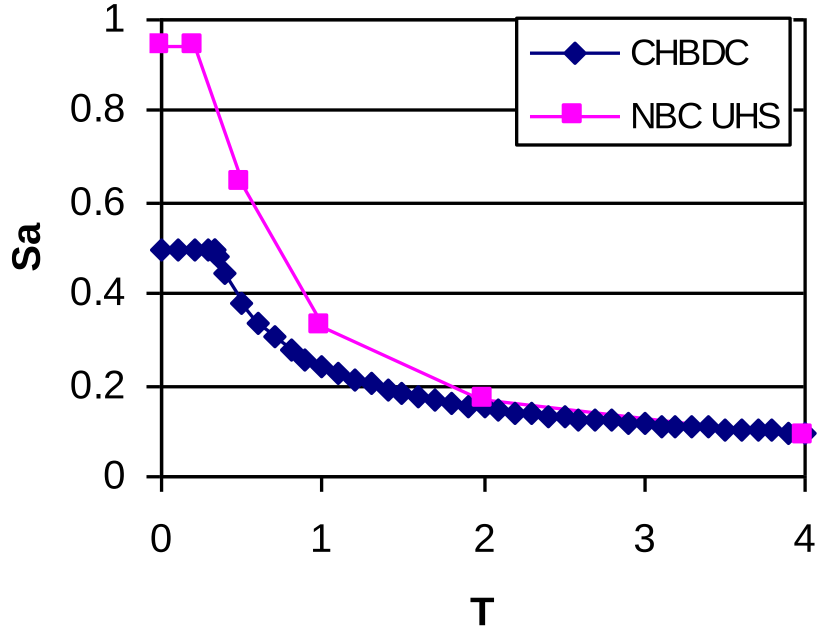
Based on AASHTO 1994

A = Zonal acceleration ratio for probability of exceedance of 10% in 50 years (475 year return period

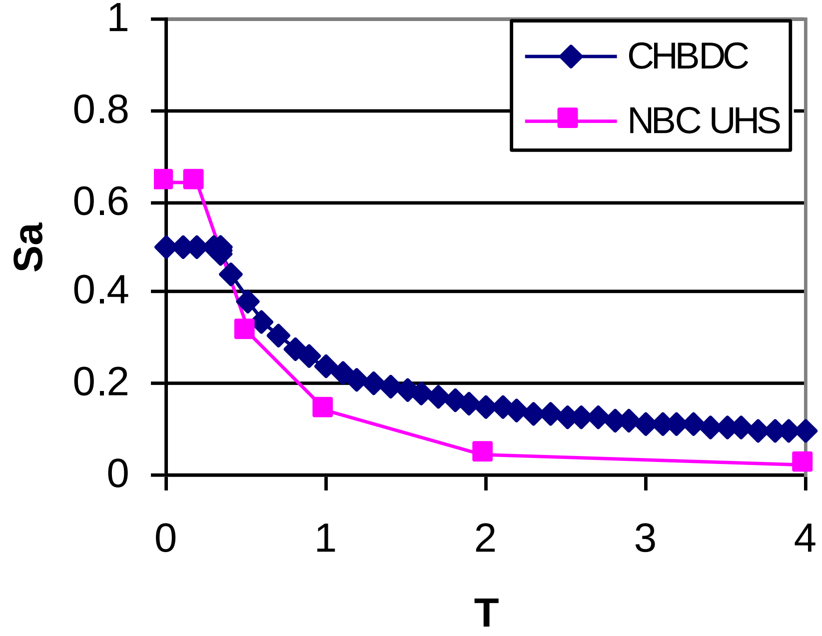
Return Period for Collapse Prevention

- 2475 return period (2% in 50 years probability of exceedance) would be used for design for collapse prevention
- Same return period as for buildings

Comparison of CHBDC (10% in 50 Years) Spectrum with 2010 NBCC UHS (2% in 50 Years)



Vancouver



Montréal

NBCC Updated UHS for 2015

- GSC developing 5th generation seismic hazard maps for SCED
  - 18 years more earthquakes
  - Probabilistic treatment of Cascadia
  - New Ground Motion relations
  - New spectral values (shorter and longer periods)
  - Adjusted reference ground condition

Seismic Hazard

10% in 50 years

- In 1985 NBC used PGA and PGV
- In 2000 CHBDC used PGA (AASHTO)

2% in 50 years

- In 2005/2010 NBC used
  - Sa at 0.2, 0.5, 1.0, 2.0 seconds
- For 2015 GSC may add 0.15, 5, 10 seconds

For bridges and long-period buildings

Importance Categories

- Lifeline bridges
- Major-route bridges
- Other bridges

- Classification includes social/survival, economic and security/defence requirements

Performance-Based Approach

- Use a Performance-Based approach with seismic design performance criteria

Seismic Ground motion Probability of Exceedance (return period)	Service Level [1]	Damage Level [2]
Lifeline Bridges		
2% in 50 years (2475 years)	Possible loss of service	Significant (No collapse)
5% in 50 years (975 years)	Limited	Repairable
10% in 50 years (475 years)	Immediate	Minimal
Major- Route Bridges		
2% in 50 years (2475 years)	Possible loss of service	Significant (No collapse)
10% in 50 years (475 years)	Limited	Repairable
Other Bridges		
2% in 50 years (2475 years)	Possible loss of service	No collapse

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- Be Prepared!
- Stations and Data
- General Information
- Products / Research

Resources
- Earthquake Search
- Hazard Calculator
- Station Book
- Waveform Data
- External Links

Site

Seismic design tools for engineers

Note The 2005 National Building Code of Canada is currently in force. The 2010 edition was released on November 30, 2010 and will be adopted by individual provinces and territories in the coming months. It is up to the designer to determine which version of the code is applicable in their jurisdiction.

Ground motion parameters for use with the National Building Code of Canada

2010 edition	2005 edition	1995 edition
Get 2010 hazard values	Get 2005 hazard values	Get 1995 hazard values
2010 National hazard maps	2005 National hazard maps	1995 National hazard maps
Open File 6761: 2010 model and values in preparation	Open File 4459: 2005 model and values	Open File 82-33: 1985/1995 model

Spectral Values

Site Coordinates:**43.64** °N **79.61**°W

User File Reference:

Requested by:,

National Building Code interpolated seismic hazard values

2%/50 years (0.000404 per annum) probability

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.234 g	**0.133 g**	**0.065 g**	**0.021 g**	**0.130 g**

Interpolated seismic hazard values at other probabilities

40%/50 years (0.01 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.028 g	0.016 g	0.008 g	0.003 g	0.009 g

10%/50 years (0.0021 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.088 g	0.049 g	0.027 g	0.009 g	0.040 g

5%/50 years (0.001 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.148 g	0.079 g	0.042 g	0.013 g	0.072 g

Different Design Approaches

- The force-based, R factor approach, will be used with optional design methods
- Time-history and push-over analysis permitted (required for lifeline bridges)
- Displacement-based approach will be described in the Commentary with a reference to the AASHTO Guidelines

Non-Linear Dynamic Analysis – Lifeline Bridges

- Minimum requirements would be specified, including scaling of records and the number of records
- Peer Review would be required to check the methodology

Shear Keys

- Shear keys can be designed to remain elastic at the design hazard level or can be designed to act as fuses, limiting the forces in the shear keys.

Shear Keys

- Compute the overstrength shear key capacity
- The overstrength shear capacity is used in assessing the loads applied to adjacent capacity-protected elements

Likely Changes

- New multi-hazard levels
- Seismic Performance Zones
- Performance-Based Design Approach
- Damage indicators for “service” limits
- Triggers for required type of analysis
- Revised modification factors for force-based approach
- Force-based approach for regular bridges only
- New section on Ductile Diaphragms
- Requirements for fill settlement for approach slabs
- Evaluation to follow the performance-based approach
- Guidance on soil-structure interaction
- Requirements for non-linear analysis

- Updating of section on Seismic Base Isolation/Energy Dissipation

CSA S6-14
Clause 4.10 Base Isolation

R. Tremblay

Workshop on the Seismic Isolation and Damping of Bridge Structures

April 30, 2012

CSRN Researchers – Bridge Isolation

Najib Bouaanani
École Polytechnique de Montréal

Luc Chouinard*
McGill University

Constantin Christopoulos (Project Leader)
University of Toronto

Frédéric Légeron
Université de Sherbrooke

Patrick Paultre*
Université de Sherbrooke

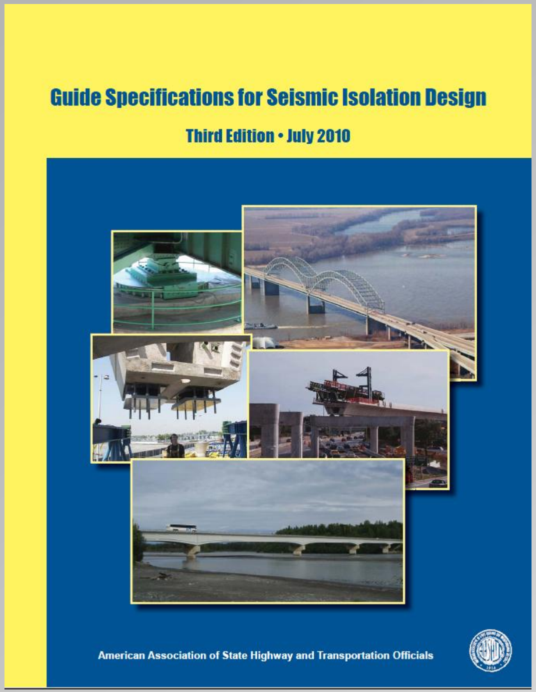
Robert Tremblay*
Ecole Polytechnique de Montréal

Carlos Ventura
University of British Columbia

**** Member, CSA-S6 Sub-Committee on Chapter 4)***

CLAUSE 4.10 – Seismic base isolation

1. General
2. Zonal acceleration ratios
3. Seimic performance zones
4. Site effects and site coefficients
5. Response modification factors & design requirements
6. Analysis procedures
7. Clearance and design displacements for seismic and other loads
8. Design forces for seismic performance Zones 1
9. Design forces for seismic performance Zones 2-4
10. Other requirements
11. Required tests of isolation system
12. Elastomeric bearings - Design
13. Elastomeric bearings- Construction
14. Sliding bearings - Design
15. Sliding bearings - Construction



1. Applicability
2. Definitions
3. Seimic hazard
4. Design respone spectrum
5. Seismic zones
6. Response modification factors
7. Analysis procedures
8. Design properties of isolation system
9. Clearance
10. Design forces for seismic zone 1
11. Design forces for seismic zones 2, 3 & 4
12. Other requirements
13. Required tests
14. Elastomeric bearings
15. Elastomeric bearings – construction
16. Sliding bearings
17. Sliding bearing – construction
18. Other isolation systems

4.10 Seismic base isolation

4.10.1 General

Clause 4.10 specifies requirements for isolator units and for the seismic isolation design of highway bridges.

Design requirements for isolation bearings are specified in Clauses 4.10.2 to 4.10.10. These requirements provide a revised design procedure for isolation bearings that allows for the possibility of large displacements resulting from the seismic response. General test requirements are specified in Clause 4.10.11. Requirements for elastomeric isolators are specified in Clauses 4.10.12 and 4.10.13. Additional requirements for sliding isolators are specified in Clauses 4.10.14 and 4.10.15. The requirements of Section 11 shall also apply.

Isolation systems without self-centring capabilities shall not be used.

- ***Scope: Seismic base isolation <u>and damping systems</u>***

  Note: STU = energy dissipating device restrainer?

- ***Minimum re-centering capability required?***

4.10 Seismic base isolation

4.10.1 General

Clause 4.10 specifies requirements for isolator units and for the seismic isolation design of highway bridges.

Design requirements for isolation bearings are specified in Clauses 4.10.2 to 4.10.10. These requirements provide a revised design procedure for isolation bearings that allows for the possibility of large displacements resulting from the seismic response. General test requirements are specified in Clause 4.10.11. Requirements for elastomeric isolators are specified in Clauses 4.10.12 and 4.10.13. Additional requirements for sliding isolators are specified in Clauses 4.10.14 and 4.10.15. The requirements of Section 11 shall also apply.

Isolation systems without self-centring capabilities shall not be used.

- ***Need for a separate section on added energy dissipation systems ?***
    - ***Added viscous dampers (include STU)***
    - ***Hysteretic dampers***
    - ***Friction dampers***

4.10.2 Zonal acceleration ratio

The zonal acceleration ratio, A, shall be as specified in Table 4.1 but not less than 0.1.

Note: *The zonal acceleration ratio specified in Table 4.1 for seismic isolation design is the same as that for conventional design.*

4.10.3 Seismic performance zones

The seismic performance zones, which delineate the method of analysis and the minimum design requirement, are the same as those for conventional design and are specified in Table 4.1.

4.10.4 Site effects and site coefficient

The site coefficient for seismic isolation design, S_i, which accounts for the site condition effects on the elastic response coefficient, shall be as specified in Table 4.7.

Table 4.7
Site coefficient for seismic isolation design, S_i
(See Clauses 4.10.4 and 4.10.6.2.1.)

Soil profile type (see Clauses 4.4.6.2 to 4.4.6.5)	Site coefficient for seismic isolation design, S_i
I	1.0
II	1.5
III	2.0
IV	2.7*

**Site-specific studies should be used for isolated bridges on Type IV soils.*

- ***Harmonization with Section 4***
- ***Use of different F_a & F_v values (longer T)***

4.10.3 Seismic performance zones

The seismic performance zones, which delineate the method of analysis and the minimum design requirement, are the same as those for conventional design and are specified in Table 4.1. Additional requirements of Clause 4.10.3.1 apply.

4.10.3.1. Damage level performance requirements

The damage level performance levels to 4.4.3 apply to the structures. The damage level performance requirements for the isolation system are:

(a) Minimal Damage: no damage during the design earthquake.

(b) Repairable Damage (no span or component collapse): minor damage may occur such as to non-structural protection of bearing. Vertical bearing capacity of the isolator is maintained. Displacement capacity is not exceeded. Permanent offsets of up to 1/3? seismic design displacement is permitted.

(c) Significant Damage (No Collapse): Damage does not cause collapse of any span or part of the structure. Bearing/isolation system may be damaged and re-instatement of the structure may require complete replacement of the bearing/isolation system.

Commentary:

The commentary should give examples for different systems

The device should be able to function after the earthquake and be able to resist aftershocks and may require only minor repair (no change of device necessary)

- *For isolator and dissipater, minimal damage means no damage since the bridge is fully elastic. There is no residual displacement in the isolator and dissipater. This means that for the system not behaving elastically, the non-elastic threshold is not exceeded.*
- *Examples of repairable damage are seismic fuse that may be broken, and minor damage to the anchor bolts. Displacement capacity is not exceeded and residual displacement may require some centering after an earthquake.*
- *Significant damage means that the bearing may have exceeded its displacement capacity, anchor bolt may be broken, but the bridge is not collapsed either partially or fully.*

4.10.5 Response modification factors and design requirements for substructure

Response modification factors, *R*, for all substructures shall be limited to 1.5, whereas substructures for lifeline and emergency-route bridges shall be designed to remain elastic (*R* = 1.0). For all isolated bridges, the design and detailing requirements for substructures in Seismic Performance Zones 2, 3, and 4 shall, at a minimum, be equivalent to the requirements for structures in Seismic Performance Zone 2.

- ***Essentially elastic response to activate isolation***
- ***Design forces based on maximum expected displacement ?***
- ***Forces when isolation and/or ED system have limited displacement capacity (see 4.10.7)***
- ***Use factored or probable resistance?***
- ***Need for ductility requirements:***
  - ***Lower R factor + relaxed ductility requirements (useful for existing structures); or***
  - ***Higher R factor + minimum ductility requirements (to accommodate uncertainty in demand)***

4.10.6 Analysis procedures

4.10.6.1 General

Table 4.2 shall be used to determine the applicable analysis procedure. The application of the applicable analysis procedure to isolated bridges shall be as specified in Clause 4.10.6.2 or 4.10.6.3. However, for isolation systems where the effective damping (expressed as a percentage of critical damping) exceeds 30%, a three-dimensional non-linear time-history analysis shall be performed using the hysteresis curves of the isolation system unless the value of *B* in Clause 4.10.6.2.1 is limited to 1.7.

- ***Harmonization with non-isolated bridges***
- ***Need for separate table for analysis methods?***
- ***Minimum % of column mass in model (SDOF method with B no longer applies)***
- ***NLTH required if no re-centering, in all cases?***
- ***Criteria for 1D vs 2D vs 3D analysis (vertical accelerations)***

4.10.6 Analysis procedures

4.10.6.1 General

Table 4.2 shall be used to determine the applicable analysis procedure. The application of the applicable analysis procedure to isolated bridges shall be as specified in Clause 4.10.6.2 or 4.10.6.3. However, for isolation systems where the effective damping (expressed as a percentage of critical damping) exceeds 30%, a three-dimensional non-linear time-history analysis shall be performed using the hysteresis curves of the isolation system unless the value of B in Clause 4.10.6.2.1 is limited to 1.7.

4.10.6.2 Uniform-load/single-mode spectral analysis

Note: *See Clauses 4.5.3.1 and 4.5.3.2 for the uniform-load and single-mode spectral methods.*

4.10.6.2.1 Statically equivalent seismic force and coefficient

Except for the case where a soil profile for the bridge site is Type IV, the statically equivalent seismic force, F, shall be

$$F = C'_{sm} W$$

where

C'_{sm} = elastic seismic response coefficient for isolated structures

$$= \frac{AS_i}{BT_e} \leq 2.5\frac{A}{B}$$

W = dead load of the superstructure segment supported by isolation bearings

The displacement, d_i, across the isolation bearings (in millimetres) shall be

$$d_i = \frac{250AS_iT_e}{B}$$

where

A = zonal acceleration ratio from Table 4.1

B = damping coefficient from Table 4.8 for the direction under consideration

S_i = dimensionless site coefficient for isolation design for the given soil profile, as specified in Table 4.7

T_e = period of vibration, s

$$= 2\pi\sqrt{\frac{W}{\Sigma k_{eff}\, g}}$$

where

Σk_{eff} = sum of the effective linear stiffnesses of all bearings and substructures supporting the superstructure segment, calculated at displacement d_i

g = acceleration due to gravity

Table 4.8
Damping coefficient, *B*
(See Clauses 4.10.6.2.1 and 4.10.11.2.)

Equivalent viscous damping, β (% of critical)	Damping coefficient, B
≤2	0.8
5	1
10	1.2
20	1.5
30	1.7
40	1.9
50	2

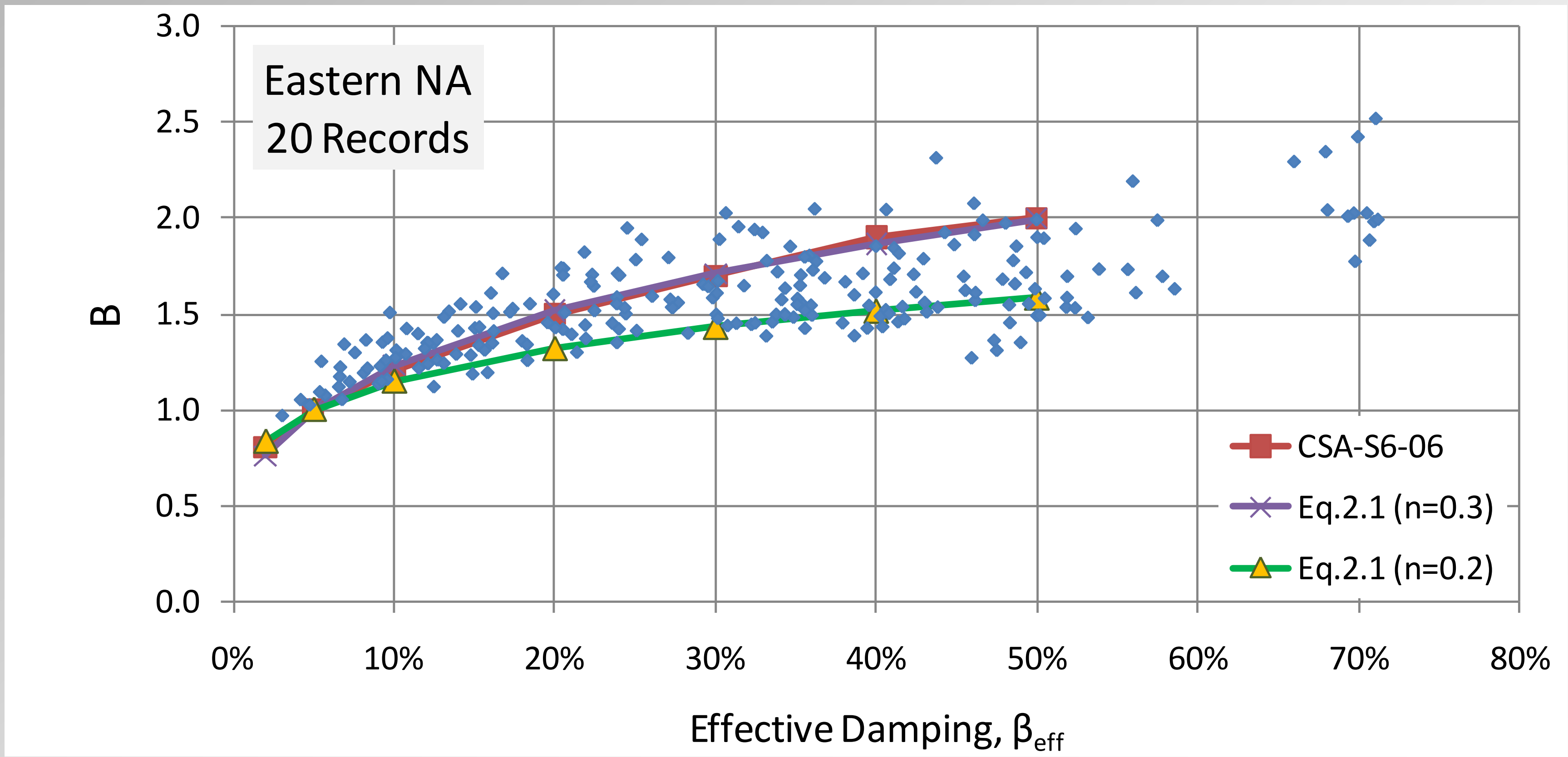
Note: *The percentage of critical damping shall be verified by a test of the isolation system's characteristics as specified in Clause 4.10.11.3.3. The damping coefficient shall be based on linear interpolation for damping levels other than those specified in this Table. For isolation systems where the effective damping exceeds 30% of critical, a three-dimensional non-linear time-history analysis shall be performed using the hysteresis curves of the system, unless B is limited to 1.7.*

C'_{sm} = elastic seismic response coefficient for isolated structures

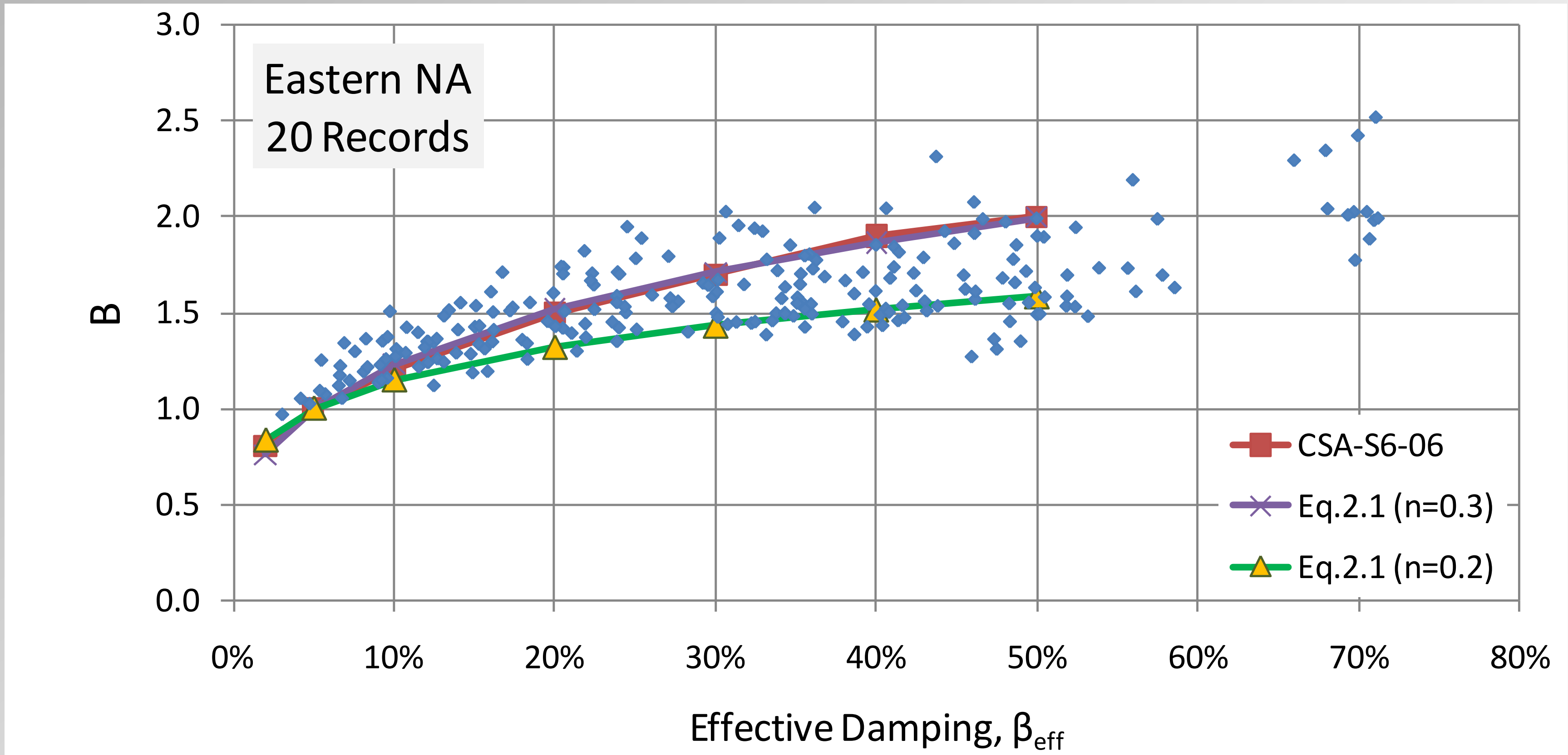
$$= \frac{AS_i}{BT_e} \leq 2.5\frac{A}{B}$$

- ***C'_{sm} replaced by UHS values***
- ***B revisited:***
  - ***Displacement spectra***
  - ***Forces obtained from displacement (or NLTH)***
  - ***Eastern and western Canada ground motions***
  - ***Influence of bridge period***
  - ***Site coefficients***
  - ***Include other dampers or ED systems***
  - ***…***

$$B = \left(\frac{\beta_{eff}}{5\%} \right)^{n}$$



$$B = \left(\frac{\beta_{eff}}{5\%}\right)^{n}$$



4.10.6.2.2 Application of uniform-load/single-mode method of analysis

The statically equivalent force determined in accordance with Clause 4.10.6.2.1, which is associated with the displacement across the isolation bearings, shall be applied using either the uniform-load method or the single-mode spectral method of analysis independently along two perpendicular axes and combined as specified in Clause 4.4.9.2. The effective linear stiffness of the isolators used in the analysis shall be calculated at the design displacement.

4.10.6.3 Multi-mode spectral analysis

Note: *See Clause 4.5.3.3 for the multi-mode spectral method.*

Where the appropriate ground motion response spectrum for the isolated modes is specified by Clause 4.10.6.2.1, an equivalent linear response spectrum analysis shall be performed in accordance with Clause 4.5.3. The ground motion response spectrum specified in Clause 4.4.7 shall be used for all other modes of vibration. The effective linear stiffness of the isolators shall be calculated at the design displacements.

The combination of orthogonal seismic forces shall be in accordance with Clause 4.4.9.2.

- ***Harmonization with non-isolated bridges***
- ***Clarification for damped vs undamped modes***

4.10.6.4 Time-history analysis

Note: *See Clause 4.5.3.4 for the time-history method.*

For isolation systems requiring a time-history analysis, the following requirements shall apply:

(a) The isolation system shall be modelled using the non-linear deformational characteristics of the isolators determined and verified by test in accordance with Clause 4.10.11.

(b) Pairs of horizontal ground motion time-history components shall be selected from different recorded events and modified to be compatible with the design spectra of Clause 4.4.7. The following methods may be used to achieve this modification:

  (i) time histories may be scaled so that their 5%-damped response spectra do not fall below the design spectra of Clause 4.4.7 by more than 10% in the period range of 1 to 5 s or by more than 20% in the range below 1 s; or

  (ii) time histories may be scaled so that the square root of the sum of the squares (SRSS) of the 5%-damped spectrum of the scaled components does not fall below 1.3 times the design spectra of Clause 4.4.7 for the period range of 1 to 5 s.

(c) At least three appropriate pairs of time histories shall be developed and each pair shall be applied simultaneously to the model. The maximum response of the parameter of interest shall be used for the design.

- ***Harmonization with Clause 4.5.3.4***
- ***Mean response with 5 or more records***
- ***Only one adjustment method***
- ***Range of T for scaling based on T_{eff}, T_t ?***
- ***Need for special additional requirements ?***
- ***Effects of vertical accelerations (friction) ?***

4.10.7 Clearance and design displacements for seismic and other loads

The design displacements in the two orthogonal directions for clearance purposes shall be the maximum displacement determined in each direction from the analysis. The required clearance for lifeline and emergency-route bridges shall be 1.25 times the maximum displacements calculated.

The total design displacement for the testing requirements of Clause 4.10.11 shall be the maximum of 50% of the elastomer shear strain in an elastomeric-based system or the maximum displacement that results from the combination of loads specified in Clause 4.4.9.2.

Horizontal deflections in the isolators resulting from load combinations involving wind loads on structure and traffic, braking forces, and centrifugal forces, as specified in Table 3.1, as well as thermal movements, shall be calculated and adequate clearance shall be provided.

- ***Design seismic displacement = 1.25(?) x d_i***
- ***Factor varies with bridge categories?***
- ***Maximum expected displacement:***
  - ***Q + K, K with 50%(?) T***
  - ***Q_r + K, K with 100%(?) T***
- ***Combination depends on return period?***
- ***Clearance based on maximum expected displacement***

Table 3.1
Load factors and load combinations

(See Clauses 3.5.1, 3.10.1.1, 3.10.5.2, 3.13, 3.16.3, 4.10.7, 4.10.10.1, 7.6.3.1.1, 7.7.3.1.1, 9.4.2, and 15.6.2.4.)

	Permanent loads			Transitory loads					Exceptional loads			
Loads	*D*	*E*	*P*	*L**	*K*	*W*	*V*	*S*	*EQ*	*F*	*A*	*H*
Fatigue limit state												
FLS Combination 1	1.00	1.00	1.00	1.00	0	0	0	0	0	0	0	0
Serviceability limit states												
SLS Combination 1	1.00	1.00	1.00	0.90	0.80	0	0	1.00	0	0	0	0
SLS Combination 2†	0	0	0	0.90	0	0	0	0	0	0	0	0
Ultimate limit states‡												
ULS Combination 1	α_D	α_E	α_P	1.70	0	0	0	0	0	0	0	0
ULS Combination 2	α_D	α_E	α_P	1.60	1.15	0	0	0	0	0	0	0
ULS Combination 3	α_D	α_E	α_P	1.40	1.00	0.50§	0.50	0	0	0	0	0
ULS Combination 4	α_D	α_E	α_P	0	1.25	1.65§	0	0	0	0	0	0
ULS Combination 5	α_D	α_E	α_P	0	1.00	0	0	0	1.00	0	0	0
ULS Combination 6**	α_D	α_E	α_P	0	0	0	0	0	0	1.30	0	0
ULS Combination 7	α_D	α_E	α_P	0	0	0.90§	0	0	0	0	1.30	0
ULS Combination 8	α_D	α_E	α_P	0	0	0	0	0	0	0	0	1.00
ULS Combination 9	1.35	α_E	α_P	0	0	0	0	0	0	0	0	0

**For the construction live load factor, see Clause 3.16.3.*
†For superstructure vibration only.
‡For ultimate limit states, the maximum or minimum values of α_D, α_E, and α_P specified in Table 3.2 shall be used.
§For wind loads determined from wind tunnel tests, the load factors shall be as specified in Clause 3.10.5.2.
***For long spans, it is possible that a combination of ice load F and wind load W will require investigation.*

Legend:

A = ice accretion load
D = dead load
E = loads due to earth pressure and hydrostatic pressure, including surcharges but excluding dead load
EQ = earthquake load
F = loads due to stream pressure and ice forces or to debris torrents
H = collision load arising from highway vehicles or vessels
K = all strains, deformations, and displacements and their effects, including the effects of their restraint and the effects of friction or stiffness in bearings. Strains and deformations include strains and deformations due to temperature change and temperature differential, concrete shrinkage, differential shrinkage, and creep, but not elastic strains
L = live load (including the dynamic load allowance, when applicable)
P = secondary prestress effects
S = load due to differential settlement and/or movement of the foundation
V = wind load on traffic
W = wind load on structure

Note pointing to 4.10.7

Table 3.1
Load factors and load combinations
(See Clauses 3.5.1, 3.10.1.1, 3.10.5.2, 3.13, 3.16.3, 4.10.7, 4.10.10.1, 7.6.3.1.1, 7.7.3.1.1, 9.4.2, and 15.6.2.4.)

Loads	Permanent loads			Transitory loads					Exceptional loads			
	D	*E*	*P*	*L**	*K*	*W*	*V*	*S*	*EQ*	*F*	*A*	*H*
Fatigue limit state												
FLS Combination 1	1.00	1.00	1.00	1.00	0	0	0	0	0	0	0	0
Serviceability limit states												
SLS Combination 1	1.00	1.00	1.00	0.90	0.80	0	0	1.00	0	0	0	0
SLS Combination 2†	0	0	0	0.90	0	0	0	0	0	0	0	0
Ultimate limit states‡												
ULS Combination 1	α_D	α_E	α_P	1.70	0	0	0	0	0	0	0	0
ULS Combination 2	α_D	α_E	α_P	1.60	1.15	0	0	0	0	0	0	0
ULS Combination 3	α_D	α_E	α_P	1.40	1.00	0.50§	0.50	0	0	0	0	0
ULS Combination 4	α_D	α_E	α_P	0	1.25	1.65§	0	0	0	0	0	0
ULS Combination 5	α_D	α_E	α_P	0	1.00	0	0	0	1.00	0	0	0
ULS Combination 6**	α_D	α_E	α_P	0	0	0	0	0	0	1.30	0	0
ULS Combination 7	α_D	α_E	α_P	0	0	0.90§	0	0	0	0	1.30	0
ULS Combination 8	α_D	α_E	α_P	0	0	0	0	0	0	0	0	1.00
ULS Combination 9	1.35	α_E	α_P	0	0	0	0	0	0	0	0	0

**For the construction live load factor, see Clause 3.16.3.*
†*For superstructure vibration only.*
‡*For ultimate limit states, the maximum or minimum values of α_D, α_E, and α_P specified in Table 3.2 shall be used.*
§*For wind loads determined from wind tunnel tests, the load factors shall be as specified in Clause 3.10.5.2.*
***For long spans, it is possible that a combination of ice load F and wind load W will require investigation.*

Legend:

A = ice accretion load
D = dead load
E = loads due to earth pressure and hydrostatic pressure, including surcharges but excluding dead load
EQ = earthquake load
F = loads due to stream pressure and ice forces or to debris torrents
H = collision load arising from highway vehicles or vessels
K = all strains, deformations, and displacements and their effects, including the effects of their restraint and the effects of friction or stiffness in bearings. Strains and deformations include strains and deformations due to temperature change and temperature differential, concrete shrinkage, differential shrinkage, and creep, but not elastic strains
L = live load (including the dynamic load allowance, when applicable)
P = secondary prestress effects
S = load due to differential settlement and/or movement of the foundation
V = wind load on traffic
W = wind load on structure

Note pointing to 4.10.7

4.10.10 Other requirements

4.10.10.1 Non-seismic lateral forces

Isolated structures shall resist all non-seismic lateral load combinations applied above the isolation system, including load combinations involving wind loads on the structure and the traffic, braking forces, and centrifugal forces specified in Table 3.1.

An elastic restraint system shall be provided to limit lateral displacements of the isolation system caused by non-seismic forces, to a value satisfactory to the design Engineer.

4.10.10.2 Lateral restoring force

The isolation system shall be configured to produce a lateral restoring force such that the lateral force at the design displacement is at least $0.025W$ greater than the lateral force at 50% of the design displacement.

4.10.10.3 Vertical load stability

The isolation system shall provide a factor of safety of at least 3.0 for vertical loads (dead load plus live load) in its laterally undeformed state. It shall also be designed to be stable under the dead load plus or minus any vertical load resulting from seismic effects at a horizontal displacement of 1.5 times the total design displacement for isolation systems with a lateral restoring force. If the design is based on maximum credible response spectra, the 1.5 and 3.0 coefficients shall be reduced to 1.1 and 2.2, respectively.

4.10.10.4 Cold weather requirements

Cold weather performance shall be considered in the design of all types of isolation systems in sustained low-temperature zones.

4.10.10 Other requirements

4.10.10.1 Non-seismic lateral forces

Isolated structures shall resist all non-seismic lateral load combinations applied above the isolation system, including load combinations involving wind loads on the structure and the traffic, braking forces, and centrifugal forces specified in Table 3.1.

An elastic restraint system shall be provided to limit lateral displacements of the isolation system caused by non-seismic forces, to a value satisfactory to the design Engineer.

- ***Inherent friction used as elastic restraint ?***

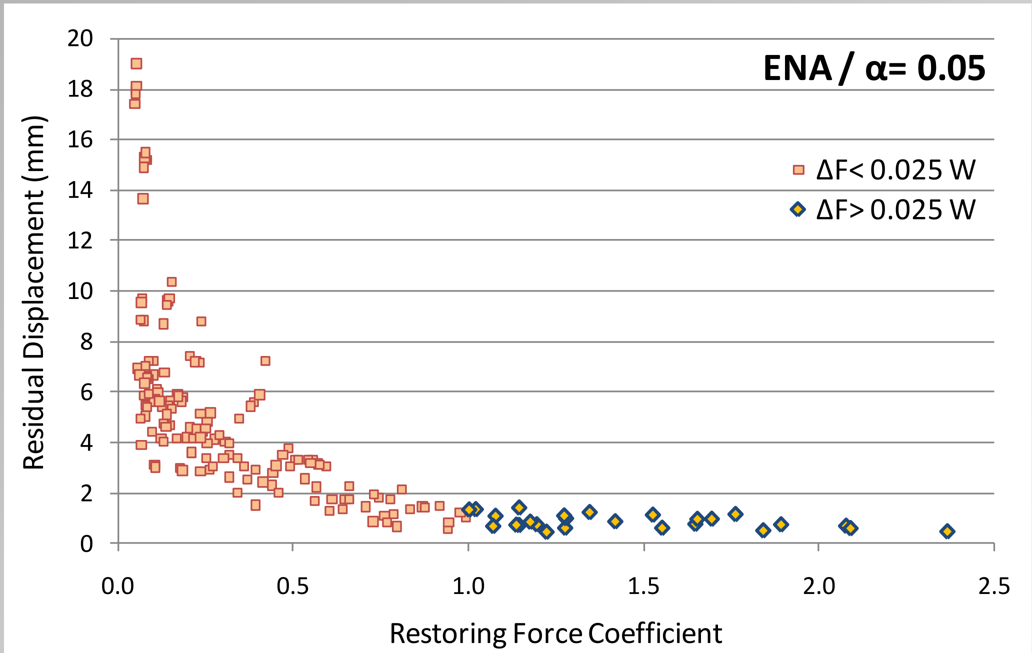
4.10.10.2 Lateral restoring force

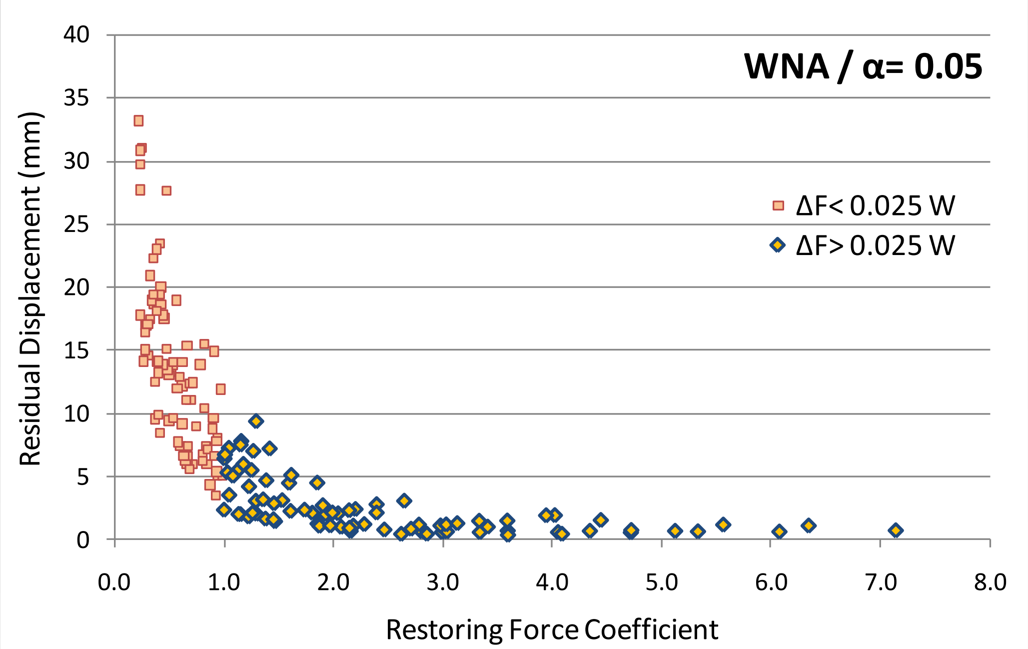
The isolation system shall be configured to produce a lateral restoring force such that the lateral force at the design displacement is at least 0.025*W* greater than the lateral force at 50% of the design displacement.

- ***Criteria is being revisited***
  - $K_d\ d_i \geq 0.025\ W$?
  - $T_t < 6$ s ?
  - NLTH if $T_t > 6$ s ?

12.2—Lateral Restoring Force

The isolation system shall be configured to produce a lateral restoring force such that the period corresponding to its tangent stiffness based on the restoring force alone at any displacement, Δ, up to its total design displacement (TDD), d_t, shall be less than 6 s (Figure C12.2-1). Also the restoring force at d_t shall be greater than the restoring force at $0.5d_t$ by not less than $W/80$. Isolation systems with constant restoring force need not satisfy the requirements above. In these cases, the combined constant restoring force of the isolation system shall be at least equal to 1.05 times the characteristic strength (Q_d) of the isolation system under service conditions.


ENA / α= 0.05
ΔF< 0.025 W
ΔF> 0.025 W
Residual Displacement (mm)
Restoring Force Coefficient
0
2
4
6
8
10
12
14
16
18
20
0.0
0.5
1.0
1.5
2.0
2.5


WNA / α= 0.05
ΔF< 0.025 W
ΔF> 0.025 W
Residual Displacement (mm)
Restoring Force Coefficient
0
5
10
15
20
25
30
35
40
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0

4.10.8 Design forces for Seismic Performance Zone 1

The seismic design force of the connection between superstructure and substructure at each isolator for bridges in Seismic Performance Zone 1, F_A, shall be

$$F_A = k_{eff} d_i$$

where

k_{eff} = effective linear stiffness of the isolation bearing calculated at displacement d_i

d_i = displacement of the isolated superstructure as specified in Clause 4.10.6.2.1, using a minimum zonal acceleration ratio, A, of 0.10

4.10.9 Design forces for Seismic Performance Zones 2, 3, and 4

The requirements of Clauses 4.4.10.3 and 4.4.10.4 and the response modification factor and design requirements of Clause 4.10.5 shall apply in Seismic Performance Zones 2, 3, and 4.

The seismic design forces for columns and piers shall not be less than the forces resulting from the yield level of a softening system, the friction level of a sliding system, or the ultimate capacity of a sacrificial seismic-restraint system. In all cases, the larger of the static or dynamic conditions shall apply.

- ***Include forces due to additional damping***
- ***Design forces based on maximum expected displacement (NLTH)***
- ***Consistency with design of substructure (4.10.5)***

4.10.10.3 Vertical load stability

The isolation system shall provide a factor of safety of at least 3.0 for vertical loads (dead load plus live load) in its laterally undeformed state. It shall also be designed to be stable under the dead load plus or minus any vertical load resulting from seismic effects at a horizontal displacement of 1.5 times the total design displacement for isolation systems with a lateral restoring force. If the design is based on maximum credible response spectra, the 1.5 and 3.0 coefficients shall be reduced to 1.1 and 2.2, respectively.

- ***Vertical acceleration demand examined***
- ***Amplification factors to be revisited based on return period and bridge category***

4.10.10.4 Cold weather requirements

Cold weather performance shall be considered in the design of all types of isolation systems in sustained low-temperature zones.

- ***Consistency with E + T combination***

4.10.11 Required tests of isolation system

4.10.11.1 General

The deformation characteristics and damping values of the isolation system used in the design and analysis shall be based on the tests specified in Clause 4.10.11. Tests on similarly sized isolators may be used to satisfy the requirements of Clause 4.10.11. Such tests shall validate design properties that can be extrapolated to the actual sizes used in the design.

The design shall also be based on manufacturers' pre-Approved or certified test data.

4.10.11.2 Prototype tests

The following requirements shall apply to prototype tests:

(a) Prototype tests shall be performed on two full-size specimens of each type and size similar to that used in the design. The tests specimens shall include the elastic restraint system, if such a system is used in the design. The specimens tested shall not be used for construction.

(b) For each cycle of tests, the force-deflection and hysteretic behaviour of the test specimens shall be recorded.

(c) The following sequence of tests shall be performed for the prescribed number of cycles at a vertical load similar to the typical or average dead load on the isolators of a common type and size. The total design displacement of these tests shall be in accordance with Clause 4.10.7:

   (i) 20 fully reversed cycles of loading at a lateral force corresponding to the maximum non-seismic design force;

   (ii) three fully reversed cycles of loading at each of the following increments of the total design displacement: 0.25, 0.50, 0.75, 1.0, and 1.25; and

   (iii) 15 S_i/B cycles, but not fewer than 10 fully reversed cycles of loading at 1.0 times the total design displacement and a vertical load similar to dead load. B shall be determined from Table 4.8.

(d) The vertical load-carrying elements of the isolation system shall be statically tested at the displacements resulting from the requirements of Clause 4.10.10.3. In these tests, the maximum downward force shall be taken as the load of 1.25D plus the increased vertical load due to earthquake effects, and the minimum download force shall be taken as 0.8D minus the vertical load due to earthquake effects, where EQ is any vertical load resulting from horizontal seismic loads.

(e) If a sacrificial elastic restraint system is used, the ultimate capacity shall be established by test.

4.10.11.3 Determination of force-deflection characteristics

4.10.11.3.1 General

The following requirements shall apply:

(a) The force-deflection characteristics of the isolation system shall be based on the cyclic load test results for each fully reversed cycle of loading.

(b) The effective stiffness of an isolator unit, k_{eff}, shall be calculated for each cycle of loading as follows:

$$k_{eff} = \frac{F_p - F_n}{\Delta_p - \Delta_n}$$

where F_p and F_n are the maximum positive and maximum negative forces, respectively, and Δ_p and Δ_n are the maximum positive and maximum negative test displacements, respectively.

If the minimum effective stiffness is to be determined, $F_{p,min}$ and $F_{n,min}$ shall be used in the equation.

4.10.11.3.2 System adequacy

The performance of the test specimens shall be deemed to be adequate if the following conditions are satisfied:

(a) The force-deflection plots of all tests specified in Clause 4.10.11.2 have a positive incremental force-carrying capacity.

(b) For each increment of test displacement specified in Clause 4.10.11.2(c)(ii), the following conditions are met:

  (i) there is less than a ±10% change from the average effective stiffness of a given test specimen over the required three cycles of test; and

  (ii) there is not more than a 10% difference in the average values of effective stiffness of the two test specimens of a common type and size of the isolator unit over the required three cycles of test.

(c) There is not more than a 20% increase or 20% decrease in the effective stiffness between the first cycle and any subsequent cycle of each test specimen for the cyclic tests specified in Clause 4.10.11.2(c)(iii).

(d) There is not more than a 20% decrease in the effective damping of each test specimen for the cyclic tests specified in Clause 4.10.11.2(c)(iii).

(e) All specimens of vertical load-carrying elements of the isolation system remain stable at the displacements specified in Clause 4.10.10.3 for the static loads specified in Clause 4.10.11.2(d).

Table 4.1
Seismic Design Performance Criteria

Seismic Ground motion Probability of Exceedance (return period)	**Service Level**	**Damage Level**
Lifeline Bridges		
2% in 50 years (2475 years)	Possible loss of service	Significant (No collapse)
5% in 50 years (975 years)	Limited	Repairable
10% in 50 years (475 years)	Immediate	Minimal
Emergency Route Bridges		
2% in 50 years (2475 years)	Possible loss of service	Significant (No collapse)
10% in 50 years (475 years)	Limited	Repairable
Other Bridges		
2% in 50 years (2475 years)	Possible loss of service	No collapse

- ***Test displacement***
  = 1.25 x design seismic displacement ?
- ***Return period vs Performance criteria (for other bridges)?***

4.10.11.3.3 Design properties of the isolation system

The following requirements shall apply to the design properties of the isolation system:

(a) The minimum and maximum effective stiffness of the isolation system shall be determined as follows:

  (i) The value of k_{min} shall be based on the minimum effective stiffnesses of individual isolator units as determined by the cyclic tests of Clause 4.10.11.2(c)(ii) at a displacement amplitude equal to the design displacement.

  (ii) The value of k_{max} shall be based on the maximum effective stiffnesses of individual isolator units as determined by the cyclic tests of Clause 4.10.11.2(c)(ii) at a displacement amplitude equal to the design displacement.

(b) The equivalent viscous damping ratio, β, of the isolation system shall be calculated as follows:

$$\beta = \frac{1}{2\pi} \bullet \frac{\text{Total area}}{\Sigma k_{max} d_i^2}$$

where the total area represents the energy absorbed by the isolation system in one cycle and shall be taken as the sum of the areas inside the hysteresis loops of all isolators. The hysteresis loop area of each isolator shall be taken as the minimum area of one cycle obtained from the three hysteresis loops established by the cyclic tests of Clause 4.10.11.2(c)(ii) at a displacement amplitude equal to the design displacement.

- ***To be reviewed:***
  - ***demand from eastern and western Canada ground motions (number of cycles)***
  - ***static or dynamic***
  - ***Tests at low temperature (consistent with Q + K combination)***
  - ***Need for 3 series of tests:***

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13—REQUIRED TESTS OF ISOLATION SYSTEMS

All isolation systems shall have their seismic performance verified by testing. In general, there are three types of tests to be performed on isolation systems: 1) system characterization tests, described in Article 13.1; 2) prototype tests, described in Article 13.2; and 3) quality control tests, described in Sections 15, 17, and 18.

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13.1—System Characterization Tests

The fundamental properties of the isolation system shall be evaluated by testing prior to its use. The purpose of system characterization tests is to substantiate the properties of individual isolator units as well as the behavior of an isolation system. Therefore, these tests include both component tests of individual isolator units and shake table tests of complete isolation systems.

At a minimum, these tests shall consist of:

- Tests of individual isolator units in accordance with nationally recognized guidelines approved by the Engineer, and
- Shaking table tests at a scale no less than one-fourth full scale. Scale factors must be approved by the Engineer.

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13.2—Prototype Tests

The deformation characteristics and damping values of the isolation system used in the design and analysis shall be verified by prototype tests. Tests on similarly sized isolator units may be used to satisfy the requirements of this section. Such tests must validate design properties that can be extrapolated to the actual sizes used in the design.

13.2.1—Test Specimens

Prototype tests shall be performed on a minimum of two full-size specimens of each type and size similar to that used in the design. Prototype test specimens may be used in construction, if they have the specified stiffness and damping properties and they satisfy the project quality control tests after having successfully completed all prototype tests.

Reduced-scale prototype specimens shall only be allowed when full-scale specimens exceed the capacity of existing testing facilities and approval is granted by the Engineer.

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15.2—Quality Control Tests

The following quality control tests shall also be performed on elastomeric bearings.

15.2.1—Compression Capacity

A 5-min sustained proof load test shall be conducted on each bearing. The compressive load for the test shall be 1.5 times the maximum (dead load plus live load). If

...

15.2.2—Combined Compression and Shear

All bearings shall be tested in combined compression and shear. The bearings may be tested in pairs. The compressive load shall be the average dead load of all bearings of that type, and the bearings shall be subjected to three fully reversed cycles of loading at the larger of the TDD or 50 percent of the elastomer's thickness.

For each bearing, the effective stiffness and *EDC* shall be averaged over the three cycles of the test. For each group of similar bearings of the same type and size, the effective stiffness and *EDC* shall be averaged. The results shall not differ from the design values by more than the limits given in Table 15.2.2-1. Alternative

...

4.10.12 Elastomeric bearings — Design

4.10.12.1 General

Isolator units that use elastomeric bearings shall be designed in accordance with Clause 4.10.12. Additional test requirements are specified in Clause 4.10.13.

The requirements of Clause 4.10.12 shall be considered supplemental to those of Section 11. The requirements of Clause 4.10.12 shall govern in the event of a conflict with those of Section 11.

The design procedures specified in Clause 4.10.12 are based on service loads excluding impact. Elastomeric bearings used in isolation systems shall be reinforced using integrally bonded steel reinforcement. Fabric reinforcement shall not be permitted.

4.10.12.2 Shear strain components for isolation design

The four components of shear strain in the bearing shall be calculated as follows:

(a) Shear strain due to compression by vertical loads, ε_{sc}, shall be calculated as follows:

$$\varepsilon_{sc} = 6S\varepsilon_c$$

...

4.10.13 Elastomeric bearings — Construction

4.10.13.1 General

Isolator units that use elastomeric bearings shall be constructed in accordance with Clause 4.10.13.

The requirements of Clause 4.10.13 shall be considered supplemental to Section 11. The requirements of Clause 4.10.13 shall govern in the event of a conflict with those of Section 11.

Elastomeric bearings used in isolation systems shall be reinforced using integrally bonded steel reinforcement. Fabric reinforcement shall not be permitted.

Seismic isolation bearings shall meet the requirements of Clause 4.10.13.2 and Section 11.

4.10.13.2 Additional requirements for elastomeric isolation bearings

4.10.13.2.1 General

In addition to the material and bearing tests required by Clauses 4.10.10 and 4.10.11, the tests specified in Clauses 4.10.13.2.2 and 4.10.13.2.3 shall be performed on elastomeric isolation bearings.

...

4.10.14 Sliding bearings — Design

Sliding bearings may be used for isolation systems if Approved. The Regulatory Authority shall specify appropriate materials and design parameters. The requirements of Clause 11.6 for PTFE bearing surfaces shall be satisfied.

4.10.15 Sliding bearings — Construction

Isolator units that use sliding bearings shall be constructed in accordance with Section 11.

- ***In appendix ?***
- ***Other systems ?***

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The isolation system shall also be designed to be stable under 1.2 times the dead load plus any vertical load resulting from seismic live load plus overturning, at a horizontal displacement equal to the offset displacement plus the larger of the following displacements:

- 1.1 times the TDD for the maximum considered earthquake,
- In Seismic Zones 1 and 2, 2.0 times the TDD for a 1,000-yr return period earthquake, or
- In Seismic Zones 3 and 4, 1.5 times the TDD for a 1,000-yr return period earthquake.

8.1.2—Minimum and Maximum K_d and Q_d

The minimum and maximum values of K_d and Q_d shall be determined as:

$$K_{d,max} = K_d \ \lambda_{max, Kd} \qquad (8.1.2\text{-}1)$$

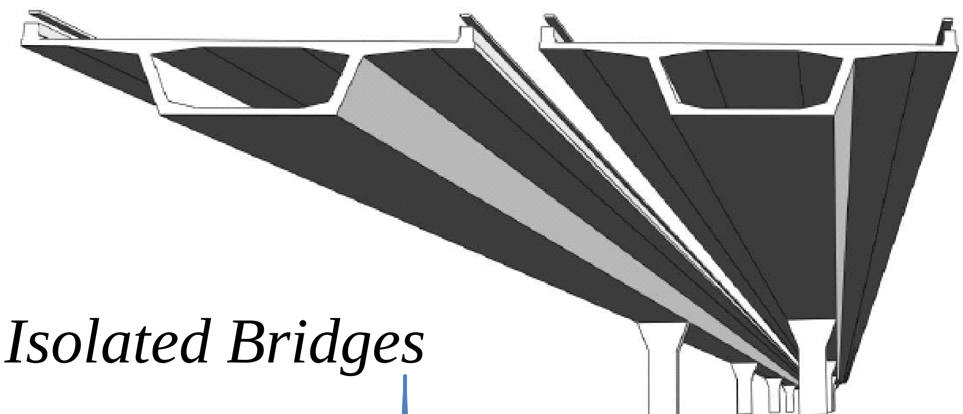
$$K_{d,min} = K_d \ \lambda_{min, Kd} \qquad (8.1.2\text{-}2)$$

$$Q_{d,max} = Q_d \ \lambda_{max, Qd} \qquad (8.1.2\text{-}3)$$

$$Q_{d,min} = Q_d \ \lambda_{min, Qd} \qquad (8.1.2\text{-}4)$$

Calibration of Design Methodologies and Optimal Use of Isolation and Damping for Isolated Bridges

Constantin Christopoulos,
University of Toronto



CSRN Workshop on Damped and Isolated Bridges
Vancouver, 26 April, 2012

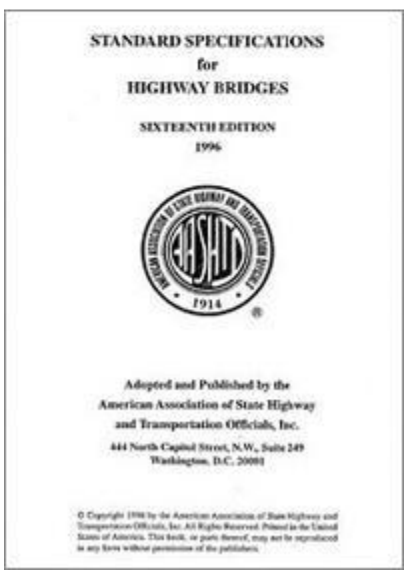
CSRN
RCRP
Canadian Seismic Research Network
Réseau canadien pour la recherche parasismique

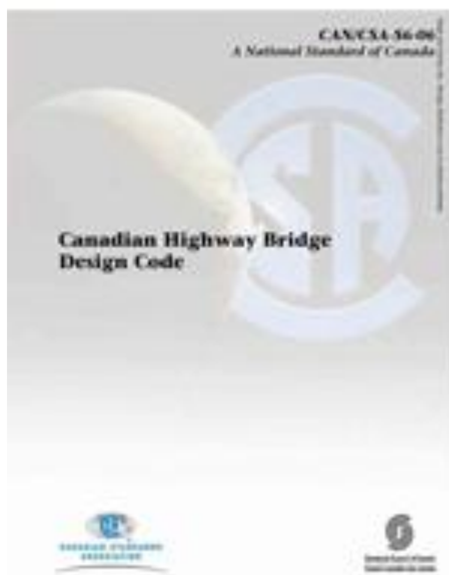
Funded by NSERC / *Subventionné par le CRSNG*

Overview of identified research needs for implementing isolation and damping in bridges in Canada

New Canadian Bridge Design Code:

- Canadian Highway Bridge Design Code CSA-S6-06 seismic provisions developed based on historical events occurred along the North American west coast and are therefore calibrated to these ground motions (essentially based on previous AASHTO LRFD Specifications)
- High risk of earthquake in Eastern Canada - Definition of Seismic Hazard in absence of historical records and impact of high frequency content on design of isolated and damped bridges.
- Methodologies for Optimal combination of isolation and damping especially for retrofit of large number of existing seismically deficient bridges





New Code
CAN/CSA-S6

New and
Retrofited Bridges

Ongoing Studies

Viacheslav Koval, Ph.D. Candidate, University of Toronto

Professor Robert Tremblay, Ecole Polytechnique de Montreal

1. Directivity effect study
2. B-factor study
3. Analytical Model for optimizing combinations of damping and isolation
4. Two-stage isolation approach for optimal multi-level hazard mitigation
5. Case studies in Quebec and Vancouver

Ground motions for time-history analyses for ENA

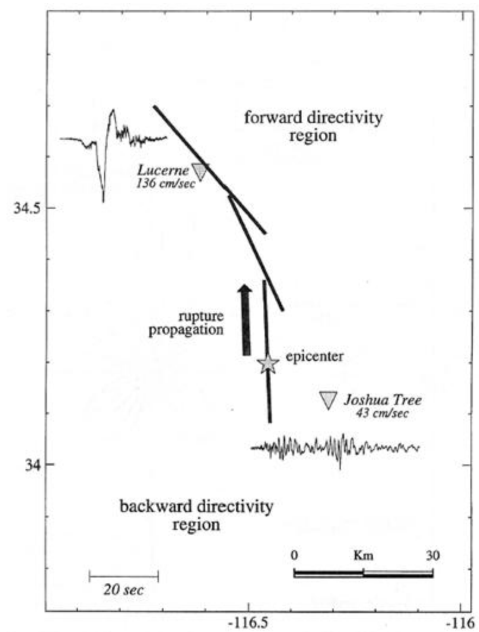
Due to the scarcity of ground motions corresponding to historical events occurred in ENA, sets of eastern artificial (Atkinson, 2009) and hybrid (McGuire et al., 2001) ground motions were adopted for the ENA regions.

- Artificial Records - Atkinson 2009
  - Time histories generated by using the new stochastic finite-fault approach (code EXSIM model) to hazard 2% in 50 years with calibration based on past large events and recordings of small-to-moderate events
  - Possibility of linear scaling to NBCC 2005 UHS
  - Four magnitude-distance sets to match entire UHS (M6 at 10-15 km, M6 at 20-30 km, M7 at 15-25 km, M7 at 50-100 km)
  - Accounting for faulting geometry, distributed rupture, and rupture non-homogeneity – seismic directivity effects
- Hybrid Records – McGuire 2001
  - Time-histories developed by modifying historic records (primarily from California strong motion) to reflect the particularities of CEUS (Central and Eastern US) ground motions
  - Possibility of linear scaling to target UHS
  - Consider effect of earthquake duration, frequency-to-frequency variation, and effects of rupture directivity

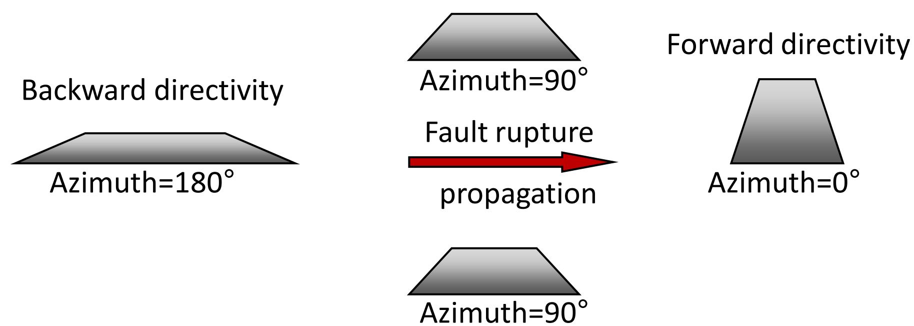
Directivity Effect Study

- Effect of azimuth on amplitude and duration of ground motion is well-known (e.g. Tremblay and Atkinson 2001).
- Effect of directivity is generally due to shear dislocation that does not occur instantaneously affecting arrival time of waves traveling from different parts of the fault (Stein and Wysession 2003).
- Influence of seismic directivity on structural response is frequently omitted during ground motion record selection.

Forward and backward rupture directivity.
1992 Landers earthquake (Somerville 1997)



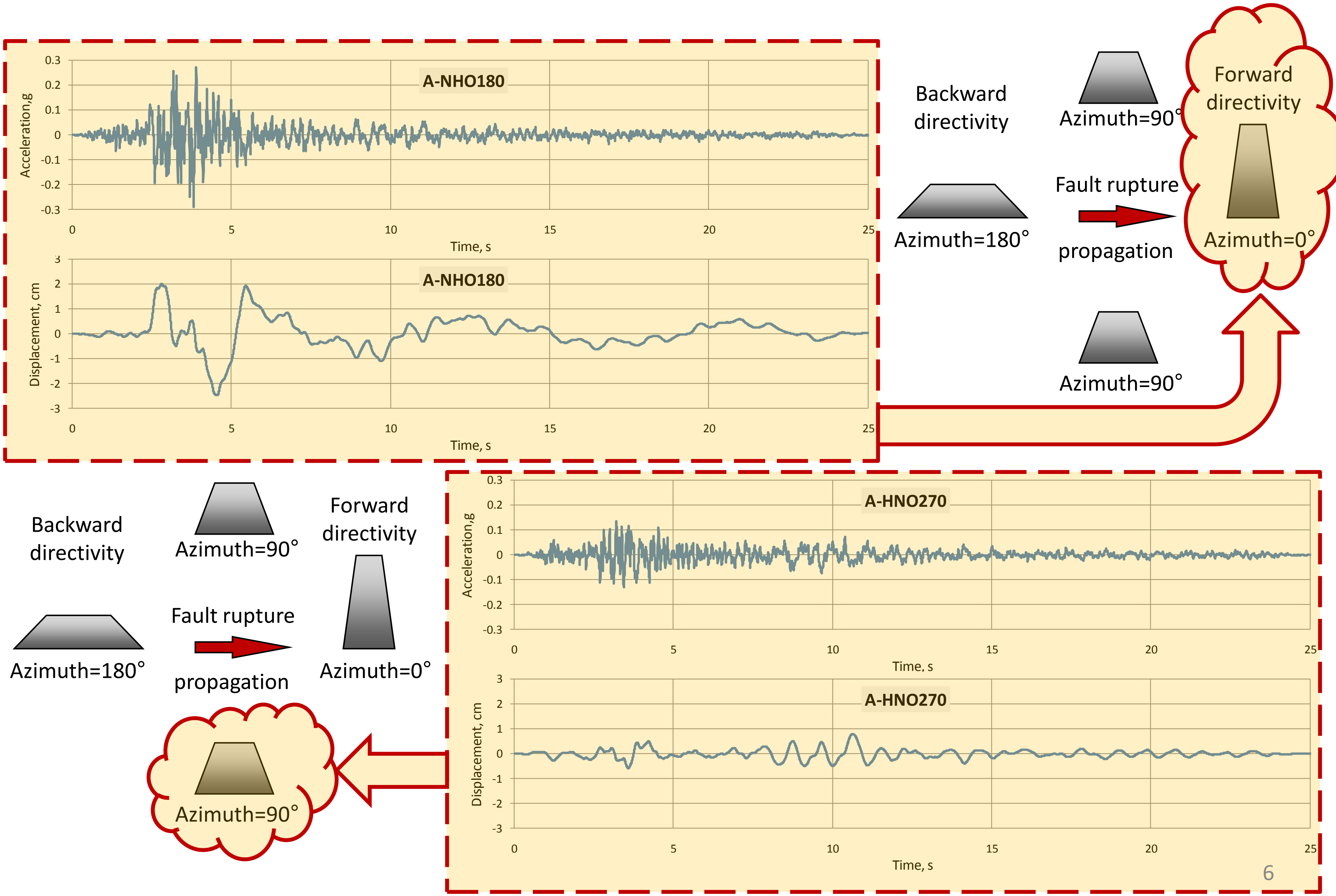
Directivity effect on the ground motion intensity and duration



<u>Examine in this study:</u>

- How much variability exists?
- Consequences of this variability on isolated and damped bridges
- How to consider it in design process?

Directivity effect study - Example



Directivity effect study

Directivity Effect Assessment in terms of 4 Damage Indices

Indices for Nonlinear Behaviour:

- *Number of yielding occurrence (Yield)*
- *Kinematic Damage Index (D_μ)*
- *Hysteretic Damage Index (D_h)*
- *Low-cycle Fatigue Resistance Damage Index (D_f)*

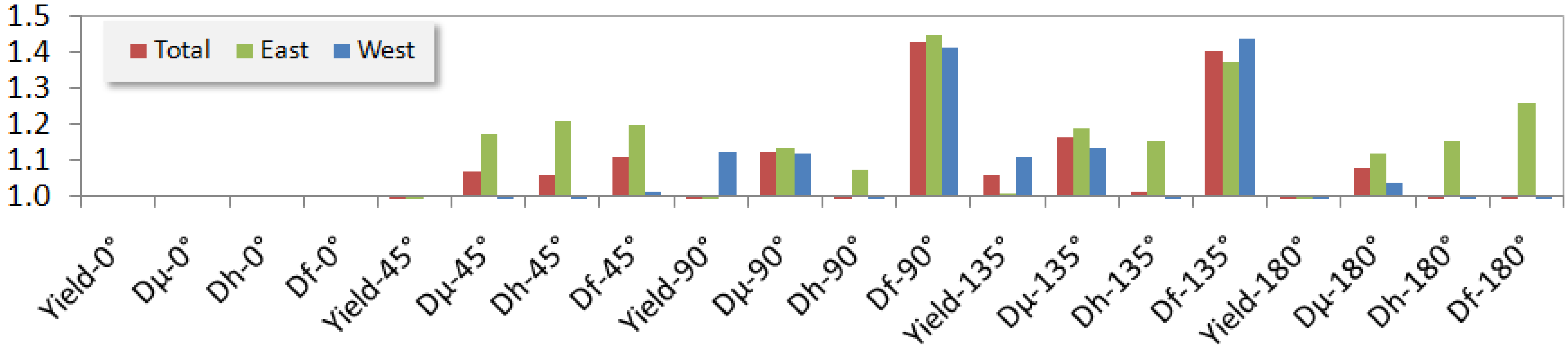
Azimuthal variability (α):

$$\alpha(\theta) = \frac{\text{Response Index}\,(\theta)}{\text{Response Index}\,(0^\circ)};$$

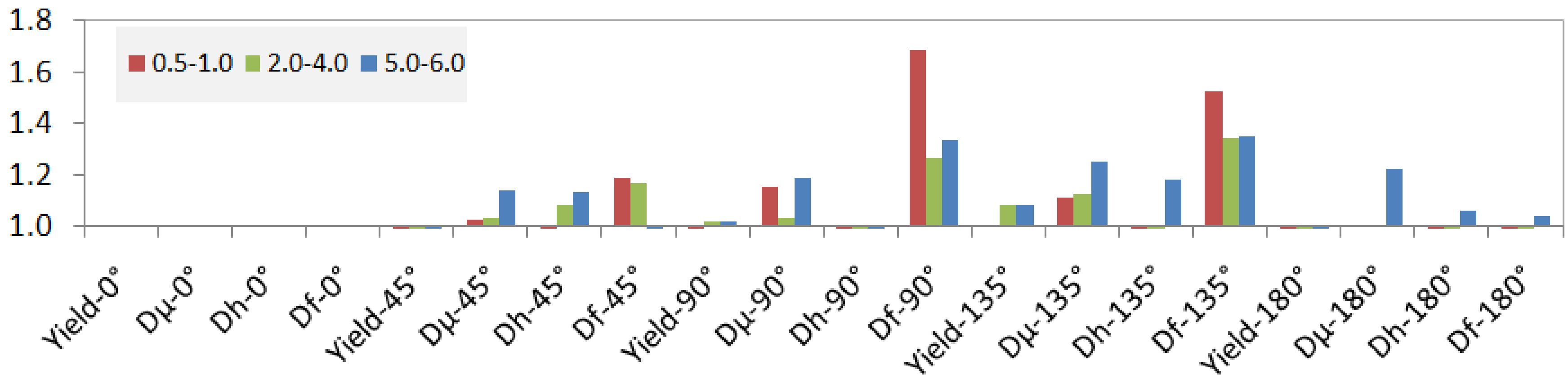
Directivity Effect Study

Azimuthal variability (α) for ENA and WNA:

$$\alpha(\theta) = \frac{\text{Response Index}\,(\theta)}{\text{Response Index}\,(0°)}$$



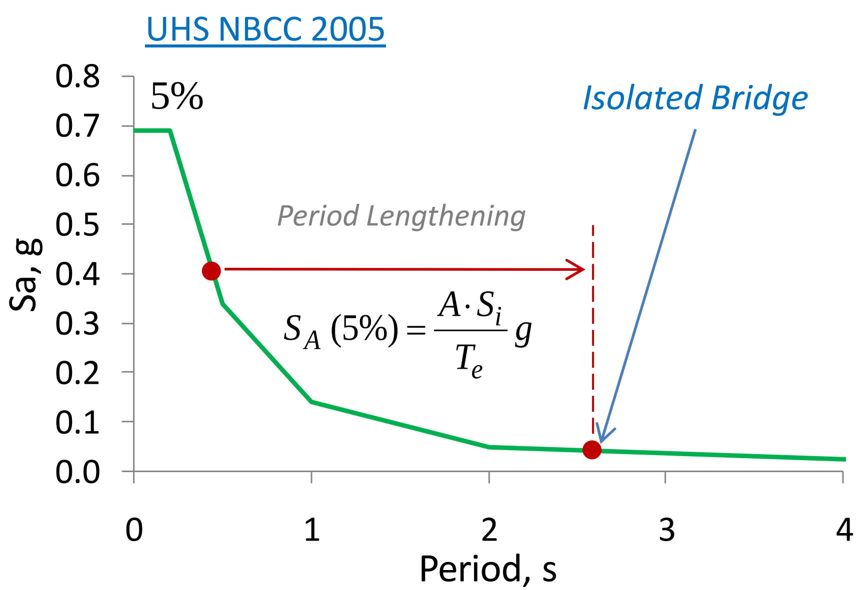
Azimuthal variability (α) for elastic period ranges (0.5-1.0s; 2.0-4.0s and 5.0-6.0s):

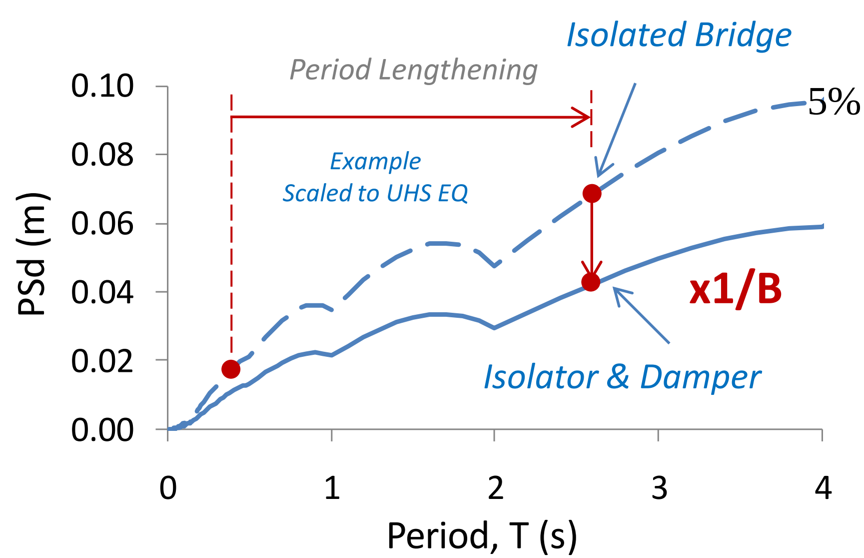


B-factor study

Current CSA-S6-Code: $F = C'_{sm} \cdot W = \dfrac{A \cdot S_i}{B \cdot T_e} \cdot W$ $\quad d_i = \dfrac{250\, A \cdot S_i \cdot T_e}{B}\ (mm)$

$$PS_D(\beta) = \frac{S_A(\beta)}{\omega^2} = \frac{S_A(5\%)}{\omega^2 B} = \frac{T_e^2}{4 \cdot \pi^2} \frac{A \cdot S_i}{B \cdot T_e} \cdot g = \frac{249 \cdot A \cdot S_i \cdot T_e}{B}\ (mm)$$





B-factors in Current Codes

CAN/CSA-S6-06

Table 4.8
Damping coefficient, *B*
(See Clauses 4.10.6.2.1 and 4.10.11.2.)

Equivalent viscous damping, β (% of critical)	Damping coefficient, B
≤2	0.8
5	1
10	1.2
20	1.5
30	1.7
40	1.9
50	2

Note: *The percentage of critical damping shall be verified by a test of the isolation system's characteristics as specified in Clause 4.10.11.3.3. The damping coefficient shall be based on linear interpolation for damping levels other than those specified in this Table. For isolation systems where the effective damping exceeds 30% of critical, a three-dimensional non-linear time-history analysis shall be performed using the hysteresis curves of the system, unless B is limited to 1.7.*

Different Code Provisions

β	AASHTO (1994)	AASHTO (2009)	EUROCODE 8	FEMA 273 (1997) FEMA 356 (2000)	FEMA 273 (1997) FEMA 356 (2000)
(%)	B	B	$1/\mu$	B_s	B_1
2	0.8	0.76	0.84	0.8	0.8
5	1.0	1.00	1.00	1.0	1.0
10	1.2	1.23	1.22	1.3	1.2
20	1.5	1.52	1.58	1.8	1.5
30	1.7	1.71	1.87	2.3	1.7
40	1.9	1.87	2.12	2.7	1.9
50	2.0	2.00	2.35	3.0	2.0

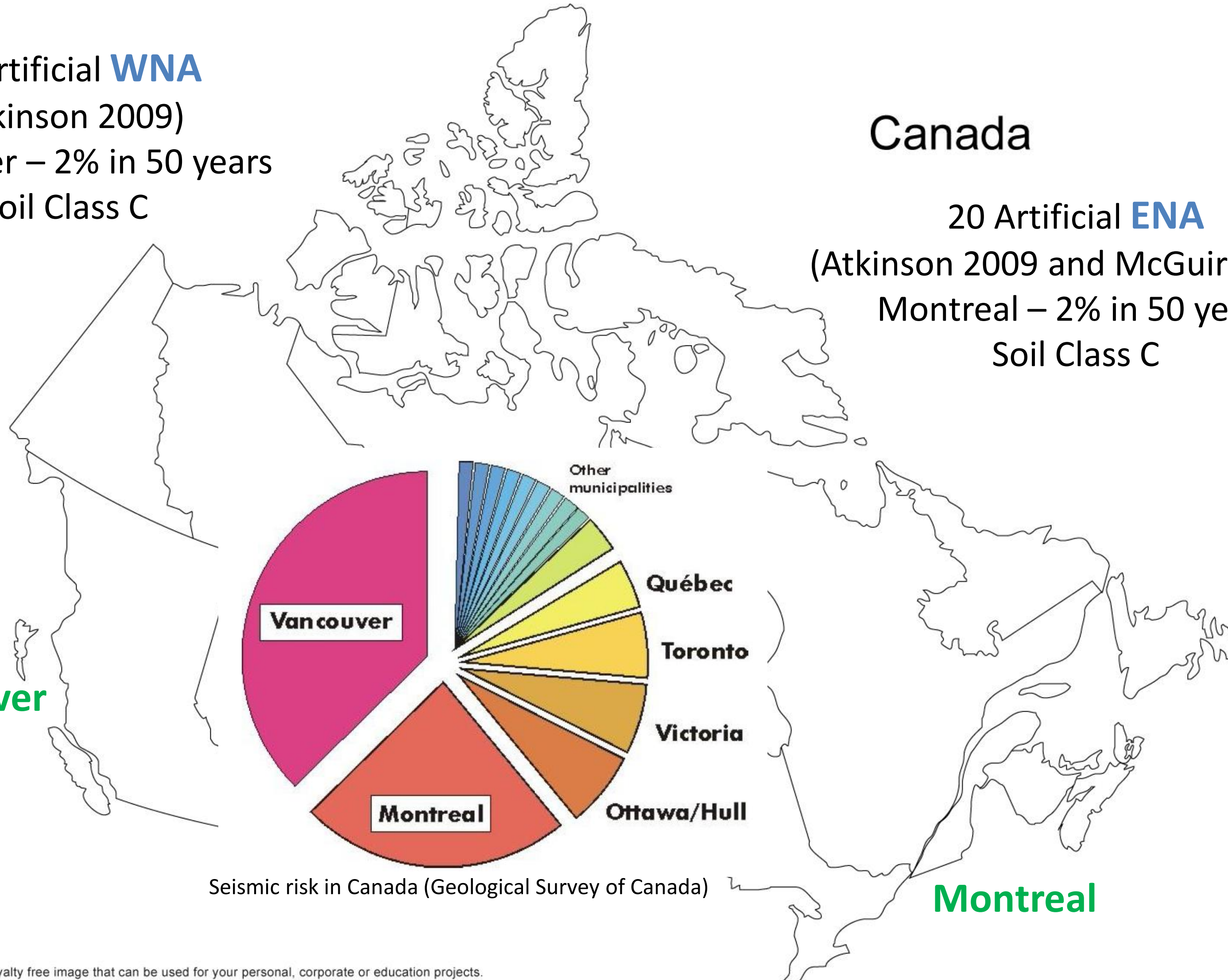
β	UBC (1994)	ATC-40 (1996)	ATC-40 (1996)	Newmark & Hall (1982)		
(%)	B	B_s	B_1	A Region	V Region	D Region
2				0.77	0.81	0.85
5	1.00	1.00	1.00	1.00	1.00	1.00
10	1.19	1.30	1.22	1.29	1.20	1.16
20	1.56	1.82	1.54	1.81	1.53	1.38
30	1.89	2.38	1.82			
40		3.03	2.08			
50						

T=0.50s T=3.33s

B-factor study

20 Artificial **WNA**
(Atkinson 2009)
Vancouver – 2% in 50 years
Soil Class C

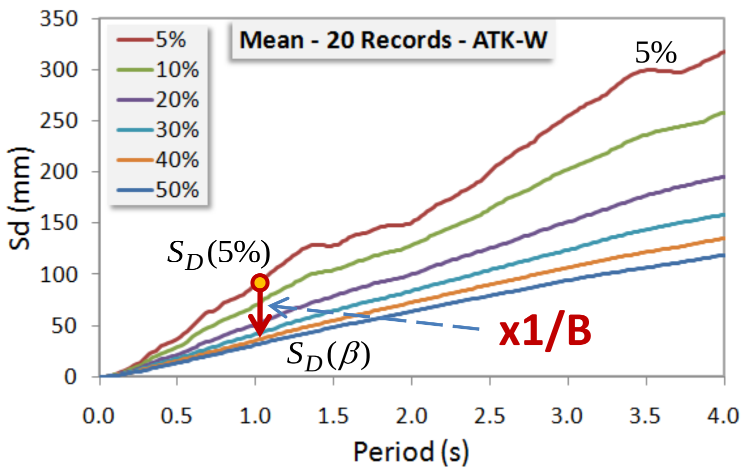
20 Artificial **ENA**
(Atkinson 2009 and McGuire 2001)
Montreal – 2% in 50 years
Soil Class C



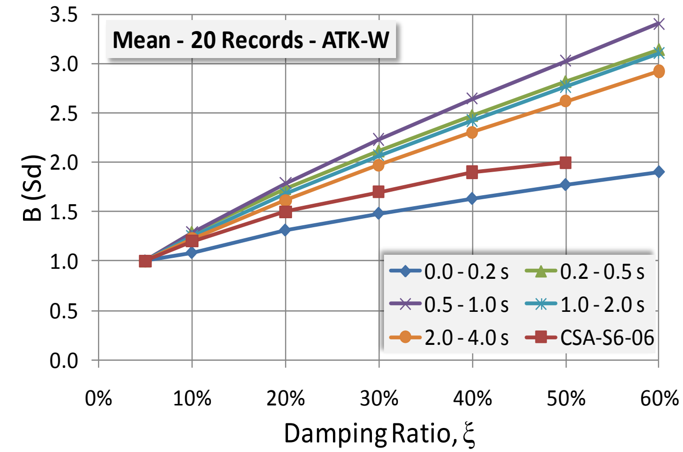
Seismic risk in Canada (Geological Survey of Canada)

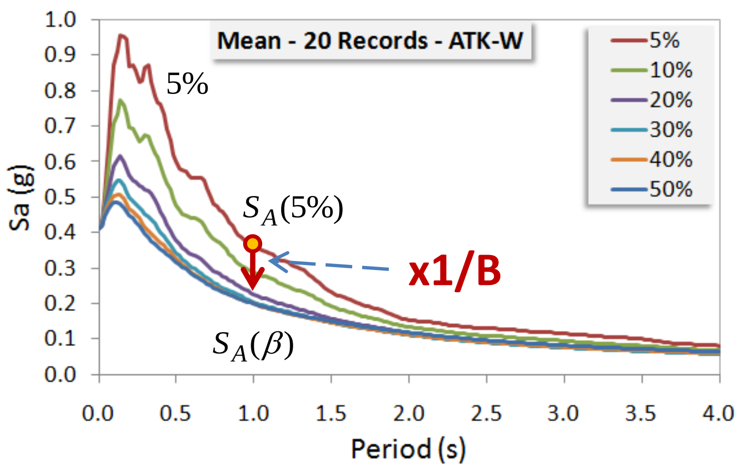
B-factor study - Linear SDOF System - WNA

WNA S_D and S_A Spectra computed from TH analyses of 20 Artificial records (Atkinson 2009) using direct integration method – scaled to Vancouver – 2% in 50 years - Soil Class C

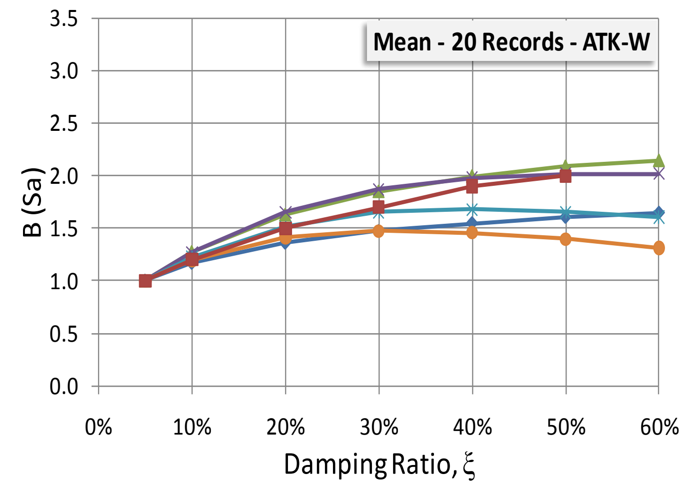


$$B(S_D) = \frac{S_D(5\%)}{S_D(\beta)}$$





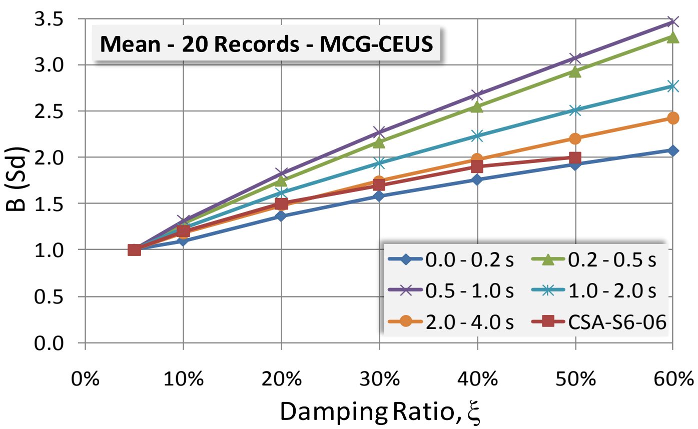
$$B(S_A) = \frac{S_A(5\%)}{S_A(\beta)}$$

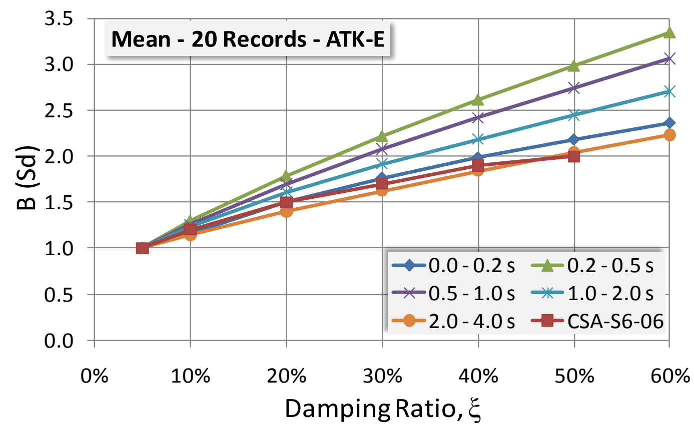


B-factor study - Linear SDOF System - ENA

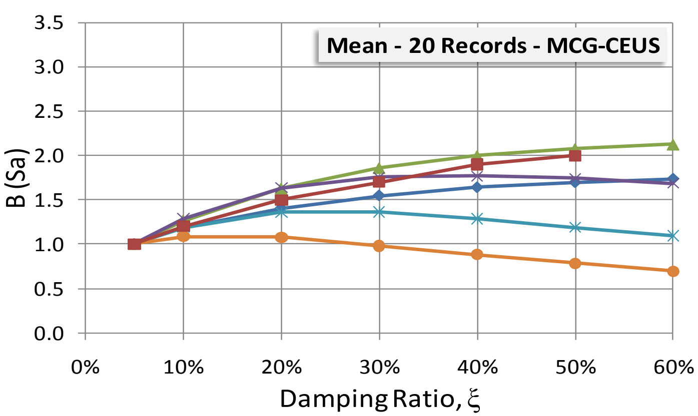
ENA S_A and S_D Spectra computed from TH analyses of 20 Artificial records (Atkinson 2009) and 20 CEUS Hybrid records (McGuire 2001) using direct integration method – linearly scaled to Vancouver – 2% in 50 years - Soil Class C

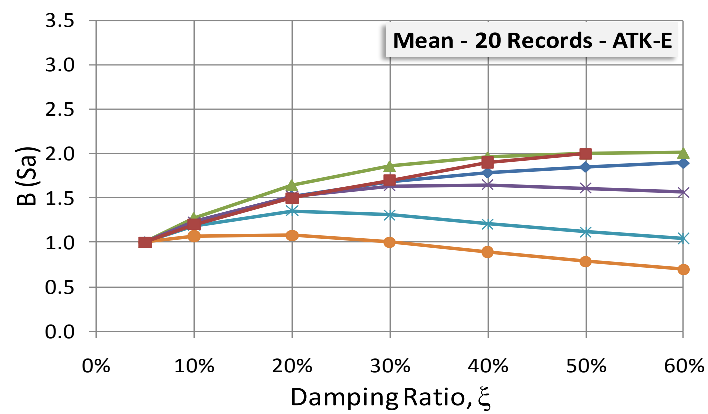
$$B(S_D) = \frac{S_D(5\%)}{S_D(\beta)}$$





$$B(S_A) = \frac{S_A(5\%)}{S_A(\beta)}$$





B-factor study - Linear SDOF Systems
Summary of Findings

- Different values of B obtained for acceleration and displacement spectra

- In general values obtained for displacement spectra were more consistent with values proposed by the code prescribed B-factors and resulted in more accurate and conservative isolator displacements

- Values of B obtained using the acceleration spectra resulted in unconservative predictions of the peak displacement when they were used in conjunction with the pseudo-displacement transformation (current codified methodology)

- Highlights a need to either adjust the B factors based on the acceleration spectrum or to define actual displacement spectra in the code

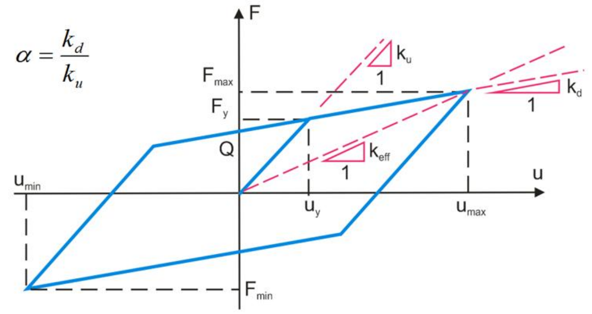
- Period dependency of B-factor (well known phenomenon) is also confirmed

- Bigger dependency on the period range and high damping values (beyond 30%) for ENA records

B-factor study - Nonlinear SDOF System – WNA and ENA

Structural parameters for nonlinear time-history analyses (12 000 analyses)

Analysis Parameter	Western NA	Eastern NA
Strength Reduction Factor	R=[4, 16, 28, 40, 52]	R=[4, 16, 28, 40, 52]
Elastic Period	T_e=[0.25, 0.5, 0.75, 1.0 s]	T_e=[0.25, 0.5, 0.75 1.0 s]
Inherent Damping Ratio	$\xi(T_e)$= [0%, 2%, 5%]; $\xi(T_{eff})$=[2%, 5%]	$\xi(T_e)$= [0%, 2%, 5%]; $\xi(T_{eff})$=[2%, 5%]
Post-Yield Stiffness Ratio	α=[0.01, 0.05, 0.1]	α=[0.01, 0.05, 0.1]
Ground-motions Records	20 Atkinson's 2009 (ATK-W)	20 Atkinson's 2009 (ATK-E)



Methodology of B-Factor Assessment:

1) Compute "exact" displacement response, D using step-by-step direct integration method

2) Determine effective period, T_{eff} :

$$T_{eff} = T_e \cdot \sqrt{\frac{\mu}{1+\alpha\mu-\alpha}}$$

3) Determine effective equivalent damping ratio, β_{eff} :

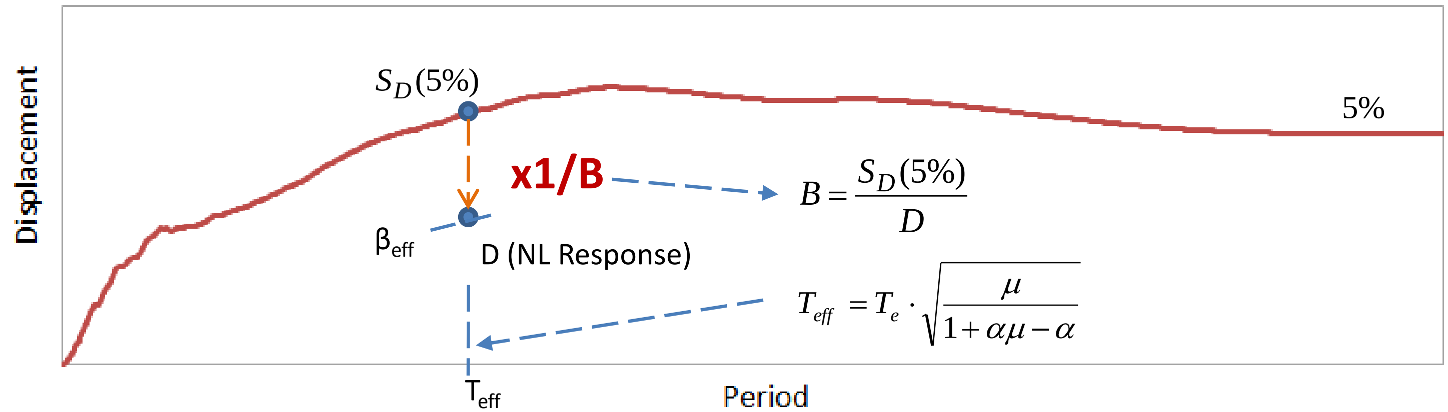
$$\beta_{eff} = \frac{W_D}{2\pi K_{eff} D^2} + \beta_v \frac{T_{eff}}{T_e} + \beta_{inh-eff}$$

4) Compute "exact" B by dividing 5% damped spectral displacement, $S_D(5\%, T_{eff})$ at effective period, T_{eff} by NL "exact" displacement response D.

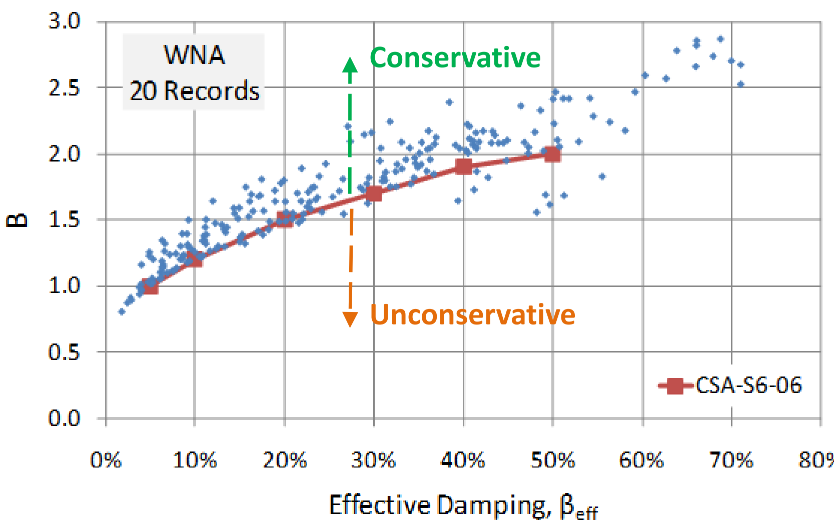
5) Compare "exact" B-factors at effective damping , β_{eff} to B specified in CSA-S6-06 code

B-factor study - Nonlinear SDOF System – WNA and ENA

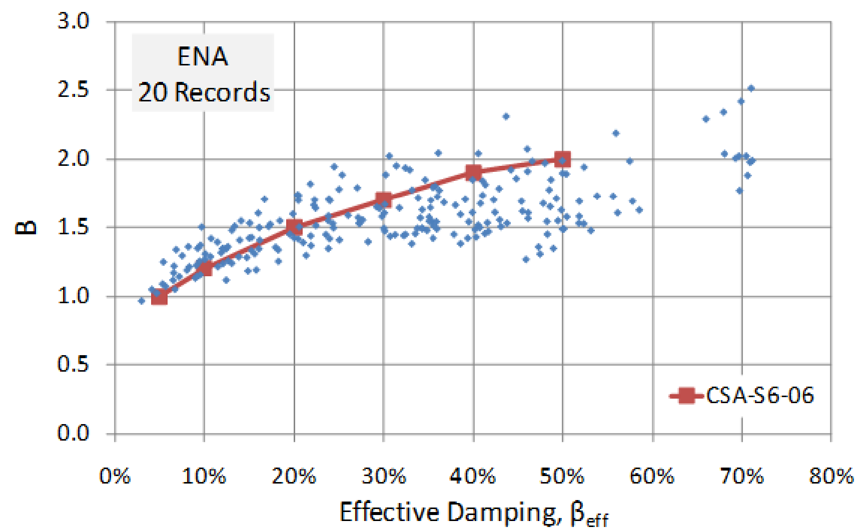
B-factor assessment from 5% damped displacement spectra – 2% in 50 years – Soil Class C



Nonlinear SDOF - WNA



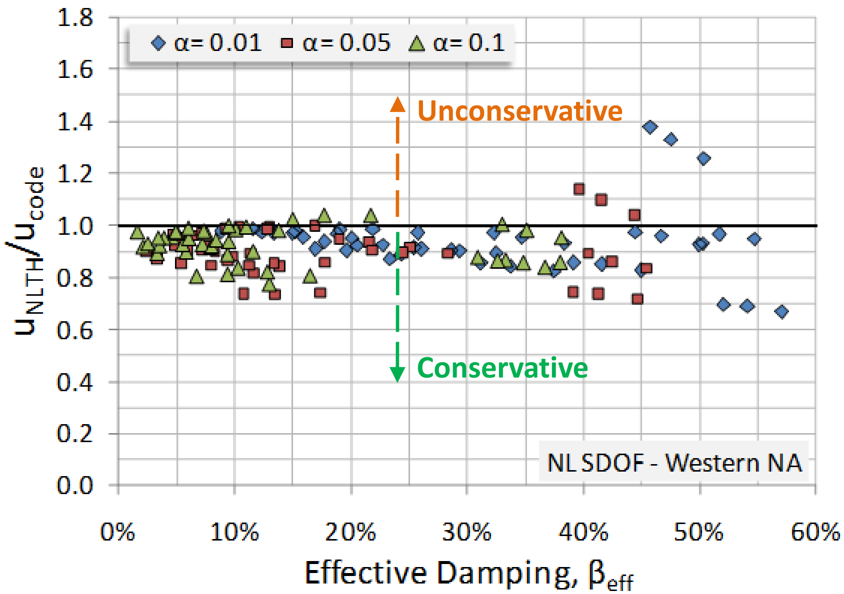
Nonlinear SDOF - ENA



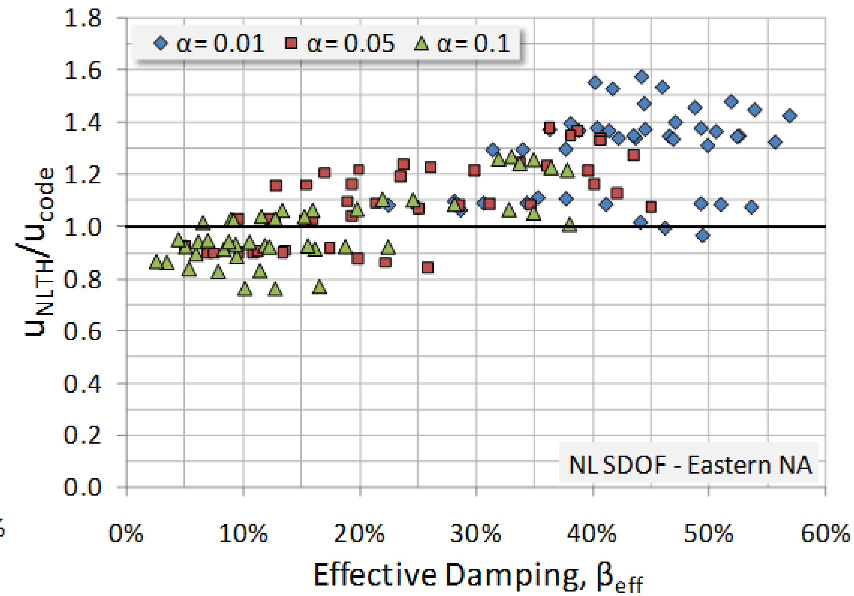
B-factor study

Comparison of NL "exact" to the response obtained by simplified method

Nonlinear SDOF - WNA



Nonlinear SDOF - ENA



- The reduction effect of equivalent damping is lower under ENA records when compared to WNA records.
- The use of the damping coefficients specified in the current CSA-S6 code results in safe designs for WNA but leads to underestimated displacement demands under ENA ground motions.
- New B-factors required for ENA.

Proposed modification B-factor study

The third edition of the AASHTO Guide Specification for Seismic Isolation Design (2010) proposes the following exponential equation for the value of *B* as a function of β_{eff} which no longer requires interpolation between tabulated values:

$$B_{Alt} = \left(\frac{\beta_{eff}}{5\%}\right)^{0.3}$$

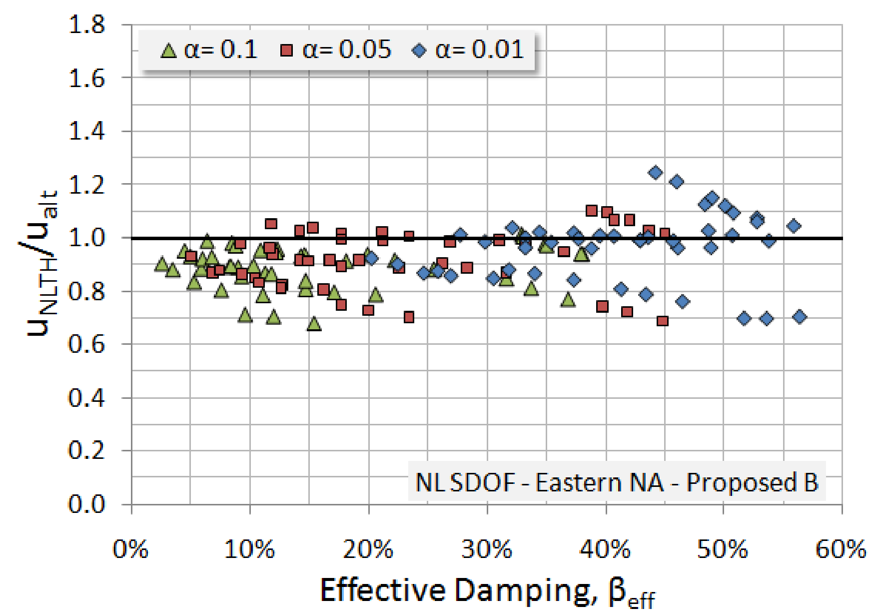
This B-function describes closely B-coefficients tabulated in current CSA-S6-06 code (generally conservative prediction for WNA)

An equation as currently used in AASHTO specification could be used with different exponents for WNA and ENA locations to compute the B factors in future edition of the CSA S6 code in Canada.

Nonlinear SDOF – ENA

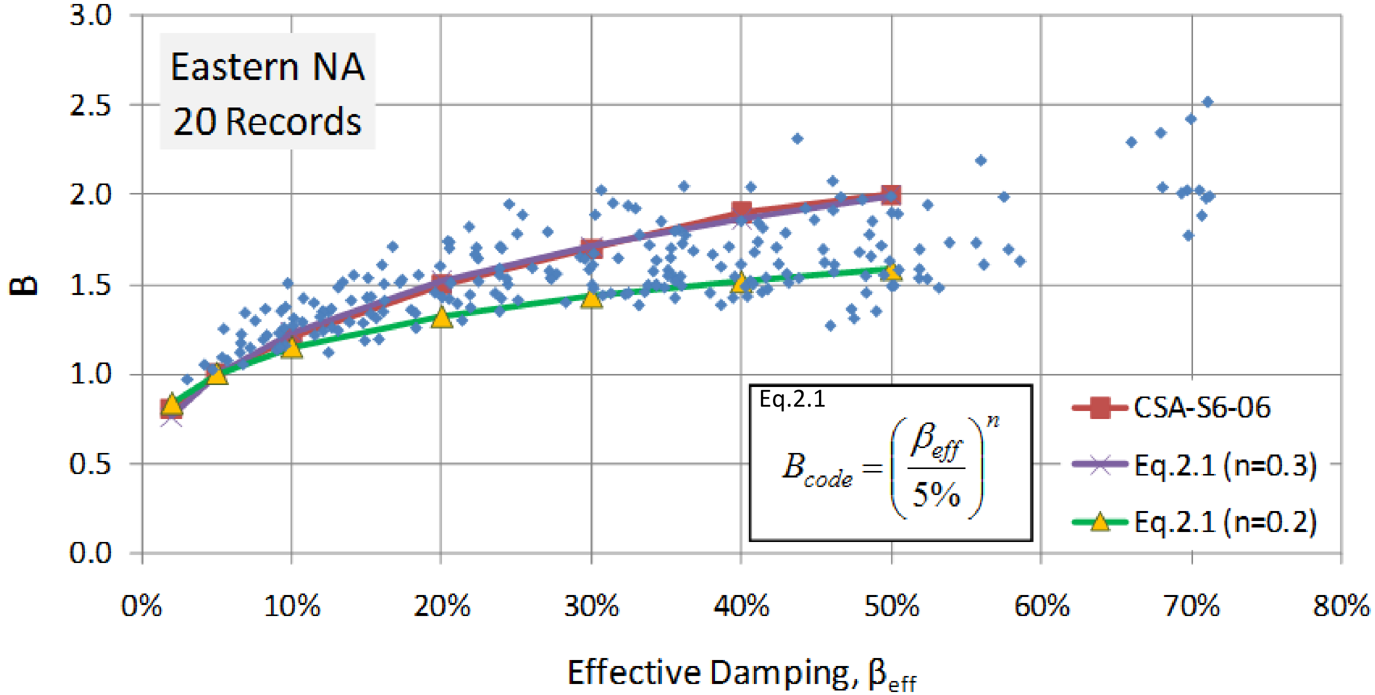
(Comparison NL "exact" to Simplified Code Method using proposed B-function for ENA)

$$\boxed{B_{Alt} = \left(\frac{\beta_{eff}}{5\%}\right)^{0.2}}$$



Essentially conservative prediction by using exponent n=0.2 for ENA

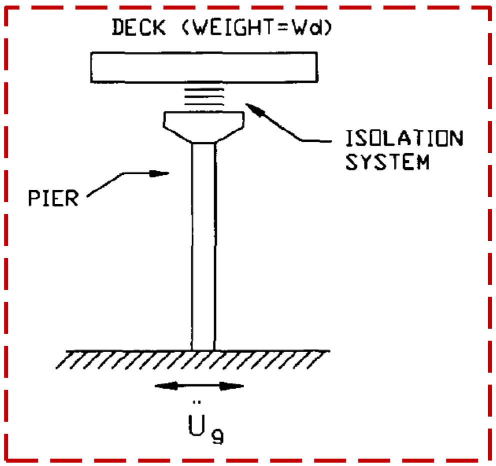
B-factor study - Proposed modification ENA



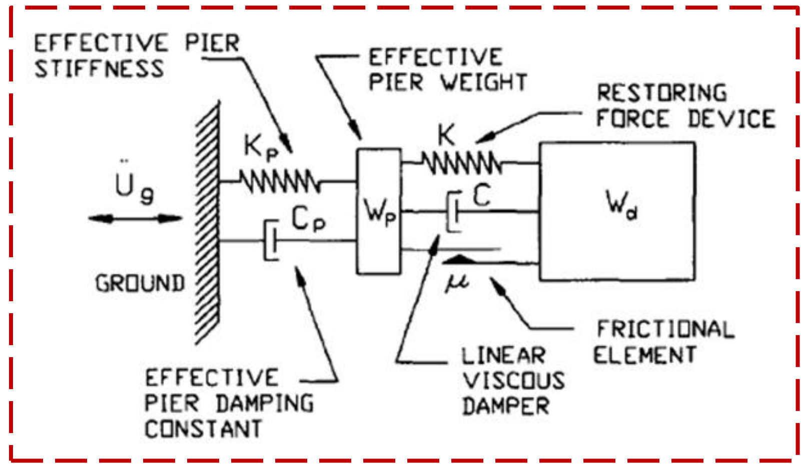
- A safe (lower bound) prediction for ENA can be obtained using the proposed equation with an exponent $n = 0.2$
- This equation could be incorporated in future edition of CSA-S6 for the *B*-coefficients with two different exponent values: $n = 0.2$ for ENA and $n = 0.3$ for WNA

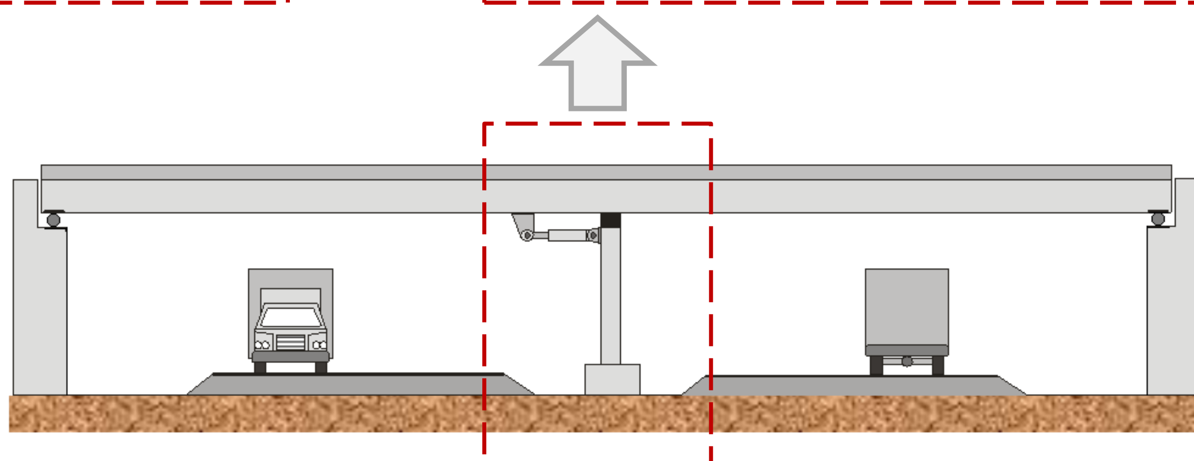
Analytical Model for optimizing combinations of damping and isolation

Isolated Bridge (Tsopelas et al. 1996)



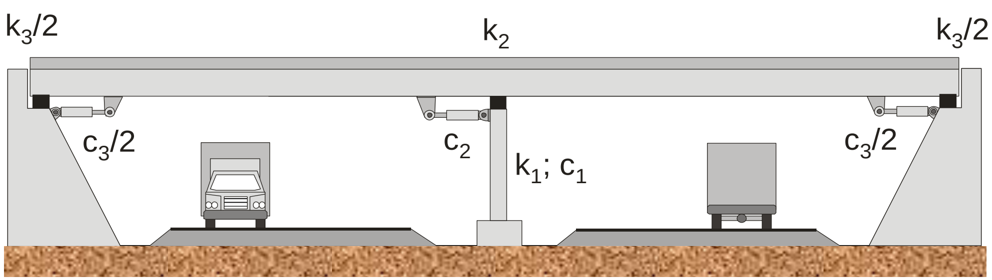
Analytical Tool – Model Assumptions



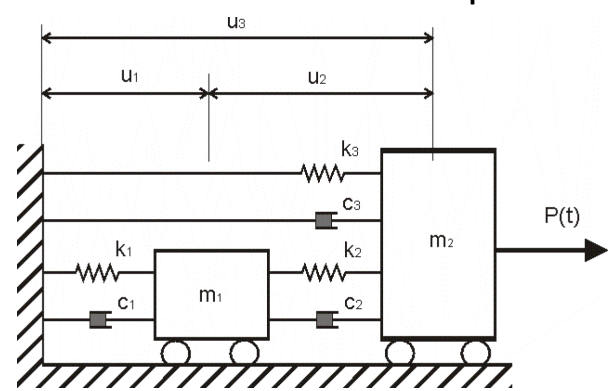


Analytical Model for optimizing combinations of damping and isolation

Isolated Bridge - Modeling



Analytical Tool – Model Assumptions



$$[M]\{\ddot{x}\}+[C]\{\dot{x}\}+[K]\{x\}=-[M]\{\ddot{x}_g\}$$

$$[M]=\begin{bmatrix} m_1 & 0 \\ 0 & m_2 \end{bmatrix}$$

$$[C]=\begin{bmatrix} (c_1+c_2) & -c_2 \\ -c_2 & (c_2+c_3) \end{bmatrix}$$

$$[K]=\begin{bmatrix} (k_1+k_2) & -k_2 \\ -k_2 & (k_2+k_3) \end{bmatrix}$$

$$\{F_I\}+\{F_D\}+\{F_S\}=\{P\}$$

$$\{F_I\}=\begin{bmatrix} m_1 & 0 \\ 0 & m_2 \end{bmatrix}\begin{Bmatrix} \ddot{x}_1 \\ \ddot{x}_2 \end{Bmatrix}=\begin{Bmatrix} m_1\ddot{x}_1 \\ m_2\ddot{x}_2 \end{Bmatrix}$$

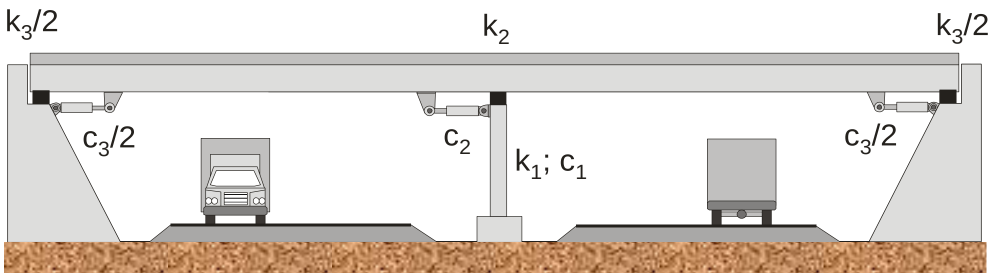
$$\{F_D\}=\begin{Bmatrix} c_1\dot{x}_1-c_2(\dot{x}_2-\dot{x}_1) \\ c_2(\dot{x}_2-\dot{x}_1)+c_3\dot{x}_2 \end{Bmatrix}=\begin{Bmatrix} F_{D1}-F_{D2} \\ F_{D2}+F_{D3} \end{Bmatrix}$$

$$\{F_S\}=\begin{Bmatrix} k_1x_1-k_2(x_2-x_1) \\ k_2(x_2-x_1)+k_3x_2 \end{Bmatrix}=\begin{Bmatrix} F_{S1}-F_{S2} \\ F_{S2}+F_{S3} \end{Bmatrix}$$

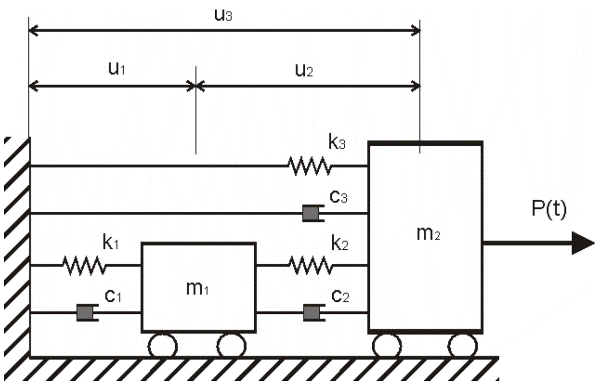
$$\begin{Bmatrix} F_{I1} \\ F_{I2} \end{Bmatrix}+\begin{Bmatrix} F_{D1}-F_{D2} \\ F_{D2}+F_{D3} \end{Bmatrix}+\begin{Bmatrix} F_{S1}-F_{S2} \\ F_{S2}+F_{S3} \end{Bmatrix}=\begin{Bmatrix} -m_1\ddot{x}_g \\ -m_2\ddot{x}_g \end{Bmatrix}$$

Analytical Model for optimizing combinations of damping and isolation

Isolated Bridge - Modeling



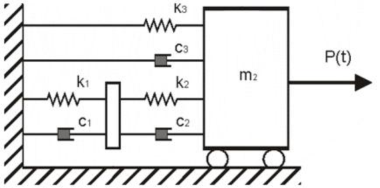
Analytical Tool – Model Assumptions



$$m_1\ddot{x}_1 + c_1\dot{x}_1 - c_2(\dot{x}_2 - \dot{x}_1) + k_1x_1 - k_2(x_2 - x_1) = -m_1\ddot{x}_g$$
$$m_2\ddot{x}_2 + c_2(\dot{x}_2 - \dot{x}_1) + c_3\dot{x}_2 + k_2(x_2 - x_1) + k_3x_2 = -m_2\ddot{x}_g$$

Assuming $m_1 = 0$:

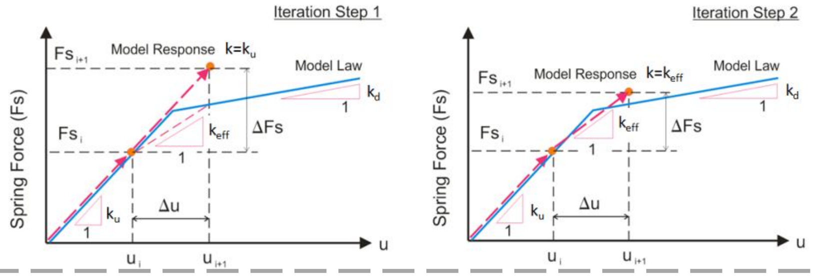
$$c_1\dot{x}_1 - c_2(\dot{x}_2 - \dot{x}_1) + k_1x_1 - k_2(x_2 - x_1) = 0$$
$$m_2\ddot{x}_2 + c_2(\dot{x}_2 - \dot{x}_1) + c_3\dot{x}_2 + k_2(x_2 - x_1) + k_3x_2 = -m_2\ddot{x}_g$$



Nonlinear viscous damping : $F_D = sign(\dot{u})\, c\, |\dot{u}|^n$

$$c_1\dot{u}_1 - c_2 sign(\dot{u}_2)\,|\dot{u}_2|^{n2} + k_1u_1 - k_2u_2 = 0$$
$$m_2(\ddot{u}_3 + \ddot{u}_g) + c_2 sign(\dot{u}_2)\,|\dot{u}_2|^{n2} + c_3 sign(\dot{u}_3)\,|\dot{u}_3|^{n3} + k_2u_2 + k_3u_3 = 0$$

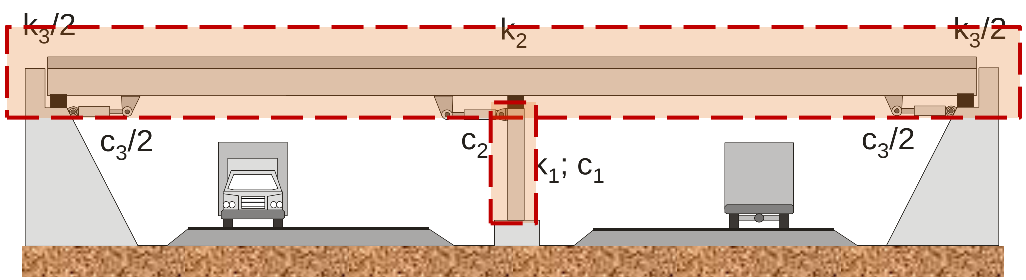
Nonlinear stiffness $k=f(k_u,k_d)$:



Analytical Model for optimizing combinations of damping and isolation - Assumptions

Isolated Bridge - Modeling

Analytical Tool – Fully Nonlinear Model

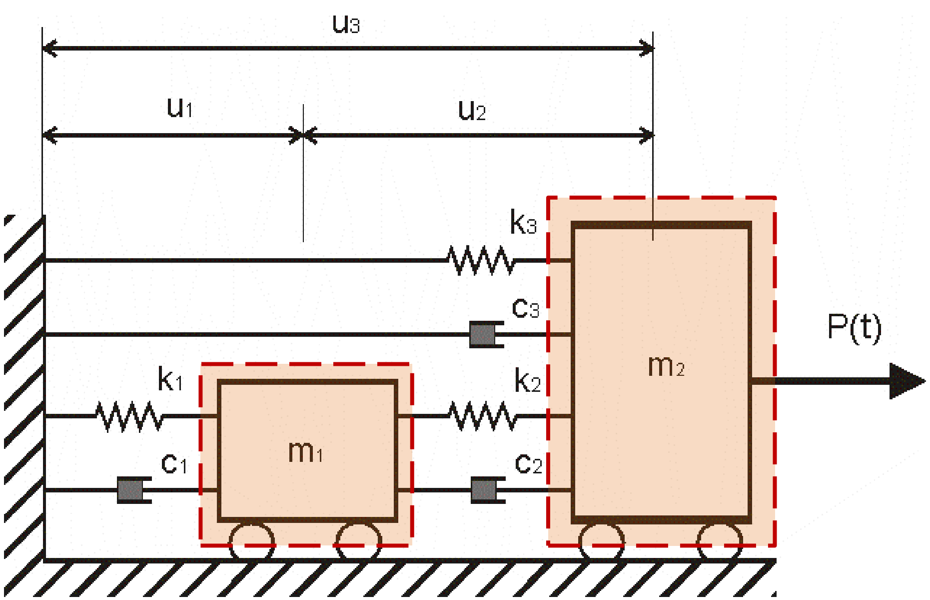


Deck Mass:

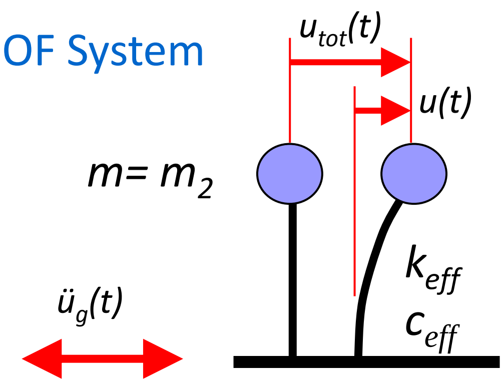
$m_2 \neq 0$

Pier Mass:

$m_1 \cong 0$



SDOF System

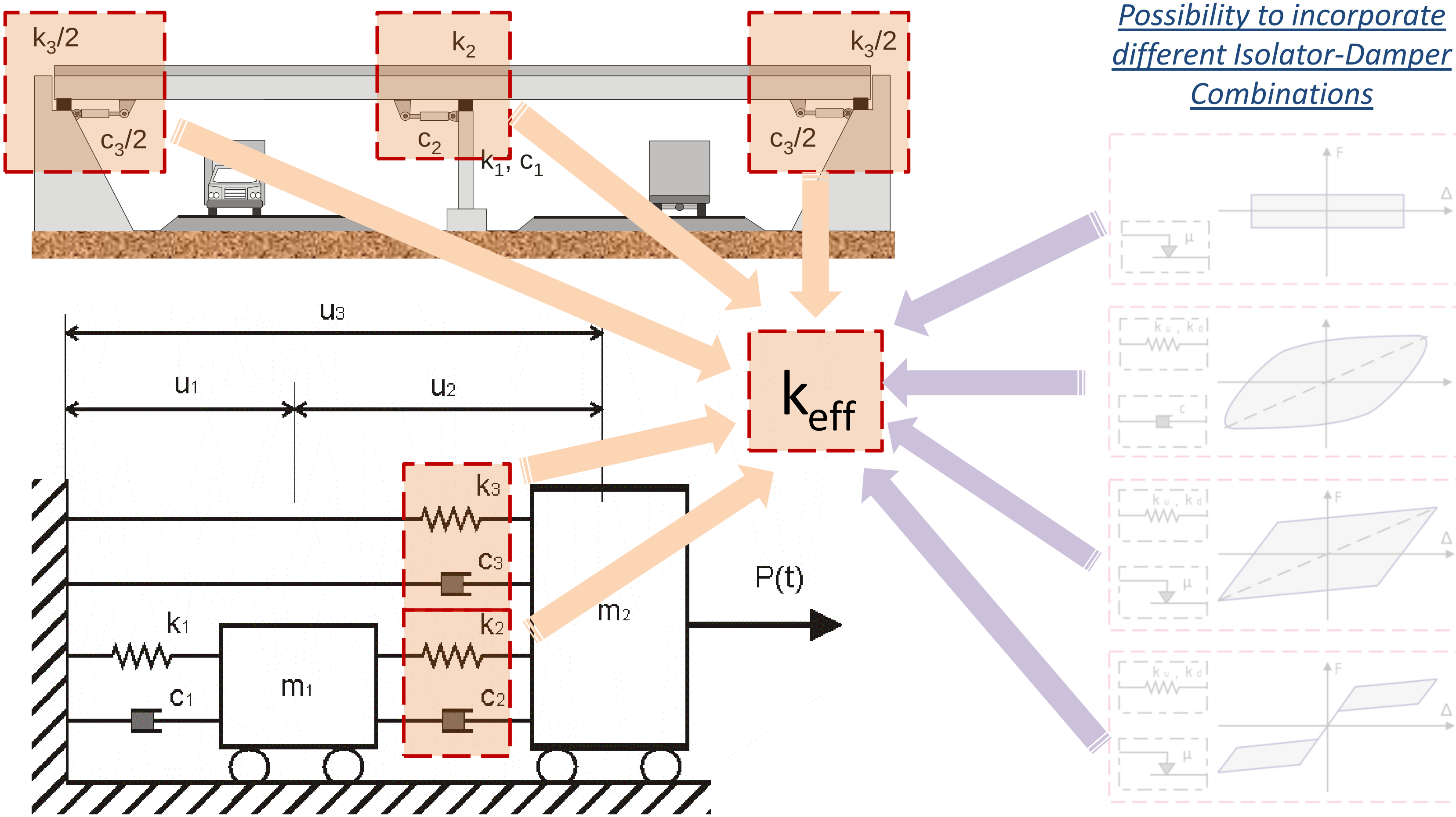


Analytical Model for optimizing combinations of damping and isolation – Effective Stiffness

Isolated Bridge - Modeling

Analytical Tool – Model Assumptions

Possibility to incorporate different Isolator-Damper Combinations

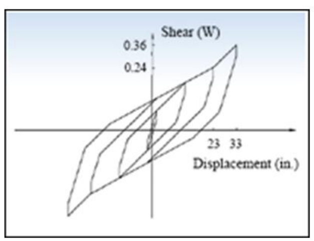


Two-stage (Multi-Stage) Isolation Approach for Optimal Multi-Level Hazard Mitigation

Sequential Activation of Isolation and Damping Devices

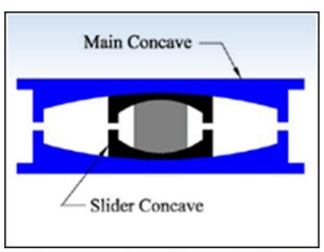
Isolation

Device Level



Triple Pendulum Force Displacement Hysteretic Loop

http://www.earthquakeprotection.com/triple_vs_single_pendulum.html



Cross Section of Triple Pendulum™ Bearing

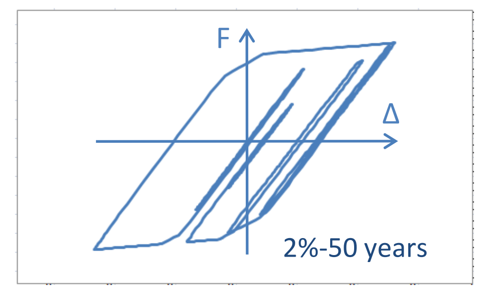
http://www.earthquakeprotection.com/triple_pendulum_bearing.html

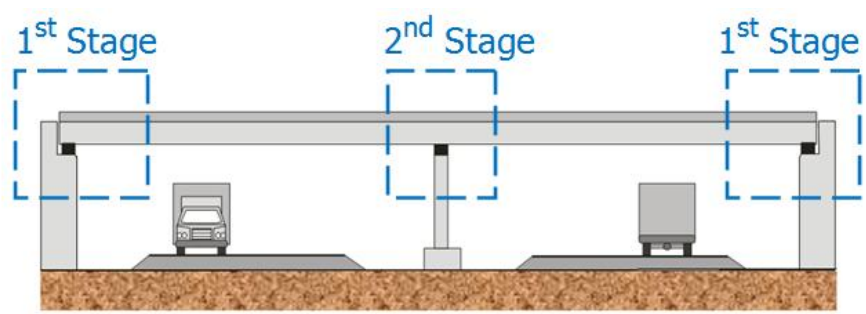
Isolation and Damping Devices

System Level (can accommodate a large range of seismic hazards through multiple combinations of isolation and damping devices)

Stage I - *(10% in 50 year hazard)* :
- No bridge damage
- Pier elastic

Stage II - *(2% in 50 year hazard)* :
- Minor damage (i.e. Expansion Joints)
- Pier elastic





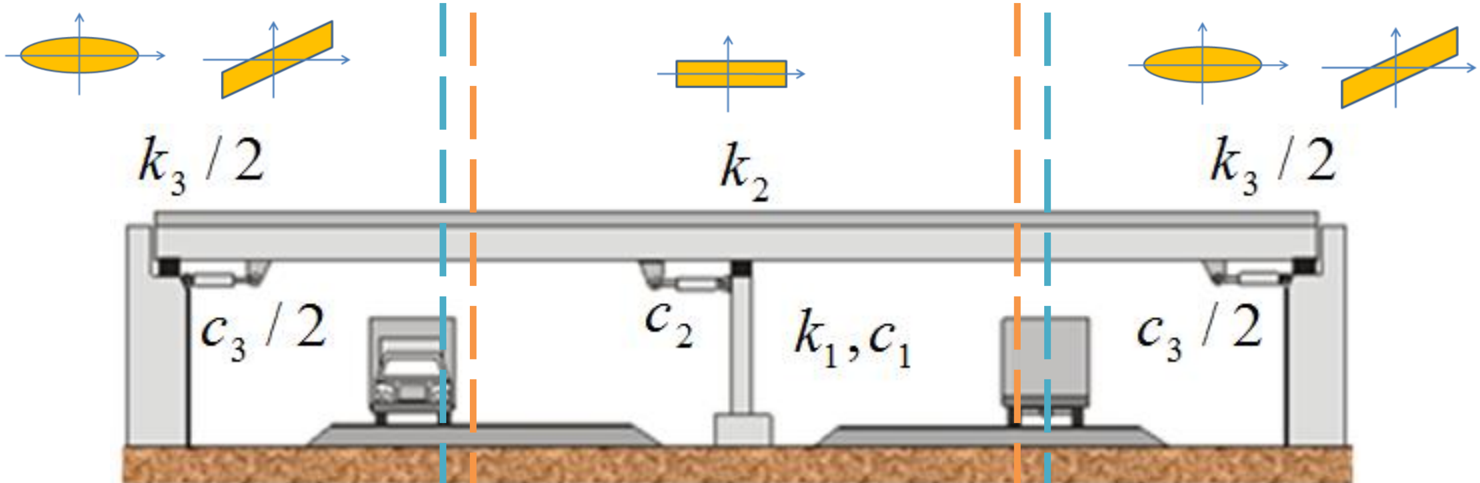
Maintaining pier elastic is very beneficial approach for existing bridges

Two-Stage (Multi-Stage) Isolation Approach for Optimal Multi-Level Hazard Mitigation

Restoring Force
$\alpha \neq 0$

Sliding
$\alpha = 0$

Restoring Force
$\alpha \neq 0$



Stage I
****Displacement-dependant and velocity-dependant Energy dissipation (10%-50 years Event)**

Stage II
*** Displacement-dependant Energy dissipation (2%-50 years Event)**

Stage I
****Displacement-dependant and velocity-dependant Energy dissipation (10%-50 years Event)**

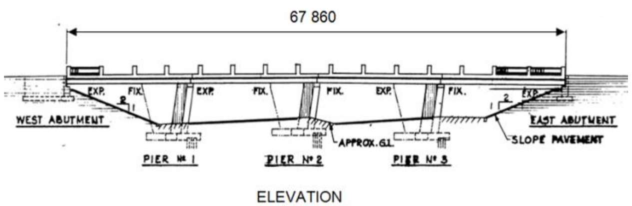
* at the level of pier because of limited ductility and force displacement capacity
** at the level of abutment more capacity to accommodate the maximum (damper + isolator) forces

Case studies in Quebec and Vancouver - Isolation only

Thousands of existing bridges over North America

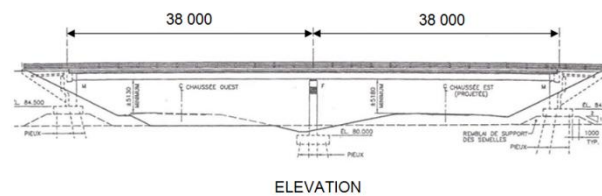
Bridge Prototype in Vancouver (4 spans – 68 m)

Photo (Google Maps - ©2012 Google)

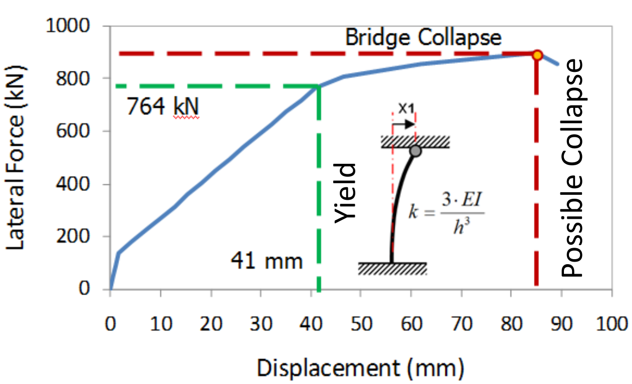


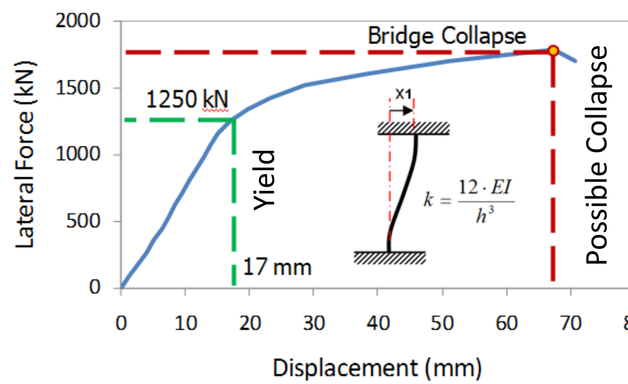
Bridge Prototype in Montreal (2 spans – 76 m)

Photo (Google Maps - ©2012 Google)

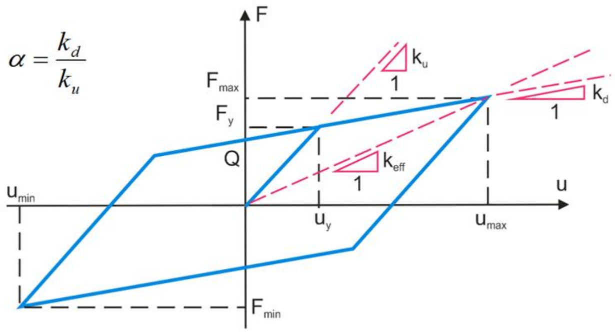


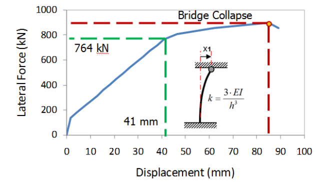
Pier Capacity computed with Response 2000





Properties of Bridges in Quebec and Vancouver





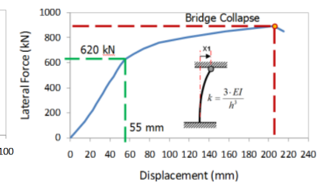


Table 1 - Original Non-isolated Bridge Properties

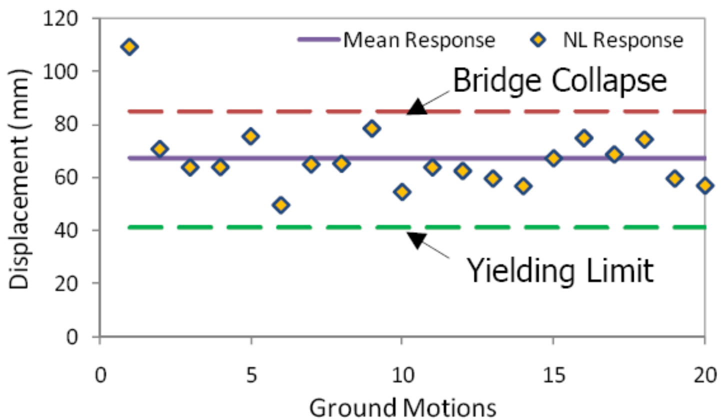
Model Parameters	Bridge in BC Vancouver	Bridge in Qc Montreal
$T_{elastic}$	0.82 s	1.04 s
Superstructure	Rigid Diaphragm	Rigid Diaphragm
k_{deck}	Infinite	Infinite
m_2	954 000 kg	2 100 000 kg
Substructure	3 Piers (fixed-pinned)	1 Pier (fixed-fixed)
m_1	0 kg	0 kg
k_{u1}	55 814 kN/m	76 600 kN/m
k_{d1}	5 821 kN/m	10 614 kN/m
F_y	764 kN (1 Pier)	1250 kN
u_y	41 mm	17 mm
u_u	85 mm	67 mm
β_{inh}	5%	5%

Table 2 - Isolated Bridge Properties

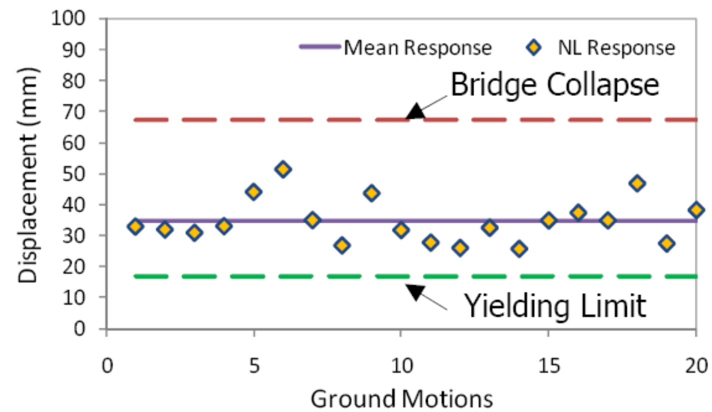
Model Parameters	Bridge in BC Vancouver	Bridge in Qc Montreal
$T_{elastic}$	0.70 s	1.24 s
Piers	3 Piers (fixed-pinned)	1 Pier (fixed-pinned)
m_1	0 kg	0 kg
k_{u1}	55 814 kN/m	11 273 kN/m
F_y	764 kN (1 Pier)	620 kN
u_y	41 mm	55 mm
u_u	85 mm	205 mm
β_{inh}	5%	5%
IS on Pier	24 Isolators	2 Isolators
k_{u2}	55 814 kN/m	100 000 kN/m
k_{d2}/k_{u2}	0	0
Q_{d2}	2106 kN (30%)	515 kN (5%)
IS on Abutment	12 Isolators	4 Isolators
k_{u3}	48 000 kN/m	44 000 kN/m
k_{d3}/k_{u3}	0.1	0.1
Q_{d3}	780 kN	880 kN

Response of Non-isolated Bridges

NL TH responses for non-isolated Bridge in Vancouver
(20 artificial records - 2%-50 years)



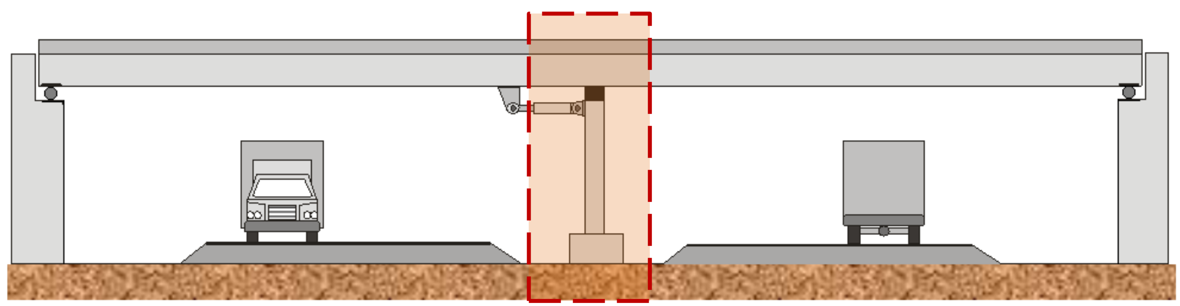
NL TH responses for non-isolated Bridge in Montreal
(20 artificial records - 2%-50 years)



Ductility demand μ=1.64 (BC)

Ductility demand μ=2.05 (QC)

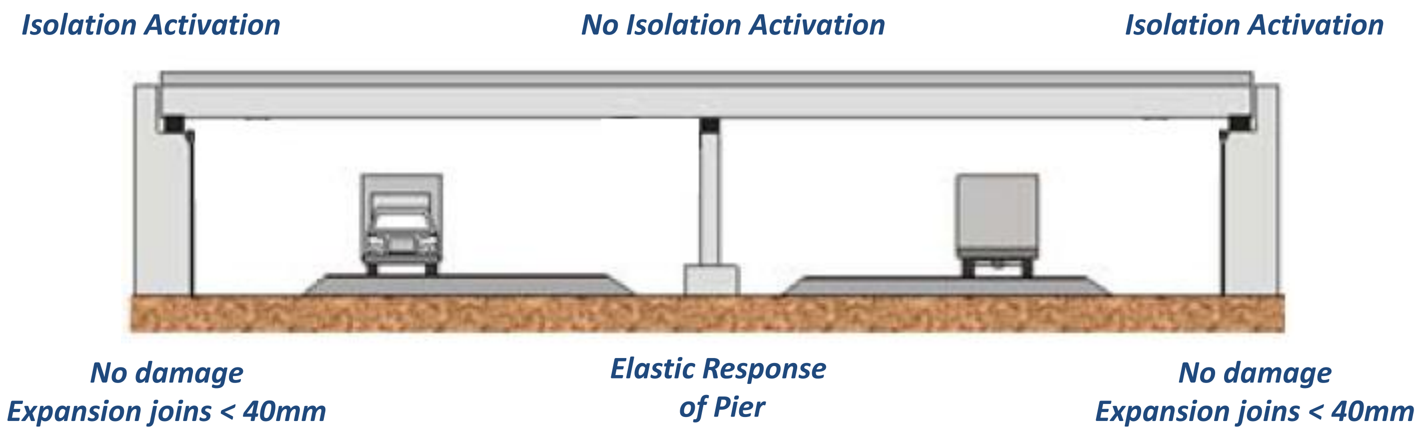
Damaged Pier



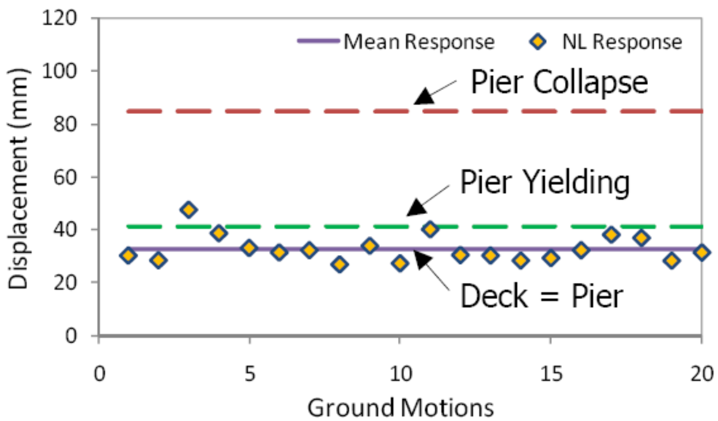
Non-isolated bridge

Response of Isolated Bridges - Stage I Activation

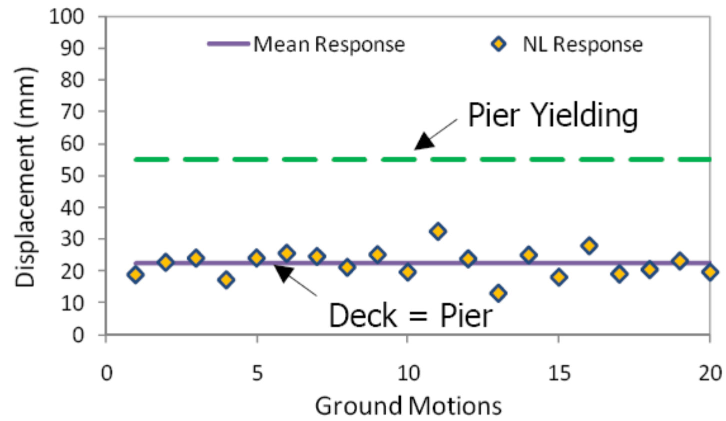
Performance objectives – Stage I – 10% in 50 years



NL TH responses for isolated Bridge in Vancouver
(20 artificial records - 10%-50 years)



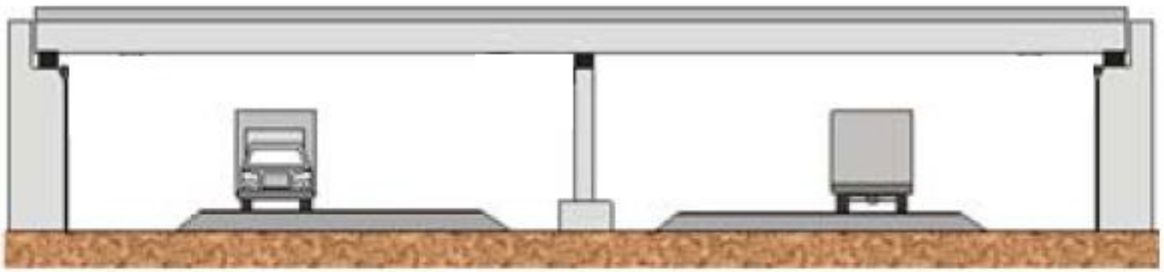
NL TH responses for isolated Bridge in Montreal
(20 artificial records - 10%-50 years)



Response of Isolated Bridges - Stage II Activation

Performance objectives – Stage II – 2% in 50 years

Isolation Activation *Isolation Activation* *Isolation Activation*

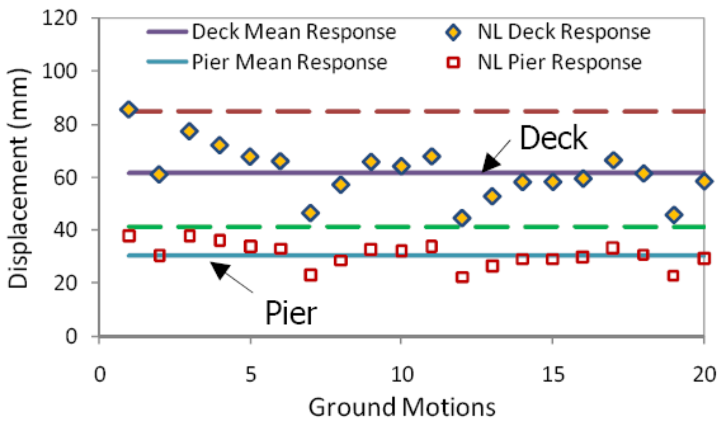


Minor damage
Expansion joins > 40mm

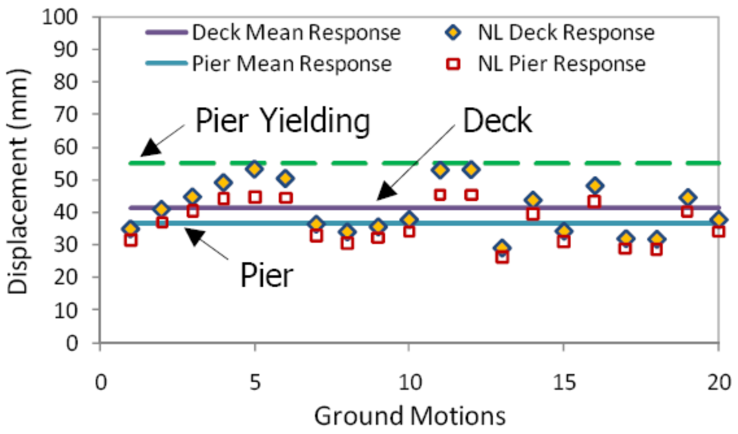
Elastic Response of Pier

Minor damage
Expansion joins > 40mm

NL TH responses for isolated Bridge in Vancouver
(20 artificial records - 2%-50 years)



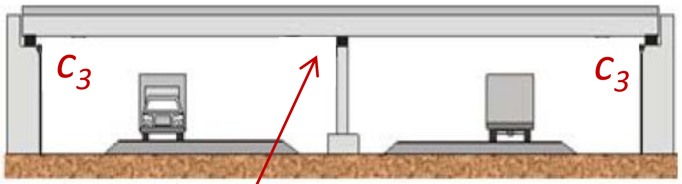
NL TH responses for isolated Bridge in Montreal
(20 artificial records - 2%-50 years)



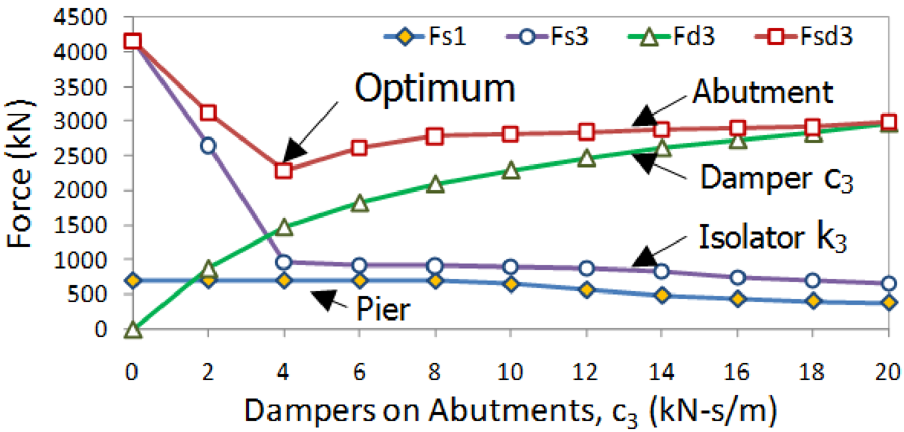
Alternate Design of Vancouver Isolated Bridge with Viscous Damper c_3 on Abutment

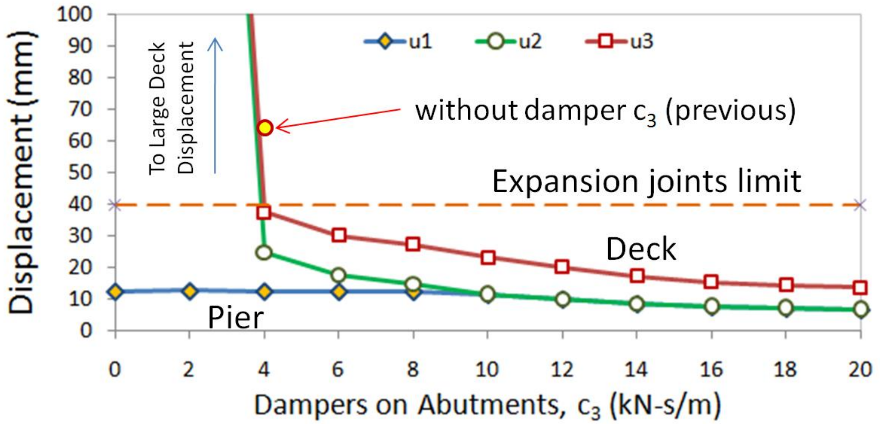
Example of Optimizing Design Process using Damper c_3 - Bridge in Vancouver

- *Re-design isolator on piers (significantly lower activation force)*
- *Variation of damping constant c_3 that reduces both pier and deck displacement while minimizing the maximum (damper + isolator) applied force on the abutment*



Significant reduction of activation force on pier





Acknowledgement

- The British Columbia Ministry of Transportation and Infrastructure
- The Ministère des Transports du Québec
- The Natural Sciences and Engineering Research Council of Canada (NSERC)
- The Fonds de recherche du Québec sur la nature et les technologies (FQRNT)

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Funded by NSERC / *Subventionné par le CRSNG*

Conclusions

1. Directivity effect has significant impact on the structural response of bridges and should be considered in the design process.
2. The use of damping coefficients given in the current CSA-S6 and AASHTO codes results in safe conservative designs for WNA. An equation similar to the one proposed by AASHTO but with different exponents for WNA and ENA could be implemented to compute the B factors in future revisions of the S6 bridge code in Canada.
3. Proposed Analytical Tool makes it possible to predict the response of Isolated Bridge under seismic demand. The model allows engineers to determine optimal seismic protection solution through an iterative optimization design process requiring minimum computational effort.
4. Optimal solution consisted in a two-stage seismic protection system involving sequential activation of the isolation and supplemental damping devices is proposed.
5. For the bridge examples examined in this study, the optimal solution consisted in a two-stage seismic protection system that prioritizes activation of protective systems installed on the abutments rather than on the piers. A fuse-type isolation system without restoring force was selected to protect the bridge piers.

Isolation System for Short and Medium Span Typical Bridges

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Funded by NSERC / Subventionné par le CRSNG

Joint CSRN-NEES Workshop on the Seismic Isolation and Damping of Bridge Structures
UBC – 30 April 2012

Frederic Legeron, PE, PhD
Professor, department of Civil Engineering
Université de Sherbrooke
Chair HQT/RTE – NSERC Canadian Seismic Research Network

UNIVERSITÉ DE SHERBROOKE

Objectives of the research team

- Develop the interest of isolation for standard bridges in new structures and existing retrofits
  - Develop experimental capacity
  - Evaluate design method and analysis requirements in relation with S6
  - Propose optimal solutions for typical bridges that would reduce seismic force and control displacement
  - Help designer with isolation/damping systems

Develop experimental capacity

Develop experimental capacity

- Objectives:
  - Better understand and characterize the behavior of isolators and dampers and relations between theoretical characteristics and actual ones. Gap the distance between design parameters and real behavior
  - Provide a realistic testing bench for testing devices under real 3D movements (hybrid testing)

Develop experimental capacity

- Horizontal capacity:
  - 500 kN
  - Stroke 1000 mm
  - Up to 2.5 Hz
- Vertical capacity:
  - 6000 kN (static)
  - 3000 kN (fluctuating)
  - Adjust the force automatically to accommodate for vertical displacement
- Dimension of the device:
  - 1200x1800mm in plan

In service since 2010 (ver 1.0)

Tests on pendulum bearings and on elastomeric bearings

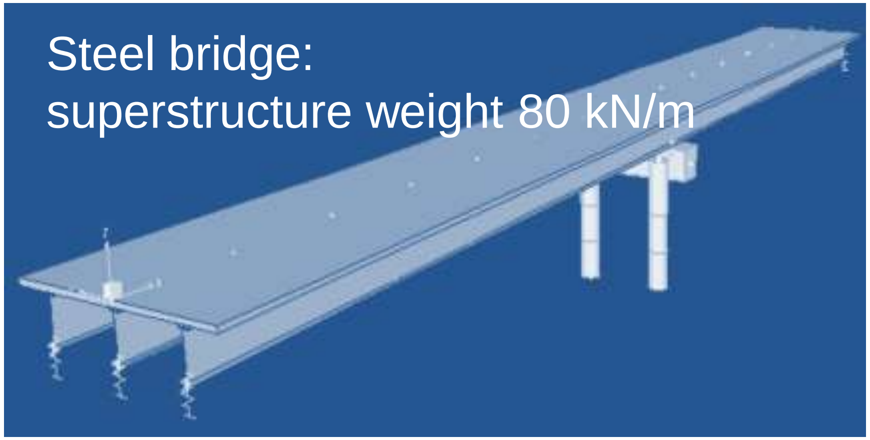
Evaluate design method and analysis

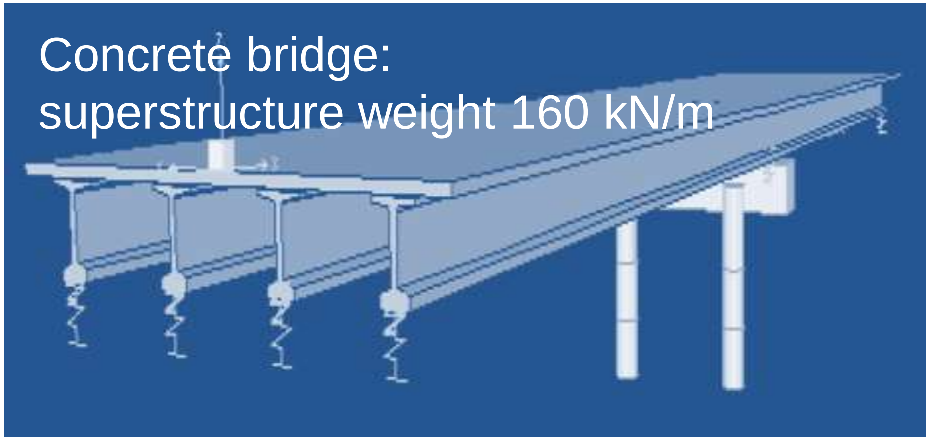
Evaluate design method and analysis

- Current S6 does not have a very detailled section on analysis:
  - Uniform load/Single-Mode analysis
  - Multimode spectral analysis
  - Time history analysis

Evaluate design method and analysis

- Parametric study to compare methods
  - Single mode spectral analysis (SMSA)
  - Multimode spectral analysis (MMSA)





Pier: 7 m in height

- 2-column bent with column diameter of 1100 mm with stiffness of 25 kN/mm
- 2-column bent with column diameter of 1600 mm with stiffness of 100 kN/mm
- A wall type pier 7200x1300mm with a stiffness of 250 kN/mm

Evaluate design method and analysis

- Type of isolator/dampers:
  - Elastomeric bearings with 5% damping
  - Lead code bearings with 10-15% damping
  - Friction pendulum bearings with 20-40% damping
  - Viscous dampers
  - Hysteretic dampers

Evaluate design method and analysis

CNB2005 – Site class type B – Steel bridge – Elastomeric bearings

	K = 25 kN/mm			K = 100 kN/mm			K = 250 kN/mm		
	SMSA	MMSA	Diff	SMSA	MMSA	Diff	SMSA	MMSA	Diff
Isolator displacement (mm)	19	21	6%	22	23	2%	23	23	1%
Column shear force (kN)	134	264	49%	154	449	66%	159	597	73%
Column bending moment (kN.m)	948	1621	41%	1094	2671	59%	1129	3354	66%

Evaluate design method and analysis

CNB2005 – Site class type B – Concrete bridge – Elastomeric bearings

	K = 25 kN/mm			K = 100 kN/mm			K = 250 kN/mm		
	SMSA	MMSA	Diff	SMSA	MMSA	Diff	SMSA	MMSA	Diff
Isolator displacement (mm)	17	17	4%	22	23	1%	24	24	1%
Column shear force (kN)	307	376	18%	413	588	30%	444	859	48%
Column bending moment (kN.m)	2182	2520	13%	2936	3796	23%	3151	5136	39%

Evaluate design method and analysis

CNB2005 – Site class type D – Concrete bridge – Elastomeric bearings

	K = 25 kN/mm			K = 100 kN/mm			K = 250 kN/mm		
	SMSA	MMSA	Diff	SMSA	MMSA	Diff	SMSA	MMSA	Diff
Isolator displacement (mm)	35	36	2%	48	48	1%	51	51	0%
Column shear force (kN)	653	747	13%	879	1076	18%	943	1427	34%
Column bending moment (kN.m)	4637	5123	9%	6238	7209	13%	6696	8950	25%

Evaluate design method and analysis

CNB2005 – Site class type D – Concrete bridge – Friction pendulum

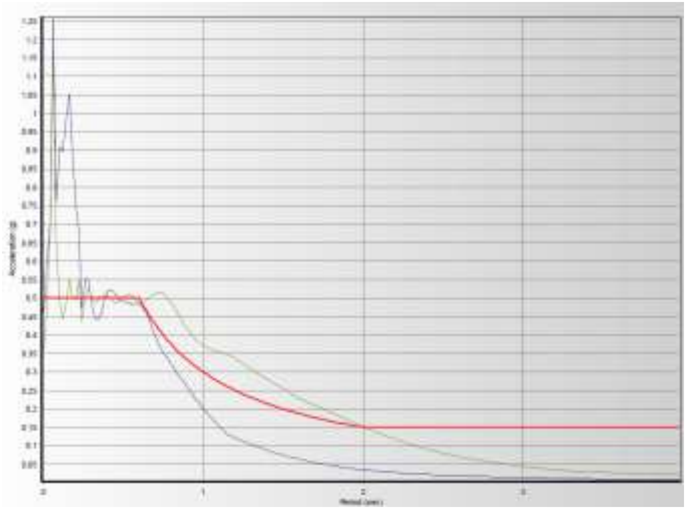
	K = 25 kN/mm			K = 100 kN/mm			K = 250 kN/mm		
	SMSA	MMSA	Diff	SMSA	MMSA	Diff	SMSA	MMSA	Diff
Isolator displacement (mm)	28	30	7%	36	37	0%	39	40	1%
Column shear force (kN)	408	516	21%	465	780	40%	485	1227	60%
Column bending moment (kN.m)	2895	3404	15%	3299	4888	33%	3447	7072	51%

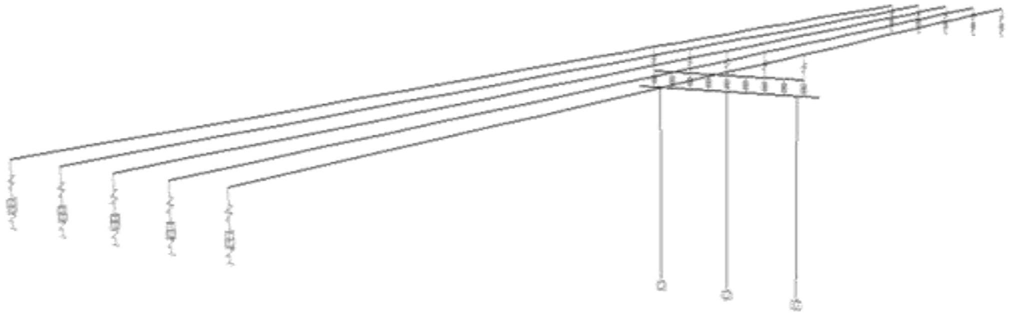
Evaluate design method and analysis

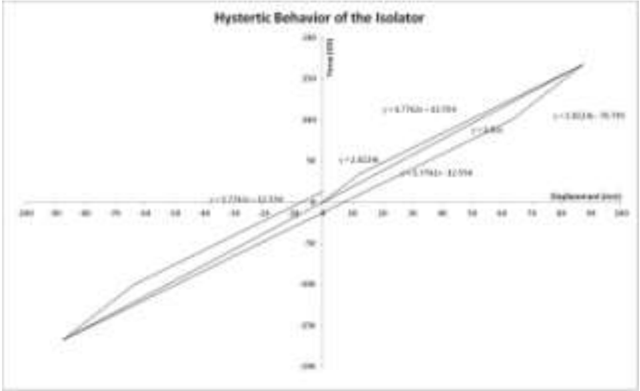
- Comparison between SMSA and MMSA
  - Displacement demand at the isolator:
    - SMSA is reasonable to estimate the displacement demand in most cases with difference less that 10% with CNB2005 spectra and site B and D
    - For heavy structures and for stiff piers, the SMSA method predict seismic forces significantly lower than the MMSA. An efficient way to reduce the difference is to use pier modes in a 2 mode simplified analysis

Evaluate design method and analysis

- When should we use time history analysis (THA)







Evaluate design method and analysis

- When should we use time history analysis (THA)

Analysis and structure type	Multimodal Spectral analysis		Time history analysis	
	Non-Isolated	Isolated	Non-Isolated	Isolated
Base shear (kN) in X-direction	10537	2297	10343	2228
Base shear (kN) in Y-direction	9637	2502	9016	2469

- Work just started, should be completed by the end of the year

Propose optimal solutions for typical bridges

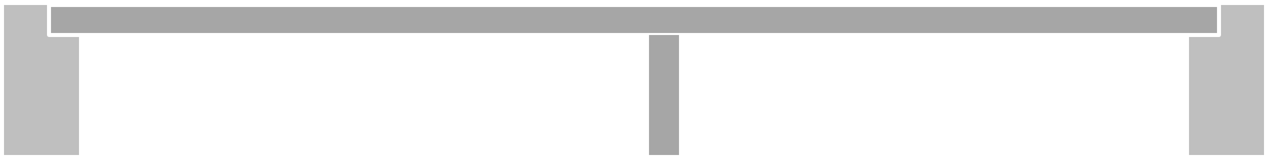
Propose optimal solutions for typical bridges

- Evaluate the interest of using elastomeric bearings for bridge overpass
- Combine elastomeric bearings with hysteretic dampers

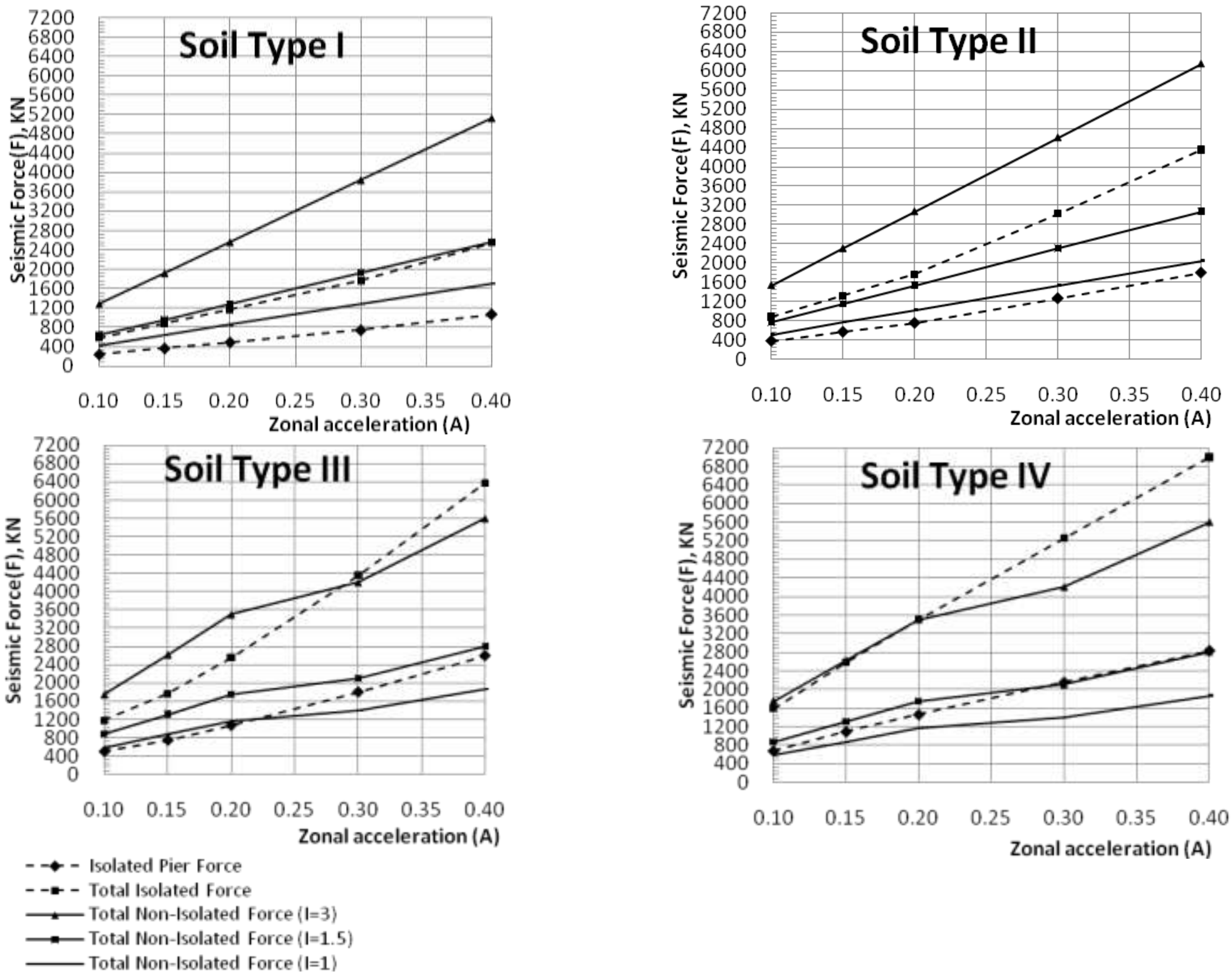
Propose optimal solutions for typical bridges

Parametric study:

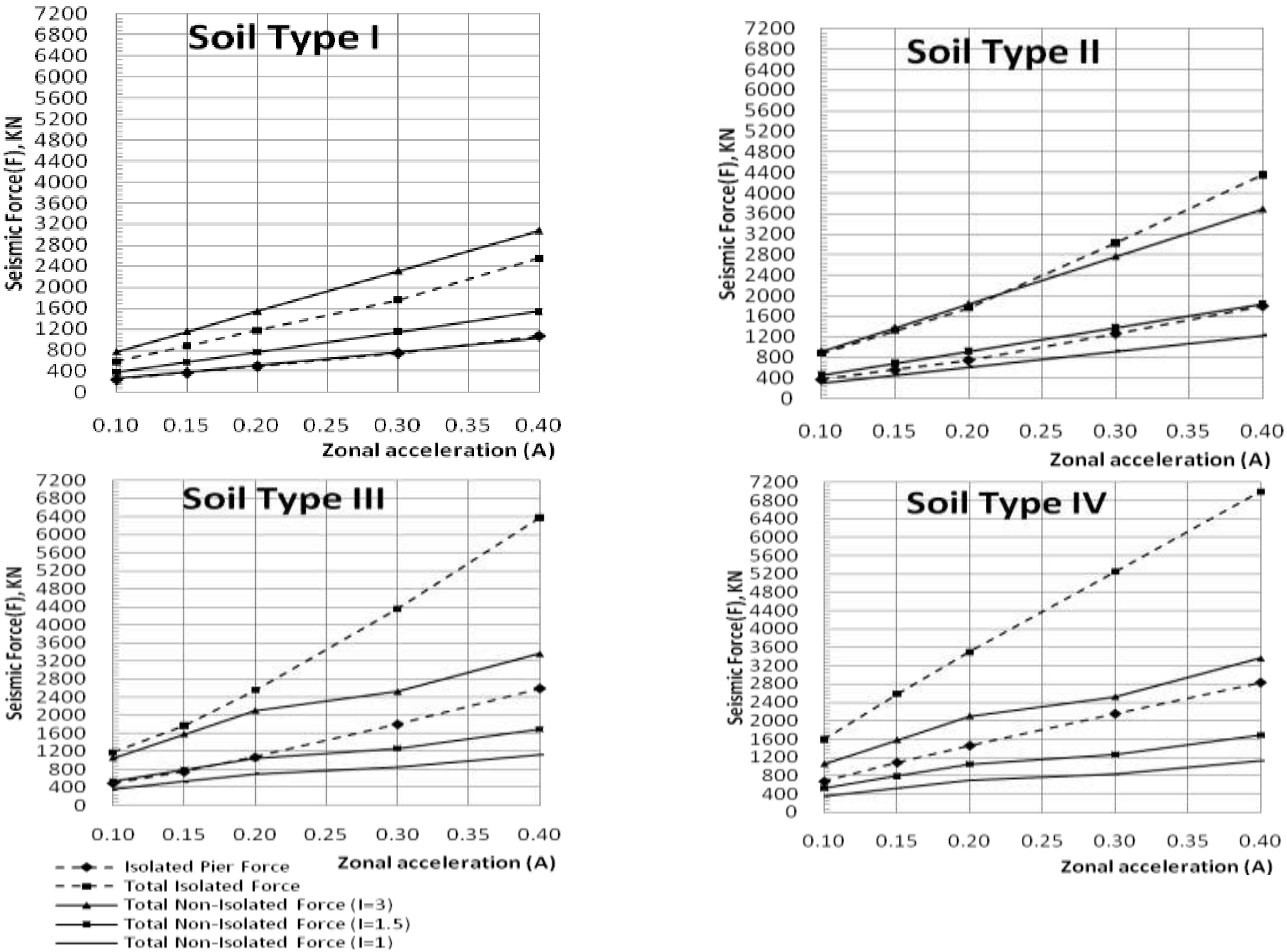
- Importance factor: I = 1.0 , 1.5 , 3.0
- Span length: 20, 35 and 50 m
- Stiffness of the pier:
  - 25,000 kN/m (corresponding to a multi-bent with 2 columns 1.2m in diameter and 5.5-m in height)
  - 100,000 kN/m (2 columns 1.5-m in diameter with about 6m in height), and
  - 250,000 kN/m (corresponding to a wall 1.2m in thickness and 6-m long and 6m in height)
- The dead weight of the bridge:
  - 50 kN/m (Typical of a light steel girder with a concrete slab),
  - 100 kN/m (typical of a heavy steel girder or a light concrete girder) and
  - 200 kN/m (typical of a heavy concrete girder solution)
- R-factor: R=3.0 or R=5.0



L = 35m; K = 100 MN/m ; w = 100kN/m ; and R=3

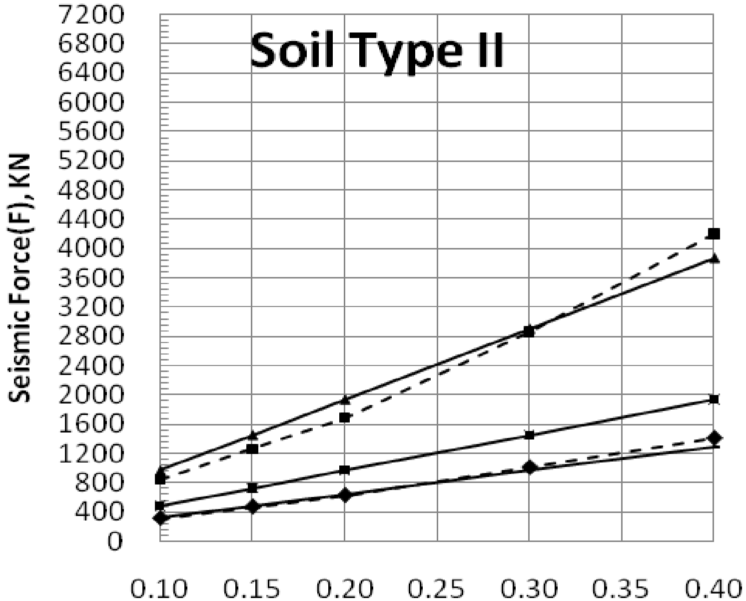


L = 35m; K = 100 MN/m ;
w = 100kN/m ; and R=5

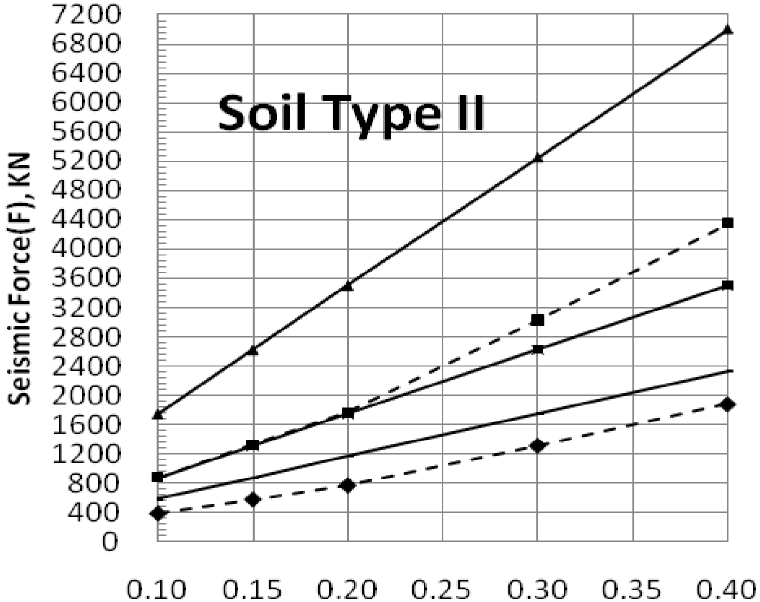


L = 35m; w = 100kN/m ; and R=3

K=25 MN/m

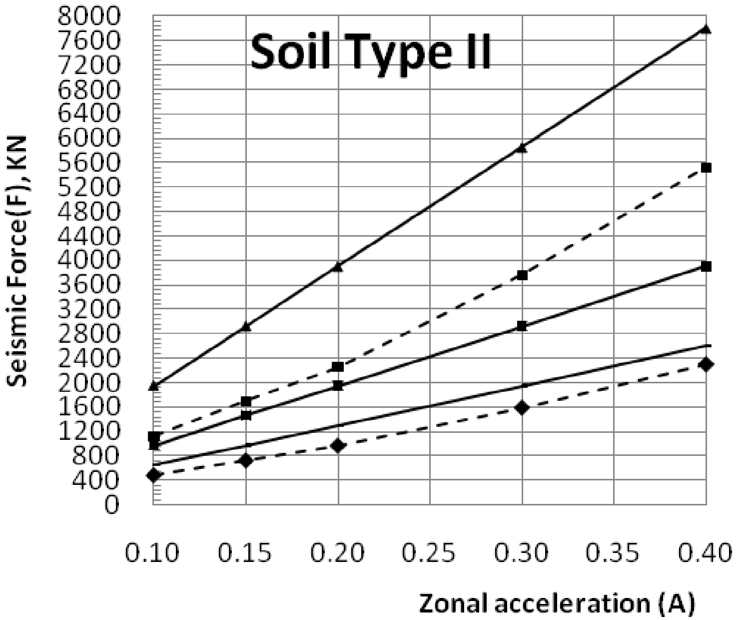


K=250 MN/m

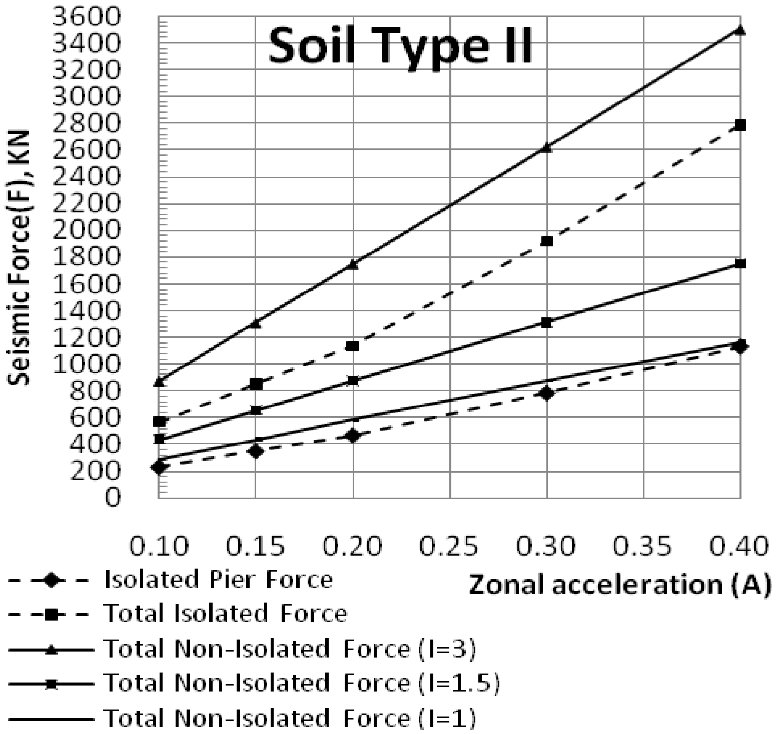


L = 50m; K=100 MN/m; and R=3

w=100 kN/m



w=50 kN/m



Propose optimal solution for typical bridges

Interest of using elastomeric bearings:

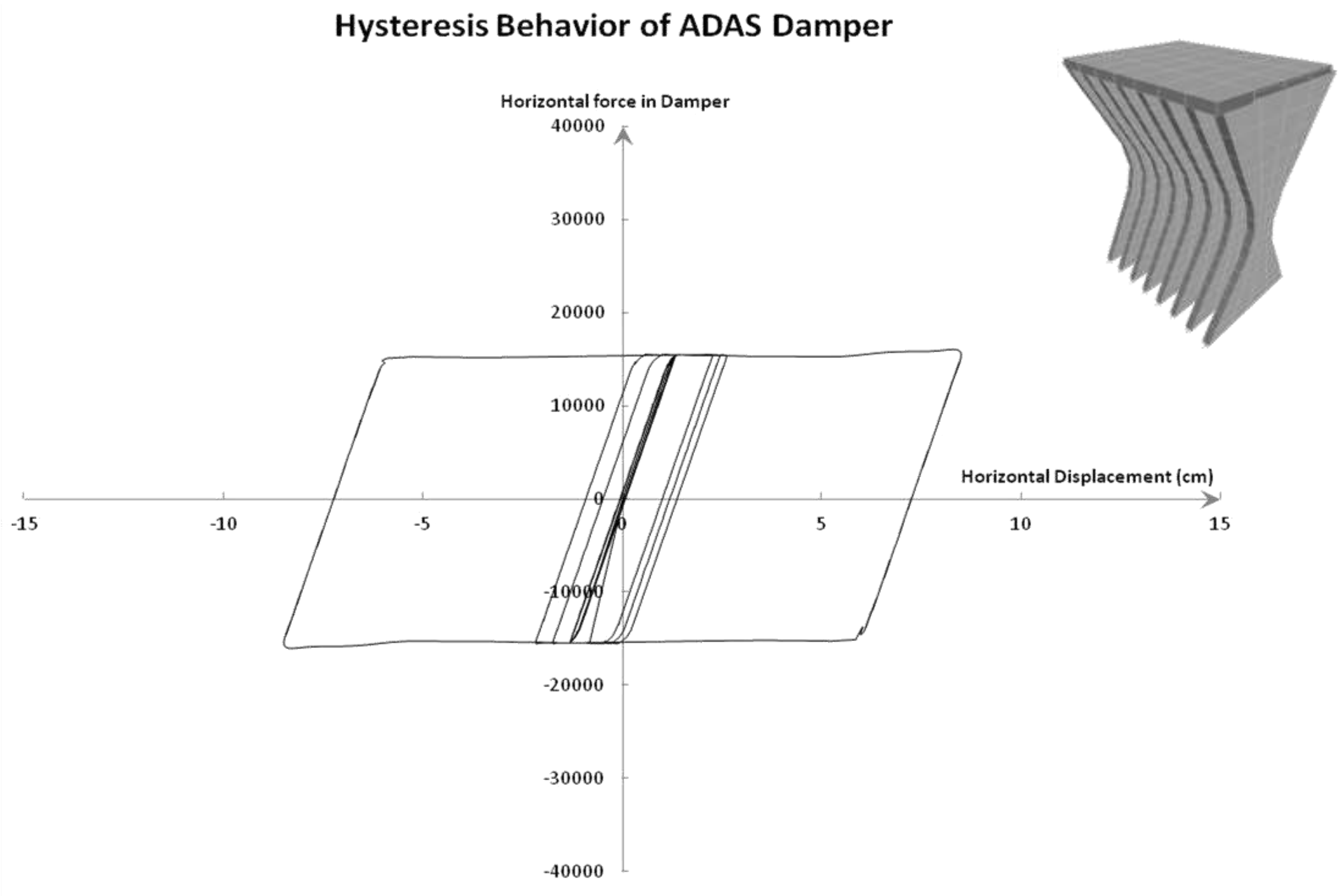
- « Other » bridges: isolation and standard design are similar solutions
- For « emergency » « lifeline » bridges, isolation provides a better solution in terms of loads in pier and foundations
- Analysis of structure with elastomeric bearings is simple and would not lead to complex design. It could « regularized » unregular bridges
- Lengthy testing requirements: a standard could be used with quality control tests

Propose optimal solution for typical bridges

Combination of elastomeric bearing and hysteretic dampers:

- Only concern with elastomeric bearing is the displacement that could be large for some bridges
- To reduce the displacement, use of hysteric dampers could be interesting: fix point could be used to dissipate energy. Under service loading, the bridge has a point of fixity provided by the elastic stiffness of the hysteretic damper and during an earthquake, the bridge is isolated with an hysteretic damper

Propose optimal solution for typical bridges



Propose optimal solution for typical bridges

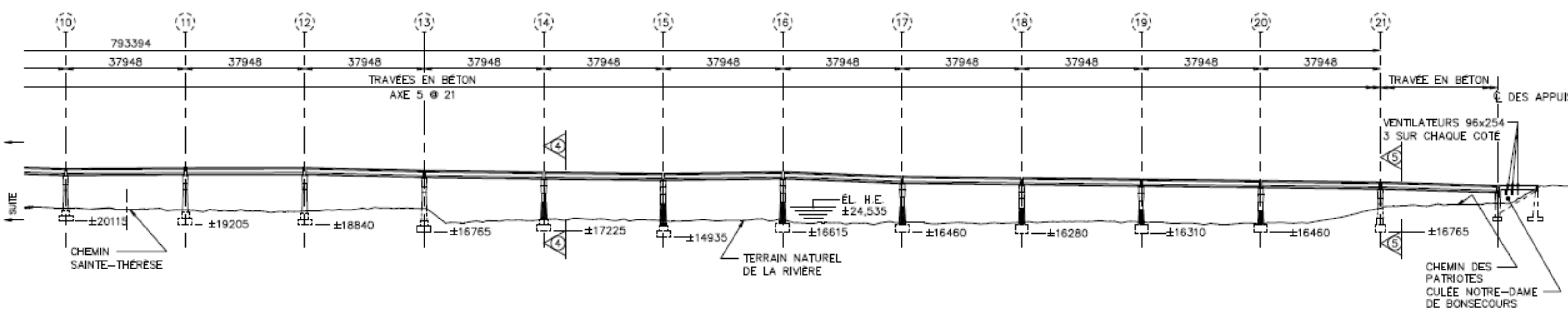
Type of Analysis	Time-history analysis	
	Isolated	Isolated &damped with damper
Period of structure (second)	1.17	1.08
Deck Transverse displacement (mm)	105	77
Deck longitudinal displacement (mm)	106	92

Help designers with isolation

Help designers with isolation

- Often, isolation with elastomeric bearing is considered too esoteric by designer whereas it simplifies bridge behavior under an earthquake in a number of cases (irregular bridges).
- Elastomeric bearings can reduce significantly the demand on substructure and this is very interesting for seismic retrofitting of existing structures
- Owners usually do not like floating systems as it has resulted in many durability issues in the past:
    - ➢ It is possible to use a fixed system with a fuse type system so for service there is a standard bearing condition (one fixed bearing for the 2-span example) and under seismic, the link is broken. It is even possible to use energy dissipating device to reduce seismic displacements.
- Elimination of importance factor in the design of elastomeric bearing design is a divisive question:
    - ➢ Principle in code request the bridge to withstand earthquake of 1000-yr return period with minor or very minor damage for emergency and lifeline bridge. It is questionable if design for 475-yr return period for lifeline bridge will ensure this performance.
    - ➢ By comparison to standard bridge design, use of I=1 and R=1 for capacity protected elements tend to assume that S6-06 consider that use of I=1 and R=1 ensure resistance of capacity protected members to 1000-year return period.
    - ➢ Recommendation of the author is to design for 1000-yr return period earthquake for lifeline bridges at least, but this should be addressed properly in revision of Section 4.10.

Help designer with isolation



ÉLÉVATION

ÉCH. 1: 750

Help designer with isolation projects

First viaduct:

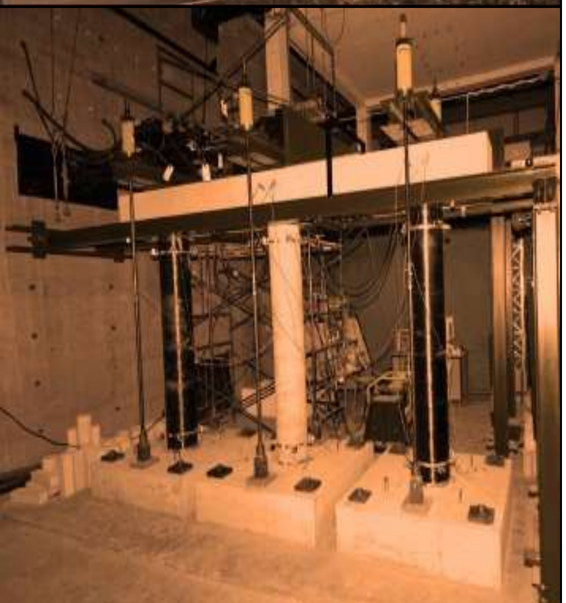
retrofitted for 0.17g instead of 0.2g

Second viaduct:

retrofitted to the full 1000-yr return period earthquake with pendulum bearings

Summary

- Ongoing research
- Objective is to complete most of the work for the end of the year to support modification of S6
- Demonstrate the great advantages of isolation for standard bridges and provide practical solutions.



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NEES

Bridge Network Fragility in Quebec with and without isolation

Patrick Paultre

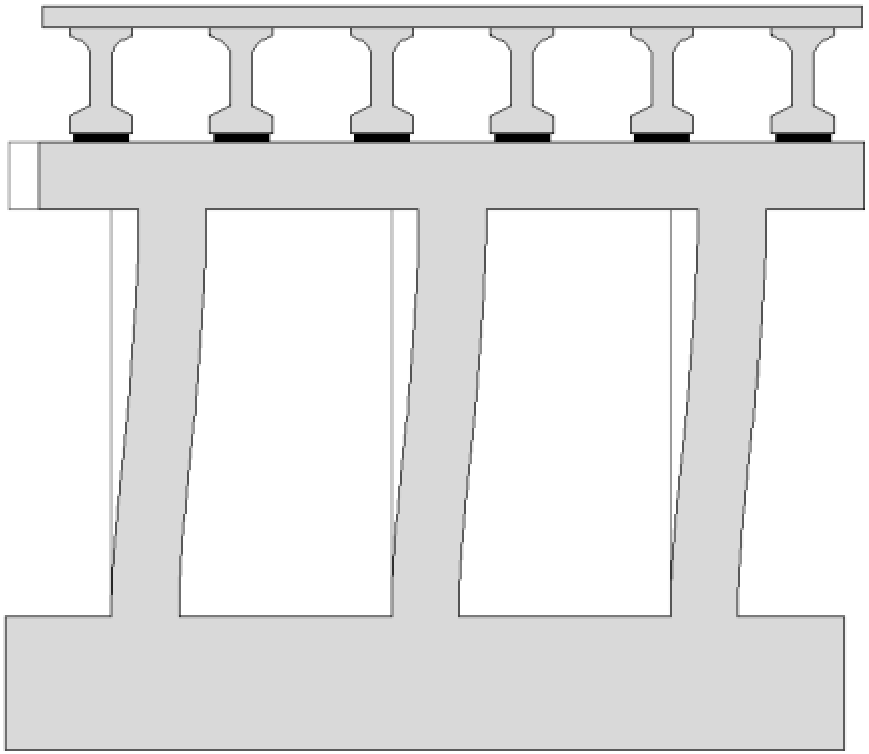
April 30, 2012
University of British Columbia

UNIVERSITÉ DE SHERBROOKE
CRGP
Earthquake Engineering and Structural Dynamics Research Center
Centre de recherche en génie parasismique et en dynamique des structures
Département de génie civil, Faculté de génie

System and Member Fragility

System fragility:

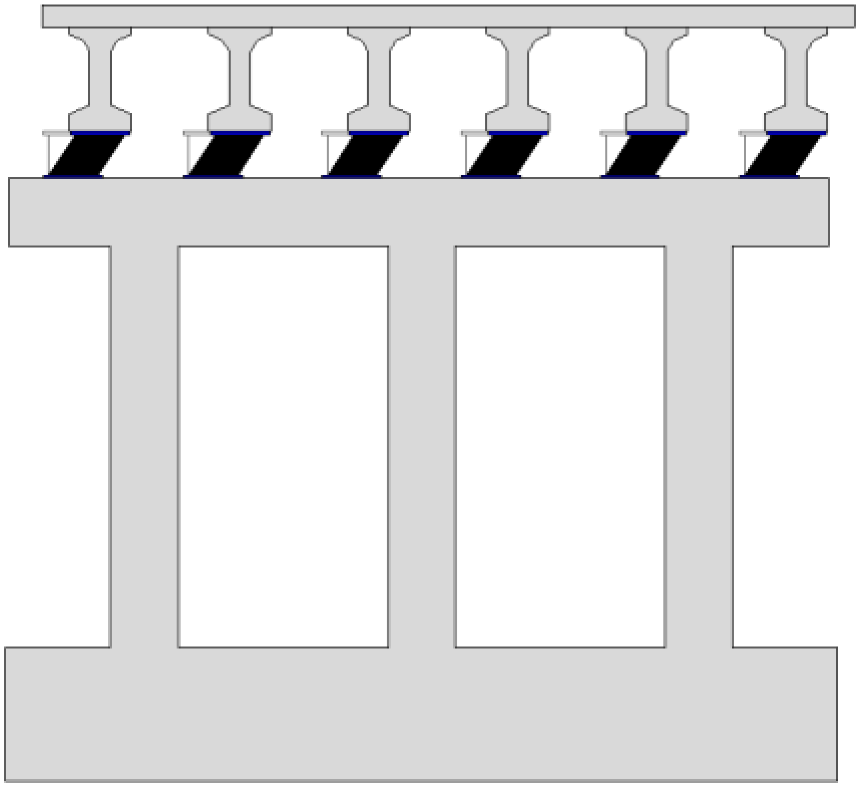
- *Depends on component fragility:*
- *Component fragility depends on limit states definition:*
  - *Columns*
  - *Elastomeric bearings*
  - *Abutment walls*
  - *Abutment foundation*
  - *Isolators*
  - *Foundation*
- *Bridge piers (columns) are usually the most fragile*



Seismic Isolation

Effective method to reduce fragility

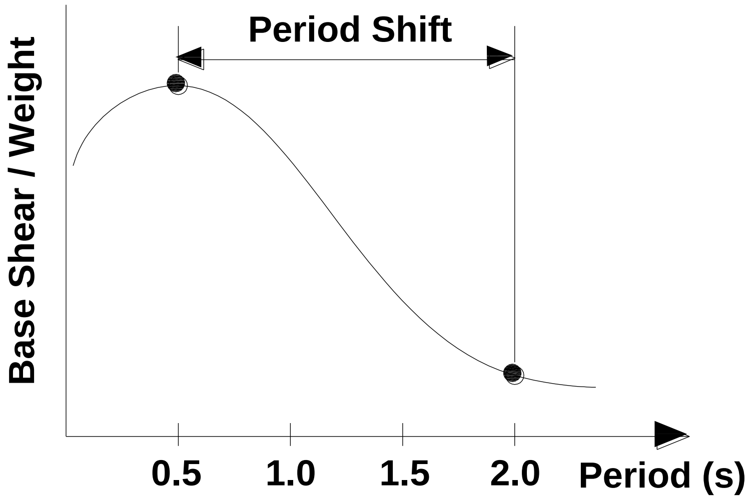
- *For new design*
- *For rehabilitation*



Seismic Isolation Retrofit

Seismic Isolation

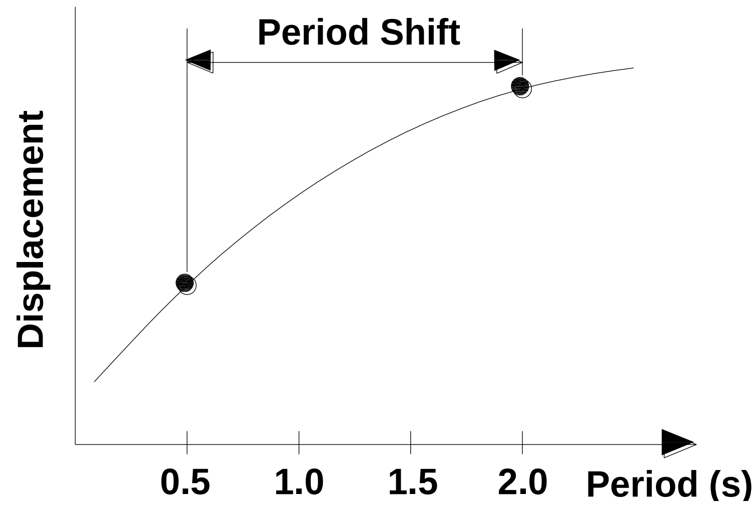
- ✓ *Effective Method:*
  - ✓ *Protection*
  - ✓ *Rehabilitation*
- ✓ *Periode Shift*
- ✓ *Protection of foundation elements*
  - ✓ *Remain in elastic range*



Seismic Isolation Retrofit

Seismic Isolation

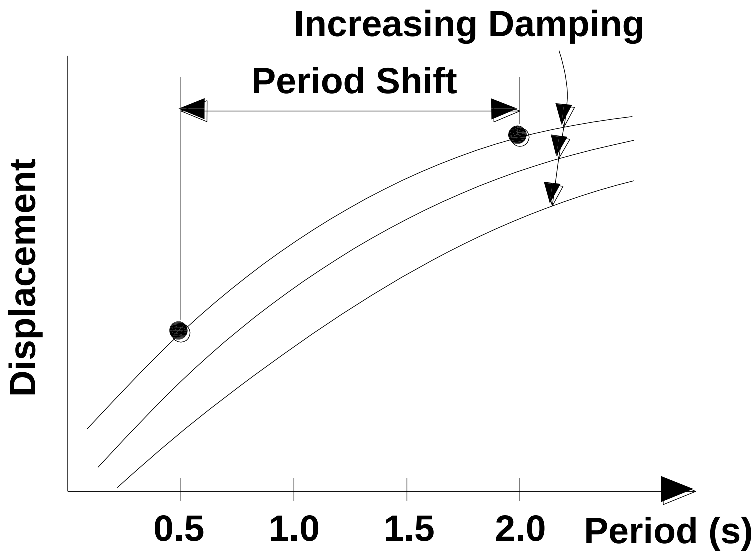
- ✓ *Effective Methode :*
  - ✓ *Protection*
  - ✓ *Rehabilitation*
- ✓ *Periode Shift*
- ✓ *Protection of foundation elements*
  - ✓ *Remain in elastic range*
- ✓ *Increase displacement*



Seismic Isolation Retrofit

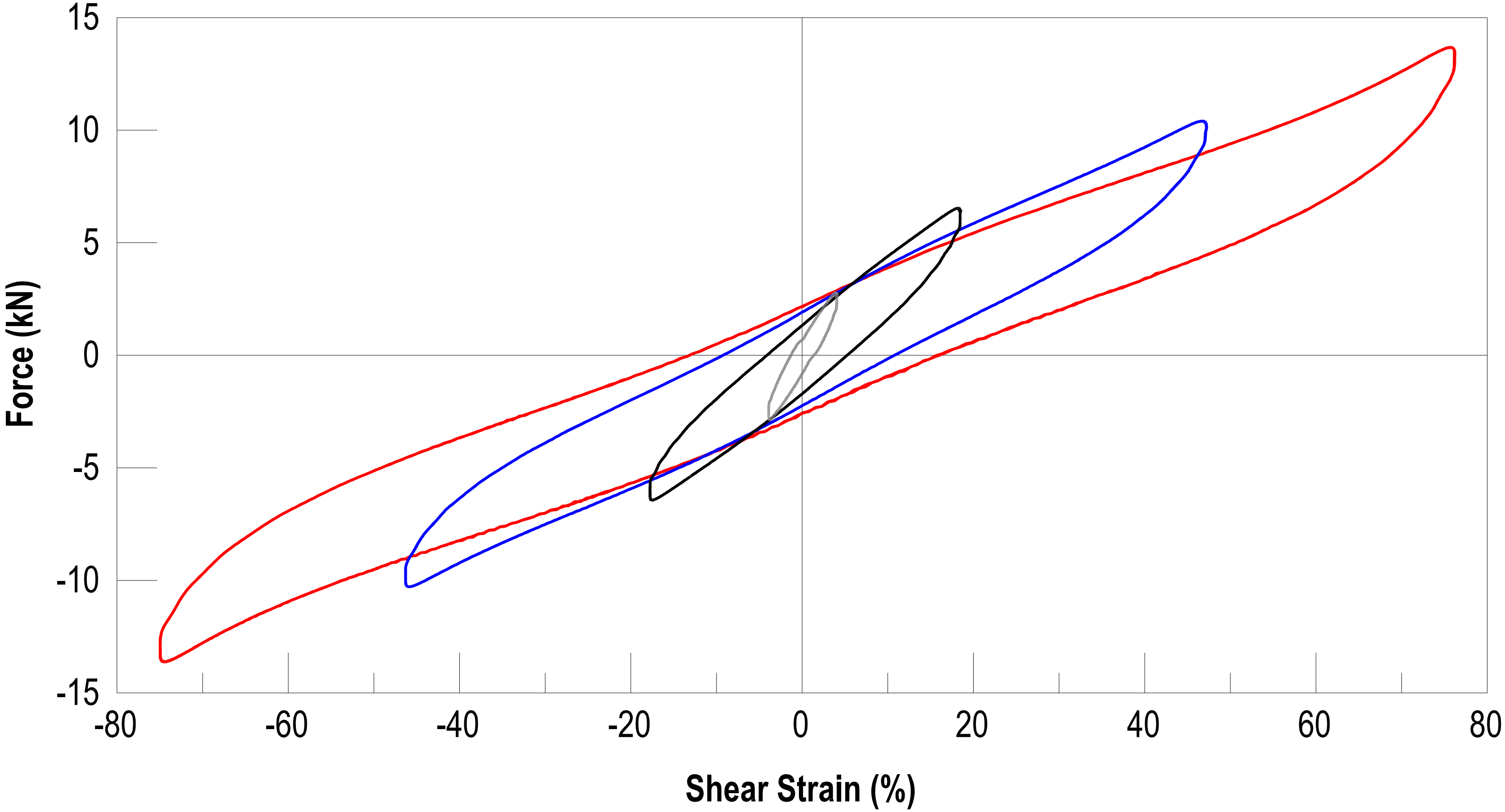
Seismic Isolation

- ✓ *Effective Methode :*
  - ✓ *Protection*
  - ✓ *Rehabilitation*
- ✓ *Periode Shift*
- ✓ *Protection of foundation elements*
  - ✓ *Remain in elastic range*
- ✓ *Increase displacement*
- ✓ *Alternative: damping*



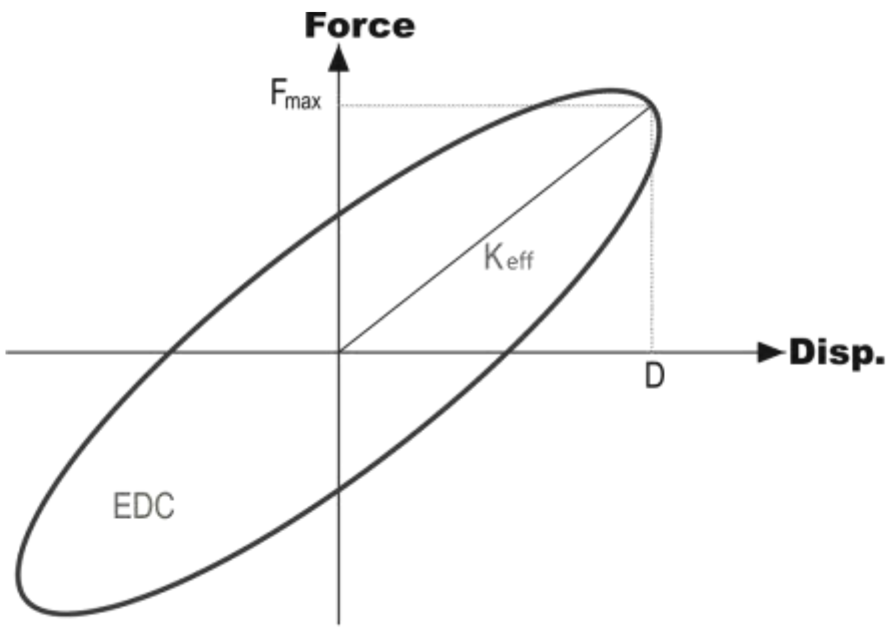
Seismic Isolation Retrofit

Behavior

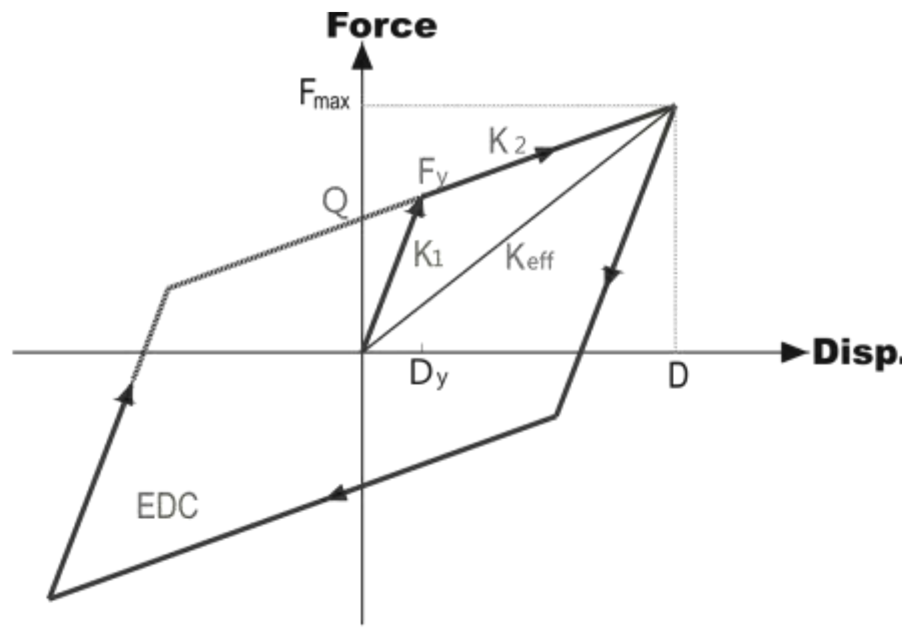


Seismic Isolation Retrofit

Behavior



(a) NRB viscous-elastic behavior



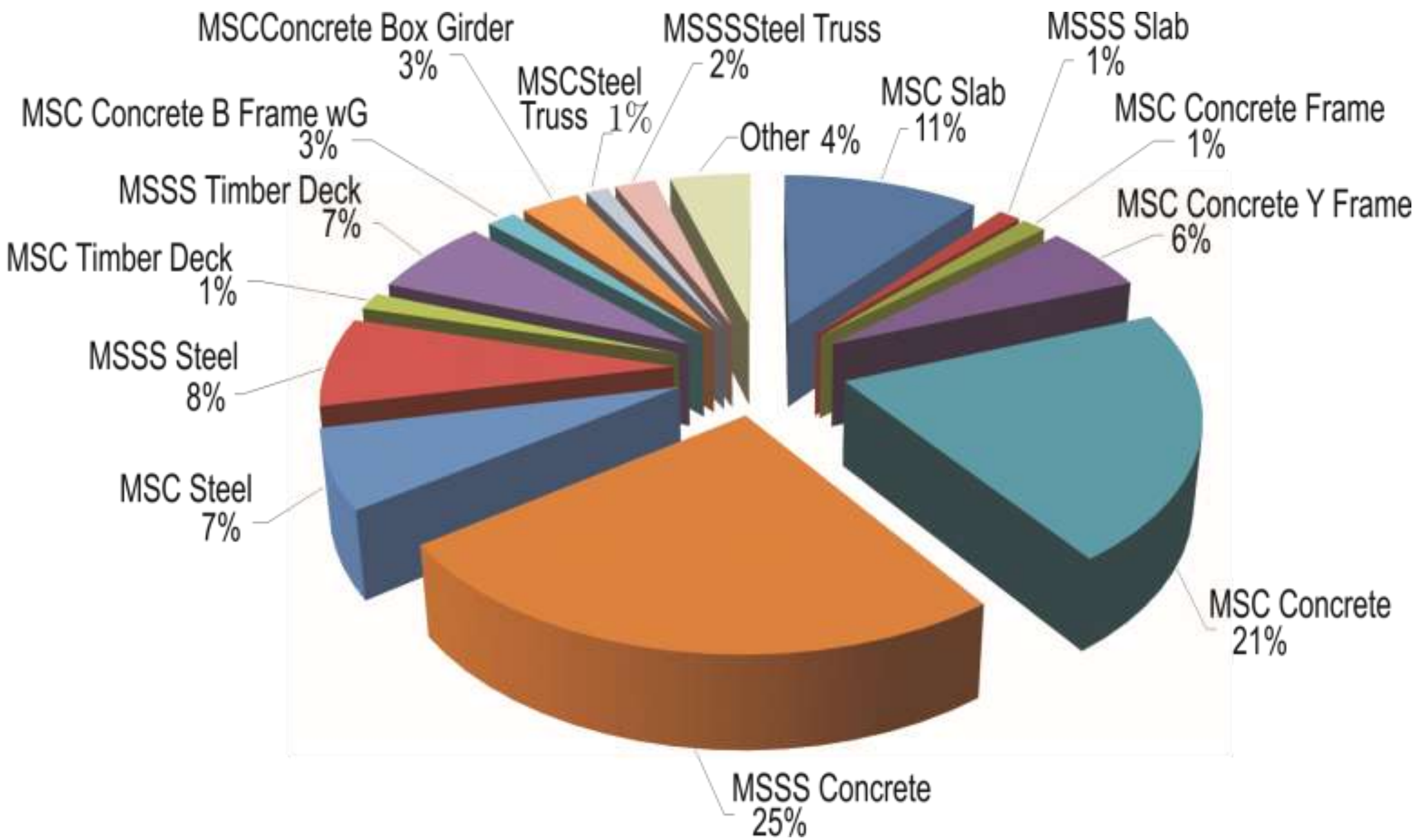
(b) Equivalent bilinear hysteretic model

$$d_i = \frac{250 A S_i T_e}{B}$$

$$T_e = 2\pi \sqrt{\frac{W}{K_{eff} \times g}}$$

Methodology

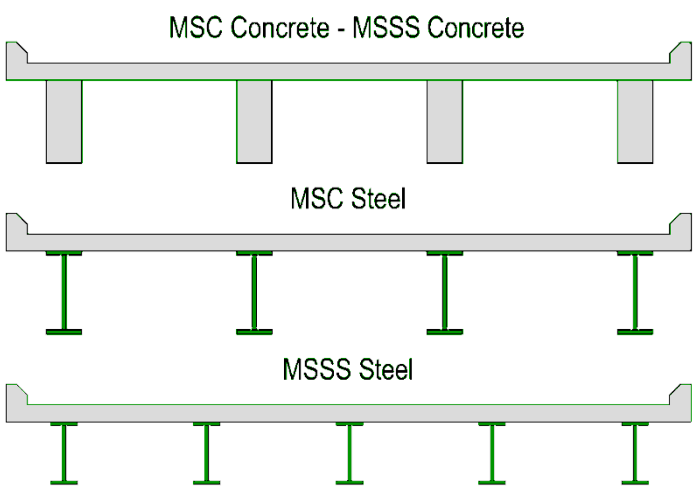
Quebec Bridge Distributions: 2672 bridges surveyed



Methodology

Quebec Bridge Distributions

Superstructure

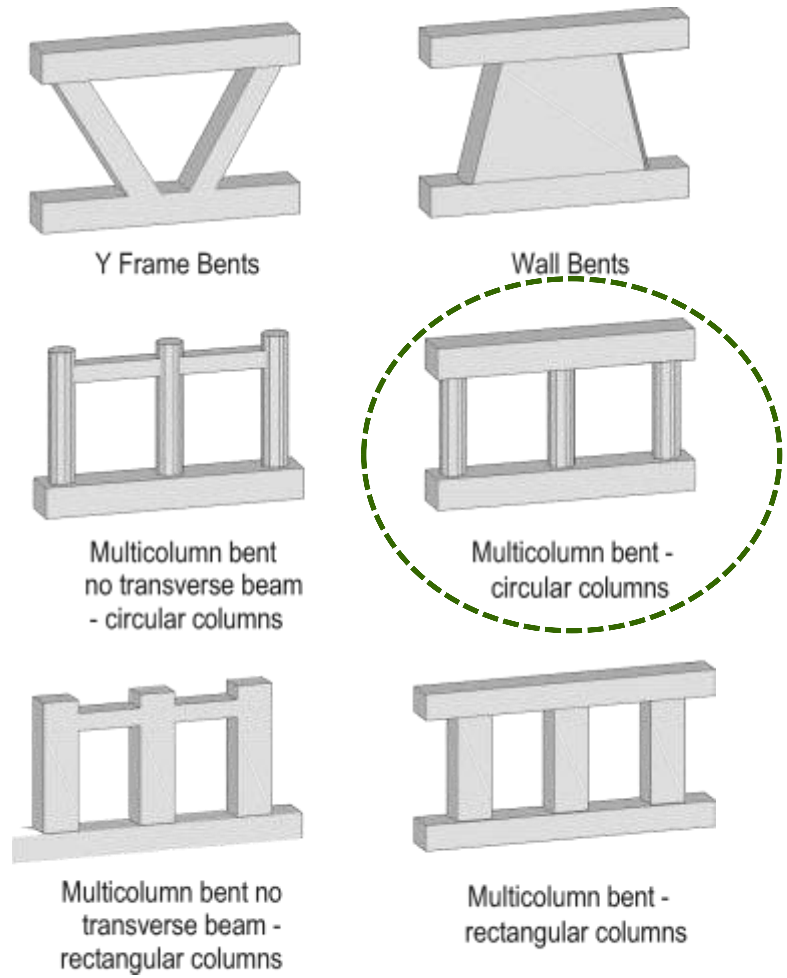


MSC Concrete – 21%
MSSS Concrete – 25%
MSC Steel – 7%
MSSS Steel – 8%

Total – 61%

Excellent candidates to seismic isolation due to their configuration

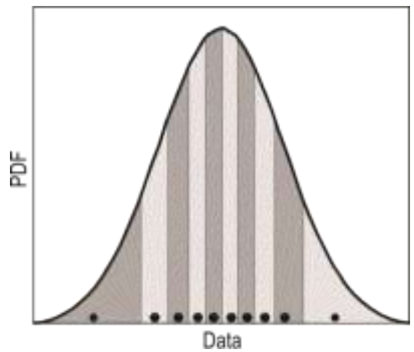
Bents



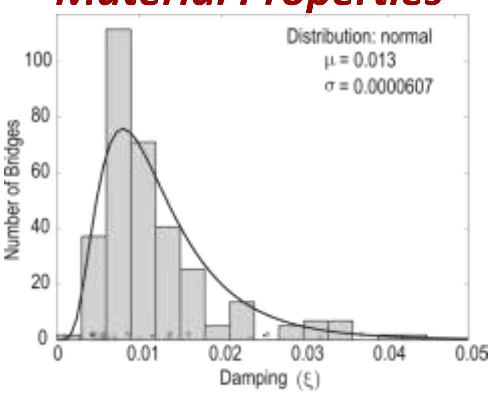
Methodology

Class Definition – Parameters distribution

Using statistical tools



Bridge Components and Material Properties



Parameter	Distribution
Geometric	
Total Length	LogNormal
Total Width	LogNormal
Total Height	LogNormal
Main Span/Total Length (Lmr)	Normal
Materials	
Concrete Strength	Normal
Steel Strength	LogNormal
Isolator Effective Stiffness	Uniform
Abutment Passive Initial Stiffness	Uniform
Abutment Active Initial Stiffness	Uniform
Foundation Rotational Stiffness	Uniform
Foundation Translational Stiffness	Uniform
Others	
Mass Variability	Uniform
Damping	Normal
Abutment/Deck Gap	Normal
Skew Angle	Normal

Methodology

Class Definition – Block definition

Each block is analyzed 15 times varying parameter properties and GMTH pairs

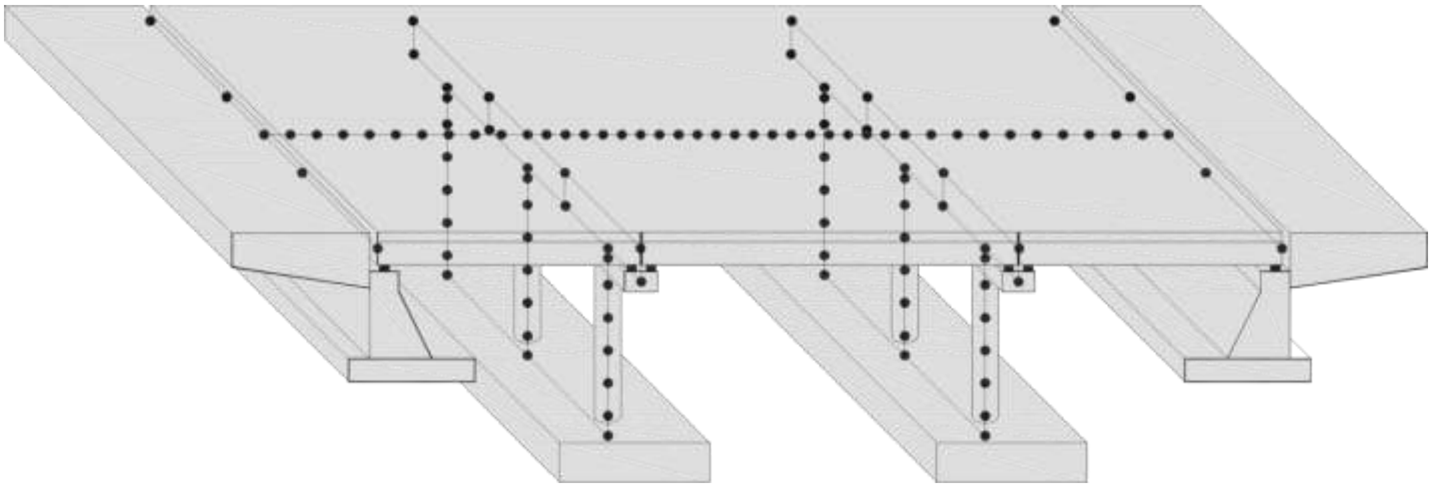
MSCConcrete	Length	Width	Height	Lmr	MSSSConcrete	Length	Width	Height	Lmr
Block 1	100.98	13.04	6.72	0.30	Block 1	115.57	14.99	2.50	0.21
Block 2	64.79	8.35	8.35	0.52	Block 2	58.60	12.50	4.84	0.41
Block 3	54.61	23.43	9.78	0.36	Block 3	43.04	15.79	3.51	0.51
Block 4	75.27	17.65	4.73	0.47	Block 4	76.88	10.33	5.93	0.40
Block 5	45.93	10.72	3.77	0.46	Block 5	44.44	8.45	5.51	0.37
Block 6	114.49	15.23	7.80	0.32	Block 6	30.47	11.91	8.22	0.58
Block 7	67.96	11.80	6.15	0.43	Block 7	70.49	24.94	11.97	0.30
Block 8	45.93	10.72	3.77	0.46	Block 8	90.16	9.03	4.36	0.43
MSCSteel	**Length**	**Width**	**Height**	**Lmr**	**MSSSSteel**	**Length**	**Width**	**Height**	**Lmr**
Block 1	59.71	9.46	4.46	0.26	Block 1	32.44	5.54	3.14	0.53
Block 2	79.67	10.13	9.81	0.31	Block 2	61.55	10.65	11.27	0.40
Block 3	90.17	14.00	7.47	0.34	Block 3	54.29	15.64	7.64	0.26
Block 4	46.75	13.51	3.57	0.45	Block 4	73.40	10.32	9.51	0.45
Block 5	64.26	11.31	10.65	0.42	Block 5	100.81	12.46	5.30	0.36
Block 6	25.90	7.91	2.72	0.49	Block 6	42.18	11.90	3.60	0.38
Block 7	56.28	18.50	6.11	0.39	Block 7	103.92	8.41	5.19	0.33
Block 8	95.72	15.46	5.35	0.35	Block 8	66.29	15.19	6.83	0.50

Methodology

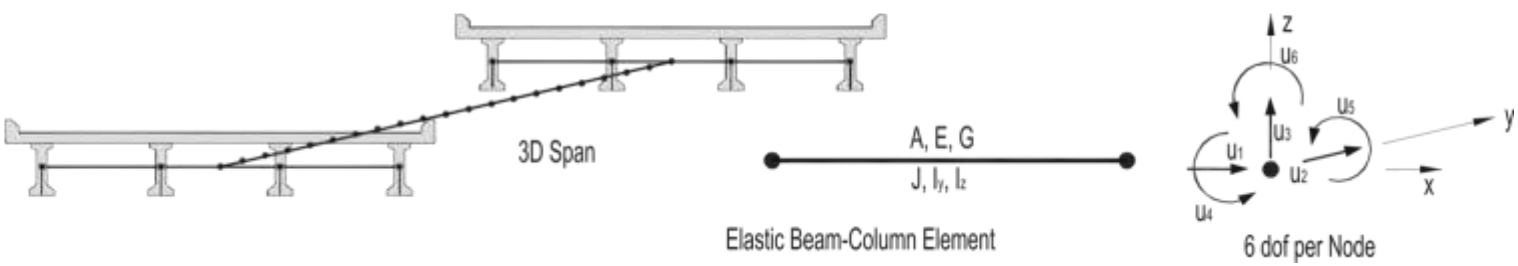
Bridge Simulation - OpenSees

Bridge Simulation - OpenSees

3D Model



Superstructure – Elastic Beam-Column Elements

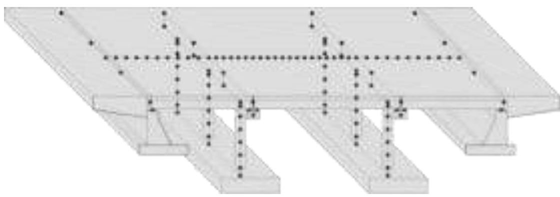


Methodology

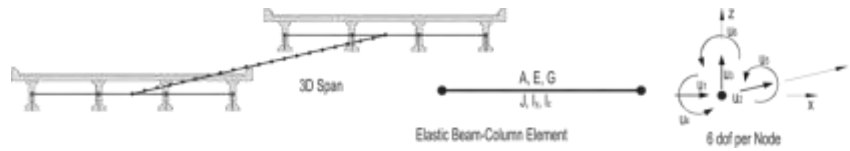
Bridge Simulation - OpenSees

Bridge Simulation - OpenSees

3D Model



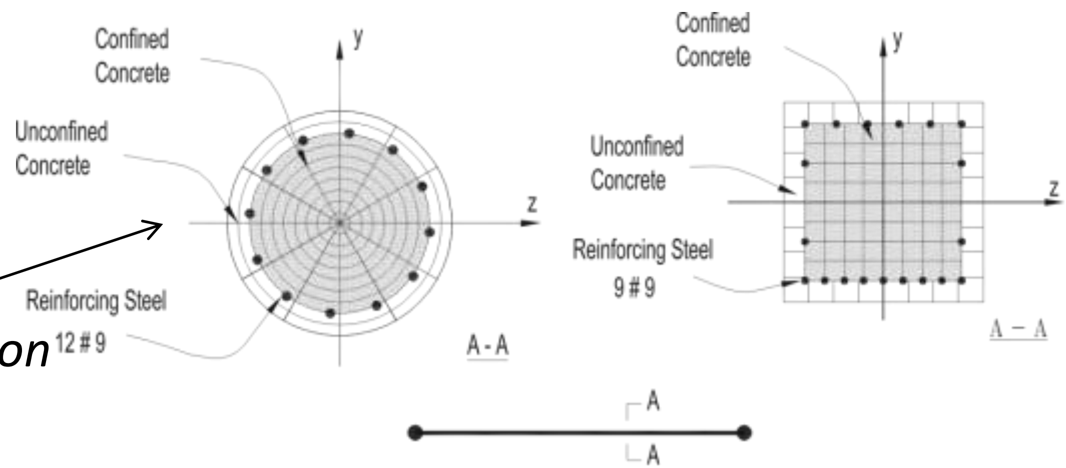
Superstructure



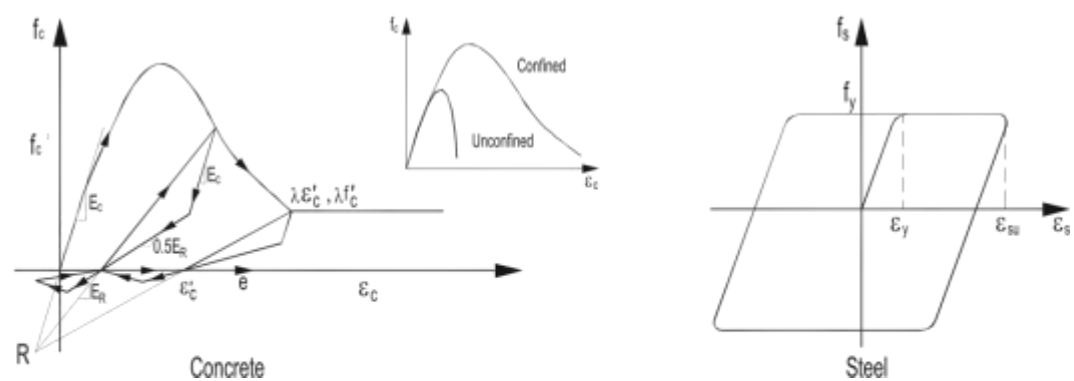
Bents

- *Transverse Beams*
- *Columns*

Nonlinear Beam Column
Elements with Fiber Section



Uniaxial Material
properties in
Fiber Sections

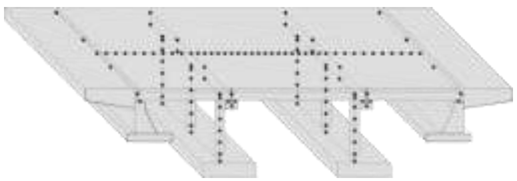


Methodology

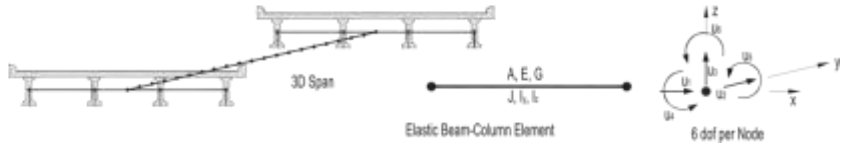
Bridge Simulation - OpenSees

Bridge Simulation - OpenSees

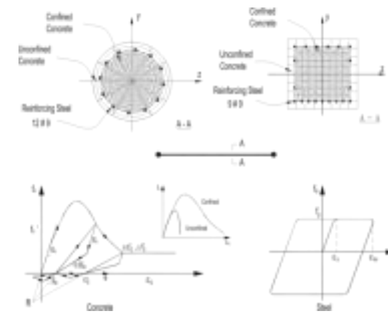
3D Model

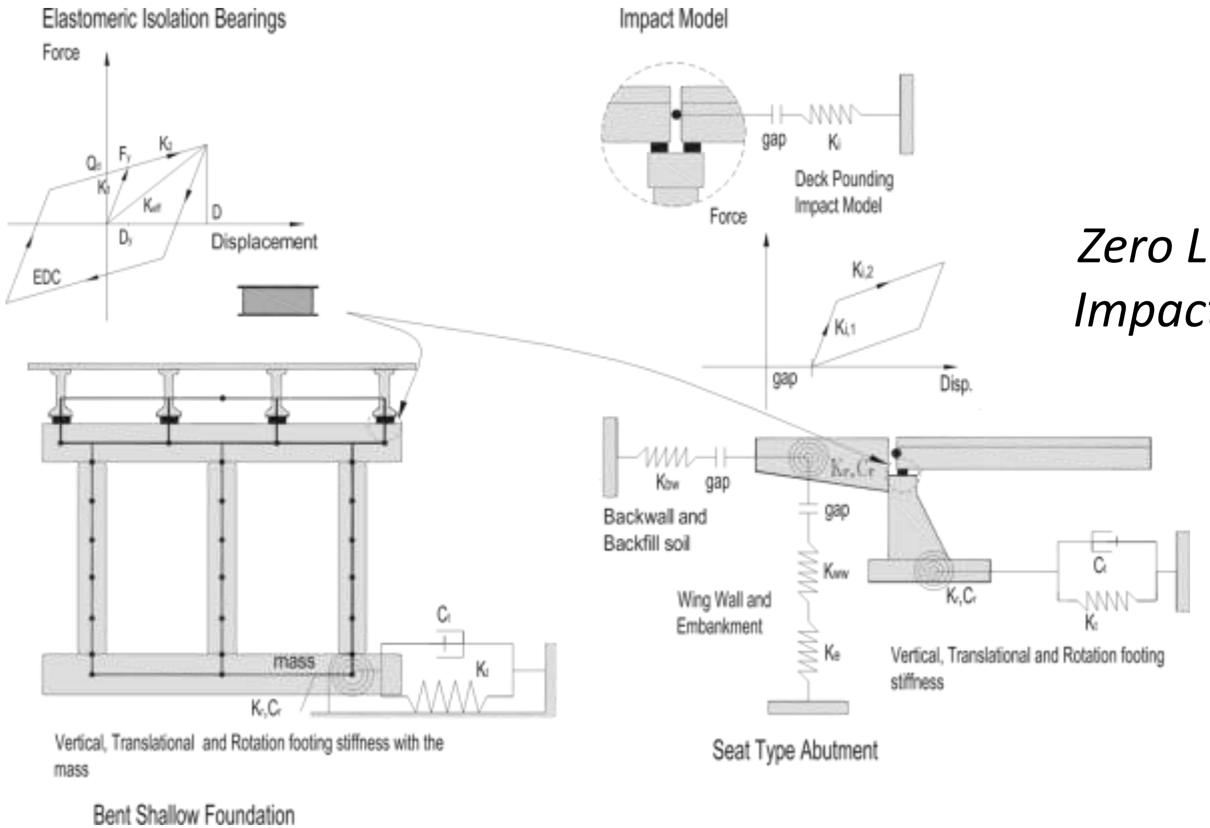


Superstructure



Bents





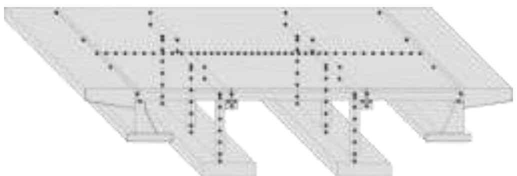
Zero Length Elements – Isolators, Impact, Abutments and Foundations

Methodology

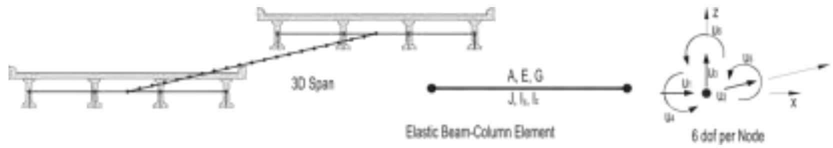
Bridge Simulation - OpenSees

Bridge Simulation - OpenSees

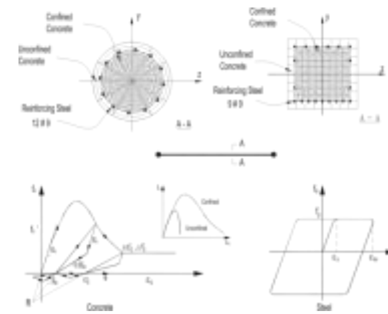
3D Model



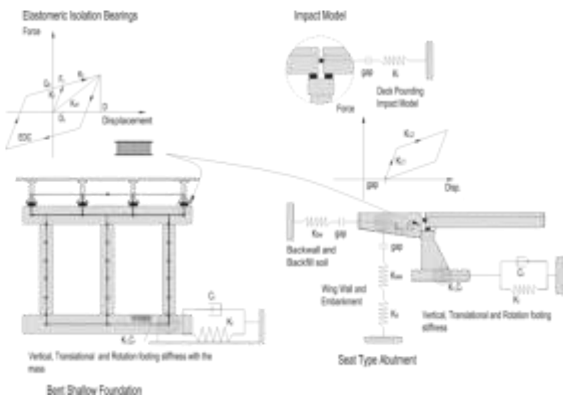
Superstructure



Bents



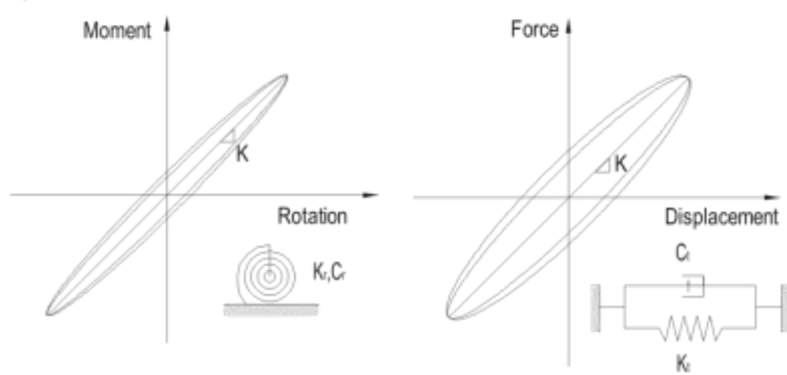
Zero Length



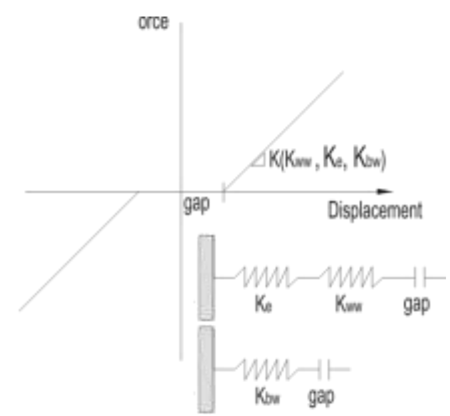
Springs Behavior

Seat type Abutments
and Shallow Foundations

Wing Wall and
Backfill Wall



Zero-length abutments and foundations springs behavior



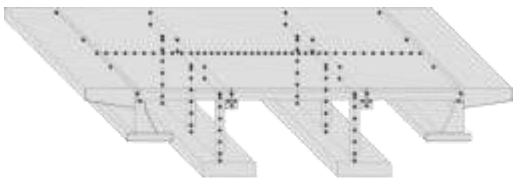
Wing Wall and Embakement and Backfill wall

Methodology

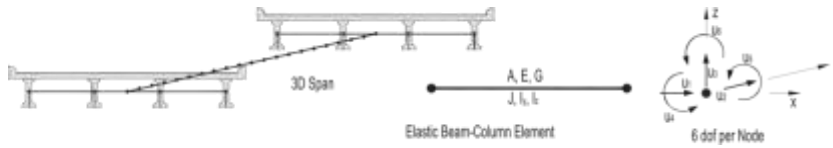
Bridge Simulation - OpenSees

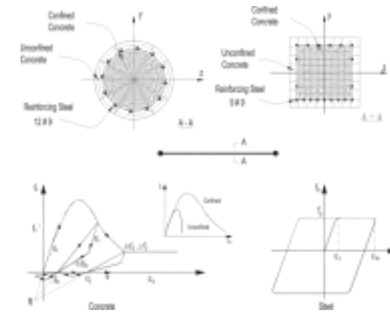
3D Model

Bents

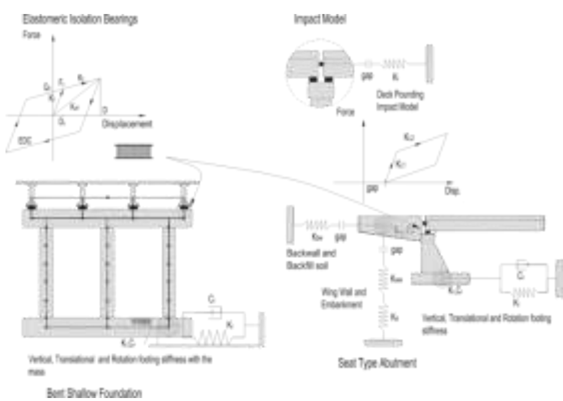


Superstructure

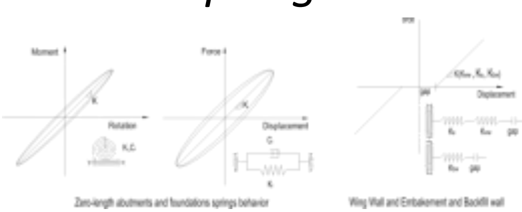




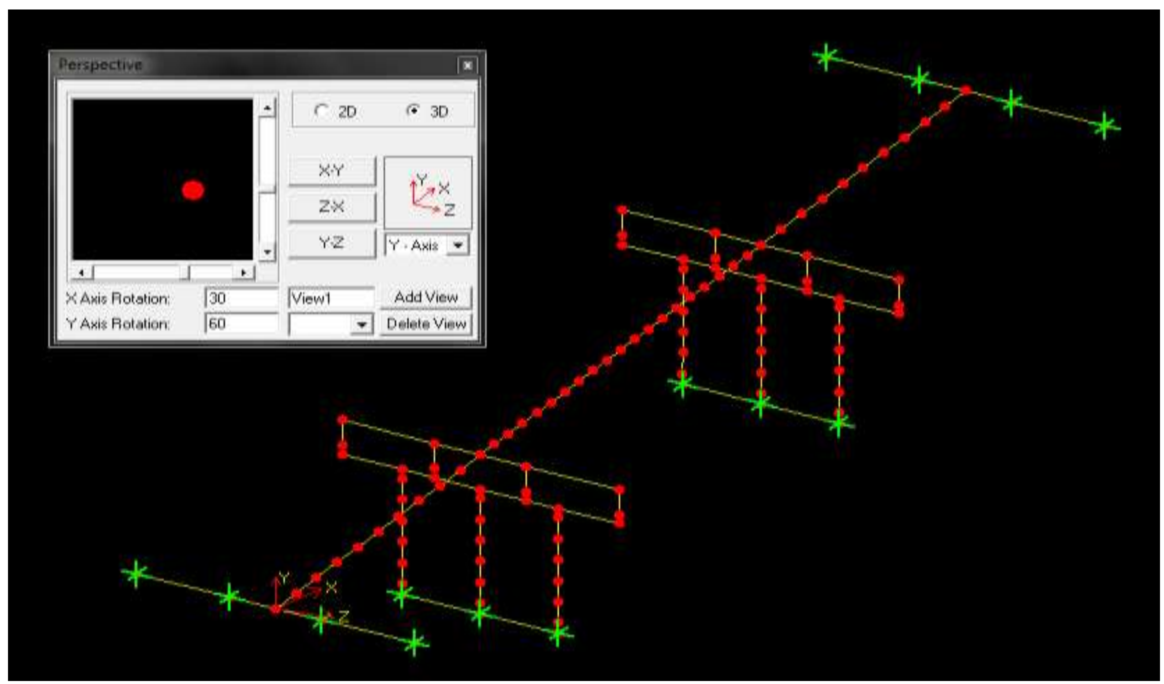
Zero Length



Springs

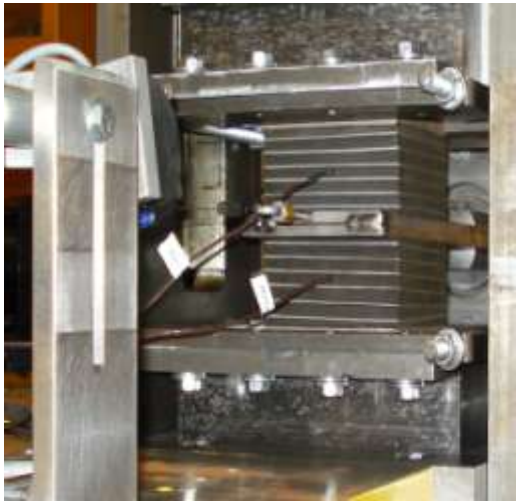


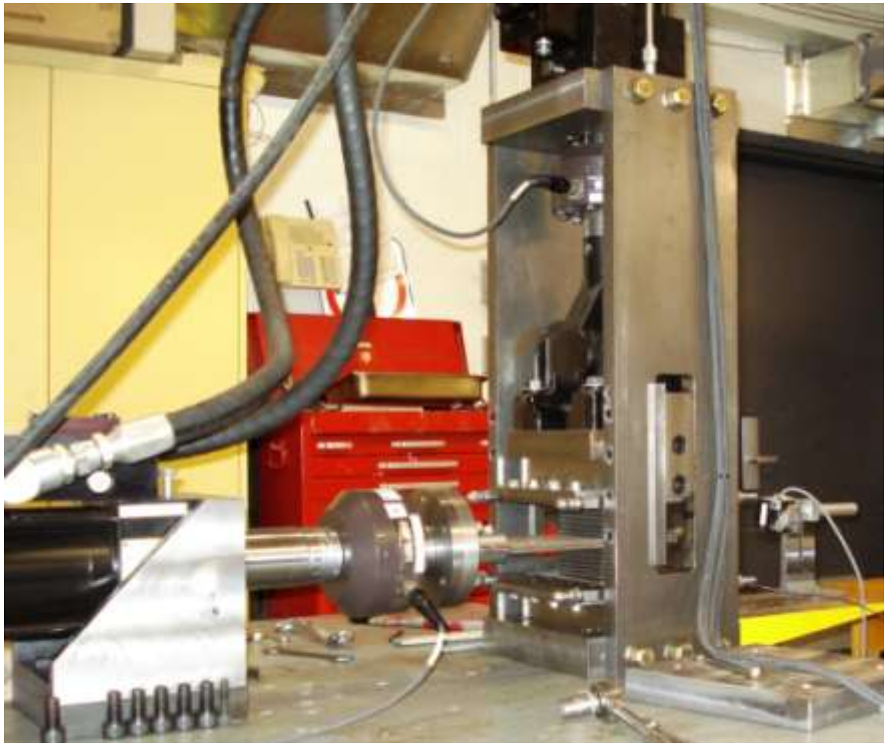
OpenSees Model



Experimental Study

Natural Rubber Devices – Reduced-scale specimens

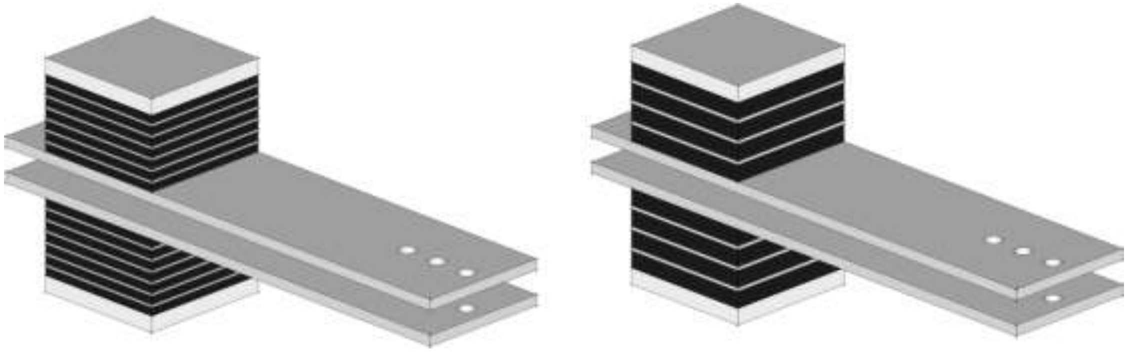




Objectives:

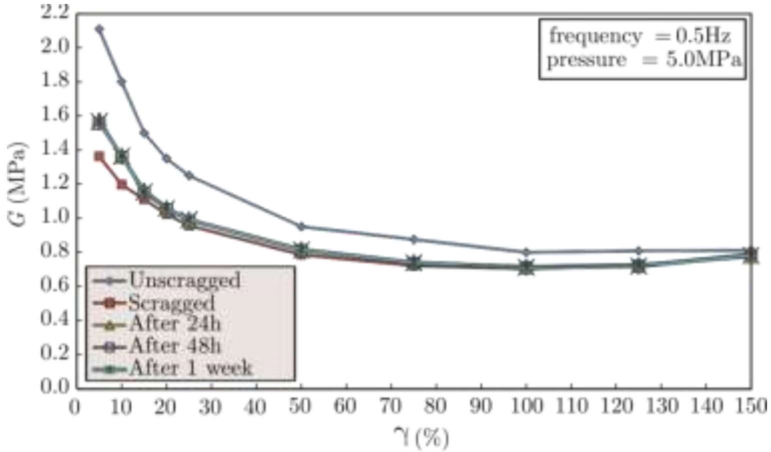
✓Verify the influence of unscragged/scragged properties for different shear deformations;

✓Verify the influence on mechanical properties of changes in specimen size, shape factor and axial load;

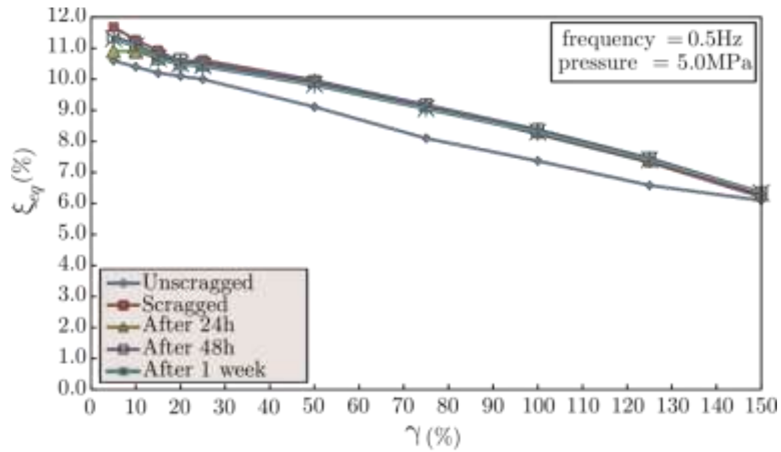


Experimental Study

Natural Rubber Devices –Reduced-scale specimens

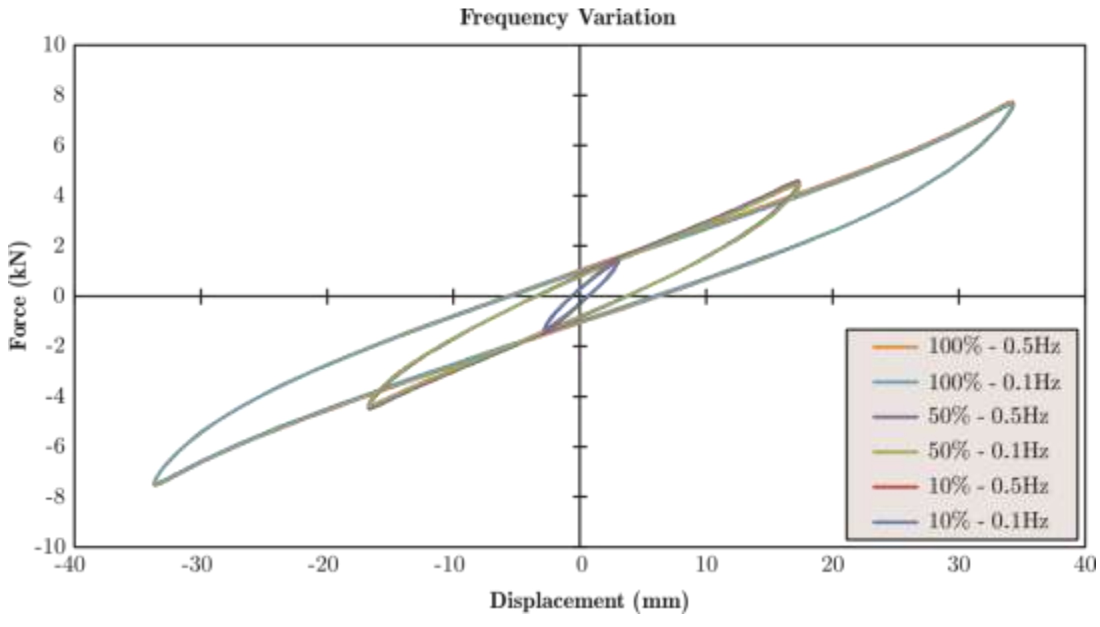


Shear Modulus



Damping

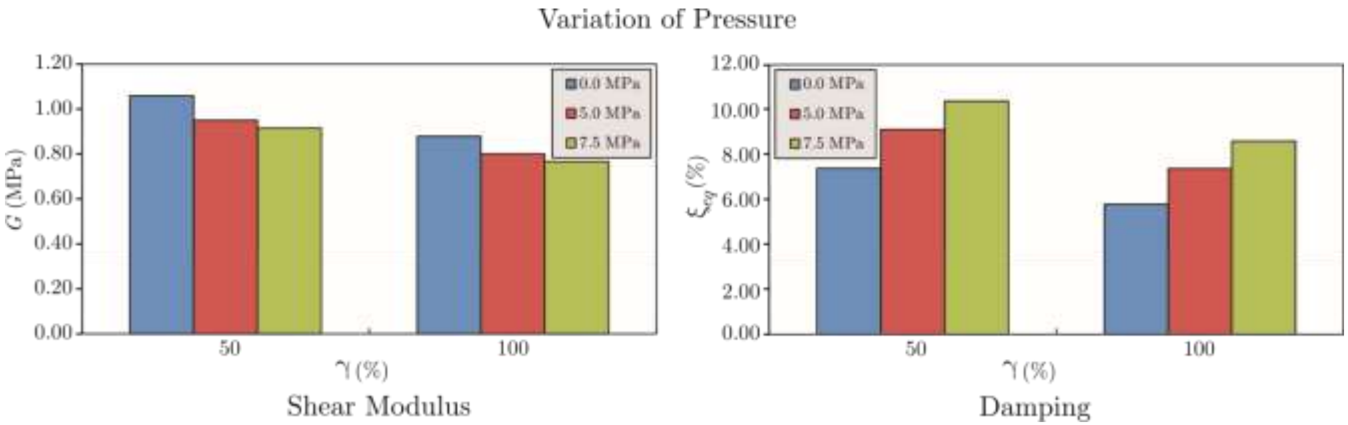
Objectives:



- ✓ *Verify the influence of unscragged/scragged properties for different shear deformations;*
- ✓ *Verify the influence on mechanical properties of changes in specimen size, shape factor and axial load;*

Experimental Study

Natural Rubber Devices –Reduced-scale specimens



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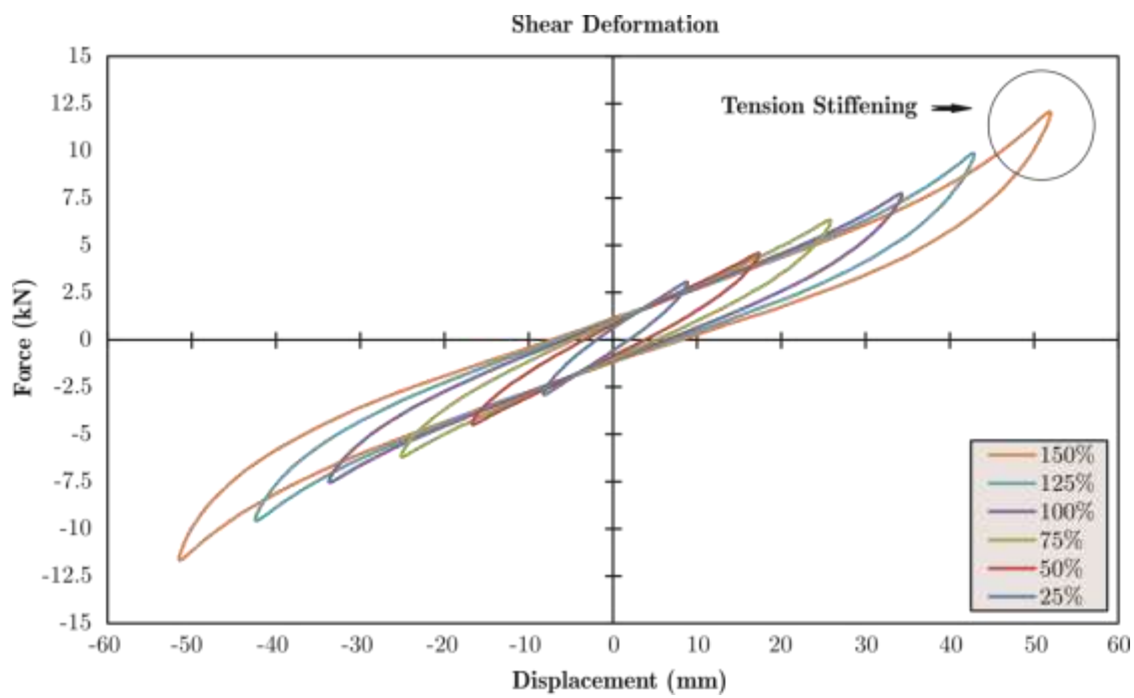
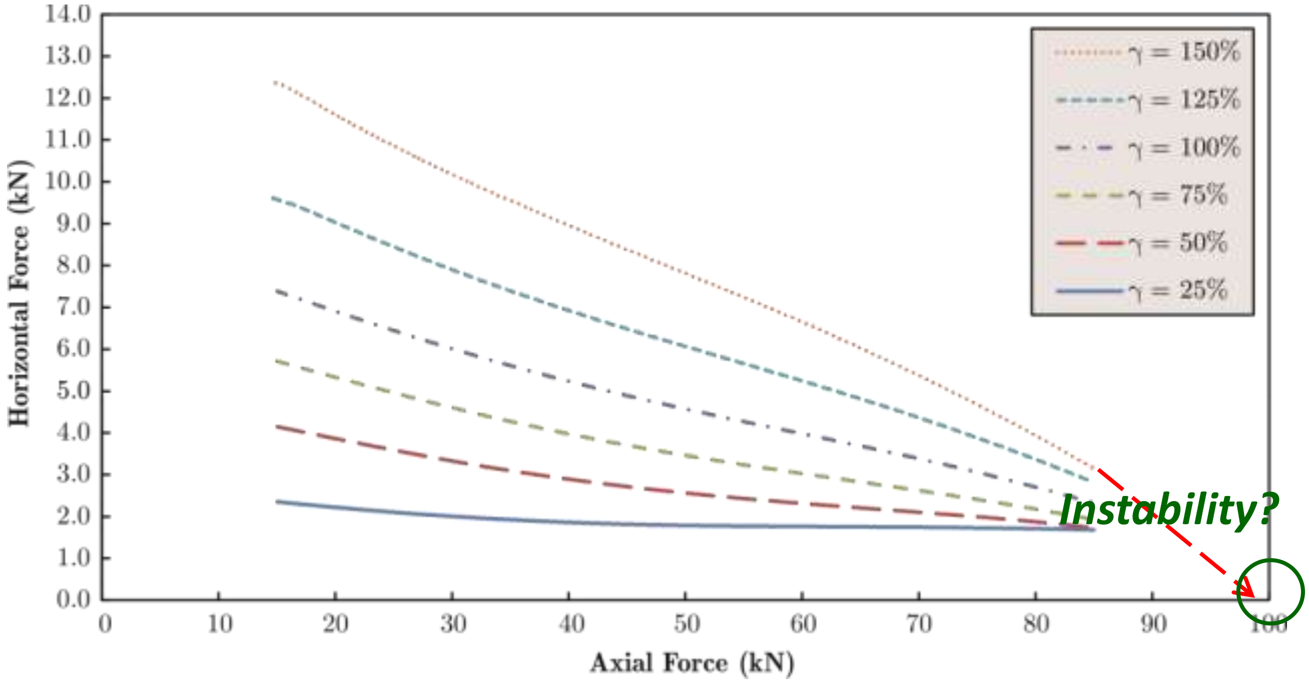


Experimental Study

Natural Rubber Devices –Reduced-scale specimens

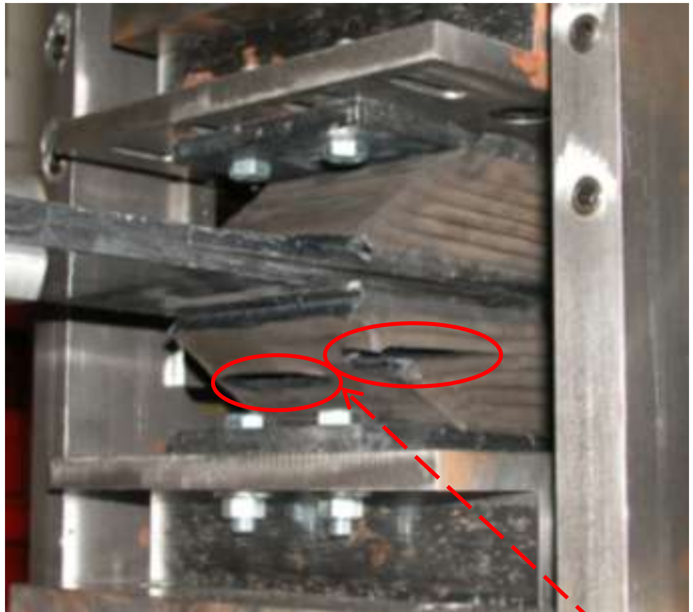


Objectives:

- ✓***Verify the influence of unscragged/scragged properties for different shear deformations;***
- ✓***Verify the influence on mechanical properties of changes in specimen size, shape factor and axial load;***
- ✓***Study of the seismic isolators in extreme conditions: Instability and Shear failure;*** → ✓***Generate experimental values for damage limit states for natural rubber seismic isolators to be used in fragility curves development for bridges in Quebec.***

Experimental Study

Natural Rubber Devices –Reduced-scale specimens



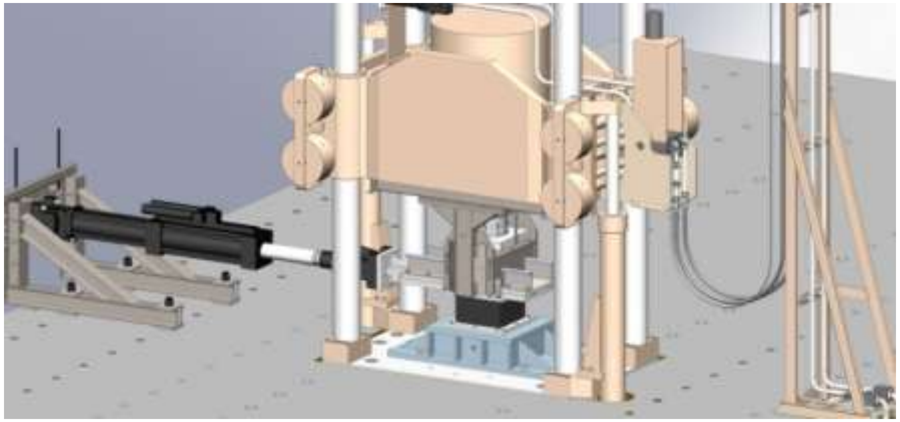
Shear Failure

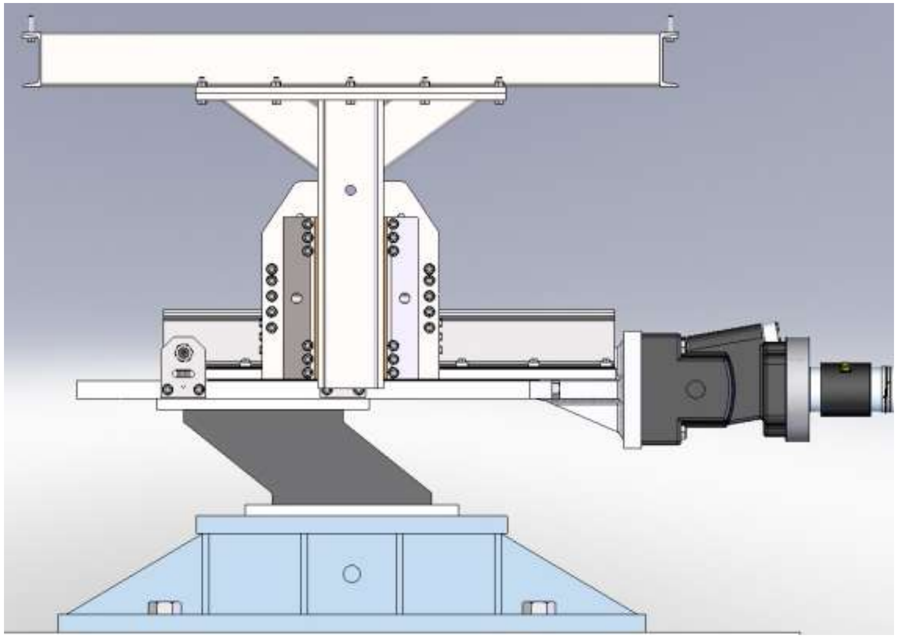
Objectives:

- ✓*Verify the influence of unscragged/scragged properties for different shear deformations;*
- ✓*Verify the influence on mechanical properties of changes in specimen size, shape factor and axial load;*
- ✓*Study of the seismic isolators in extreme conditions: Instability and Shear failure;* → ✓*Generate experimental values for damage limit states for natural rubber seismic isolators to be used in fragility curves development for bridges in Quebec.*

Experimental Study

Natural Rubber Devices – Real size





Objectives:

✓*Verify the influence of unscragged/scragged properties for different shear deformations;*

✓*Verify the influence on mechanical properties of changes in specimen size, shape factor and axial load;*

✓*Study of the seismic isolators in extreme conditions: Instability and Shear failure;* → ✓*Generate experimental values for damage limit states for natural rubber seismic isolators to be used in fragility curves development for bridges in Quebec.*

Experimental Study

Natural Rubber Devices – Real size

Experimental Study

Natural Rubber Devices – Real size

Experimental Study

Natural Rubber Devices – Real size

Analytical Fragility Curves

Seismic Fragility

C (capacity) – related to LS
D (demand) – related to the response GMTH

→ *Represented as Lognormal Distributions*

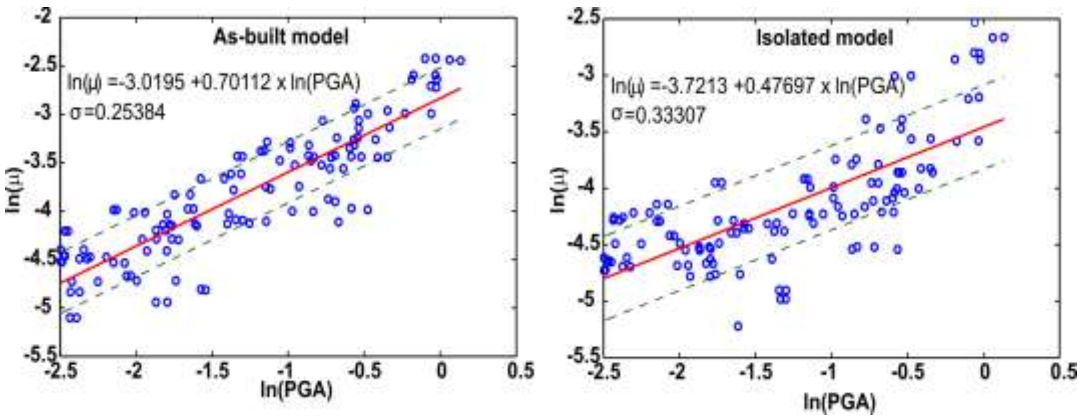
$$P[LS \mid IM] = \Phi\left[\frac{\ln(S_d / S_c)}{\sqrt{\beta_d^2 + \beta_c^2}}\right]$$

$\Phi[\cdot]$ – *standard normal distribution function*

S_d, S_c – *median structural demand and capacity*

β_d, β_c – *logarithmic standard deviation*

pairs bridge/GMTH



Peak Responses Projected Into a Lognormal Space

→

$$\ln(S_d) = \ln(a) + b \cdot \ln(IM)$$

Analytical Fragility Curves

Seismic Fragility

C (capacity) – related to LS
D (demand) – related to the response GMTH

→ *Represented as Lognormal Distributions*

$$P[LS \mid IM] = \Phi\left[\frac{\ln(S_d / S_c)}{\sqrt{\beta_d^2 + \beta_c^2}}\right]$$

Problem: establish quantitative values for different levels of damage associated to the seismic isolators based on shear deformations.

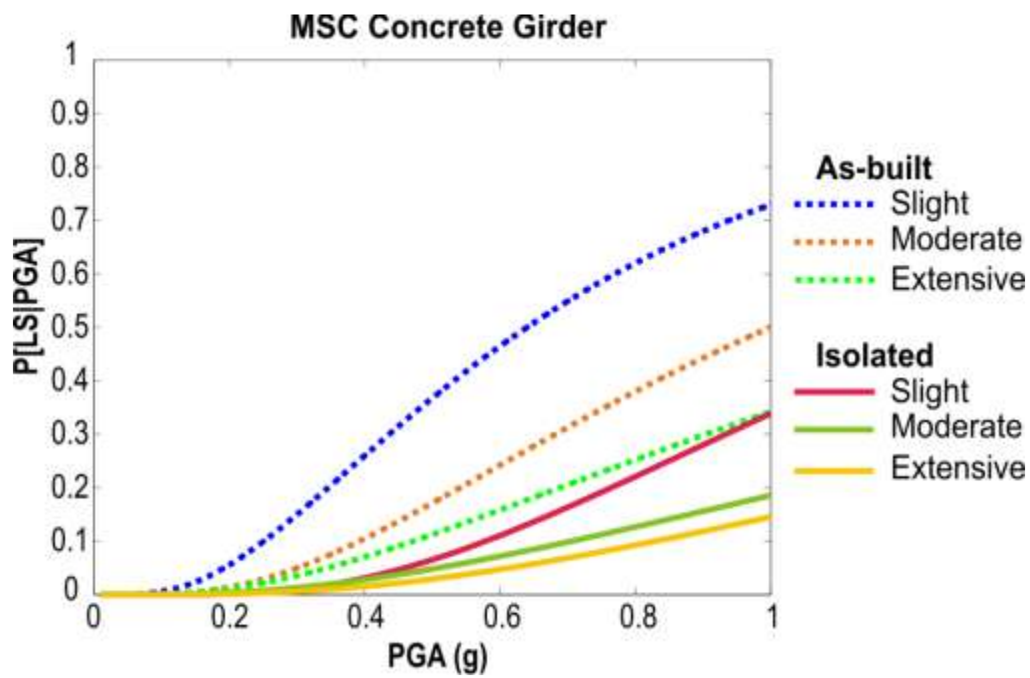
There is a lack of studies focusing on the determination of the median capacity values to be used in vulnerability assessment of structures retrofitted using natural rubber seismic isolators.

These values are necessary to generate analytical fragility curves on component levels (accounting for the damage localized in seismic isolators) which will be used to generate system fragility curves for the portfolio of bridges in Quebec considering a retrofit with NRB devices .

Analytical Fragility Curves

MSC Concrete bridges

As-Built: probable damages concentrate in columns, sliding of conventional elastomeric bearings and abutment walls.



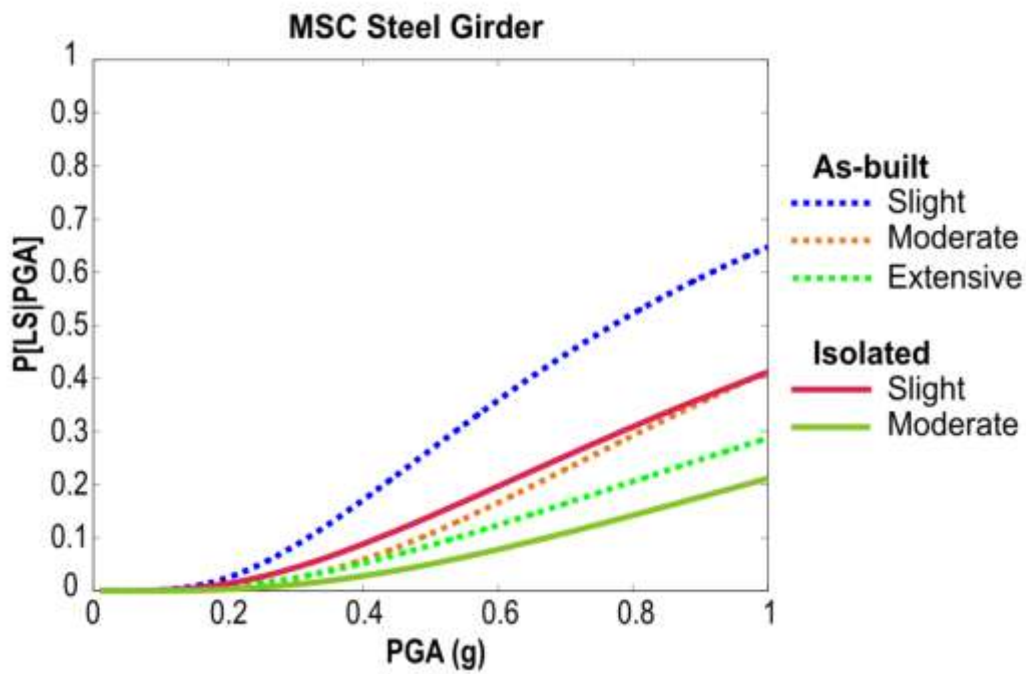
Component	Slight		Moderate		Extensive		Complete	
	Median	Disp	Median	Disp	Median	Disp	Median	Disp
Column	2.294	1.161	N/A	N/A	N/A	N/A	N/A	N/A
Ab_longW	1.175	0.565	1.777	0.565	2.588	0.603	N/A	N/A
Ab_tranW	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Ab_longF	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Ab_tranF	1.41	0.494	N/A	N/A	N/A	N/A	N/A	N/A
Isol_long	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Isol_tran	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
System	1.071	0.683	1.628	0.727	2.395	0.827	N/A	N/A

Retrofit: most probable damages concentrate only at the level of abutment wall in longitudinal direction. Columns and foundations protected.

Analytical Fragility Curves

MSC Steel bridges

As-Built: probable damages concentrate in columns, sliding of conventional elastomeric bearings and abutment walls.



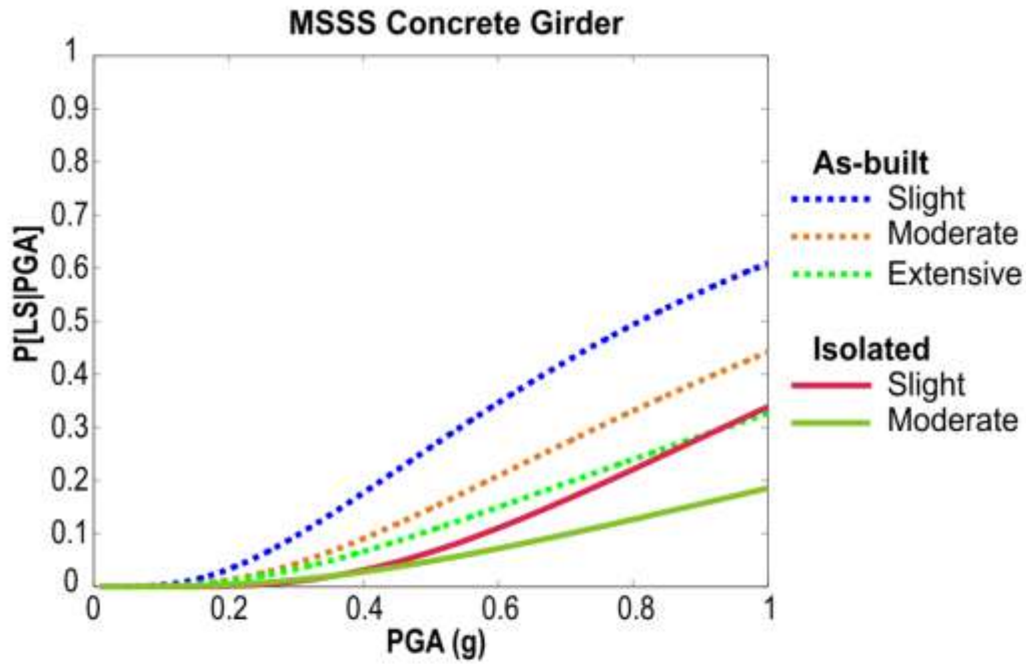
Component	Slight		Moderate		Extensive		Complete	
	Median	Disp	Median	Disp	Median	Disp	Median	Disp
Column	2.193	1.089	N/A	N/A	N/A	N/A	N/A	N/A
Ab_longW	1.401	0.731	2.263	0.731	N/A	N/A	N/A	N/A
Ab_tranW	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Ab_longF	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Ab_tranF	2.163	0.668	N/A	N/A	N/A	N/A	N/A	N/A
Isol_long	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Isol_tran	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
System	1.199	0.812	1.937	0.826	N/A	N/A	N/A	N/A

Retrofit: most probable damages concentrate only at the level of abutment wall in longitudinal direction. Columns and foundations protected.

Analytical Fragility Curves

MSSS Concrete bridges

As-Built: probable damages concentrate in columns for slight LS (spalling), sliding of conventional elastomeric bearings and abutment walls.



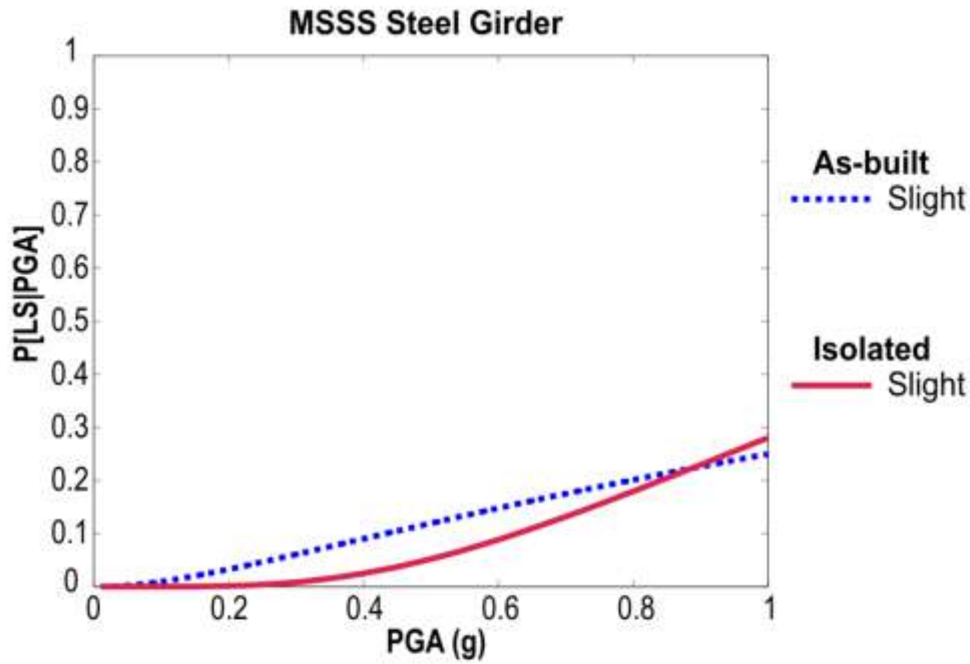
Component	Slight		Moderate		Extensive		Complete	
	Median	Disp	Median	Disp	Median	Disp	Median	Disp
Column	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Ab_longW	1.384	0.518	2.173	0.518	N/A	N/A	N/A	N/A
Ab_tranW	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Ab_longF	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Ab_tranF	1.722	0.573	N/A	N/A	N/A	N/A	N/A	N/A
Isol_long	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Isol_tran	2.596	0.761	N/A	N/A	N/A	N/A	N/A	N/A
System	1.301	0.632	2.237	1.013	N/A	N/A	N/A	N/A

Retrofit: most probable damages concentrate only at the level of abutment wall in longitudinal direction. Columns and foundations protected.

Analytical Fragility Curves

MSSS Steel bridges

As-Built: probable damages concentrate at the level of conventional elastomeric bearings with sliding and residual displacements.



Component	Slight		Moderate		Extensive		Complete	
	Median	Disp	Median	Disp	Median	Disp	Median	Disp
Column	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Ab_longW	1.580	0.521	2.519	0.521	N/A	N/A	N/A	N/A
Ab_tranW	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Ab_longF	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Ab_tranF	2.108	0.545	N/A	N/A	N/A	N/A	N/A	N/A
Isol_long	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Isol_tran	2.133	0.700	N/A	N/A	N/A	N/A	N/A	N/A
System	1.472	0.665	2.617	0.970	N/A	N/A	N/A	N/A

Retrofit: most probable damages concentrate only at the level of abutment wall in longitudinal direction. Replacing elastomeric bearings by isolators solve the problem of sliding and residual displacement.

Conclusions

As-built bridges in Quebec

- Lower LS system fragilities tend to be governed by the fragility of elastomeric bearings and columns and the highest LSs by the abutment walls
- continuous bridges are more vulnerable than simply supported bridges.
- Steel-girder bridge class (with elastomeric bearings) evidenced less fragility than concrete girder bridge class.
- The thick-slab bridge class are the most vulnerable in Quebec

Conclusions

Isolated bridges

- damages are concentrated at the level of abutment wall in longitudinal direction.
- clearance considerations for isolated bridges in transverse and longitudinal directions.
- Columns and foundations are protected.
- Fragility analyses can be used to determine the potential losses resulting from earthquakes and to prioritize retrofitting.

CSRN
RCRP

Canadian Seismic Research Network
Réseau canadien pour la recherche parasismique

Funded by NSERC / *Subventionné par le CRSNG*

Seismic Instrumentation and Structural Health Monitoring of Bridges in BC

Prof. Carlos E. Ventura
University of British Columbia

Dr. Martin Turek
BC Ministry of Transportation and Highways

Workshop on the Seismic Isolation and Damping of Bridge Structures
Vancouver, 30 April 2012

Outline

1) Value of Instrumentation of Bridges

2) Bridge Monitoring Program in BC

Comparative Study of Dynamic Response of Two Instrumented Bridges at Different Levels of Ground Shaking

Case Study 1

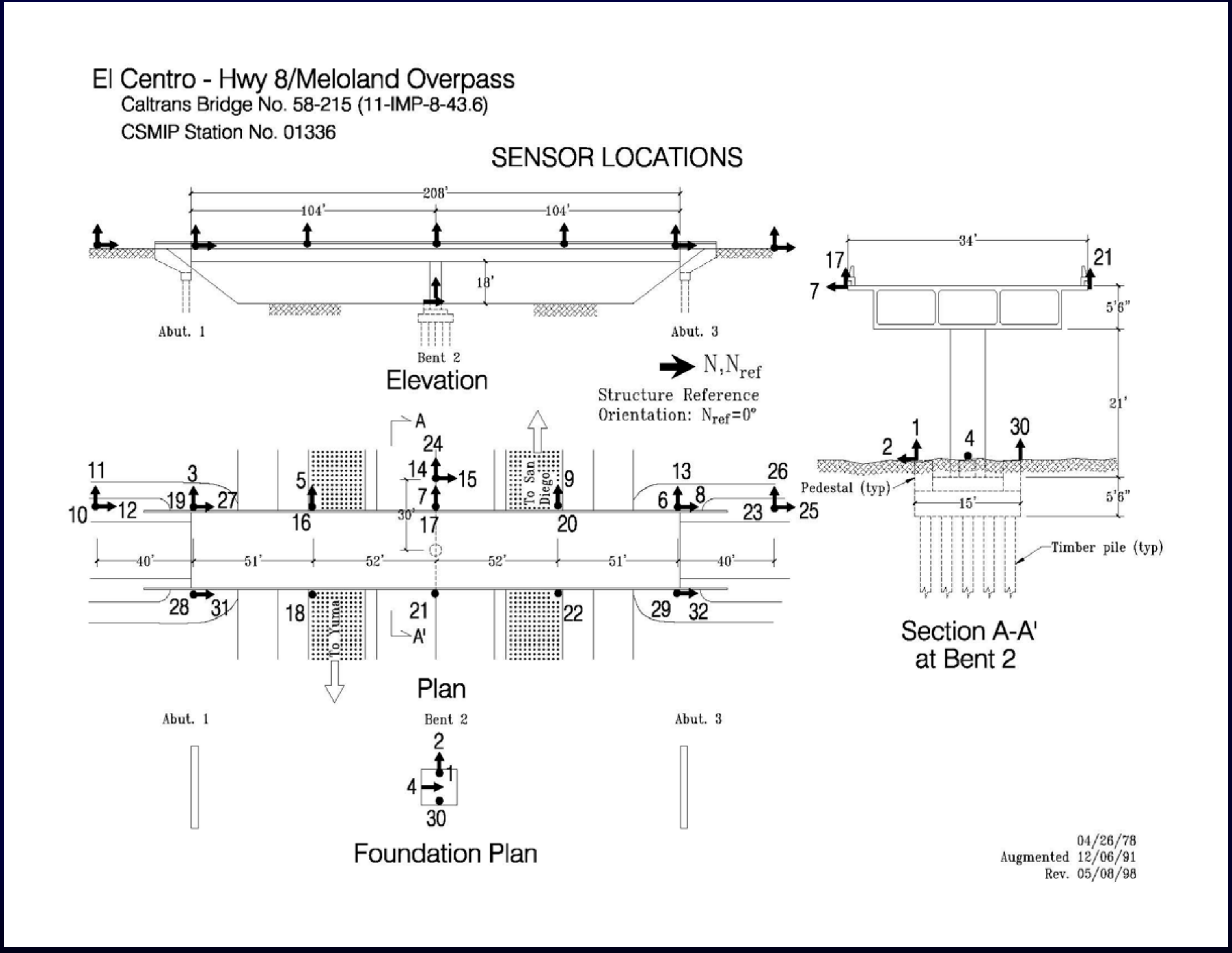
Meloland Road Overpass (MRO) in Southern California

Location of MRO


N
32.773,-115.447
Image U.S. Geological Survey
© 2011 Google
© 2011 INEGI
Google
Imagery Date: Feb 1, 2008
32°46'23.21" N 115°26'41.69" W elev -18 ft
Eye alt 1990 ft


FOR SALE

MRO Seismic Instrumentation



MRO Seismic Instrumentation

Earthquake	Epicentral Distance (km)	Horiz. Peak Acceleration. (g) Ground	Structure
ImperialValley, 1979	19.3	0.318	0.482
Calexico, 2010	58.9	0.213	0.474
SuperstitionHills, 1987	45.0	0.182	0.242
Calexico , 2009	41.2	0.174	0.509
Calexico , 2010	35.2	0.031	0.061
SuperstitionHills, 1987	46.0	0.030	0.070
Cerro Prieto, 2008	41.9	0.020	0.058
Calexico , 2008	50.4	0.017	0.027
Calexico Aftershock , 2009	34.9	0.015	0.039
CerroPrietoEvent2 , 2008	37.0	0.014	0.042
CerroPrietoEvent1 , 2008	45.0	0.012	0.035
Borrego Springs, 2010	120.2	0.012	0.054
Calexico, 2008	24.5	0.006	0.020

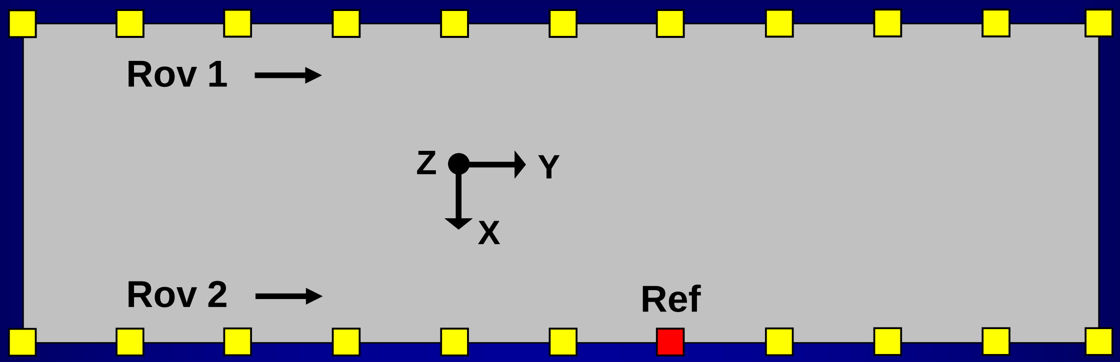
MRO Modal Parameters

(Mosquera, et al., 2009)

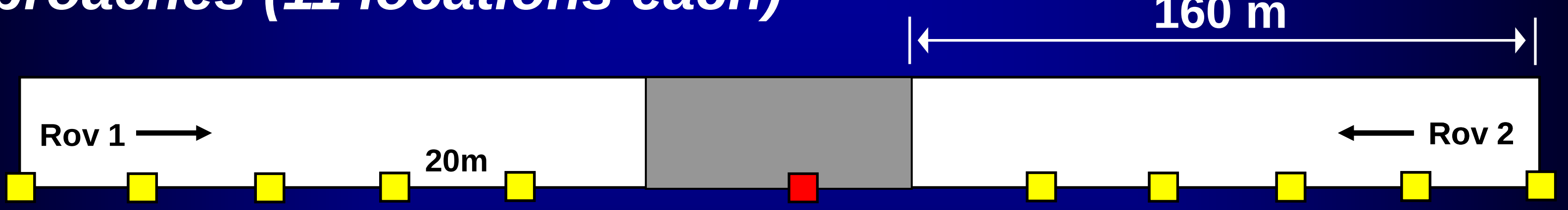
Mode	Cerro Prieto		Cerro Prieto Event 1		Cerro Prieto Event 2		Calexico		Calexico	
	f (Hz)	ζ (%)	f (Hz)	ζ (%)	f (Hz)	ζ (%)	f (Hz)	ζ (%)	f (Hz)	ζ (%)
1	3.37	1.12	3.42	1.41	3.43	1.32	3.38	1.49	3.38	1.67
2	4.45	21.4	4.31	21.27	4.47	18.70	3.98	22.79	3.97	17.40
3	4.86	3.6	4.92	2.31	4.90	2.43	4.82	2.79	4.81	3.45
4	7.14	7.4	7.32	5.67	7.29	6.33	7.21	5.18	7.23	6.93
5	10.20	5.8	10.23	4.6	10.15	5.65	9.68	5.49	9.78	6.76
6	14.69	6.15	14.69	9.04	14.79	5.59	--	--	--	--

Ambient Vibration Testing Program in April 2010

Bridge (68 locations)



Approaches (11 locations each)



- Duration at each location = 5 minutes
- Sampling rate = 100 sps (0.01 s)
- Total duration per bridge = 10 hr

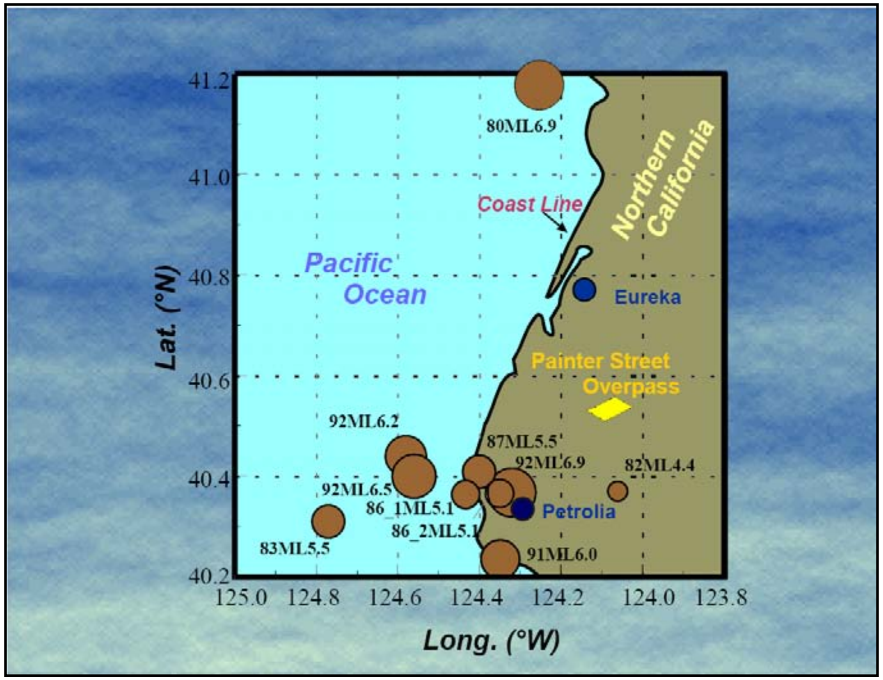
MRO Modal Parameters from AV Testing

Mode	Mode Description	Modal Parameters	
		Frequency (Hz)	Damping (%)
1	Vertical anti-symmetric mode.	3.37	1.4
2	Transverse mode.	3.63	1.0
3	Vertical symmetric mode.	4.47	2.8
4	First torsional mode.	6.74	1.8
5	Second torsional mode.	9.72	0.5
6	Second vertical anti-symmetric mode.	11.36	0.3
7	Second vertical symmetric mode.	11.82	0.4
8	Third torsional mode.	14.59	0.7
9	Third vertical anti-symmetric mode.	19.73	0.2
10	Coupled vertical and torsional mode.	23.94	0.1

Painter Street Overpass Bridge (Case Study 2)

- Structure: Two Span, Concrete box-girder and Two piers bent
- Dimension: 15.85 m wide, 80.79 m long, 7.5 m height (average)
- Skew: 38.9^0
- Abutments and piers sitting on friction piles
- Location: US Highway 101, Rio Dell, Northern California

Geographical Location of PSO Bridge



Local

Global

PSO Bridge History

- Instrumented and experienced ten significant earthquakes
- Studied by several researchers

Earthquakes Recorded

Event Code	Earthquake	Date	Mag. (M_L)	Epic. Dist. (km)	FF Accel. (g)	Struct. Accel. (g)
80ML6.9	Trinidad Offshore	8 Nov 1980	6.9	88	0.15	0.17
82ML4.4	Rio Dell	16 Dec1982	4.4	15	--	0.42
83ML5.5	Eureka	24 Aug 1983	5.5	61	--	0.22
86_1ML5.1	Cape Mendocino-1	21 Nov 1986	5.1	32	0.43	0.40
86_2ML5.1	Cape Mendocino-2	21 Nov 1986	5.1	26	0.14	0.35
87ML5.5	Cape Mendocino	31 Jul 1987	5.5	28	0.14	0.34
92ML6.9	Cape Mendocino - Petrolia	25 Apr 1992	6.9	6.4	0.54	1.09
92ML6.2	Cape Mendocino - Petrolia (AS1)	26 Apr 1992	6.2	6.2	0.52	0.76
92ML6.5	Cape Mendocino - Petrolia (AS2)	26 Apr 1992	6.5	6.4	0.26	0.31

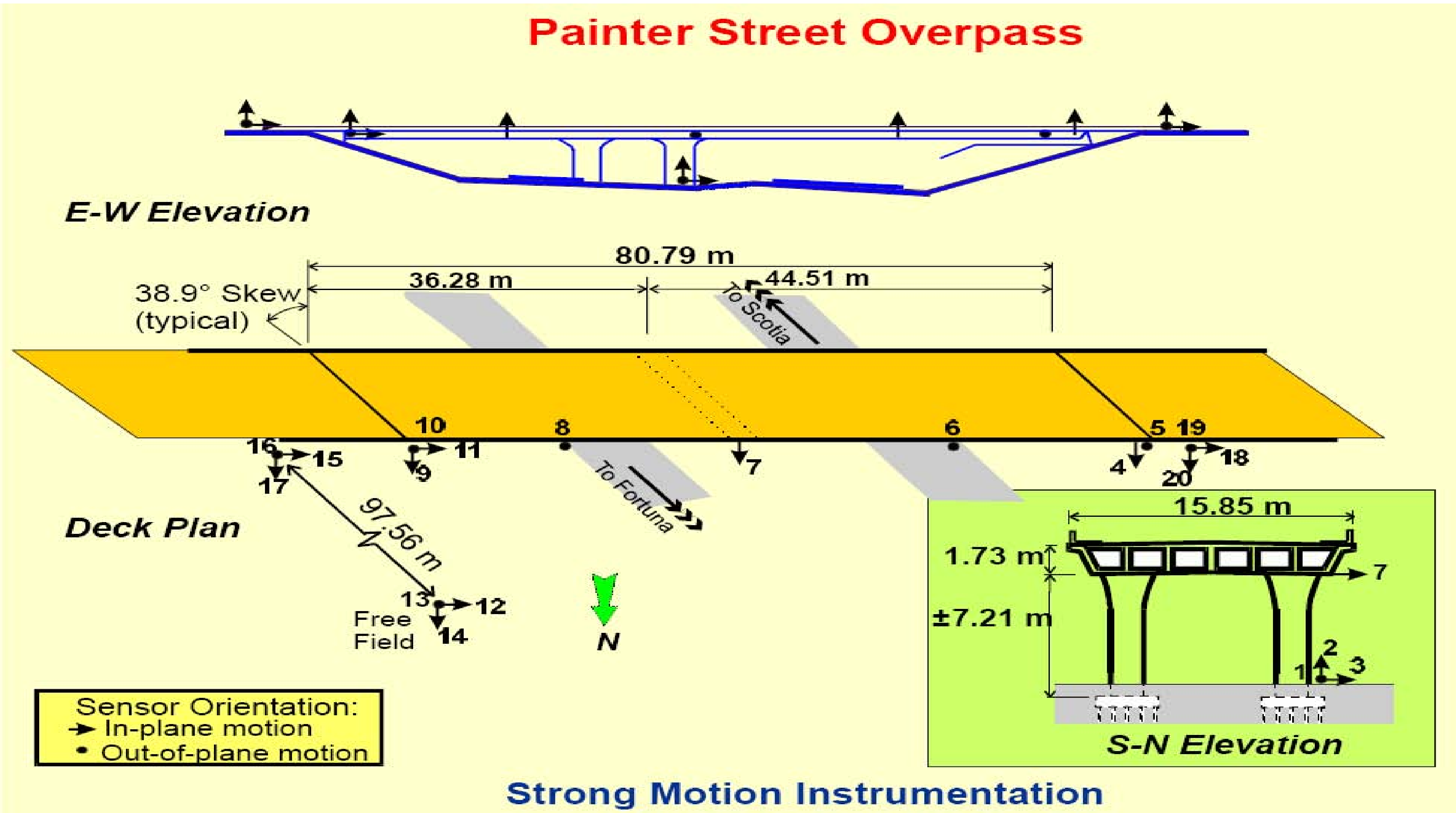
Ambient Vibration Tests & System Identification

1. Goel, 1997
2. Ventura & Felber, 1993
3. Gates & Smith, 1982

Soil-Structure Interaction

4. Zahng & Makris, 2001
5. Goel & Chopra, 1997
6. McCallen & Romstad, 1994
7. Wilson & Tan, 1990

Sensors Location



System Identification using Recorded Earthquakes

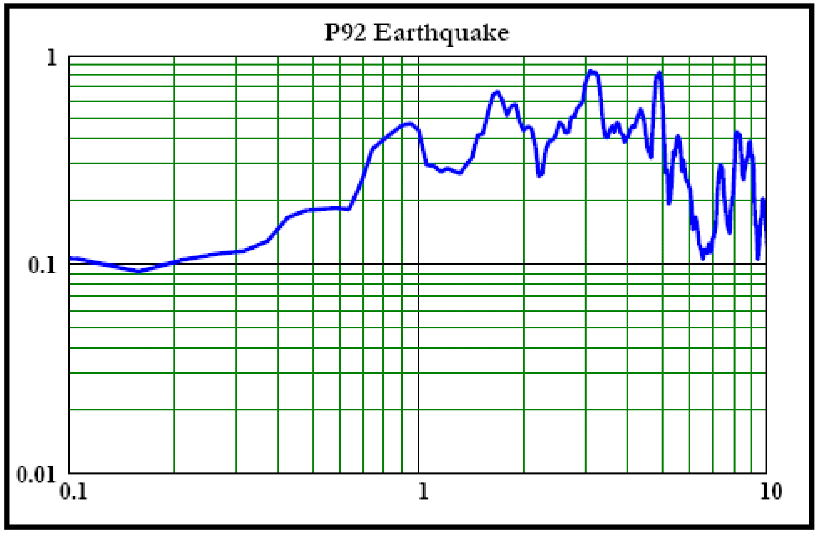
- This study was done to check the AV results of past studies (Ventura and Goel).

Earthquake (Date)	Method	1st Mode		2nd Mode		3rd Mode		4th Mode		5th Mode	
		Freq. Hz	ζ (%)	Freq. Hz	ζ (%)	Freq. Hz	ζ (%)	Freq. Hz	ζ (%)	Freq. Hz	ζ (%)
Trinidad Offshore (8 Nov 1980)	**FDD**	3.271	---	3.955	---	4.883	---	---	---	---	---
	SSI	3.194	1.17	3.864	4.65	---	---	---	---	---	---
Rio Dell (16 Dec 1982)	**FDD**	3.369	---	3.857	---	---	---	---	---	---	---
	SSI	3.395	1.60	4.097	3.56	---	---	---	---	---	---
Cape Mendocino-1 (21 Nov1986)	**FDD**	3.369	---	4.053	---	4.736	---	---	---	---	---
	SSI	3.267	3.06	3.596	4.72	4.684	1.22	---	---	---	---
Cape Mendocino-2 (21 Nov1986)	**FDD**	3.320	---	4.053	---	4.590	---	---	---	---	---
	SSI	3.375	2.10	4.021	4.12	4.779	2.24	---	---	---	---
Cape Mendocino - Petrolia (25 Apr 1992)	**FDD**	3.125	---	4.05	---	4.492	---	5.859	---	---	---
	SSI	3.147	3.92	---	---	---	---	5.468	3.39	---	---
Cape Mendocino - Petrolia (AS1) (26 Apr 1992)	**FDD**	3.027	---	---	---	4.590	---	5.713	---	6.396	---
	SSI	3.027	4.02	---	---	---	---	5.608	1.01	6.404	3.81
Cape Mendocino - Petrolia (AS2) (26 Apr 1992)	**FDD**	3.174	---	4.150	---	4.540	---	5.664	---	6.592	---
	SSI	3.060	1.471	4.013	---	---	---	5.970	4.87	6.573	0.63

System Identification of Recorded Earthquakes

Analyses done to identify the natural frequency of the site.

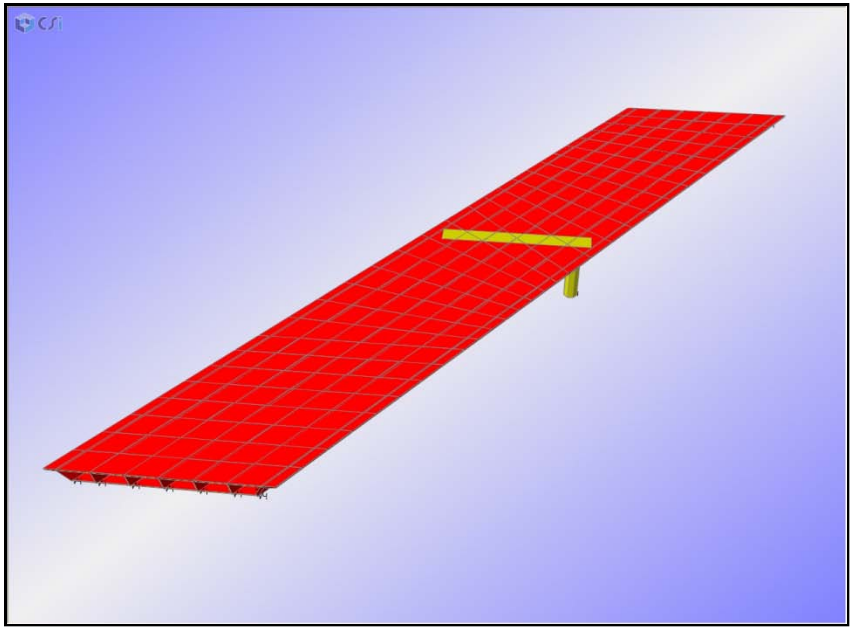
Normalized V/H Ratio vs. Frequency (Hz) by Nakamura's method



Remark:
It is always recommended to identify the site frequency in addition to the structural frequencies, otherwise data analysis may lead to erroneous conclusions

Finite Element Modeling

- As-built detail drawings were used,
- Structural geometry and properties were modeled with out any limit,
- Soil-Structure Interaction was considered,
- SAP2000® program was used.



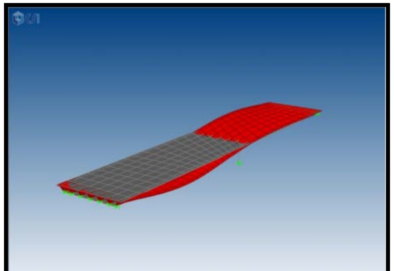
Model Calibration

- Model calibration was done based on the ambient vibration test results derived by Ventura et al, 1992.
- The values for the soil stiffness were taken from the Zhang and Makris report (2001), for the first iteration.
- These values were adjusted until the frequencies and mode shapes of the FE model and AV model were in good agreement.

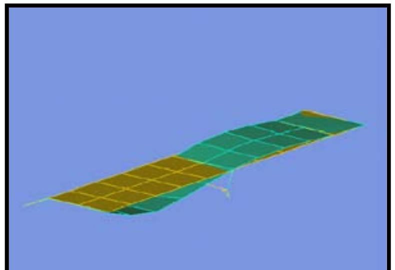
Comparison between the Ambient Vibration and the CFE model Results

Mode	**Frequency-(Period) by SAP2000 Hz - (Sec)**	**Frequency-(Period) by ARTeMIS Hz - (Sec)**
1-Vertical	**3.38-(*0.296*)**	**3.40-(*0.294*)**
2-Transverse	**4.16-(*0.240*)**	**4.10-(*0.244)***
3-Vertical	**5.07-(*0.197*)**	**4.92-(*0.203*)**
4-Vertical	**5.88-(*0.170*)**	**6.02-(*0.166*)**
5-Transverse	**6.02-(*0.166*)**	**5.97-(*0.167*)**
6-Vertical	**7.35-(*0.136*)**	**7.10-(*0.141*)**

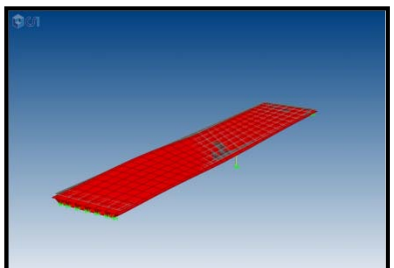
Mode Shapes of FE & AV Models



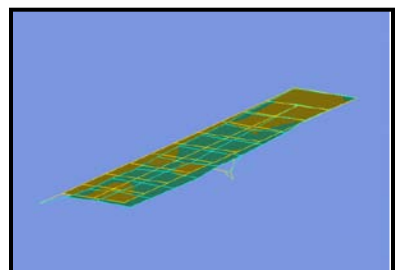
1st Mode- SAP2000



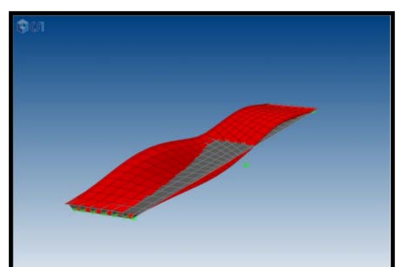
1st Mode- ARTeMIS



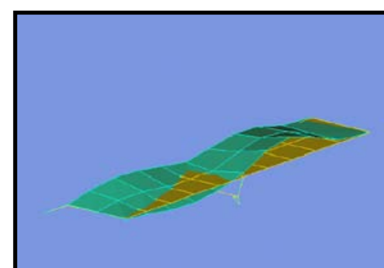
2nd Mode- SAP2000



2nd Mode- ARTeMIS



3rd Mode- SAP2000



3rd Mode- ARTeMIS

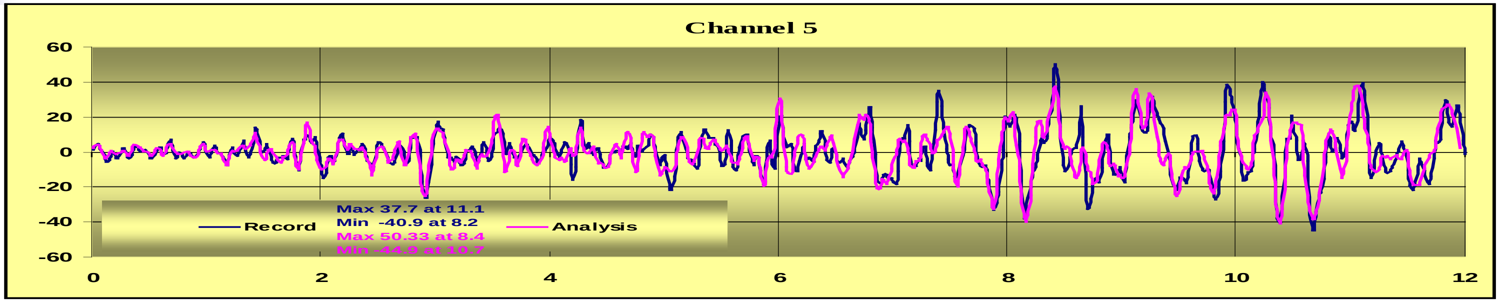
Check the CFE Model Responses

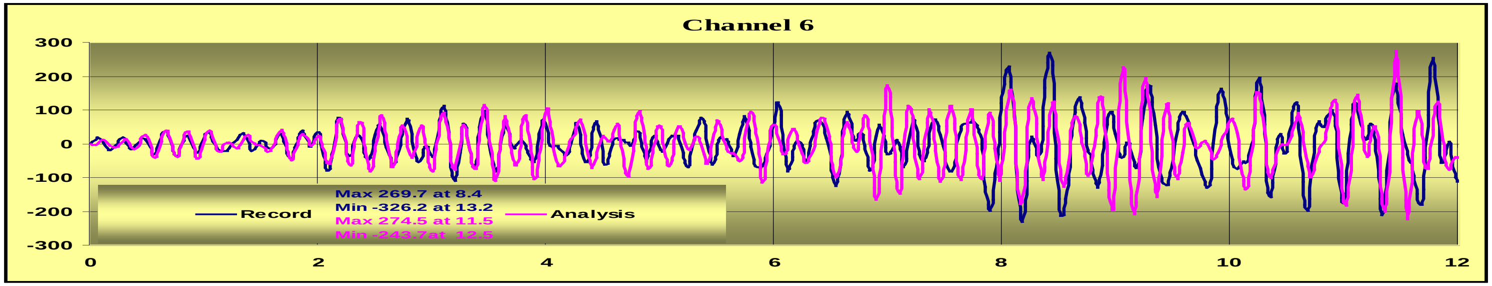
- Three earthquakes with different levels of shaking were selected for this purpose:

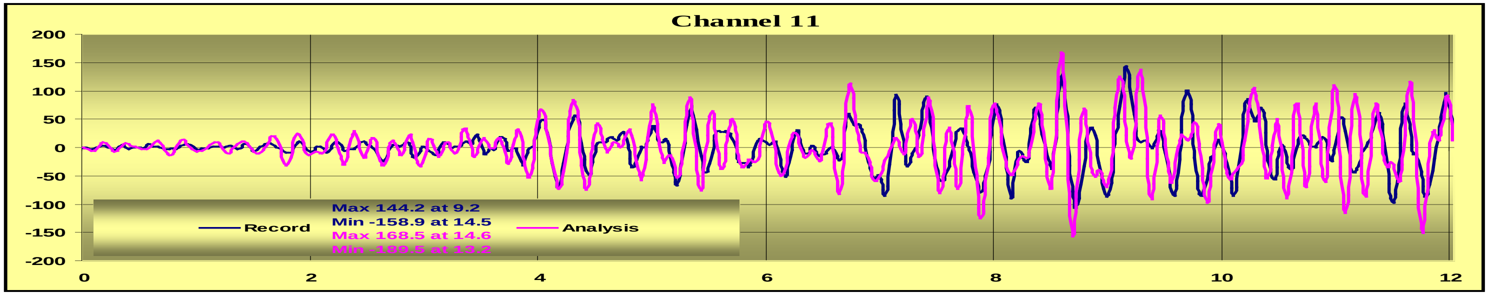
    1. Trinidad 80
    2. Cape Mendocino 86
    3. Petrolia 92

- The CFE model was analyzed with these three inputs.
- The analytical and recorded responses were compared.

Compare the Analytical and Recorded Responses

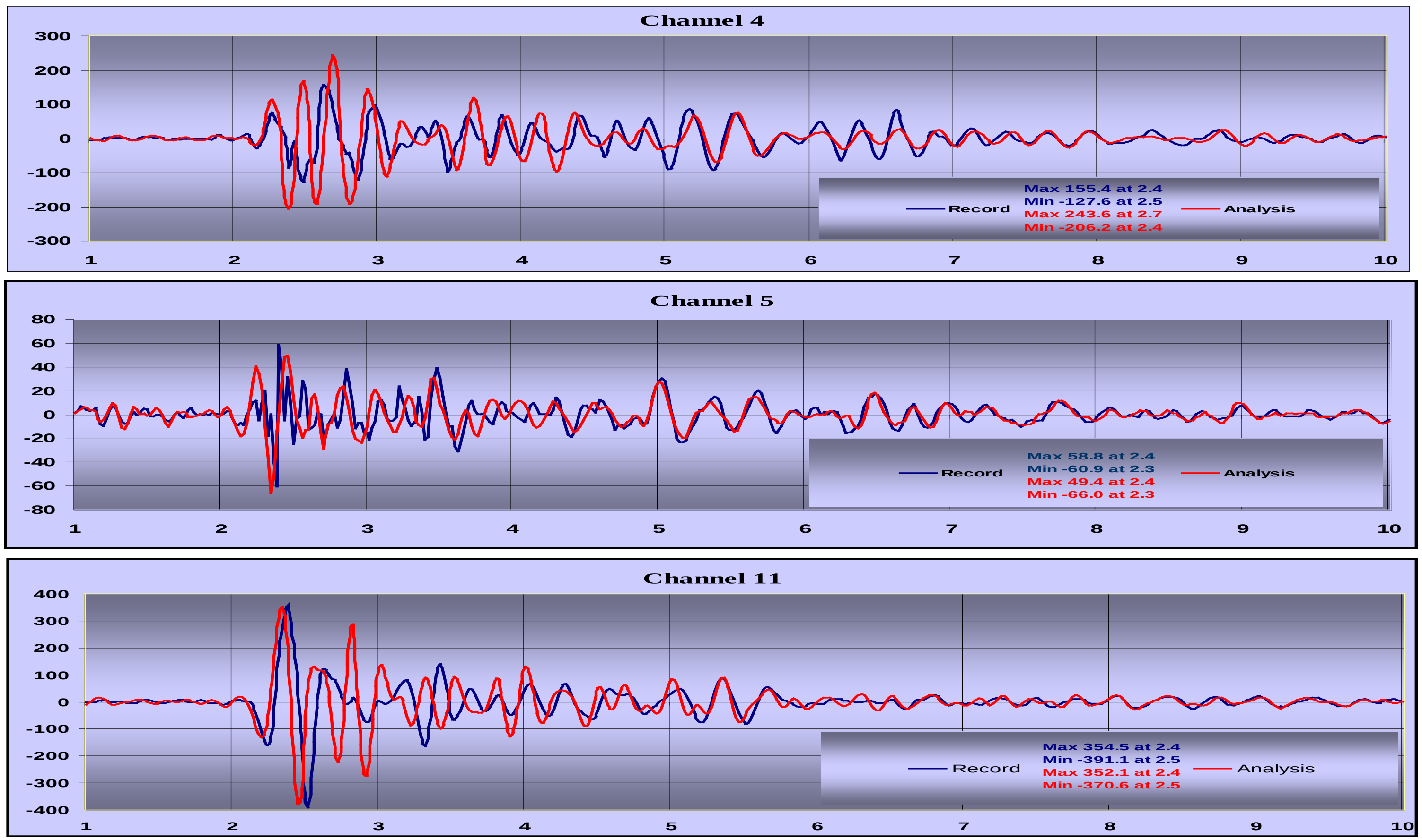






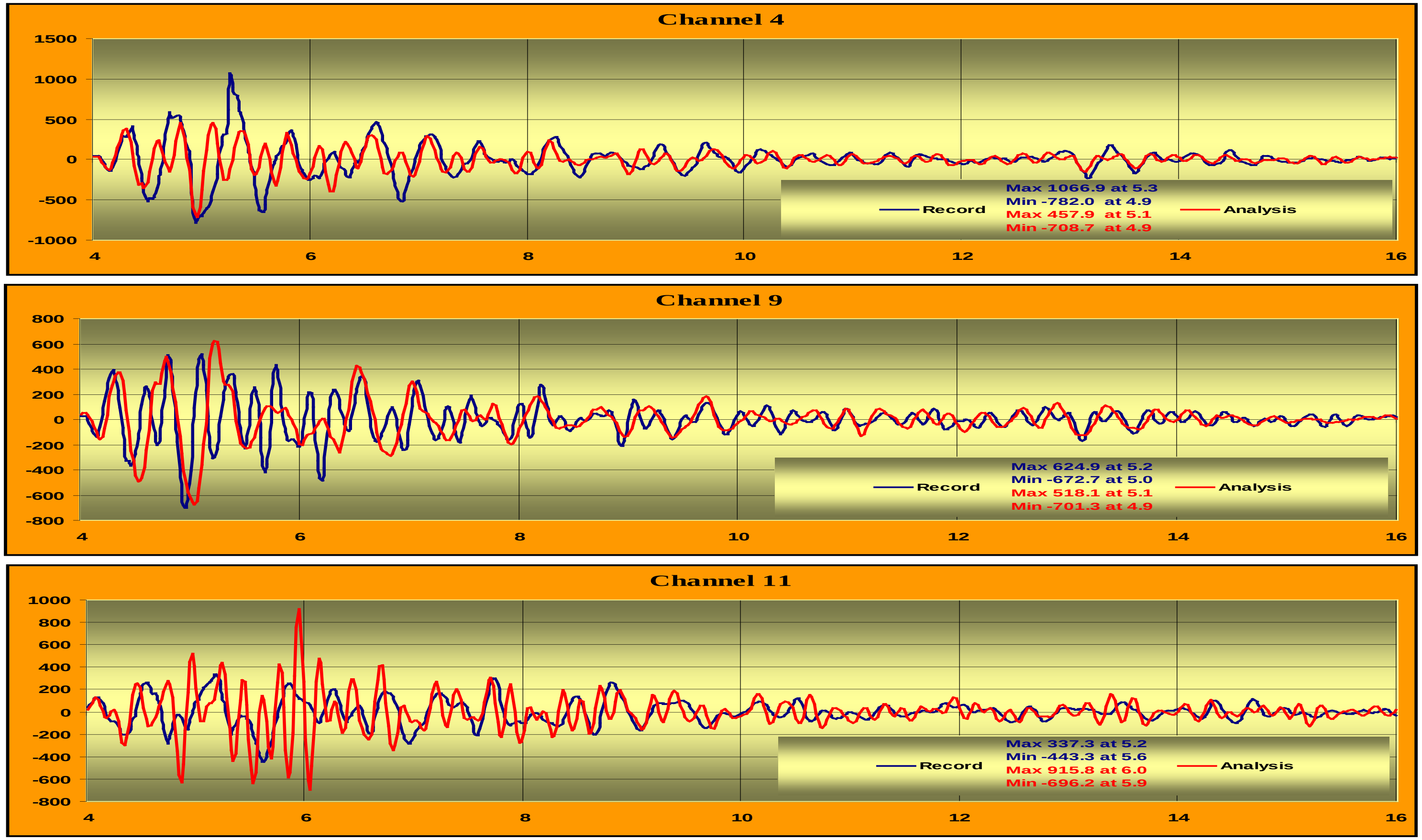
Comparison of Acceleration Time histories-Trinidad 80-(cm/s^2) vs. second

Compare the Analytical and Recorded Responses



Comparison of Acceleration Time histories-Cape Mendocino 86-(cm/s^2) vs. second

Compare the Analytical and Recorded Responses



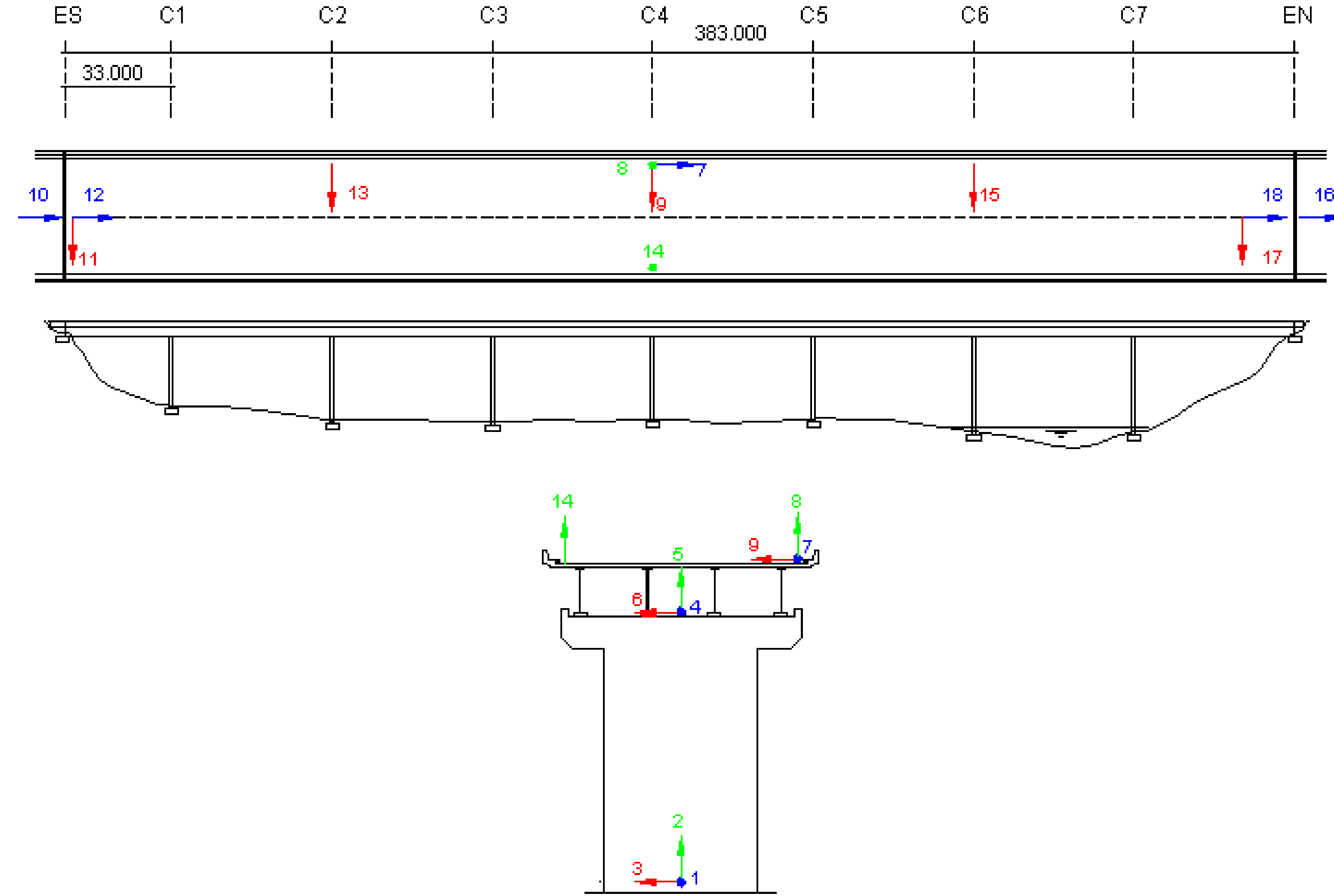
Comparison of Acceleration Time histories-Petrolia 92-(cm/s^2) vs. second

Case Study 3

Marga Marga Bridge
Viña del Mar, Chile

2010 El Maule EQ

Isolation system


ES
C1
C2
C3
C4
C5
C6
C7
EN
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33.000
10
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13
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1

Cepa C4

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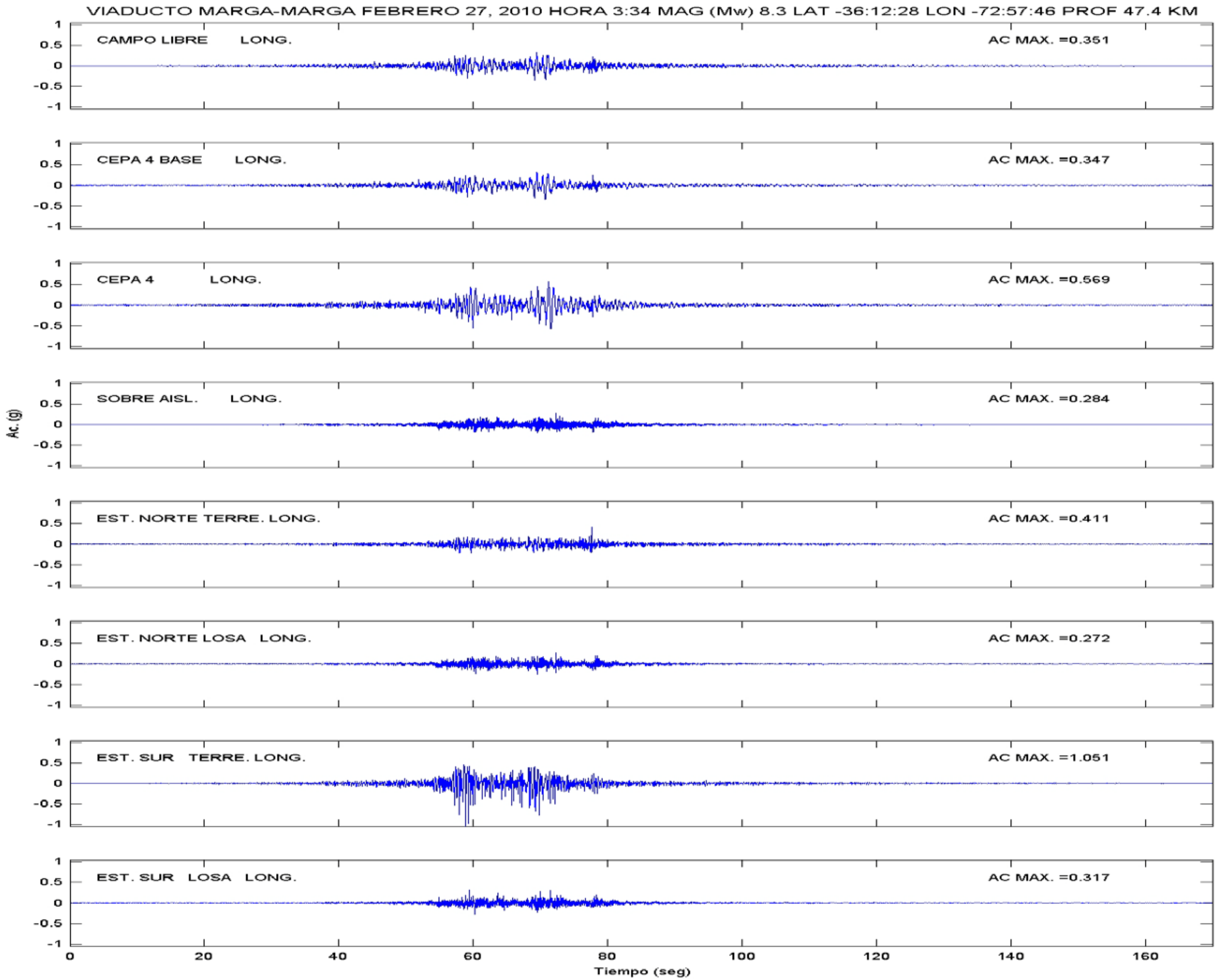

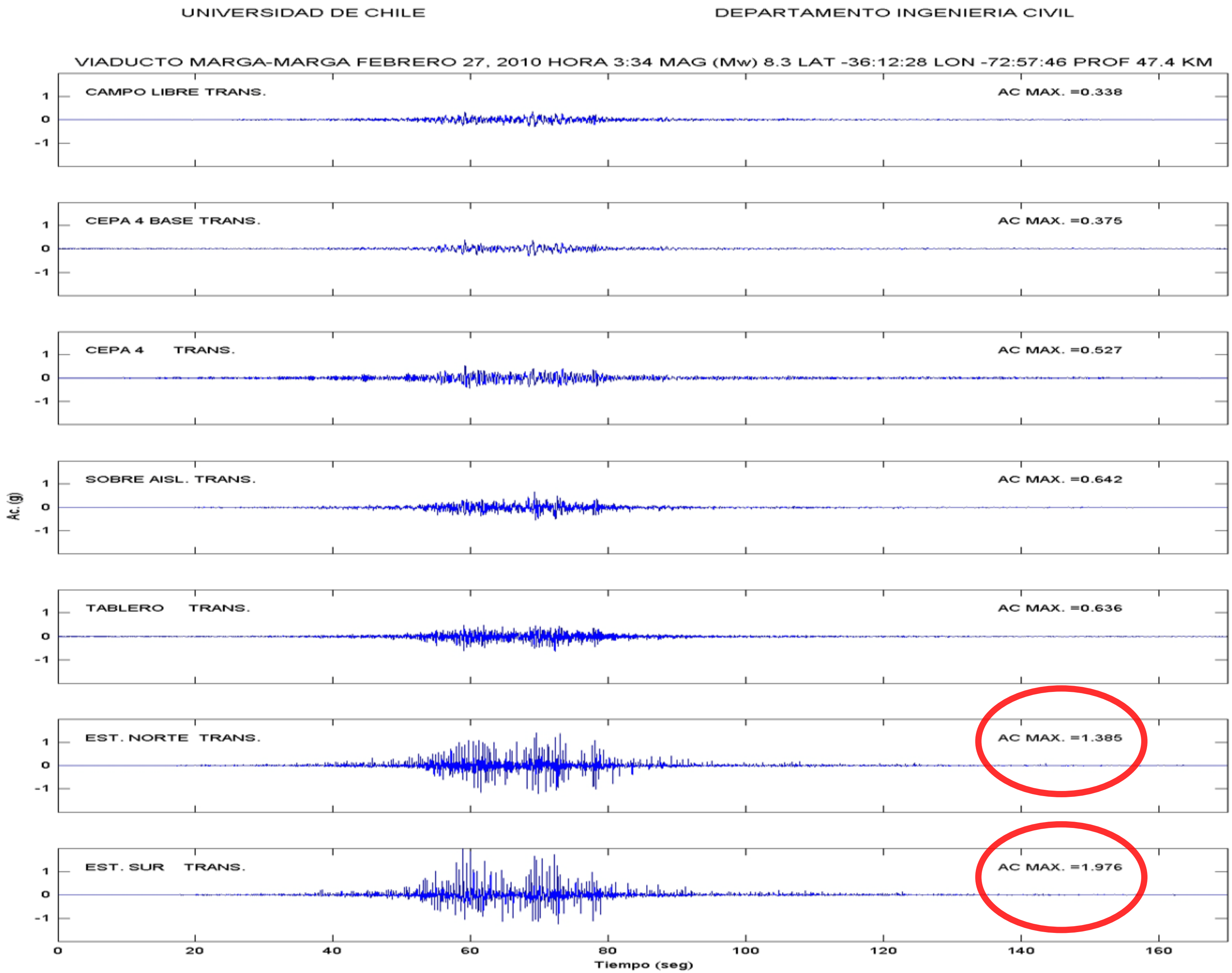

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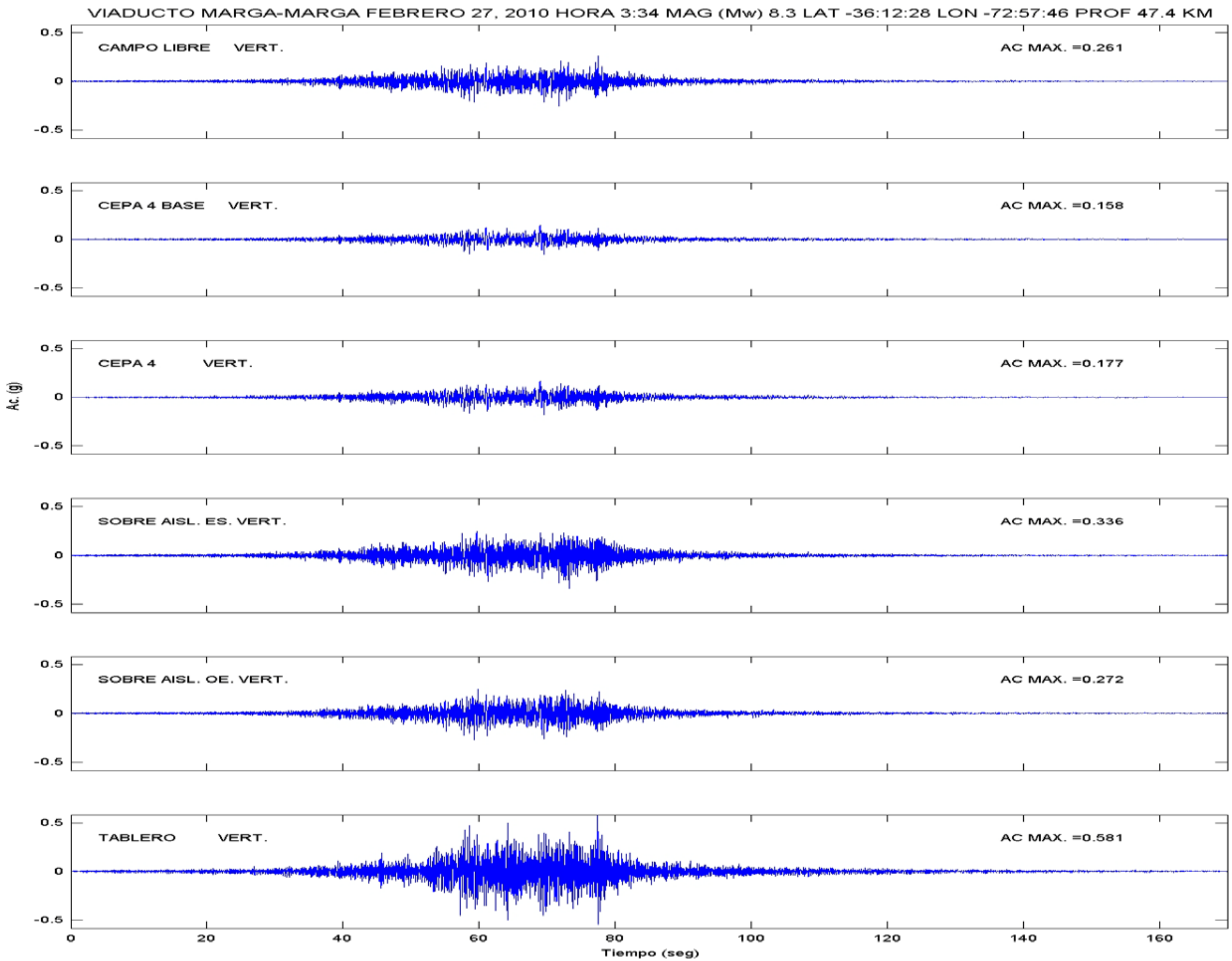
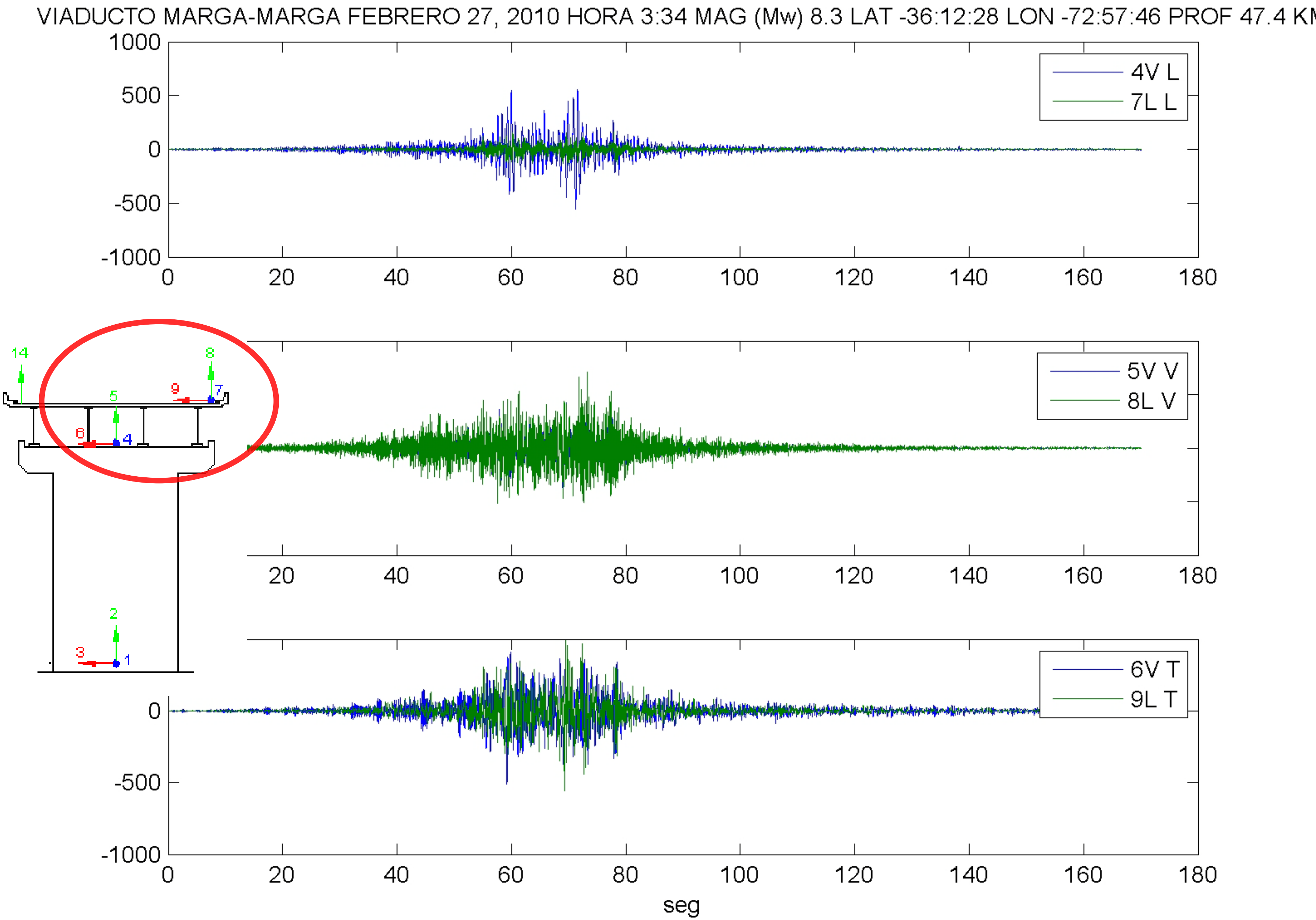
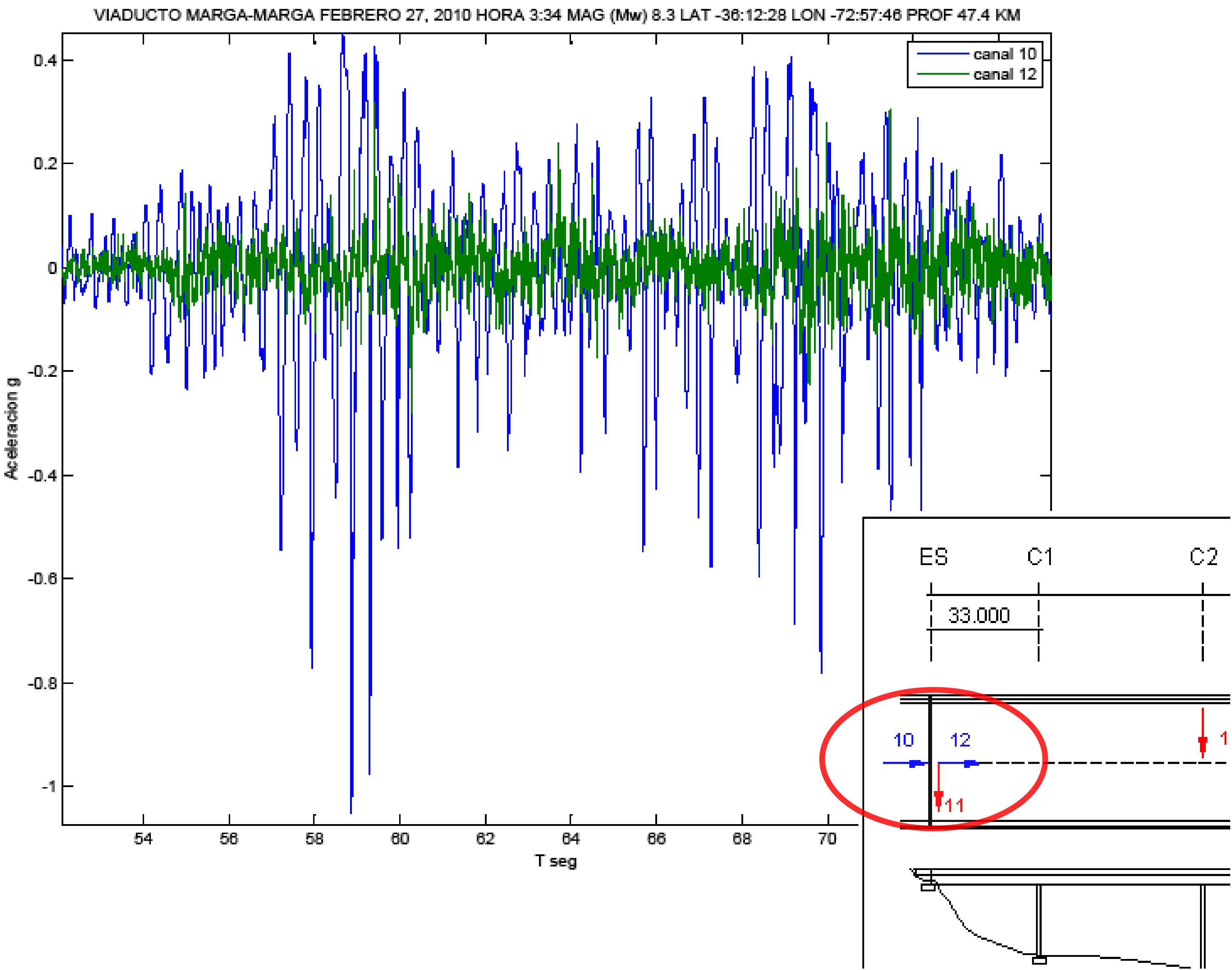
DEPARTAMENTO INGENIERIA CIVIL
VIADUCTO MARGA-MARGA FEBRERO 27, 2010 HORA 3:34 MAG (Mw) 8.3 LAT -36:12:28 LON -72:57:46 PROF 47.4 KM
CAMPO LIBRE TRANS.
AC MAX. =0.338
CEPA 4 BASE TRANS.
AC MAX. =0.375
CEPA 4 TRANS.
AC MAX. =0.527
SOBRE AISL. TRANS.
AC MAX. =0.642
TABLERO TRANS.
AC MAX. =0.636
EST. NORTE TRANS.
AC MAX. =1.385
EST. SUR TRANS.
AC MAX. =1.976
Ac. (g)
Tiempo (seg)

UNIVERSIDAD DE CHILE DEPARTAMENTO INGENIERIA CIVIL

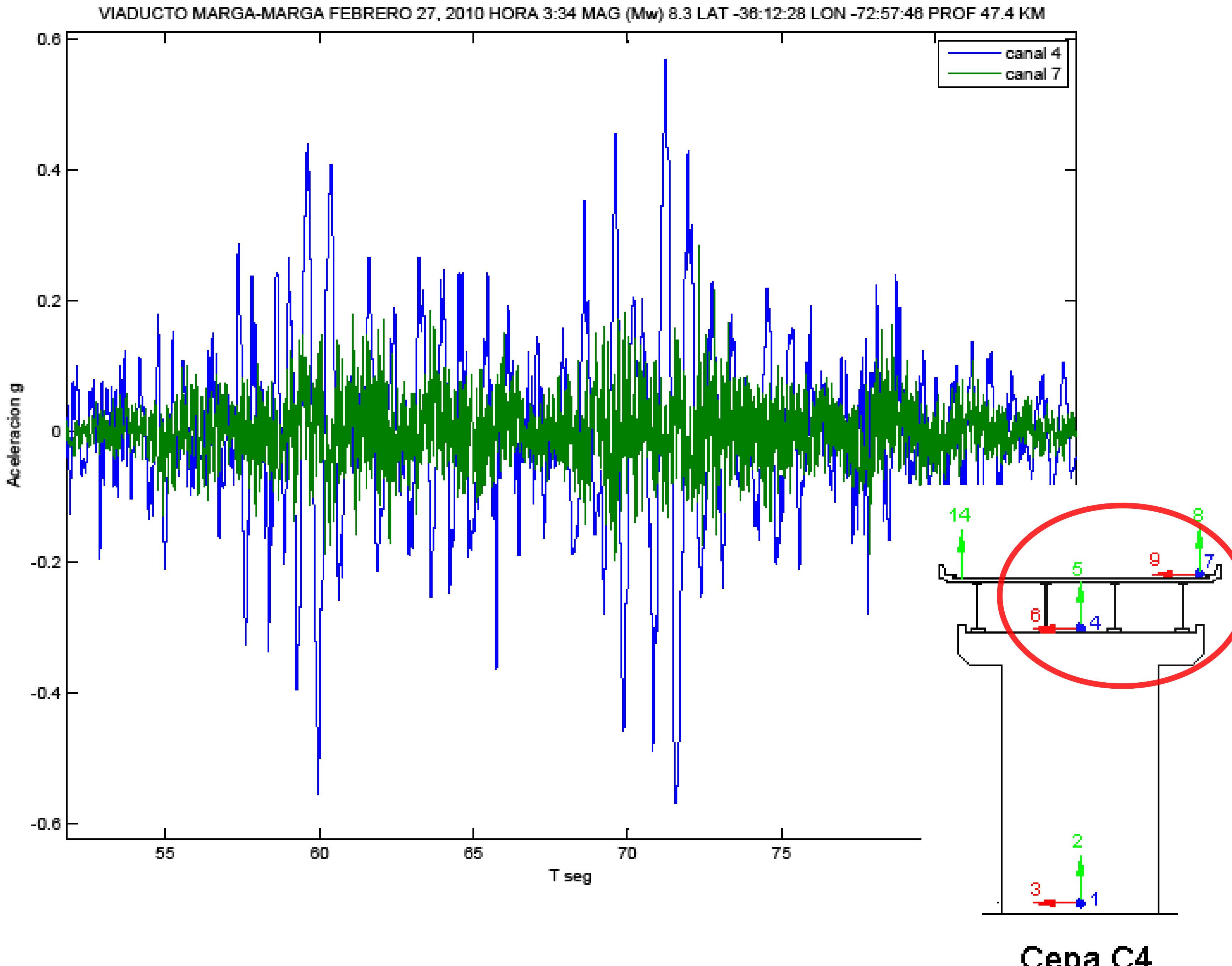




VIADUCTO MARGA-MARGA FEBRERO 27, 2010 HORA 3:34 MAG (Mw) 8.3 LAT -36:12:28 LON -72:57:46 PROF 47.4 KM
4V L
7L L
5V V
8L V
6V T
9L T
14
8
5
9
7
6
4
2
3
1
1000
500
0
-500
-1000
0
20
40
60
80
100
120
140
160
180
seg



VIADUCTO MARGA-MARGA FEBRERO 27, 2010 HORA 3:34 MAG (Mw) 8.3 LAT -36:12:28 LON -72:57:46 PROF 47.4 KM
canal 10
canal 12
Aceleracion g
T seg
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
54
56
58
60
62
64
66
68
70
ES
C1
C2
33.000
10
12
11



VIADUCTO MARGA-MARGA FEBRERO 27, 2010 HORA 3:34 MAG (Mw) 8.3 LAT -36:12:28 LON -72:57:46 PROF 47.4 KM
canal 4
canal 7
Aceleracion g
T seg
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
55
60
65
70
75
14
8
9
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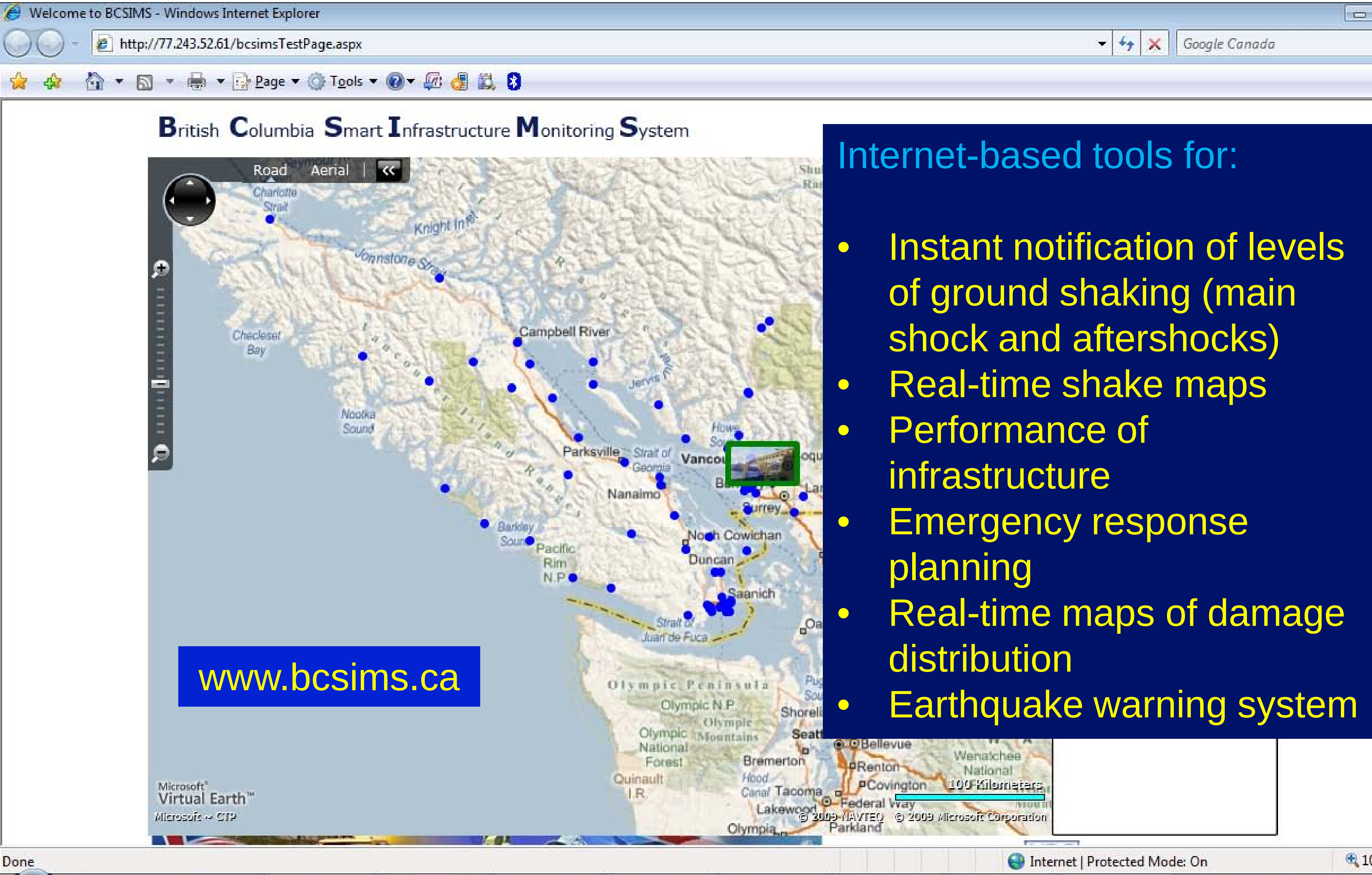
Current State of Bridge Monitoring in BC

BC Smart Infrastructure Monitoring System (BCSIMS Project)

Collaborative effort between BCMOT-UBC-GSC (and BCMOE)



www.bcsims.ca

Internet-based tools for:

- Instant notification of levels of ground shaking (main shock and aftershocks)
- Real-time shake maps
- Performance of infrastructure
- Emergency response planning
- Real-time maps of damage distribution
- Earthquake warning system

BCSIMS Summary

The technology being implemented by the Ministry and UBC will be used to:

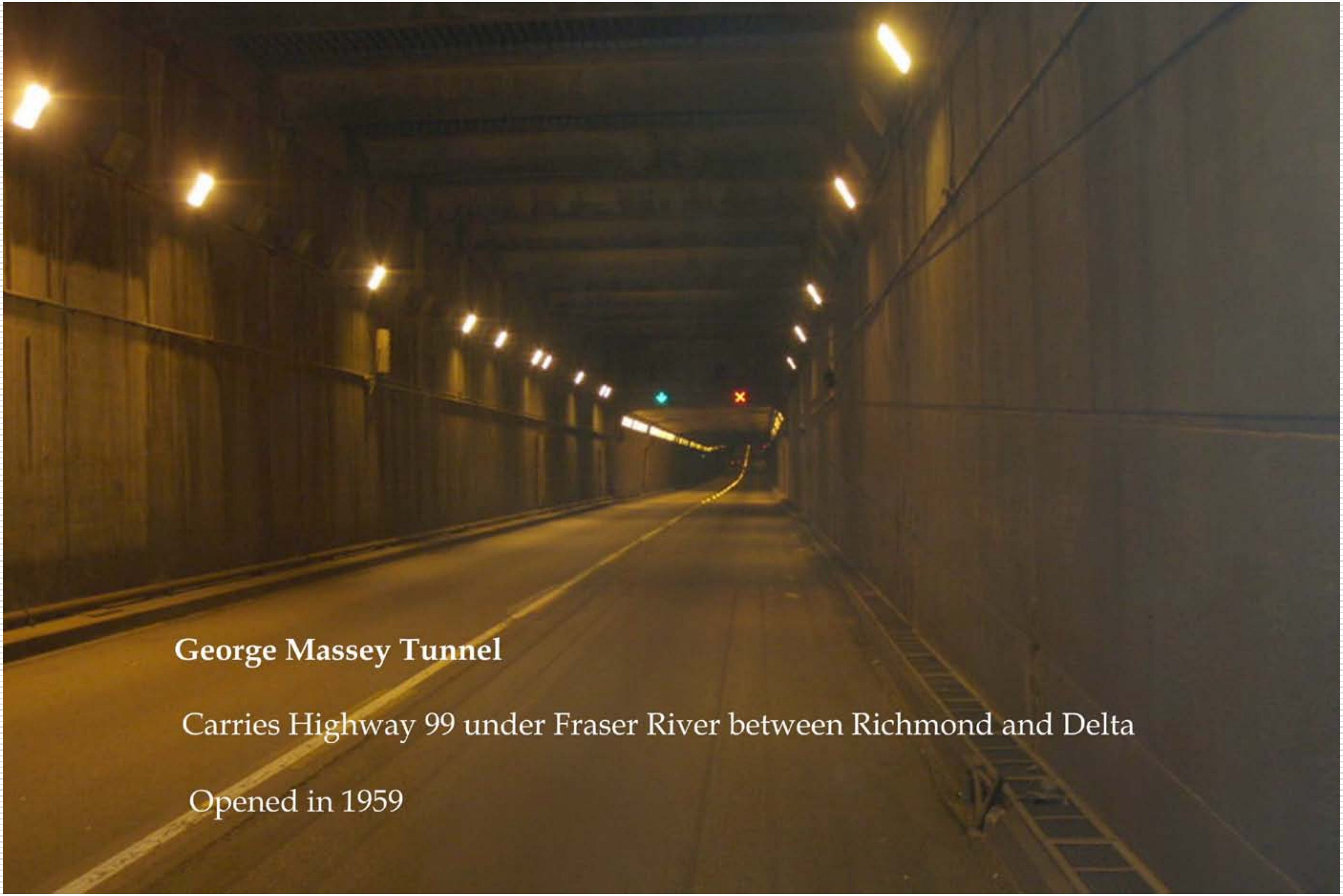
1. Detect, analyze and localize damage to structures;
2. Transmit the data regarding these structures in real time via the internet
3. Display in animated and static web pages the data as appropriate for use by the Ministry and UBC.

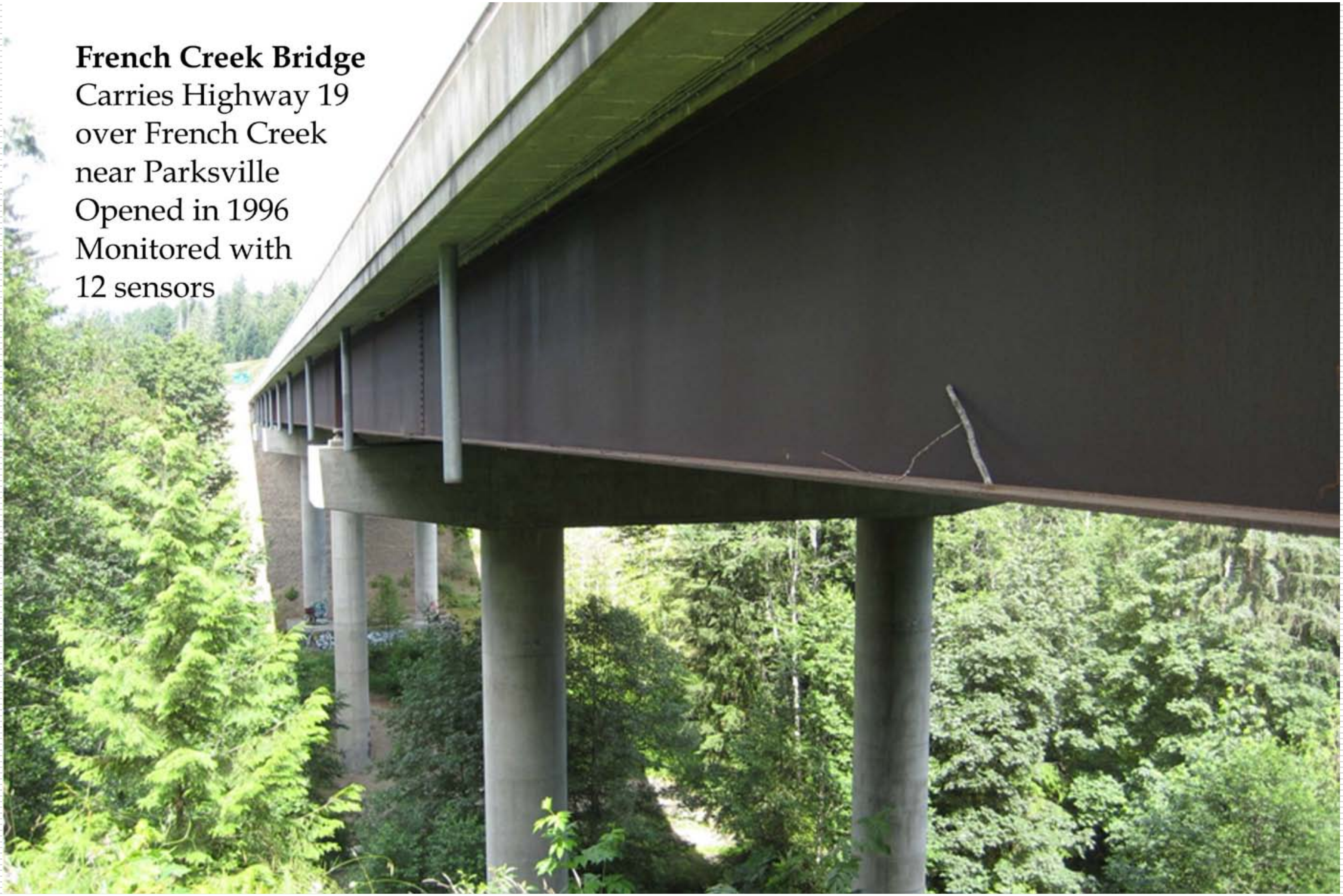
The alert systems and public access web pages will display real time seismic data from the BC Strong Motion Network to provide input for assessments by the Ministry of non-instrumented bridges.

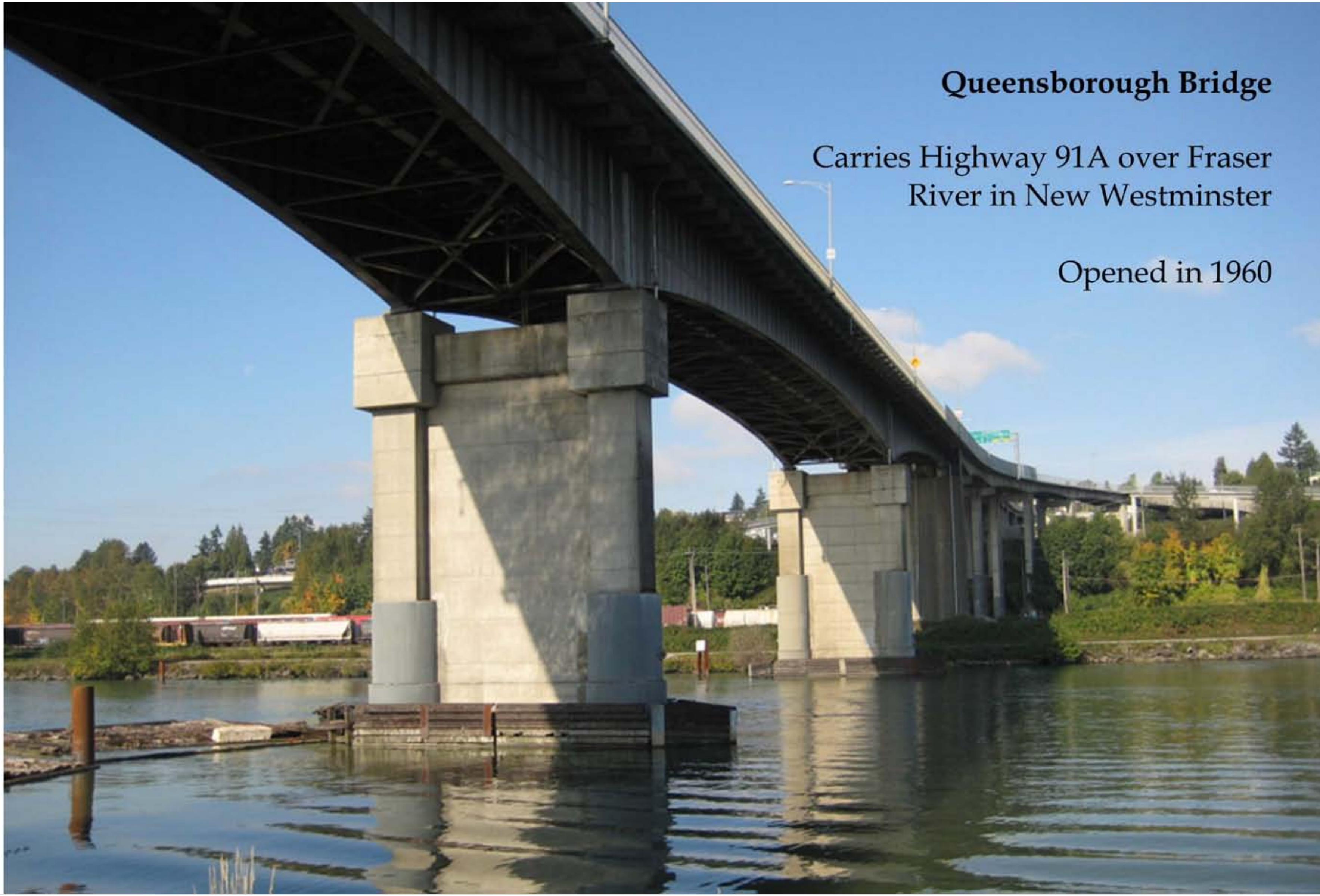
These systems may also provide other agencies, emergency responders and engineers with situational awareness

First Monitored Structures in BC

- In 1996, two bridges and one tunnel were monitored for seismic response;
- This was driven by the desire to measure the seismic inputs to the structures
- Measured few points of acceleration on the structure and several free-field or downhole sites

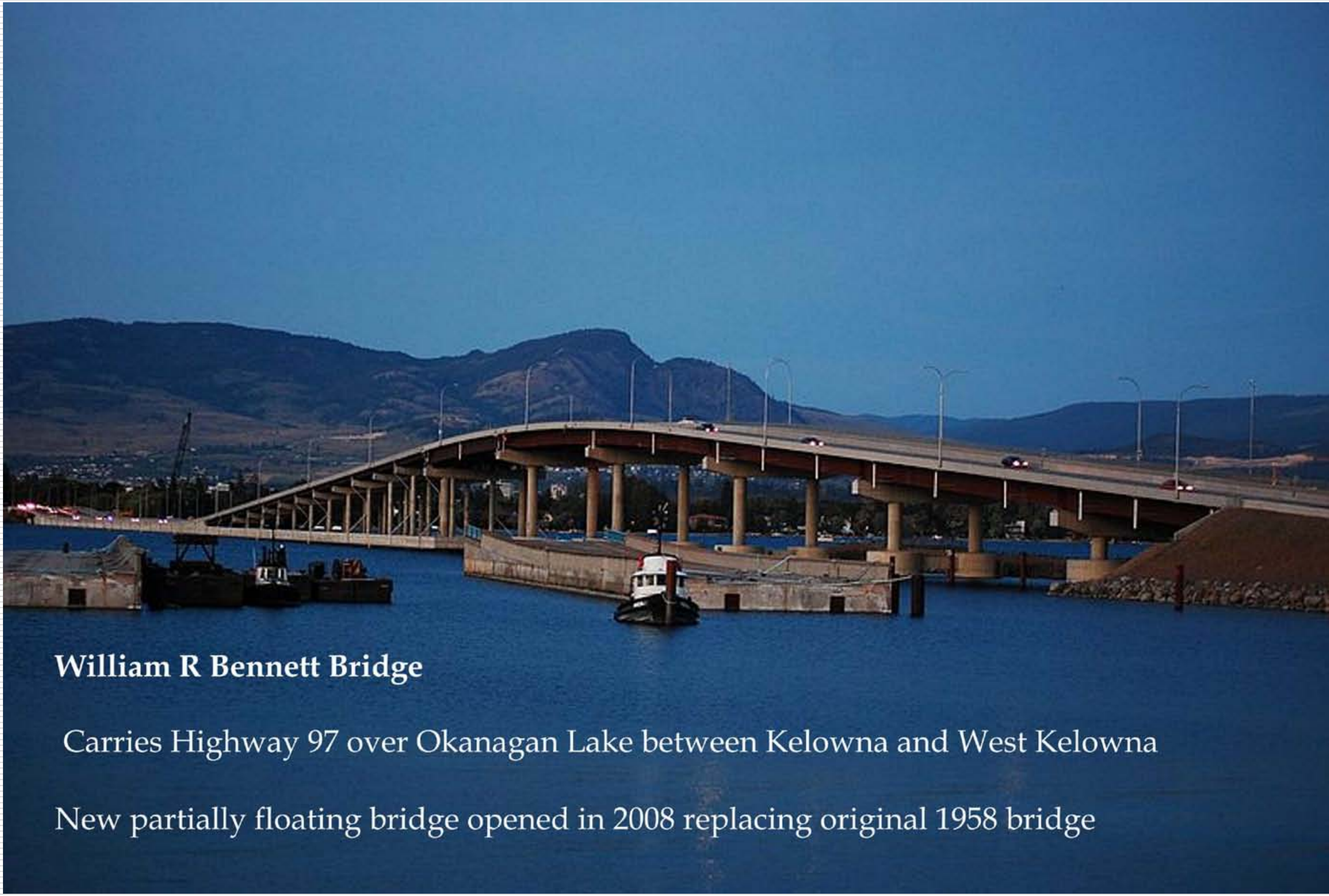

George Massey Tunnel
Carries Highway 99 under Fraser River between Richmond and Delta
Opened in 1959

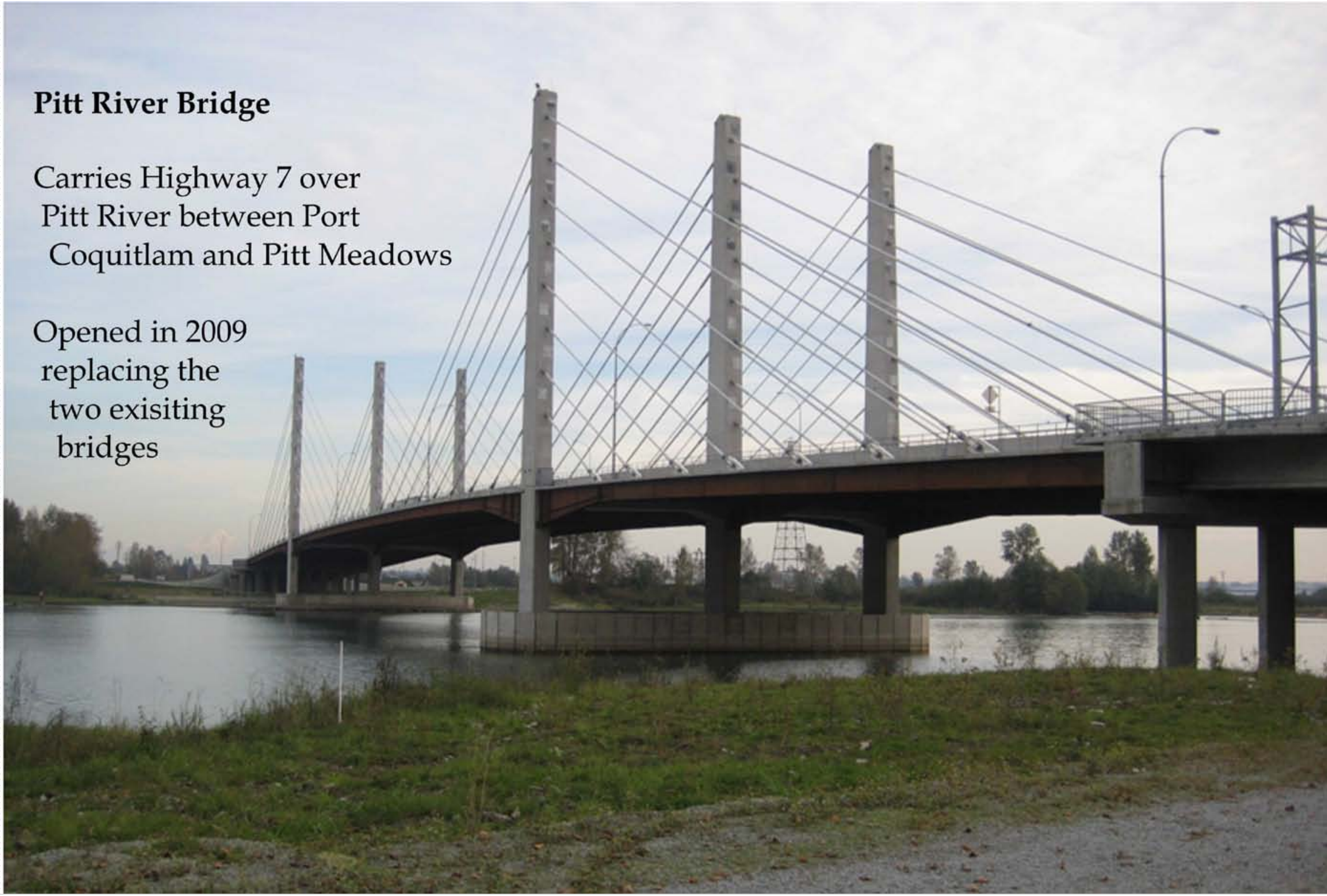

French Creek Bridge
Carries Highway 19
over French Creek
near Parksville
Opened in 1996
Monitored with
12 sensors

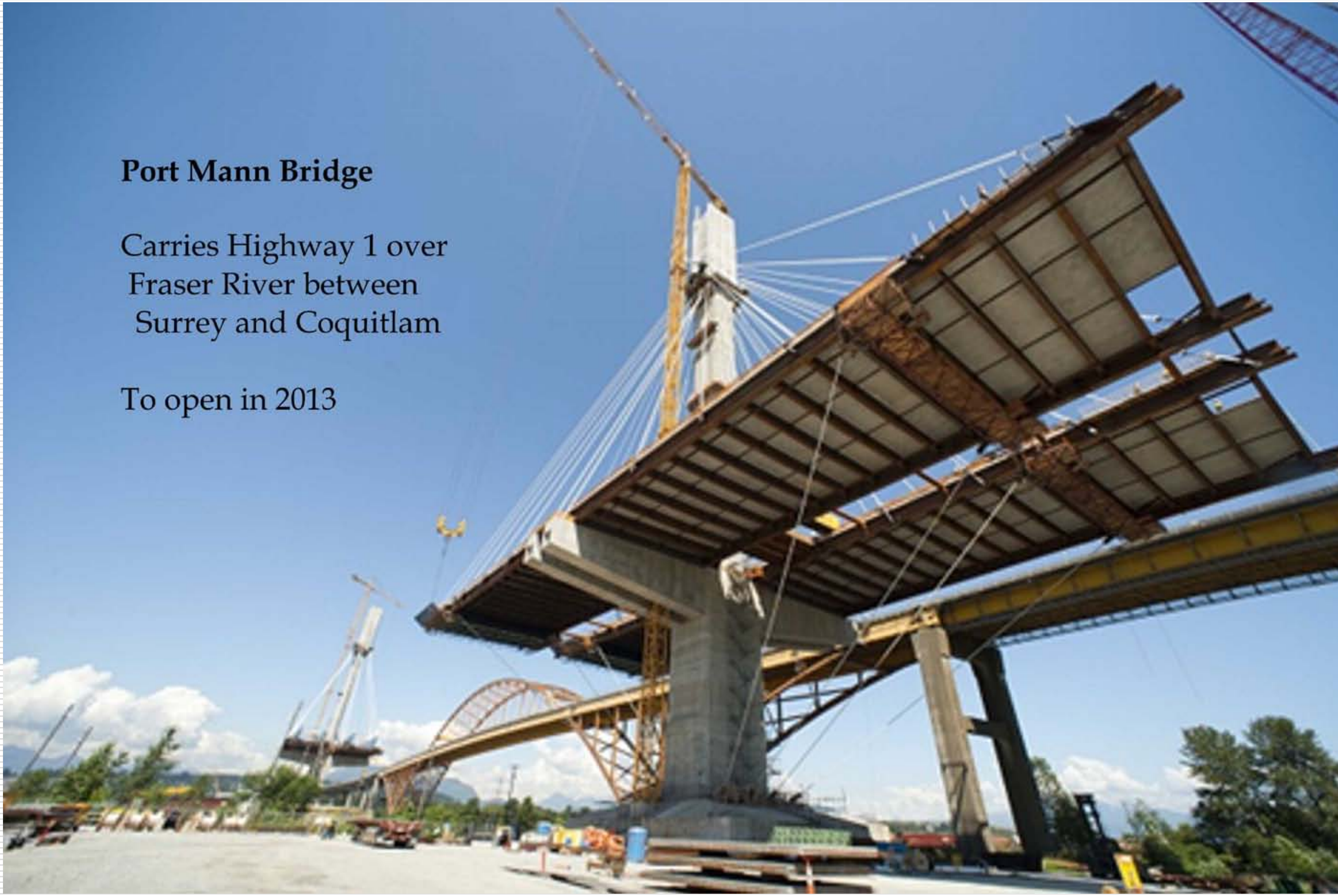

Queensborough Bridge
Carries Highway 91A over Fraser River in New Westminster
Opened in 1960

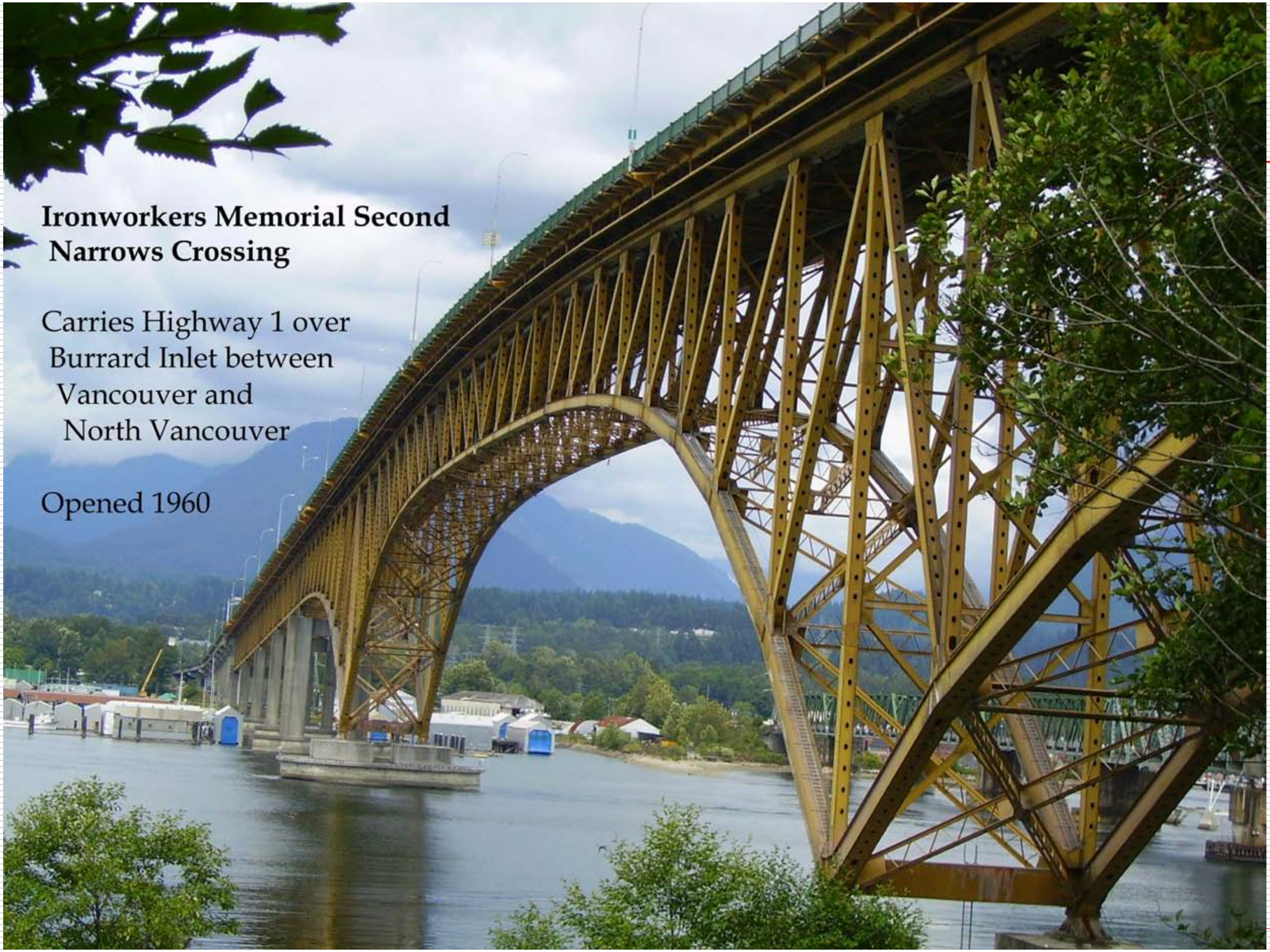
New Bridge Instrumentations

- Currently there is a significant amount of major highway construction in the lower mainland of BC: **Gateway program**
- Two of the main new cable-stayed bridges have extensive monitoring
- As part of these projects more than 50 new interchange bridges are being built; at least four will have permanent seismic monitoring
- Many new seismic monitoring stations are also being added
- Additionally a new floating bridge in the Okanagan region of BC is being monitored

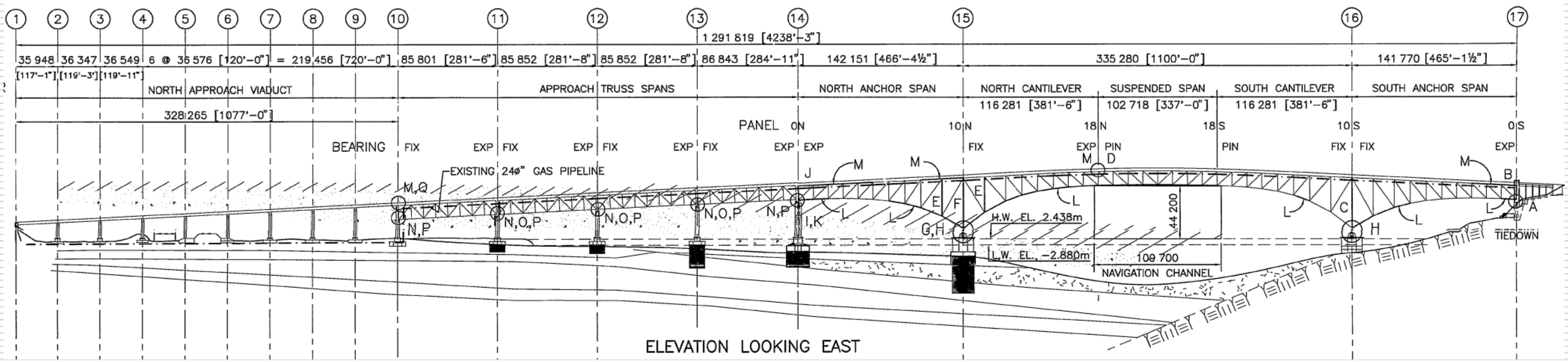

William R Bennett Bridge
Carries Highway 97 over Okanagan Lake between Kelowna and West Kelowna
New partially floating bridge opened in 2008 replacing original 1958 bridge


Pitt River Bridge
Carries Highway 7 over
Pitt River between Port
Coquitlam and Pitt Meadows
Opened in 2009
replacing the
two exisiting
bridges


Port Mann Bridge
Carries Highway 1 over
Fraser River between
Surrey and Coquitlam
To open in 2013


Ironworkers Memorial Second
Narrows Crossing
Carries Highway 1 over
Burrard Inlet between
Vancouver and
North Vancouver
Opened 1960

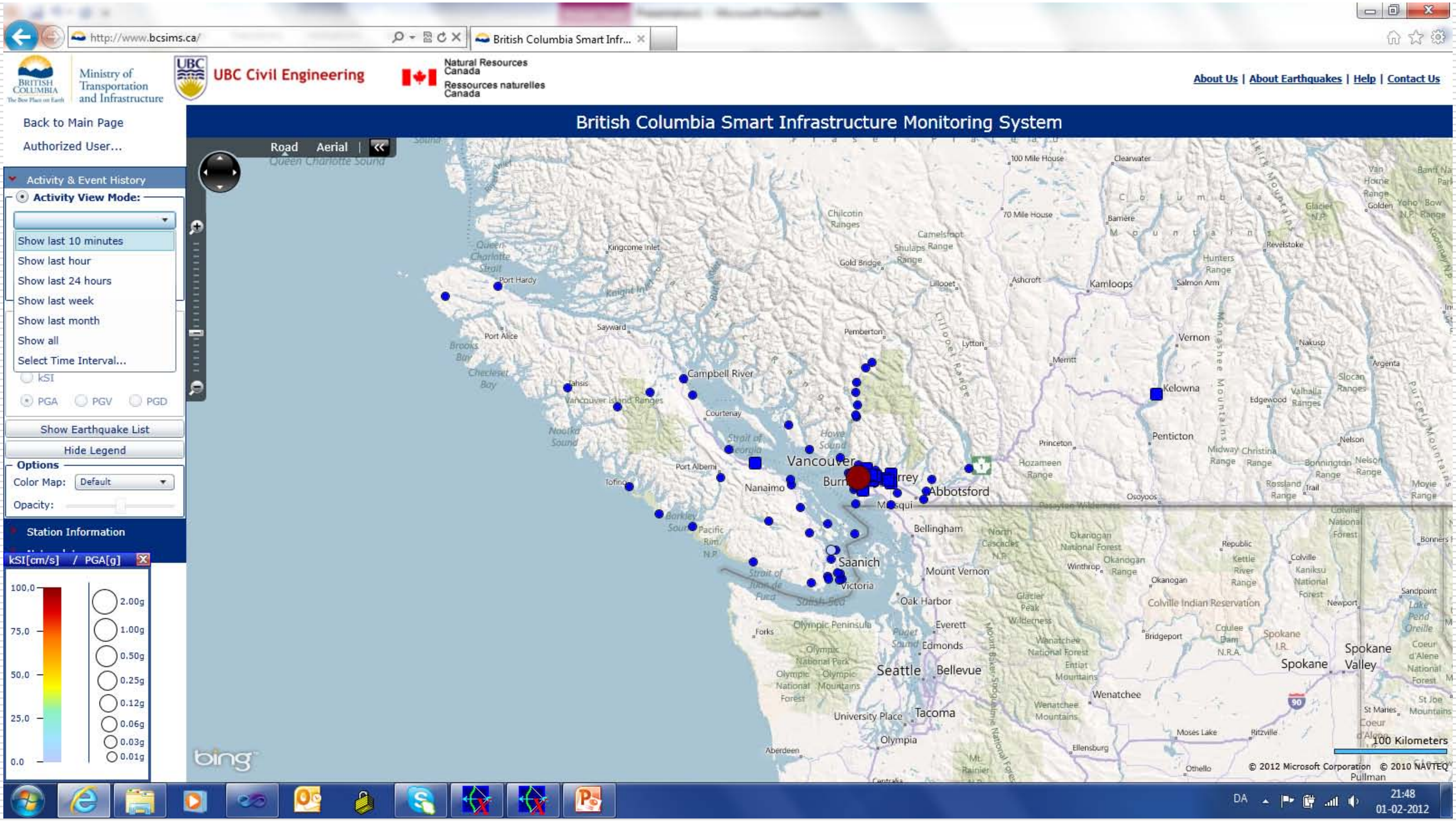
Second Narrows Bridge

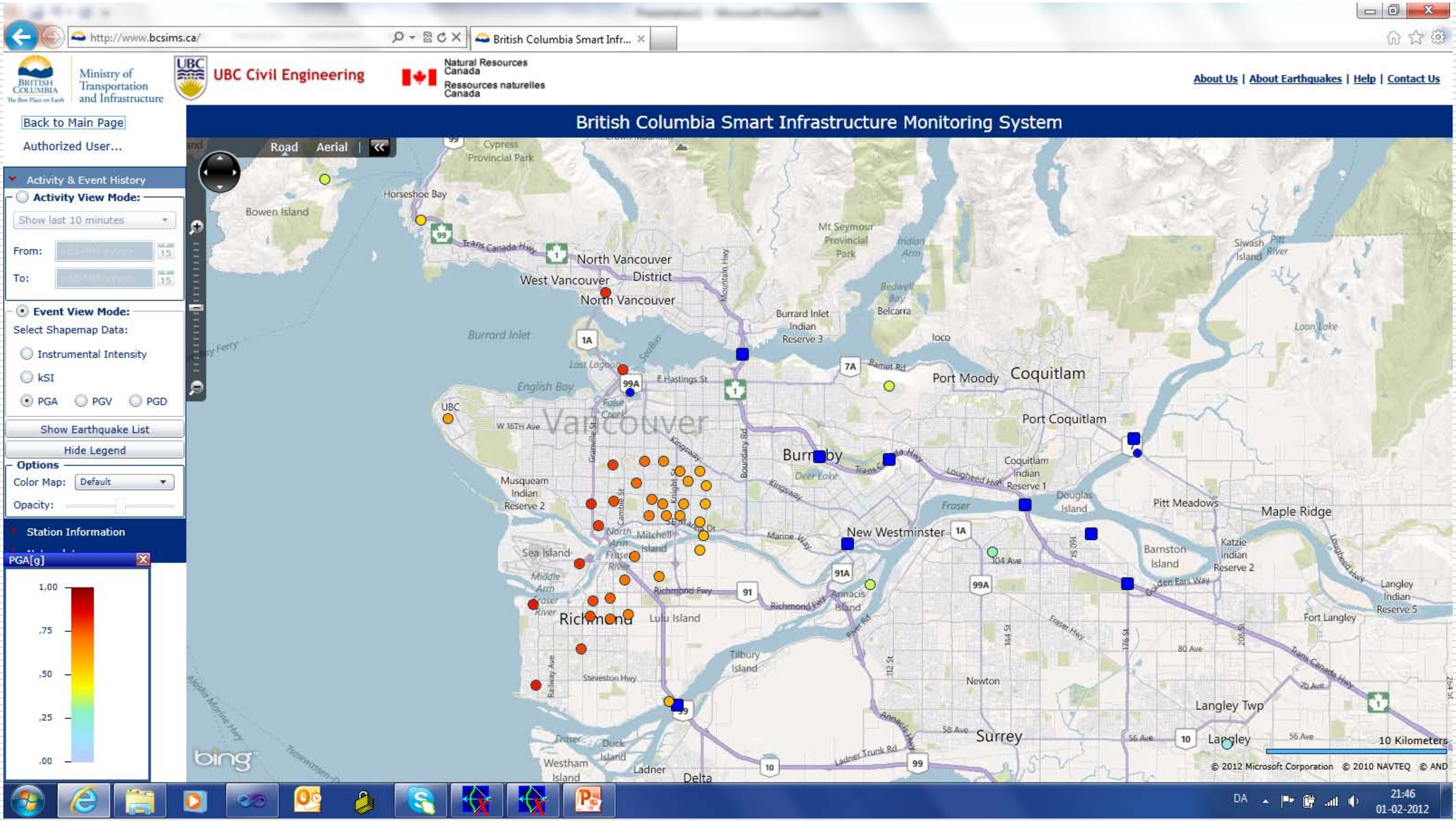


Smart Infrastructure Monitoring System (BCSIMS)

- Strong Motion Network
- Structural Health Monitoring

www.bcsims.ca





http://www.bcsims.ca/
British Columbia Smart Infr...
Ministry of Transportation and Infrastructure
UBC Civil Engineering
Natural Resources Canada
Ressources naturelles Canada
About Us | About Earthquakes | Help | Contact Us
British Columbia Smart Infrastructure Monitoring System
Back to Main Page
Authorized User...
Activity & Event History
Activity View Mode:
Show last 10 minutes
From:
To:
Event View Mode:
Select Shapemap Data:
Instrumental Intensity
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PGA
PGV
PGD
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Hide Legend
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Opacity:
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Road
Aerial
Vancouver
North Vancouver
West Vancouver
Burnaby
Richmond
New Westminster
Port Moody
Coquitlam
Port Coquitlam
Surrey
Delta
Langley
Maple Ridge
Pitt Meadows
Bowen Island
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© 2012 Microsoft Corporation © 2010 NAVTEQ
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Condition Assessment

Level 1 - from the event records:

- Peak Responses (Acc., Vel., Disp.) and Drifts
- Spectral Values (SA, SV, SD)
- Intensity, Energy and Duration
- Hysteretic Response
- Damage Indices

Level 2 - from the post-event records:

- Statistical based Damage Detection - INRIA method
- Modal Analysis based Damage Detection - ARTeMIS
- Damage Detection based on FE Model and Recorded Motions - ARTeMIS & FEMtools

Remarks

- Methodologies being developed are useful for the identification of regions of high seismic risk and the interdependencies among critical infrastructures
- Real-time information tools, such as the BCSIMS and BC SEWS project, are powerful tools for seismic risk mitigation and emergency response.
- Improving response to infrastructure failures is a necessary condition for disaster resilience

Combination of Thermal and Seismic Displacements for the Design of Base Isolation Systems of Bridges

Luc E. Chouinard
Philippe Brisebois

McGill

Outline

1 – Introduction

2 – Description of types of seismic isolation systems available for bridges in Canada

3 – Evaluation of $\Delta_{thermal}$ and $\Delta_{seismic}$ according to CHBDC CSA-S6-06

4 – Review of other bridge codes for the combination of $\Delta_{thermal}$ and $\Delta_{seismic}$

5 – Review of methods for the probabilistic combination of effects

6 – Example for regular bridges equipped with base isolators

7 – Development of simple $\Delta_{thermal}$ and $\Delta_{seismic}$ combination formula with different load combination methods

8 – Conclusions and recommendations

1 - Introduction

- Since 2000, a section addresses seismic base isolation in the CSA-S6 (Clause 4.10)

- Currently CSA-S6-06 does not provide a procedure to combine $\Delta_{seismic}$ and $\Delta_{thermal}$ for base isolation systems

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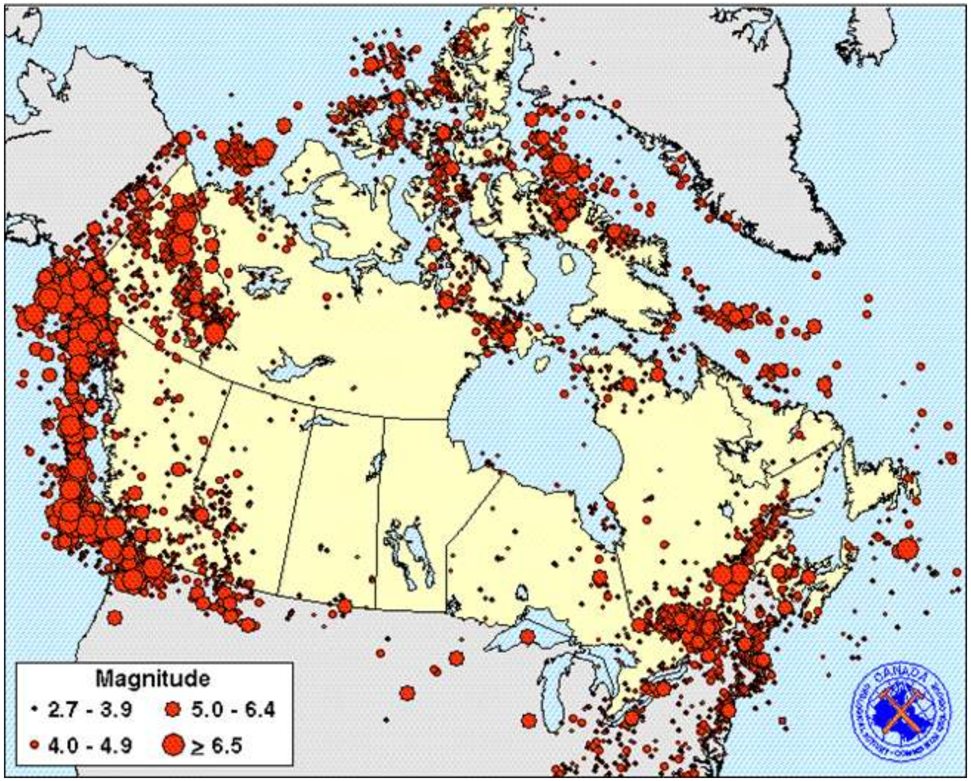
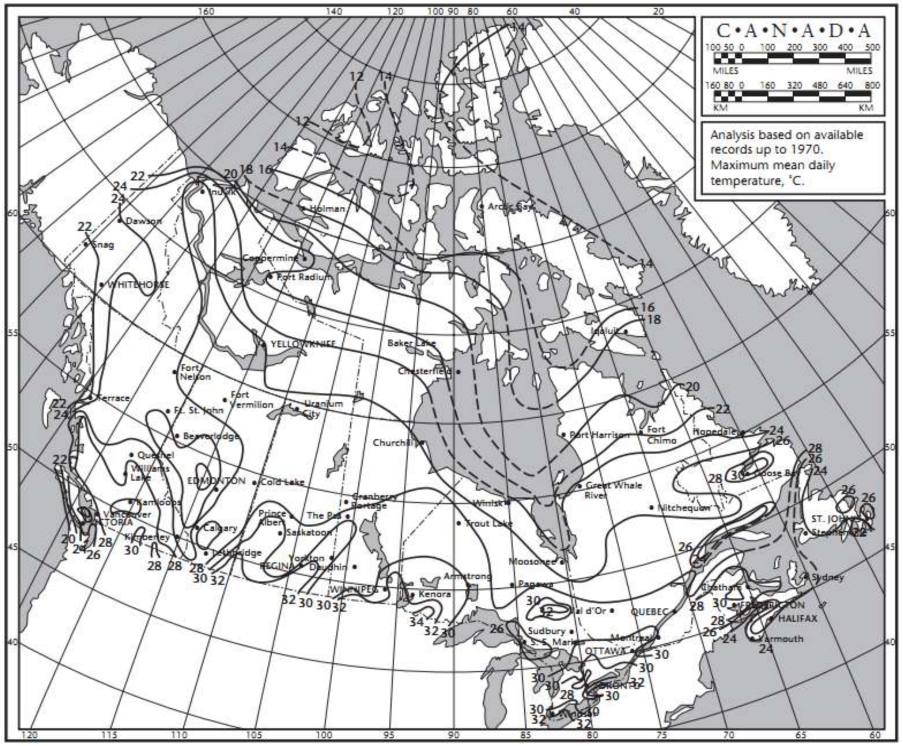
1 - Introduction



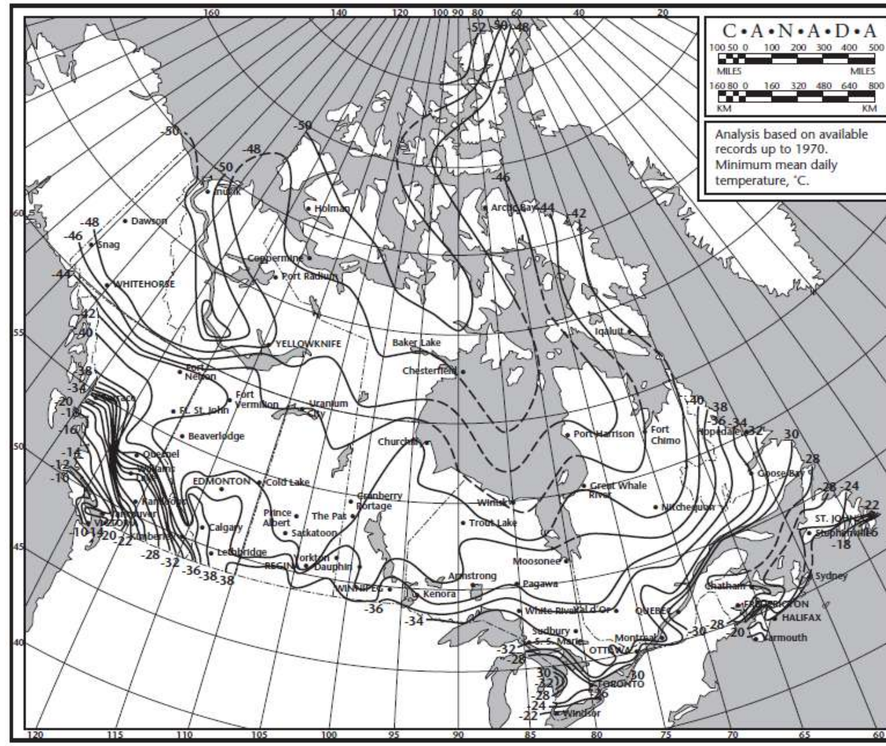
OBJECTIVE:

Develop guidelines that consider regional characterstics in:
- level of seismic activity
- climatic conditions

Maximum Mean Daily Temperatures



Minimum Mean Daily Temperatures



OBJECTIVE:

Develop guidelines that consider regional characterstics in:

- level of seismic activity
- climatic conditions

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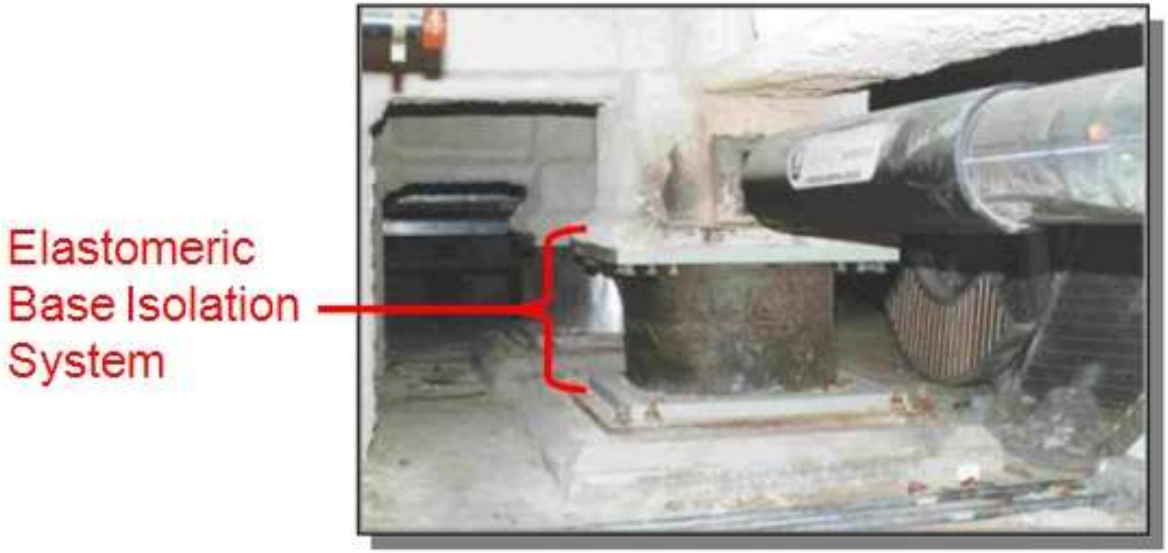
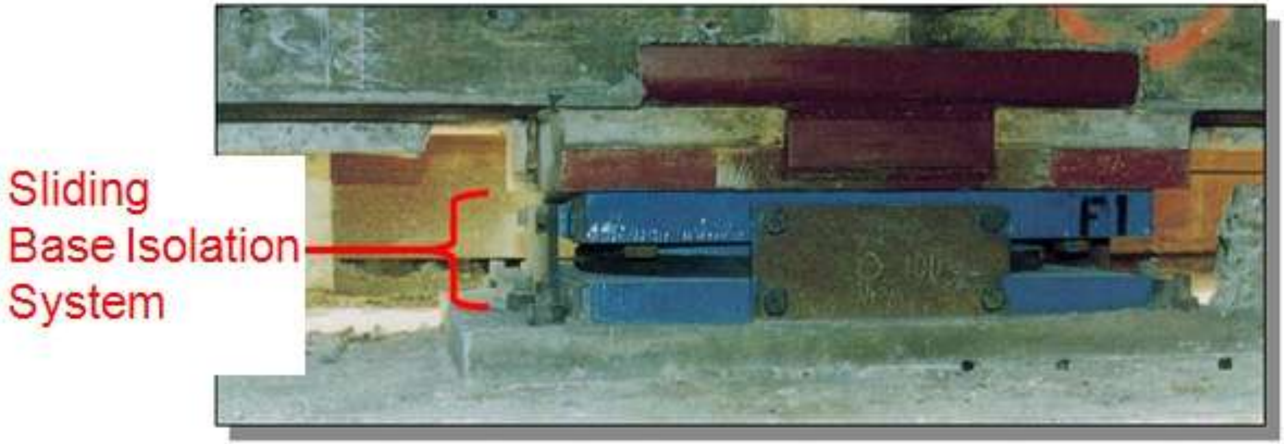
2 - Base Isolation Systems for Bridges



Low-Damping Natural or Synthetic Rubber Isolator

High-Damping Natural Rubber Isolator

Lead-Rubber Isolator



-Flat Sliding Isolator

- Spherical Sliding Isolator or

Friction Pendulum System

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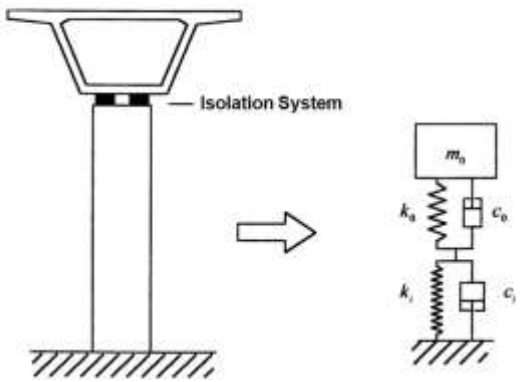
3 - $\Delta_{seismic}$ and $\Delta_{thermal}$ - CHBDC CSA-S6-06

$$\Delta_{seismic} = S_D = \frac{(250 S_a S_i T_e^2)}{B}$$



where

S_a = spectral acceleration, g

S_i = site coefficient

T_e = isolation period of the structure, sec

B = numerical coefficient related to the effective damping of the isolation system

$$\Delta_{thermal} = \alpha * L * \Delta T$$

A = material thermal coefficient, $^0C^{-1}$

L = length of the member, mm

ΔT = temperature difference after onsite installation, ^0C

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4 – Current Combination Rules for $\Delta_{seismic}$ and $\Delta_{thermal}$

National Bridge Design Code	**Combination Formula of $\Delta_{seismic}$ and $\Delta_{thermal}$**
CSA-S6-06, AASHTO-2004 and Chile-2002	None
British Columbia Ministry of Transportation Bridge Standards and Procedures Manual (2007)	$\Delta_{seismic} + 40\%\Delta_{thermal}$ (Clause 4.10.7)
New Zealand Transportation Agency Bridge Manual (2004)	$\Delta_{seismic} + 33.3\%\Delta_{thermal}$ (Clause 5.6.1)
Eurocode 8 Part 2: Bridges (2005)	$\Delta_{seismic} + 50\%\Delta_{thermal}$ (Clause 7.6.2)

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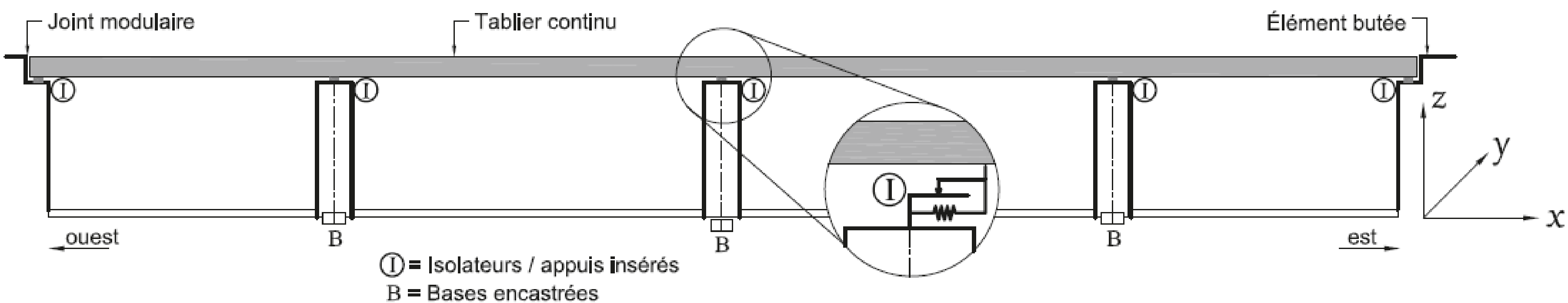
5 – Base Isolation System Analysis – Example

- 4 spans
- 2 expansion joints at abutments
- Total length = 128.8 m
- Steal beams with reinforced concrete deck
- Depth of superstructure = 1903 mm



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5.1 – $\Delta_{thermal}$ of the Example Bridge

- Effective temperatures

- Takes into consideration:
  - daily temperature changes
  - thermal gradient effects
  - material thermal coefficient
  - geometry of the superstructure
  - effective construction temperature (T_o = 15°C)

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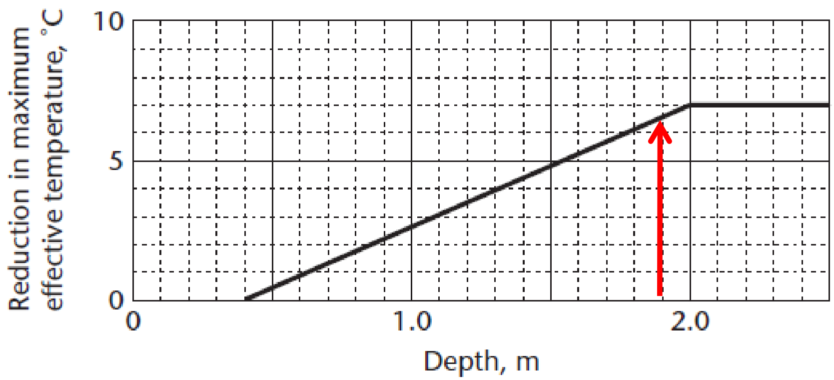
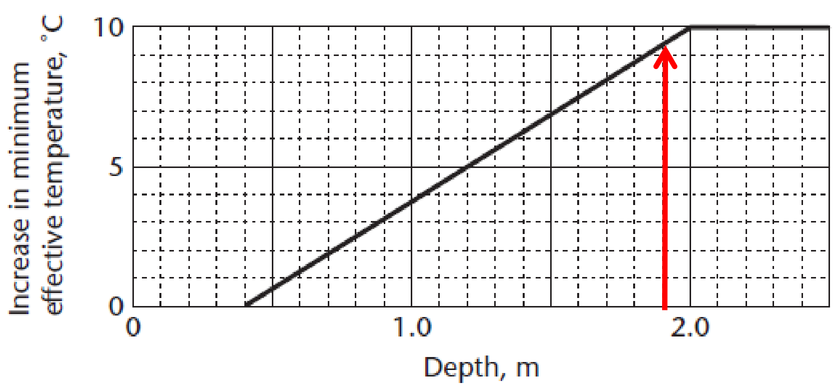
5.1 – $\Delta_{thermal}$ of the Example Bridge

Superstructure type (see Clause 3.9.3.)	Maximum effective temperature	Minimum effective temperature
A	25 °C above maximum mean daily temperature	15 °C below minimum mean daily temperature
B	20 °C above maximum mean daily temperature	5 °C below minimum mean daily temperature
C	10 °C above maximum mean daily temperature	5 °C below minimum mean daily temperature



+20°C – 6.6°C = +13.4°C



-5°C + 9.4°C = +4.4°C

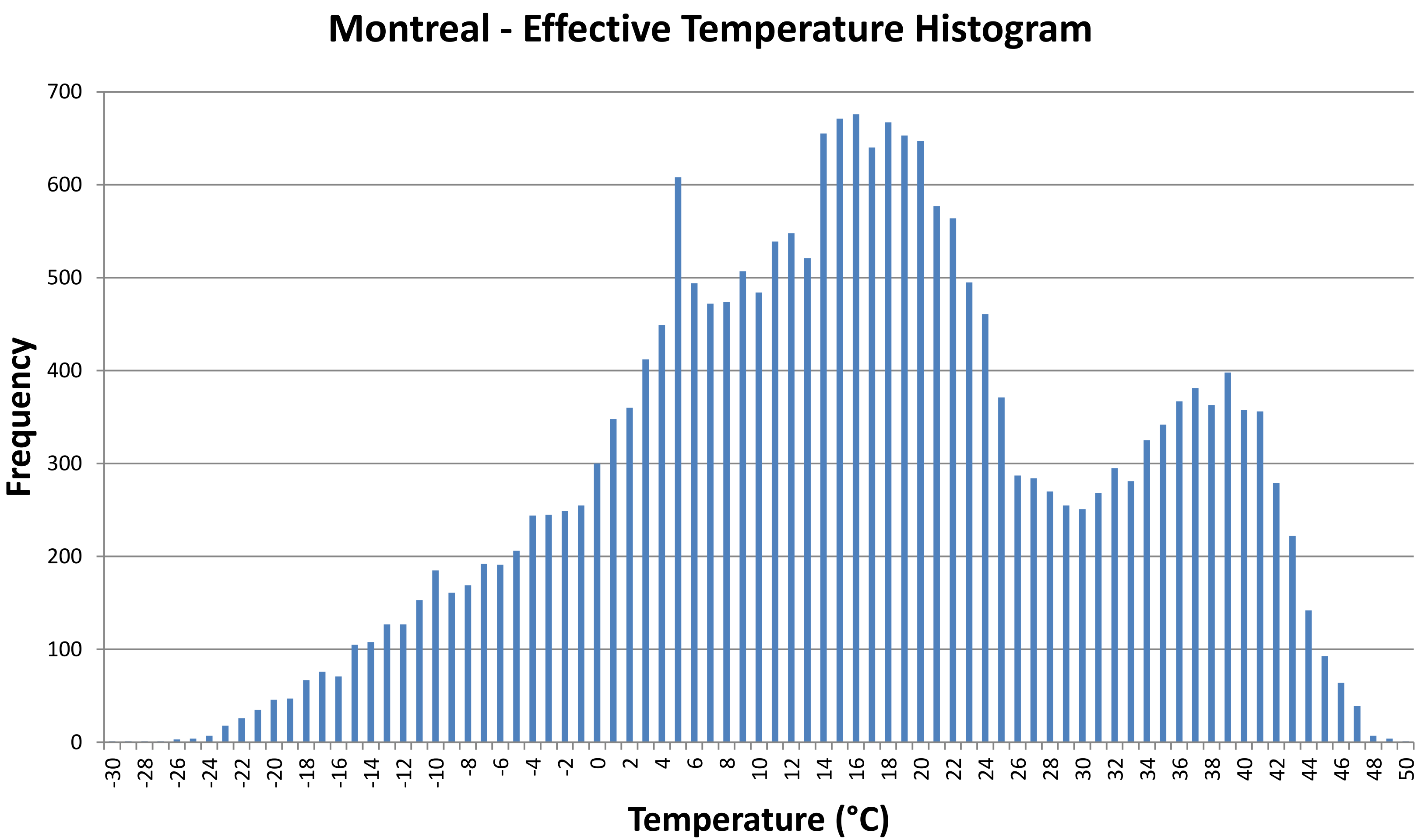
5.1 – $\Delta_{thermal}$ of the Example Bridge

- Environment Canada Daily Climatic Database

Montreal's Pierre Elliott Trudeau Airport:

Vancouver International Airport :

5.1 – ΔT_{eff} of the Example Bridge – Env. Canada



(-30°C to 50°C)

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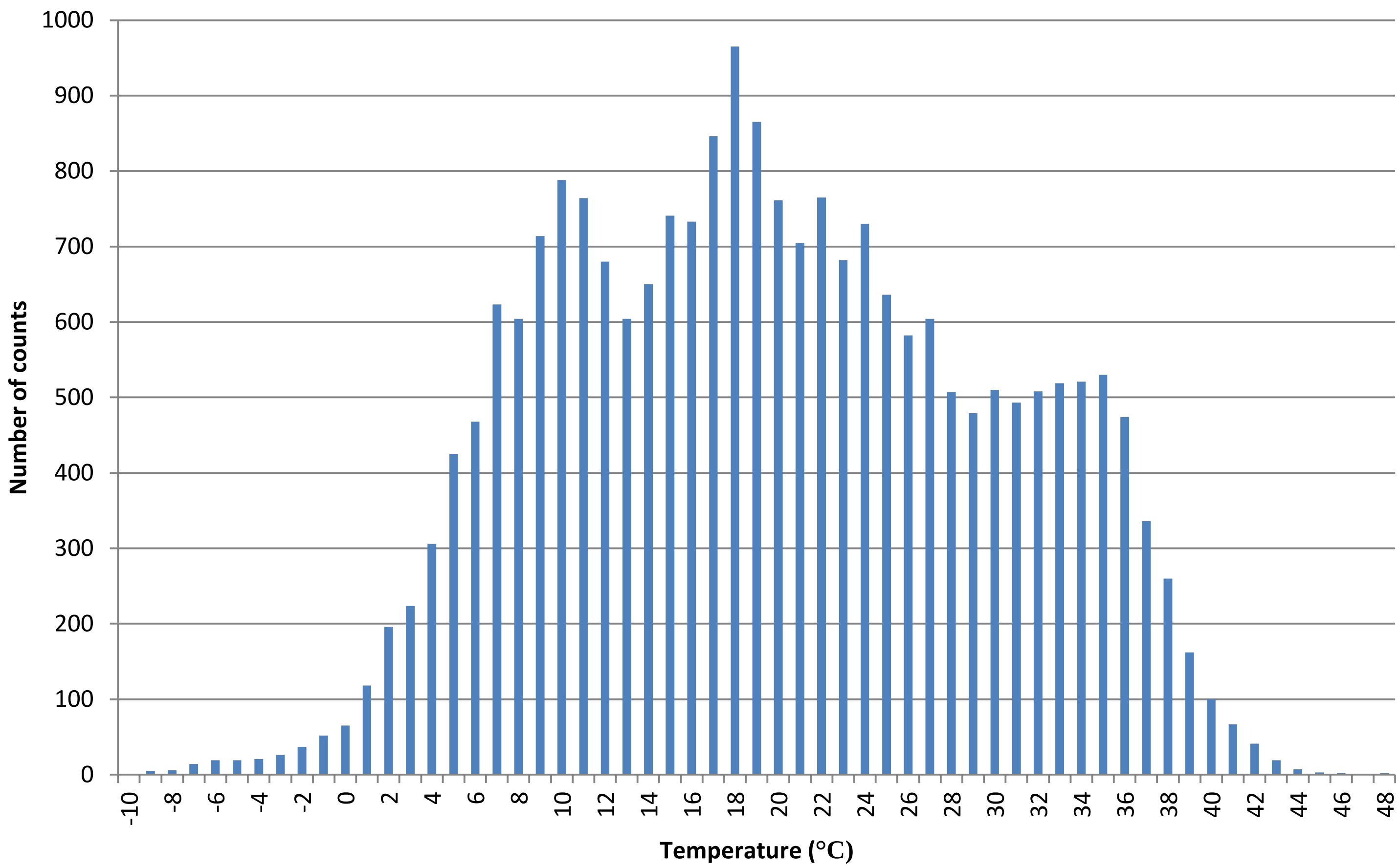
Vancouver - effective temperature



(-10°C to 48°C)

5.1 – $\Delta_{thermal}$ of the Example Bridge – Env. Canada

$\Delta_{thermal}$ (mm) = $\alpha * L * \Delta T_{max}$

where

α = 11 x 10^{-6}/ °C for steal beams and reinforced concrete deck
L = 128.8/2 = 64.4 m = 64 400 mm

(-30°C à 50°C) and T_o = 15°C
@ -30 °C: ΔT = 45°C
@ +50 °C: ΔT = 35°C

$\boxed{\Delta_{thermal}\ \textbf{max} = (11 \times 10^{-6}/\,^{\circ}C)*(64\,400\ mm)*(45^{\circ}C) = \textbf{31.9 mm}}$

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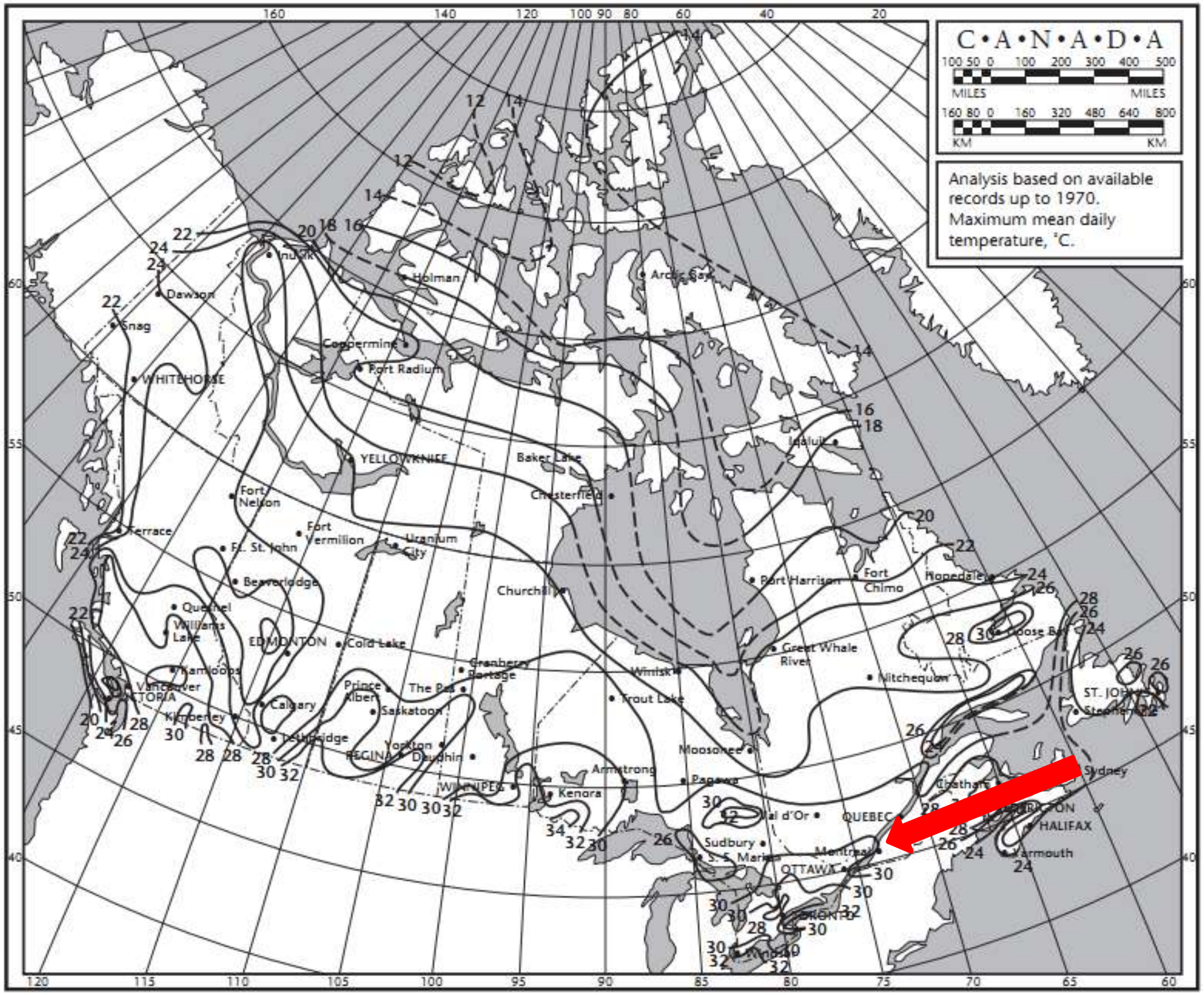
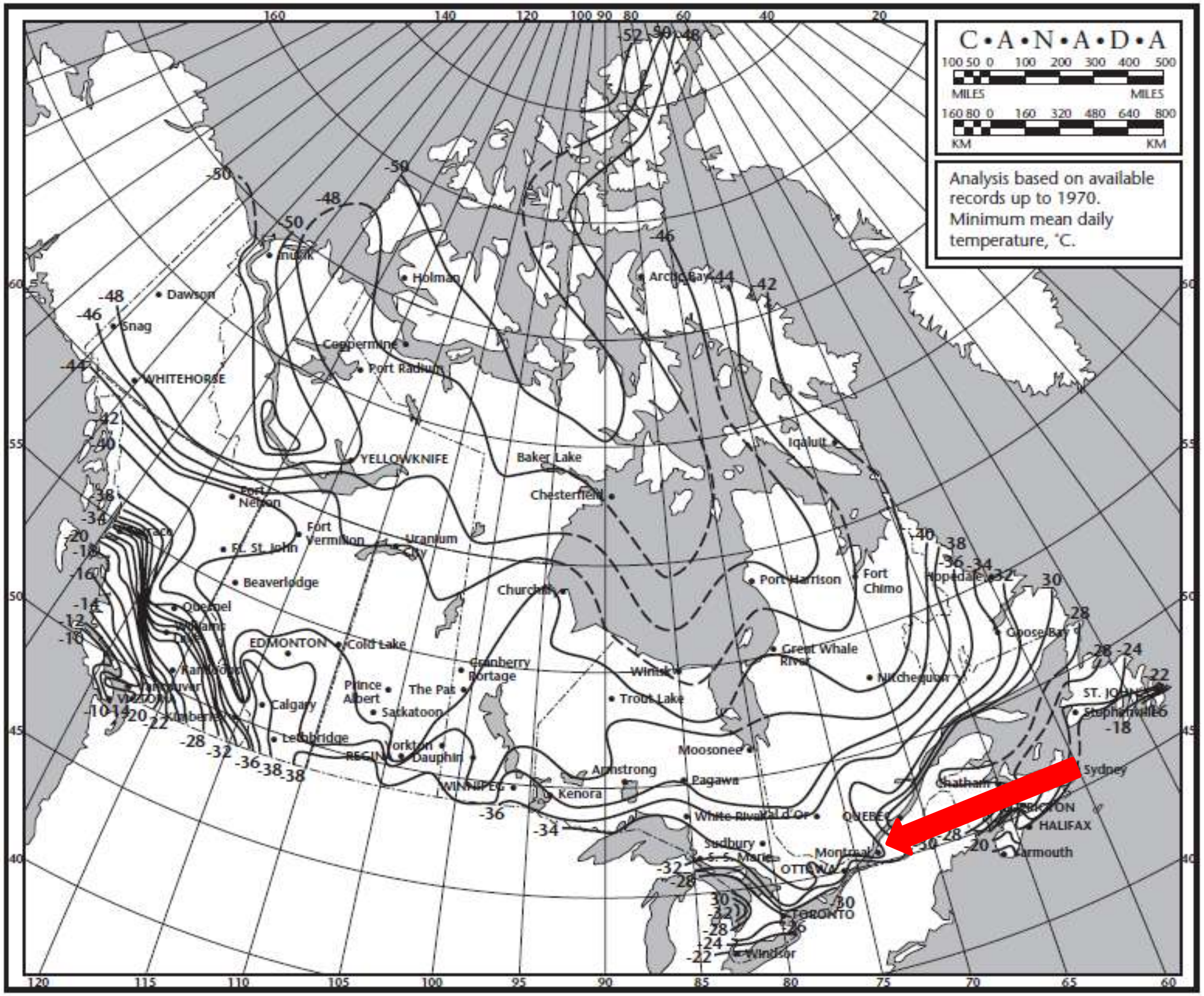
5.1 – $\Delta_{thermal}$ of the Example Bridge - CSA-S6-06

Maximum Mean Daily Temperatures



5.1 – $\Delta_{thermal}$ of the Example Bridge - CSA-S6-06

Minimum Mean Daily Temperatures



5.1 – $\Delta_{thermal}$ of the Example Bridge - CSA-S6-06

- Maximum Mean Daily Temperatures = 28°C
- Minimum Mean Daily Temperatures = -36°C

- Superstructure Type = B

28°C + 20°C = 48°C et -36°C - 5°C = -41°C

- Depth of superstructure = 1903 mm

48°C – 6.6°C = 41.4°C et -41°C + 9.4 = -31.6°C

(-31.6°C à 41.4°C) et T_o = 15°C

@ -31.6 ºC: ΔT = 46.6°C

@ +41.4 ºC: ΔT = 26.4°C

$\Delta_{thermal}$ **max** = (11 x 10^{-6}/ ºC)*(64 400 mm)*(46.6°C) = **33.0 mm**

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5.1 – $\Delta_{thermal}$ of the Example Bridge

Climatic Database	Location	ΔT_{max} (° C)	$D_{t,\ max}$ (mm)
Environment Canada	Montreal	45.0	31.9
	Vancouver	33.0	23.4
CHBDC CSA-S6-06	Montreal	46.6	33.0
	Vancouver	24.6	17.4

5.2 – $\Delta_{seismic}$ of the Example Bridge

- $\Delta_{seismic}$

  - Atkinson and Boore (2006)

  - Atkinson and Goda (2010)

Seismic events with 2% probability of exceedance in 50 years (NBCC 2010).

6 – Combination of $\Delta_{thermal}$ and $\Delta_{seismic}$ for the Example Bridge

- Probabilistic Approach

$$\lambda(\Delta_T = \Delta_S + \Delta_{Th}) = \int \lambda(\Delta_S) \cdot f_{\Delta_{Th}}(\Delta_{Th})\, d\,\Delta_{Th}$$

$$f_{\Delta_{Th}}(\Delta_{Th}) = \text{pdf of } \Delta_{Th}$$

- Turkstra's Rule

$$\mathrm{X}_{\max,\mathrm{T}} = \max \begin{Bmatrix} \max(\mathrm{X}_\mathrm{S}) + m_{Th} \\ \max(\mathrm{X}_{\mathrm{Th}}) + m_S \end{Bmatrix}$$

$$\mathrm{X}_{\max,\mathrm{T}} = \text{maximum combined effect during the period of time T.}$$

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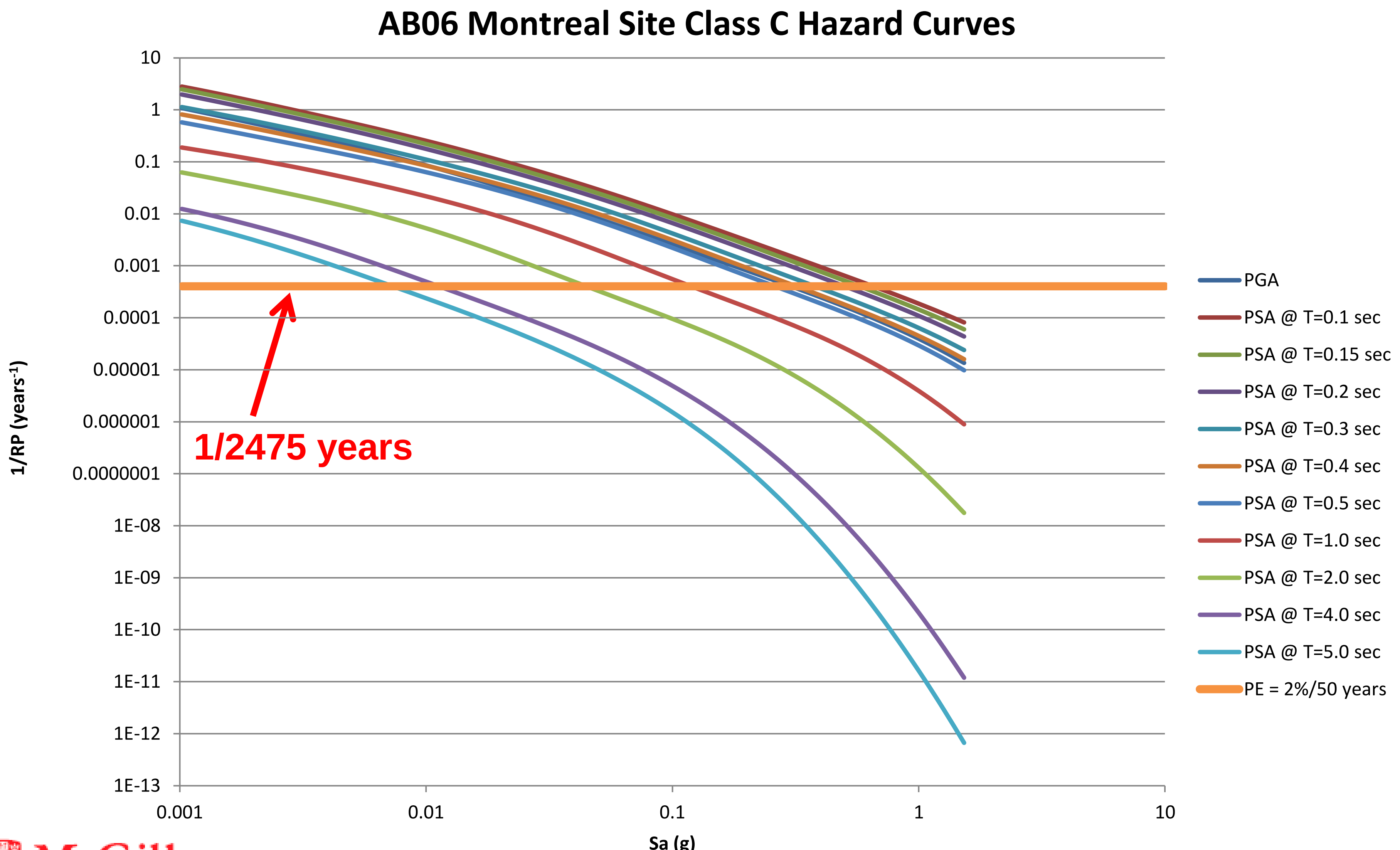
5.2 – $\Delta_{seismic}$ of the Example Bridge - CSA-S6-06



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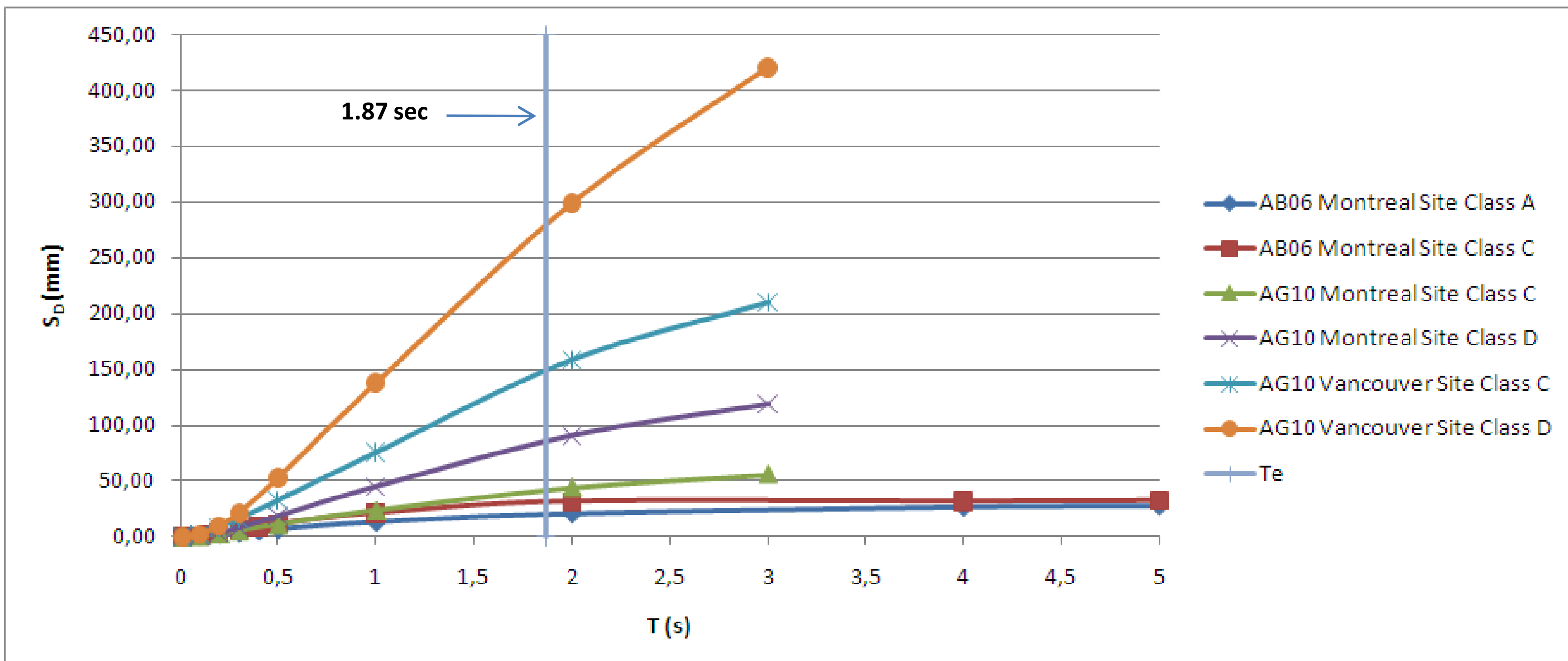
5.2 – $\Delta_{seismic}$ of the Example Bridge - CSA-S6-06

Seismic Displacements in mm of Madrid Bridge at 2%/50 years

Seismic Hazard Model	T=0.01 sec	T=0.1 sec	T=0.15 sec	T=0.2 sec	T=0.3 sec	T=0.4 sec	T=0.5 sec	T=1.0 sec	T=2.0 sec	T=3.0 sec	T=4.0 sec	T=5.0 sec	Te=1.87 sec
AB06 Montreal Site Class A	0.01	0.99	1.70	2.46	3.76	5.49	6.87	13.16	20.65	-	26.70	27.63	**19.68**
AB06 Montreal Site Class C	0.01	1.15	2.28	3.51	6.03	8.99	11.54	21.02	31.48	-	31.68	32.27	**30.12**
AG10 Montreal Site Class C	0.01	1.08	-	3.14	5.75	-	11.21	23.95	44.18	55.90	-	-	**41.55**
AG10 Montreal Site Class D	0.01	1.17	-	3.68	7.81	-	18.41	44.82	90.67	118.63	-	-	**84.71**
AG10 Vancouver Site Class C	0.01	2.10	-	8.70	16.11	-	32.71	75.56	158.84	210.10	-	-	**148.02**
AG10 Vancouver Site Class D	0.01	2.22	-	9.89	21.22	-	52.76	137.73	298.74	420.27	-	-	**277.81**



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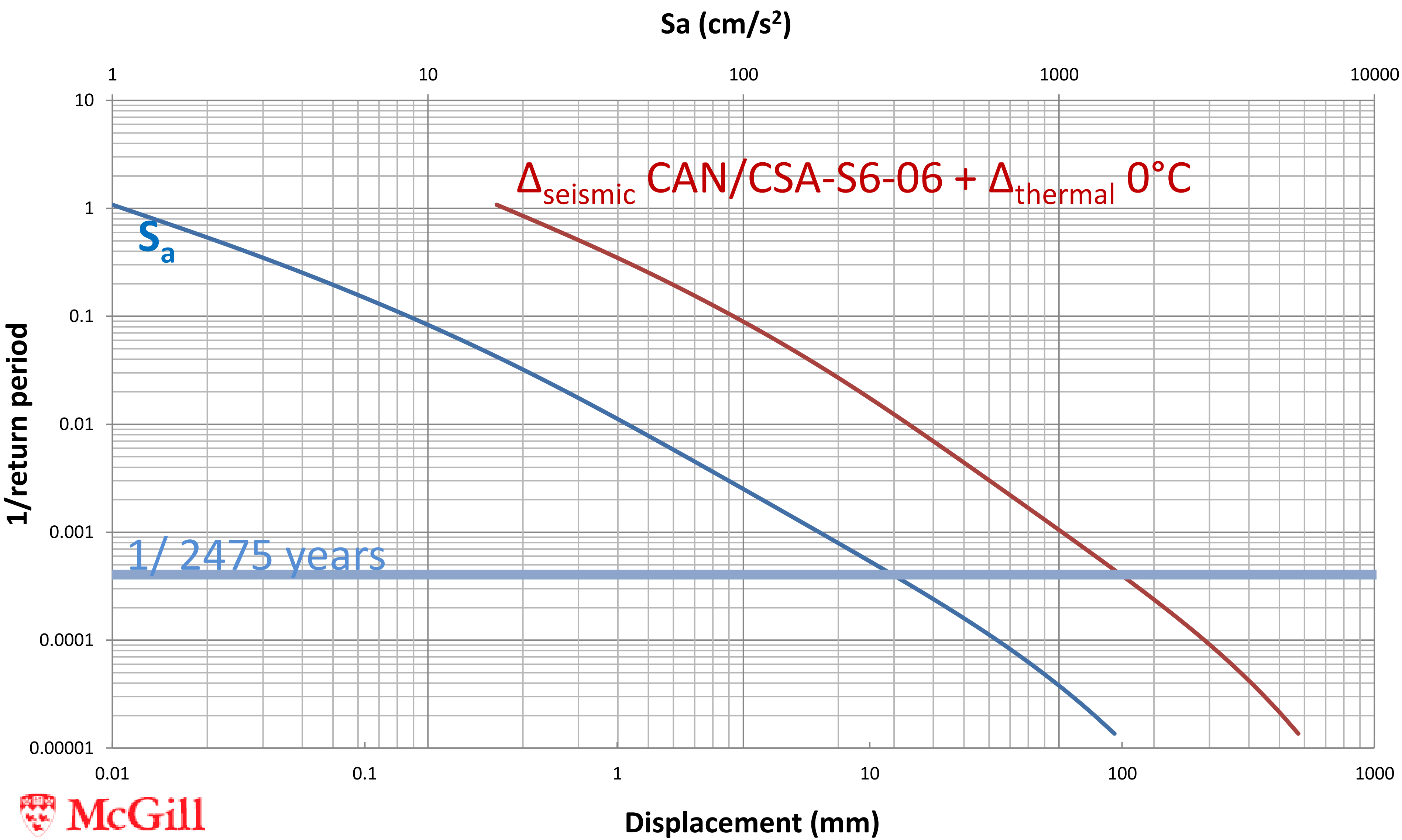
6.1 – Probabilistic Approach - Example Bridge



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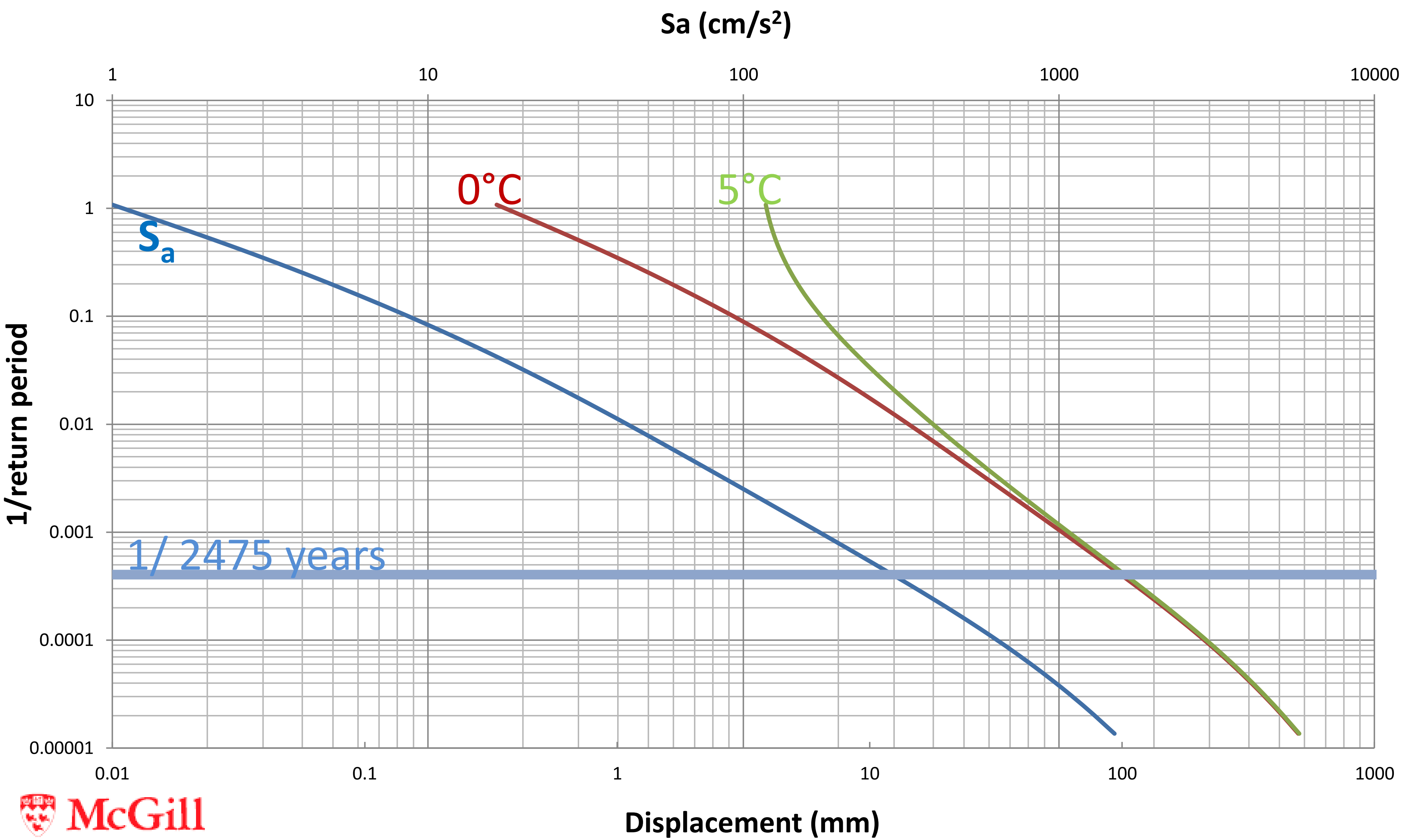
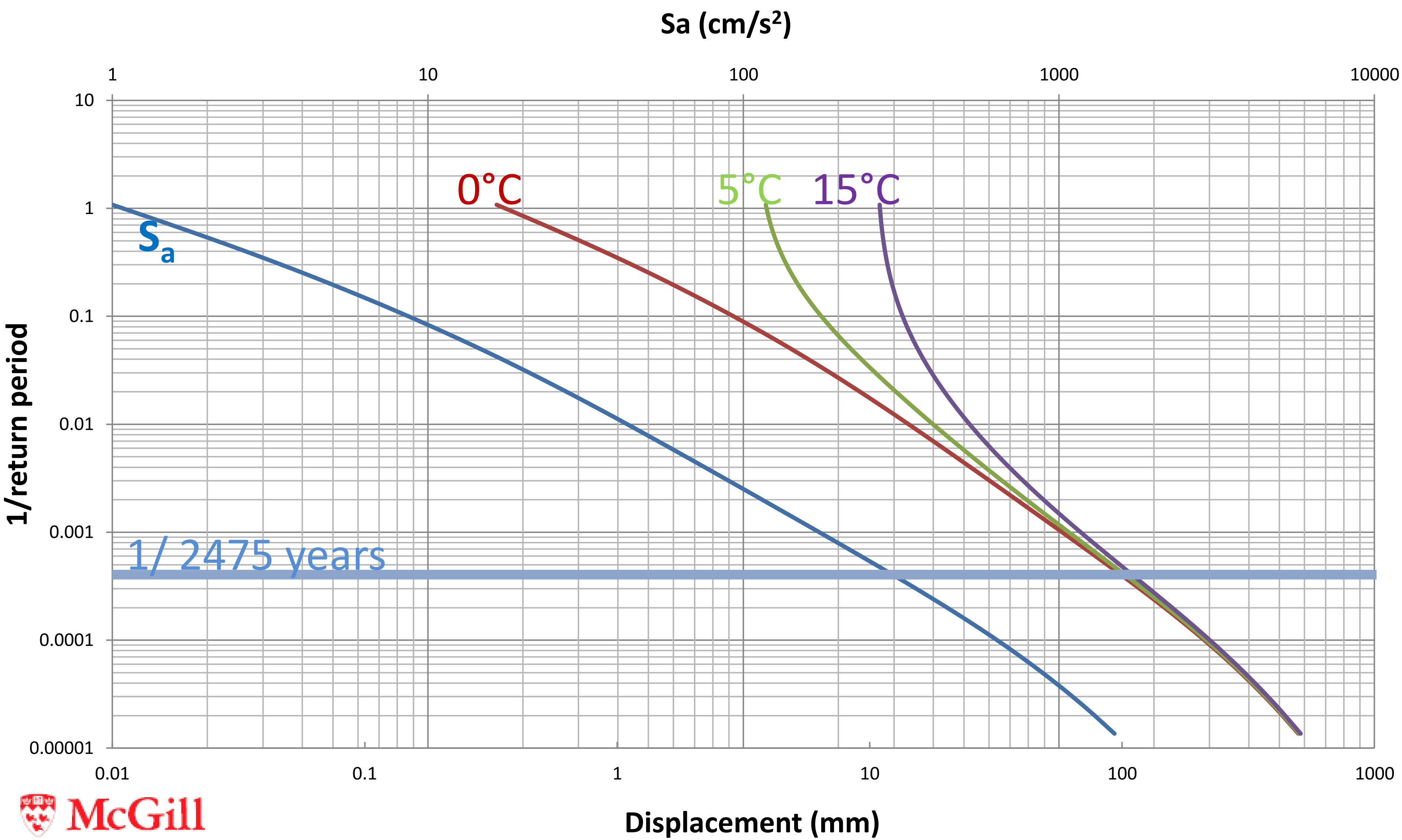
6.1 – Probabilistic Approach - Example Bridge



6.1 – Probabilistic Approach - Example Bridge



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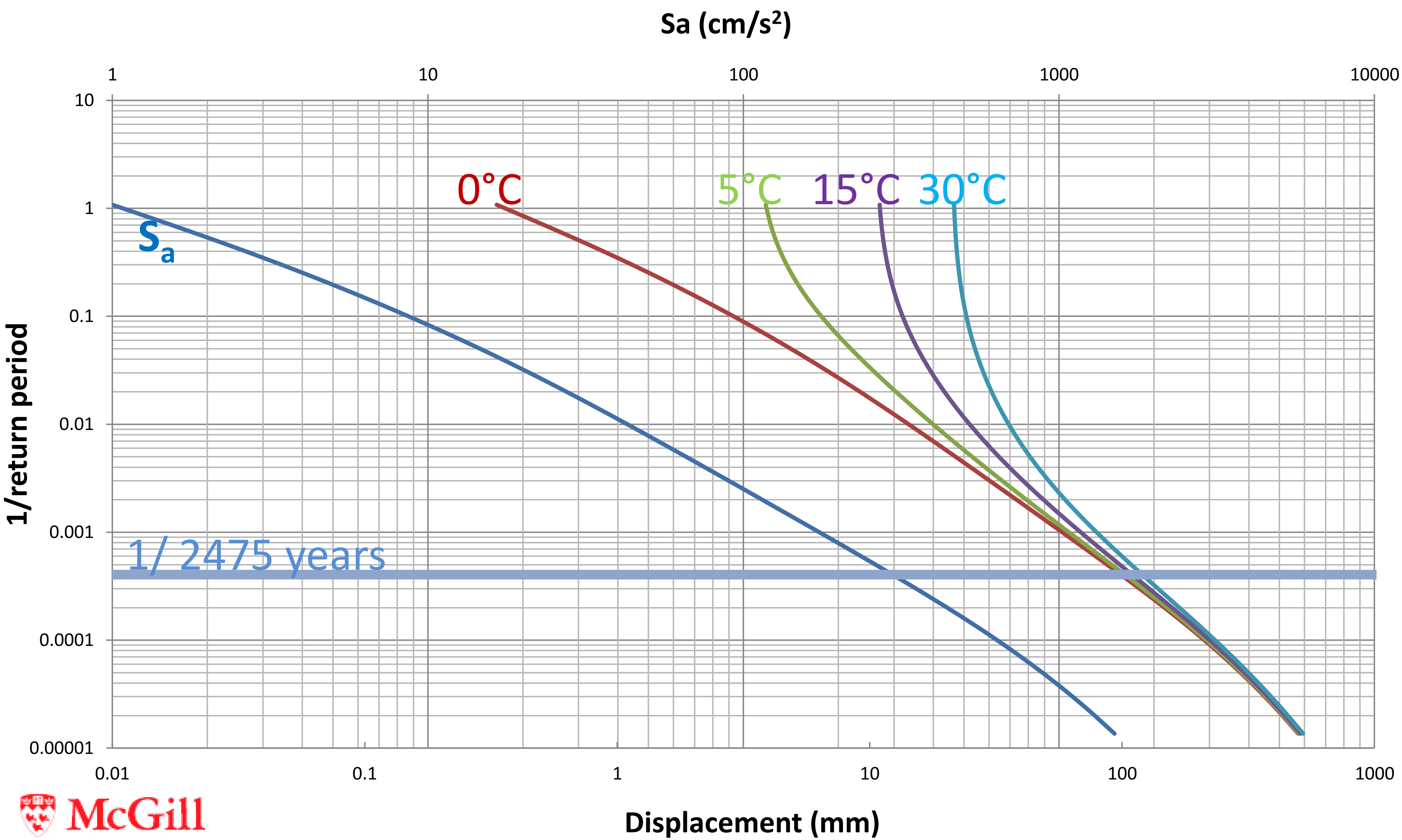
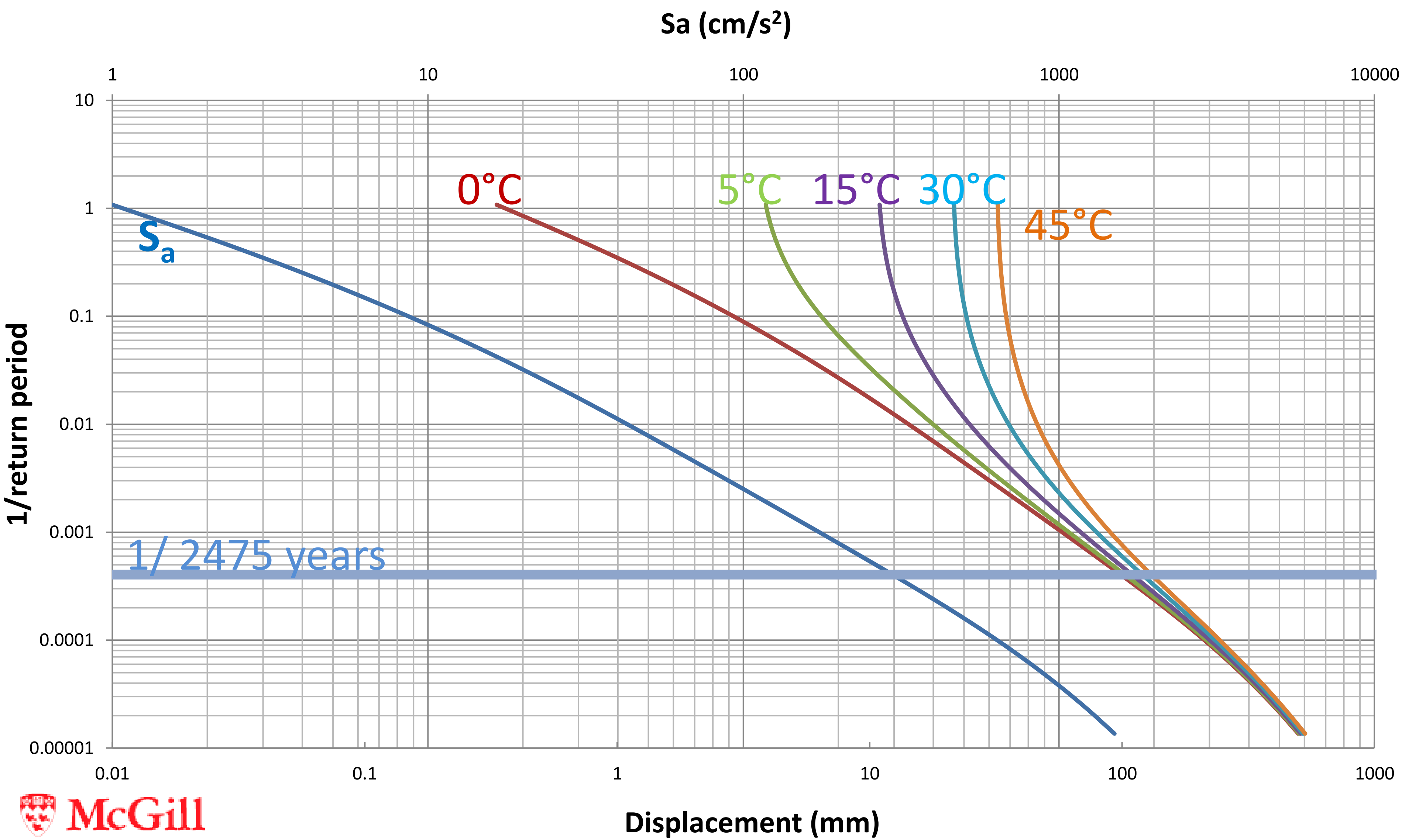
6.1 – Probabilistic Approach - Example Bridge



6.1 – Probabilistic Approach - Example Bridge



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6.1 – Probabilistic Approach - Example Bridge

$$\lambda\left(\Delta_{\mathrm{T}}^{*}\right)=\frac{1}{2475}\ \text{years}\quad(2\%\ \text{in}\ 50\ \text{years})$$

$$\lambda\left(\Delta_{\mathrm{S}}^{*}\right)=\frac{1}{2475}\ \text{years}\quad(2\%\ \text{in}\ 50\ \text{years})$$

$$\Delta_{Th}=\Delta_{T}^{*}-\Delta_{S}^{*}$$

$$\%\Delta_{Th}=\frac{\Delta_{Th}}{\Delta_{Th\,\max}}$$

$\Delta_{\text{th max}}$ → Environment Canada

$\Delta_{\text{th max}}$ → CHBDC CSA-S6-06

6.3 – Summary of $\%\Delta_t$ Results

Seismic Hazard Model	Load Combination Method	Climatic Database	T=0.5 sec	T=1.0 sec	T=2.0 sec	T=3.0 sec	T=4.0 sec	T=5.0 sec	Te=1.87 sec
AB06 Montreal Site Class A	Total Probability Theorem	Env. Canada	53.0	45.5	38.2	-	34.7	33.8	**39.2**
		CSA-S6-06	51.2	43.9	36.9	-	33.5	32.6	**37.8**
	Turkstra's Rule	Env. Canada	84.3	70.2	52.6	-	34.4	31.1	**54.9**
		CSA-S6-06	84.8	71.3	54.3	-	36.6	33.4	**56.5**
AB06 Montreal Site Class C	Total Probability Theorem	Env. Canada	49.2	37.6	33.8	-	33.3	32.7	**34.3**
		CSA-S6-06	47.5	36.3	32.6	-	32.2	31.5	**33.1**
	Turkstra's Rule	Env. Canada	73.6	52.3	27.6	-	27.2	27.2	**30.8**
		CSA-S6-06	74.5	53.9	30.1	-	26.5	26.2	**33.2**
AG10 Montreal Site Class C	Total Probability Theorem	Env. Canada	42.7	35.1	33.3	33.4	-	-	**33.6**
		CSA-S6-06	41.3	33.9	32.2	32.2	-	-	**32.4**
	Turkstra's Rule	Env. Canada	72.0	39.3	27.2	27.2	-	-	**28.7**
		CSA-S6-06	73.0	41.3	26.2	26.2	-	-	**28.2**
AG10 Montreal Site Class D	Total Probability Theorem	Env. Canada	38.9	33.9	31.2	30.6	-	-	**31.5**
		CSA-S6-06	37.6	32.7	30.1	29.5	-	-	**30.4**
	Turkstra's Rule	Env. Canada	57.4	27.2	27.2	27.2	-	-	**27.2**
		CSA-S6-06	58.9	26.2	26.2	26.2	-	-	**26.2**
AG10 Vancouver Site Class C	Total Probability Theorem	Env. Canada	33.1	30.8	31.2	31.0	-	-	**31.2**
		CSA-S6-06	44.3	41.3	41.9	41.5	-	-	**41.8**
	Turkstra's Rule	Env. Canada	27.0	27.0	27.0	27.0	-	-	**27.0**
		CSA-S6-06	36.2	36.2	36.2	36.2	-	-	**36.2**
AG10 Vancouver Site Class D	Total Probability Theorem	Env. Canada	32.0	30.6	30.6	30.0	-	-	**30.6**
		CSA-S6-06	42.9	41.1	41.1	40.2	-	-	**41.1**
	Turkstra's Rule	Env. Canada	27.0	27.0	27.0	27.0	-	-	**27.0**
		CSA-S6-06	36.2	36.2	36.2	36.2	-	-	**36.2**

6.3 – Performance of Base Isolators Under Extreme Temperatures

- Dynamic Performance Characteristics of LRBs

Performance Parameters	Cold Temperature 49 hrs @ -20°F	Ambient Temperature 70°F	Hot Temperature 23 hrs @ 120°F
Stiffness (kips/in)	17.0 (+56 %)	10.9	10.4 (-5 %)
Damping (% Critical)	36.7 (-3 %)	37.8	35.1 (-7 %)
EDC (in-kips)	2900.0 (+45 %)	2004.0	1777.0 (-11 %)

- Dynamic Performance Characteristics of FPI Bearings

Performance Parameters	Cold Temperature 49 hrs @ -40°F	Ambient Temperature 70°F	Hot Temperature 23 hrs @ 120°F
Stiffness (kips/in)	7.9 (+0 %)	7.8	7.1 (-9 %)
Damping (% Critical)	23.9 (-6 %)	25.5	22.9 (-10 %)
EDC (in-kips)	1044.0 (+8 %)	968.8	917.0 (-5 %)

HITEC, Highway Innovative Technology Evaluation Center, 1998

Seismic Hazard Model	Bearing Type	Climatic Data	T=0.5 sec	T=1.0 sec	T=2.0 sec	T=3.0 sec	T=4.0 sec	T=5.0 sec	Te=1.87 sec
AB06 Montreal Site Class A	LRB	Env. Canada	53.0	45.7	38.7	-	35.4	34.5	**39.6**
		CSA-S6-06	51.2	44.1	37.4	-	34.1	33.3	**38.3**
	FPI	Env. Canada	53.1	45.8	39.0	-	35.7	34.8	**39.9**
		CSA-S6-06	51.2	44.2	37.6	-	34.5	33.6	**38.5**
I=1.5 AB06 Montreal Site Class A	LRB	Env. Canada	51.1	39.5	34.9	-	33.1	32.8	**35.5**
		CSA-S6-06	49.4	38.2	33.7	-	31.9	31.6	**34.3**
	FPI	Env. Canada	51.1	39.7	35.3	-	33.6	33.3	**35.9**
		CSA-S6-06	49.4	38.4	34.1	-	32.4	32.1	**34.6**
AB06 Montreal Site Class C	LRB	Env. Canada	49.3	38.1	34.6	-	34.1	33.4	**35.1**
		CSA-S6-06	47.6	36.8	33.4	-	33.0	32.3	**33.9**
	FPI	Env. Canada	49.3	38.4	35.0	-	34.5	33.8	**35.4**
		CSA-S6-06	47.6	37.1	33.8	-	33.3	32.7	**34.2**
I=1.5 AB06 Montreal Site Class C	LRB	Env. Canada	40.8	33.9	32.0	-	33.3	31.8	**32.2**
		CSA-S6-06	39.4	32.8	30.9	-	32.1	30.7	**31.1**
	FPI	Env. Canada	40.9	34.3	32.5	-	33.9	32.4	**32.8**
		CSA-S6-06	39.5	33.1	31.4	-	32.7	31.3	**31.7**
AG10 Montreal Site Class C	LRB	Env. Canada	42.9	35.7	34.4	34.8	-	-	**34.6**
		CSA-S6-06	41.5	34.5	33.2	33.6	-	-	**33.4**
	FPI	Env. Canada	43.0	36.0	35.0	35.5	-	-	**35.1**
		CSA-S6-06	41.6	34.8	33.8	34.3	-	-	**33.9**
I=1.5 AG10 Montreal Site Class C	LRB	Env. Canada	40.1	34.1	32.0	31.1	-	-	**32.3**
		CSA-S6-06	38.7	32.9	30.9	30.1	-	-	**31.2**
	FPI	Env. Canada	40.2	34.5	32.9	32.2	-	-	**33.1**
		CSA-S6-06	38.9	33.3	31.8	31.1	-	-	**32.0**
AG10 Montreal Site Class D	LRB	Env. Canada	39.5	35.0	33.3	33.2	-	-	**33.5**
		CSA-S6-06	38.1	33.8	32.1	32.1	-	-	**32.3**
	FPI	Env. Canada	39.7	35.6	34.4	34.6	-	-	**34.5**
		CSA-S6-06	38.3	34.4	33.2	33.5	-	-	**33.3**
I=1.5 AG10 Montreal Site Class D	LRB	Env. Canada	35.5	31.8	33.9	33.7	-	-	**33.7**
		CSA-S6-06	34.2	30.7	32.8	32.6	-	-	**32.5**
	FPI	Env. Canada	35.8	32.6	35.6	35.9	-	-	**35.2**
		CSA-S6-06	34.6	31.5	34.4	34.7	-	-	**34.0**

AG10 Vancouver Site Class C	LRB	Env. Canada	34.0	32.8	35.7	36.7	-	-	**35.3**
		CSA-S6-06	45.6	44.1	47.9	49.2	-	-	**47.4**
	FPI	Env. Canada	34.5	33.9	38.0	39.8	-	-	**37.5**
		CSA-S6-06	46.3	45.5	51.0	53.4	-	-	**50.3**
I=1.5 AG10 Vancouver Site Class C	LRB	Env. Canada	30.3	29.8	32.9	34.7	-	-	**32.5**
		CSA-S6-06	40.7	39.9	44.1	46.6	-	-	**43.5**
	FPI	Env. Canada	31.0	31.2	35.8	38.7	-	-	**35.2**
		CSA-S6-06	41.6	41.8	48.1	52.0	-	-	**47.3**
AG10 Vancouver Site Class D	LRB	Env. Canada	33.4	34.3	38.9	41.7	-	-	**38.3**
		CSA-S6-06	44.8	46.1	52.2	55.9	-	-	**51.4**
	FPI	Env. Canada	34.2	36.3	43.2	47.6	-	-	**42.3**
		CSA-S6-06	45.9	48.7	58.0	63.9	-	-	**56.8**
I=1.5 AG10 Vancouver Site Class D	LRB	Env. Canada	33.3	34.7	38.1	42.4	-	-	**37.6**
		CSA-S6-06	44.7	46.6	51.1	56.9	-	-	**50.5**
	FPI	Env. Canada	34.5	37.8	43.8	50.6	-	-	**43.0**
		CSA-S6-06	46.3	50.8	58.7	67.9	-	-	**57.7**

Conclusion

- Probabilistic approcah has been implemented

  -Distribution of temperatures
  -Earthquake hazards
  -Perfromance of isolators

- Applied to a sample of isolated bridges in various climatic and seismic zones

- 50% of the thermal dispalcements appears to cover all cases

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THANK YOU

QUESTIONS?

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Group for Research in
STRUCTURAL ENGINEERING

On some ingredients for performance-based assessment of isolated bridges in Canada

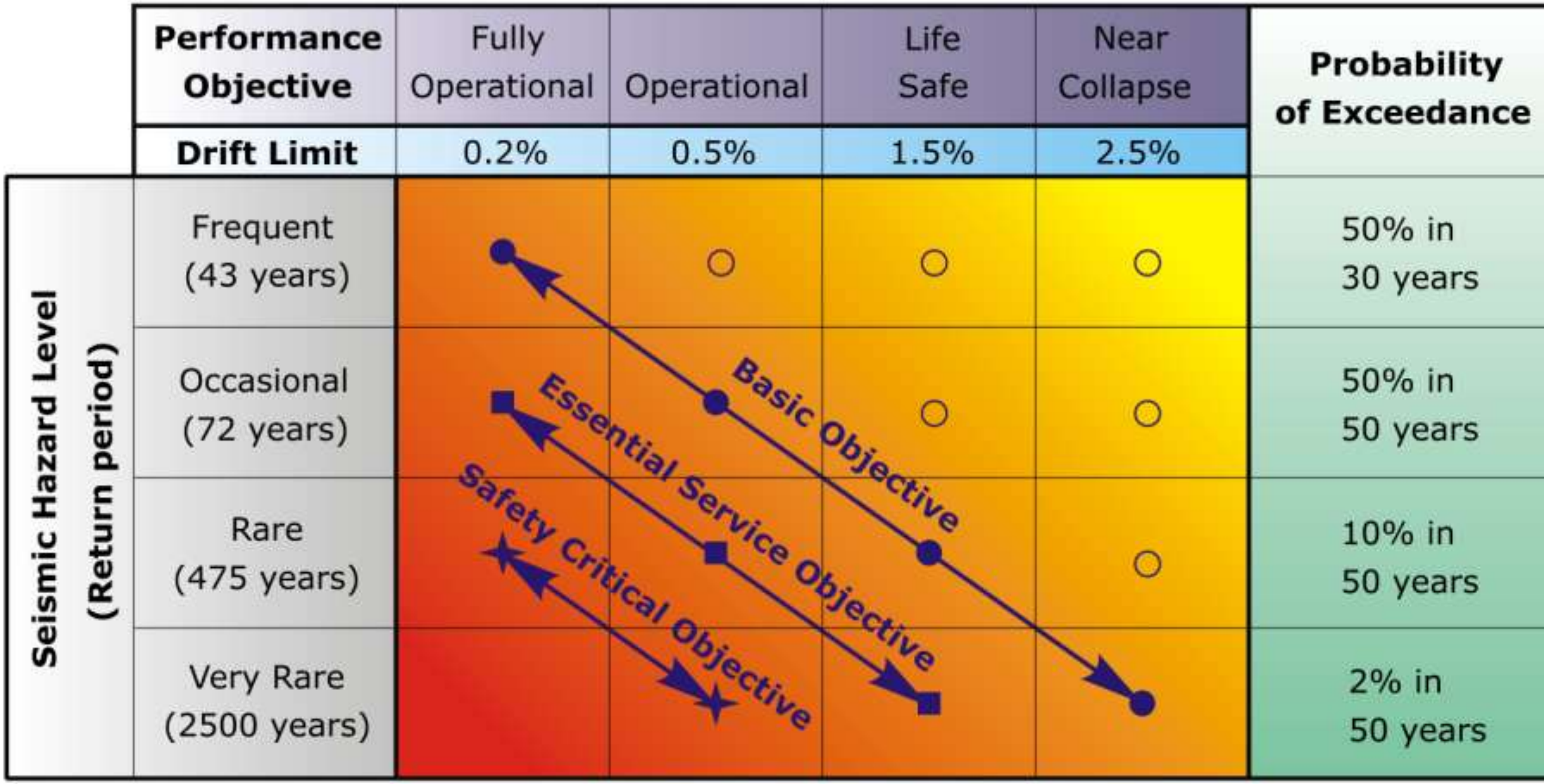
Najib Bouaanani, Eng., Ph.D.

Associate Professor

Department of Civil, Geological and Mining Engineering

- Destroying Loma Prieta (1989), Northridge (1994) and Kobe (1995) earthquakes enhanced interest in Performance-Based Seismic Design (PBSD) as an alternative to prescriptive building codes which depend on Force-Based approaches.

- PBSD gives the designer more flexibility to meet target performance and economic objectives.



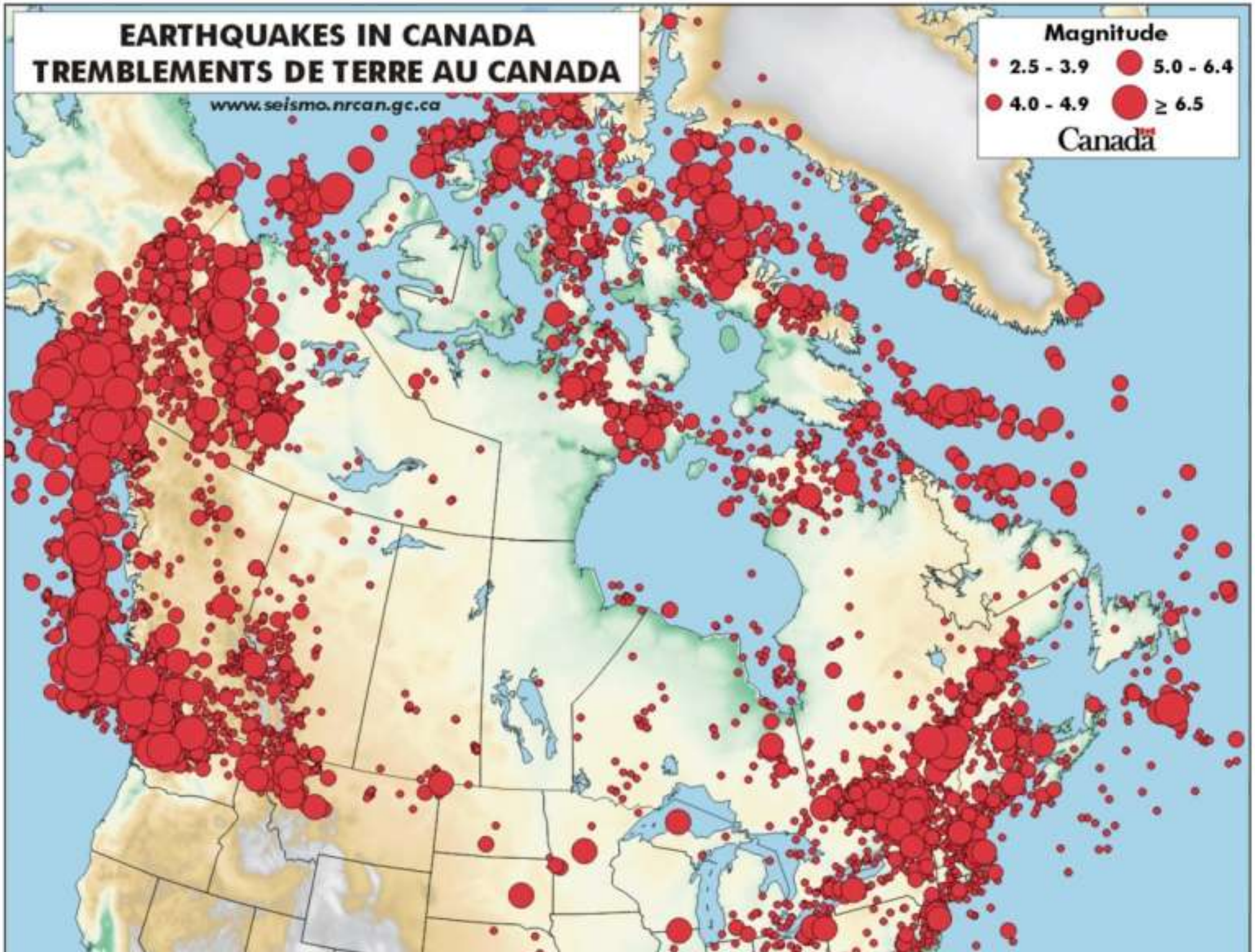
[*Adapted from Vision 2000 Blue Book (1995) and DeVall (2003)*]

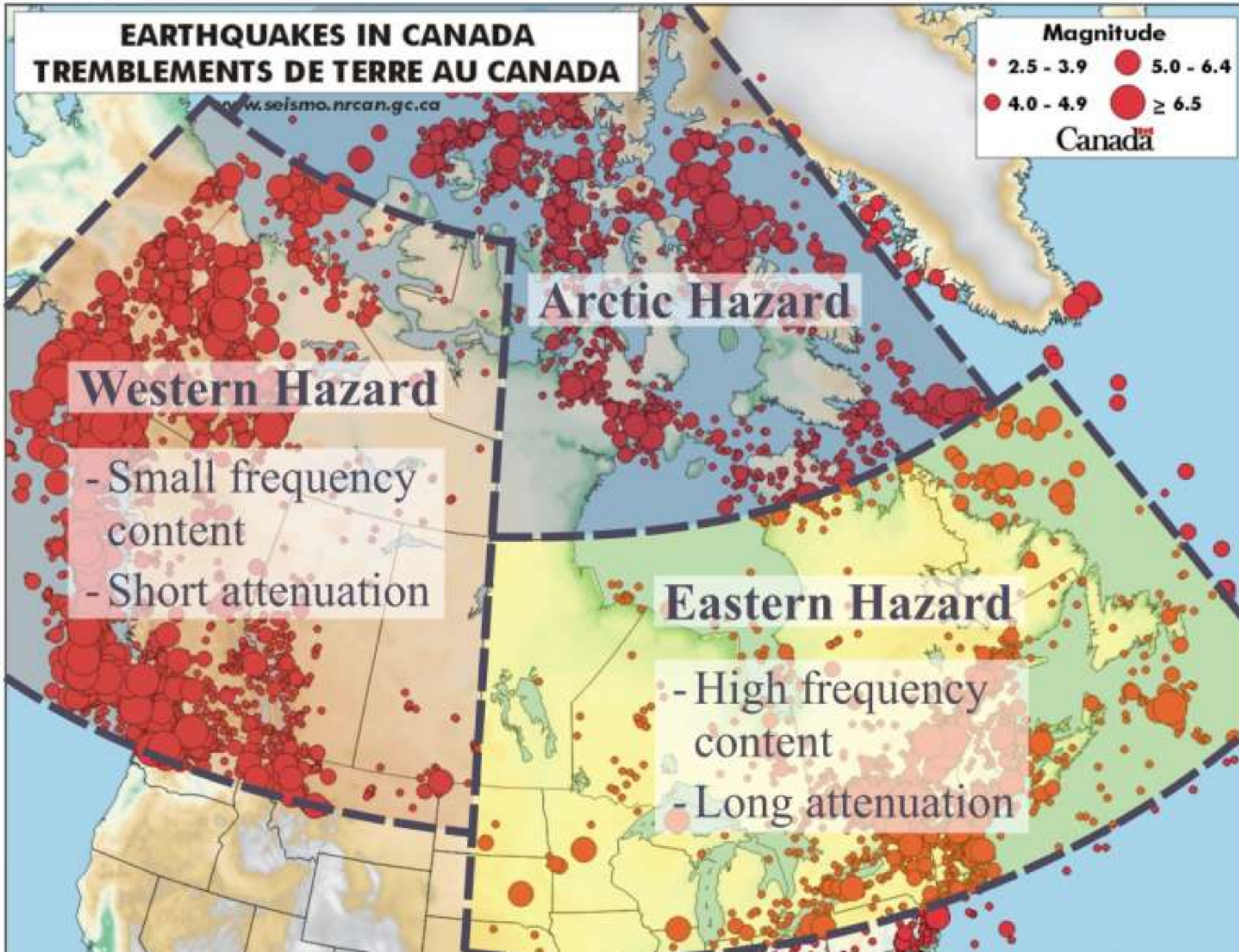
Main objective

- Assessment of the structural seismic performance of isolated bridges considering specificities of seismic hazard in Canada

Focus on the following important ingredients

- Displacement demands in eastern and western Canada
- Damping effects
- Ductility effects
- 3D effects

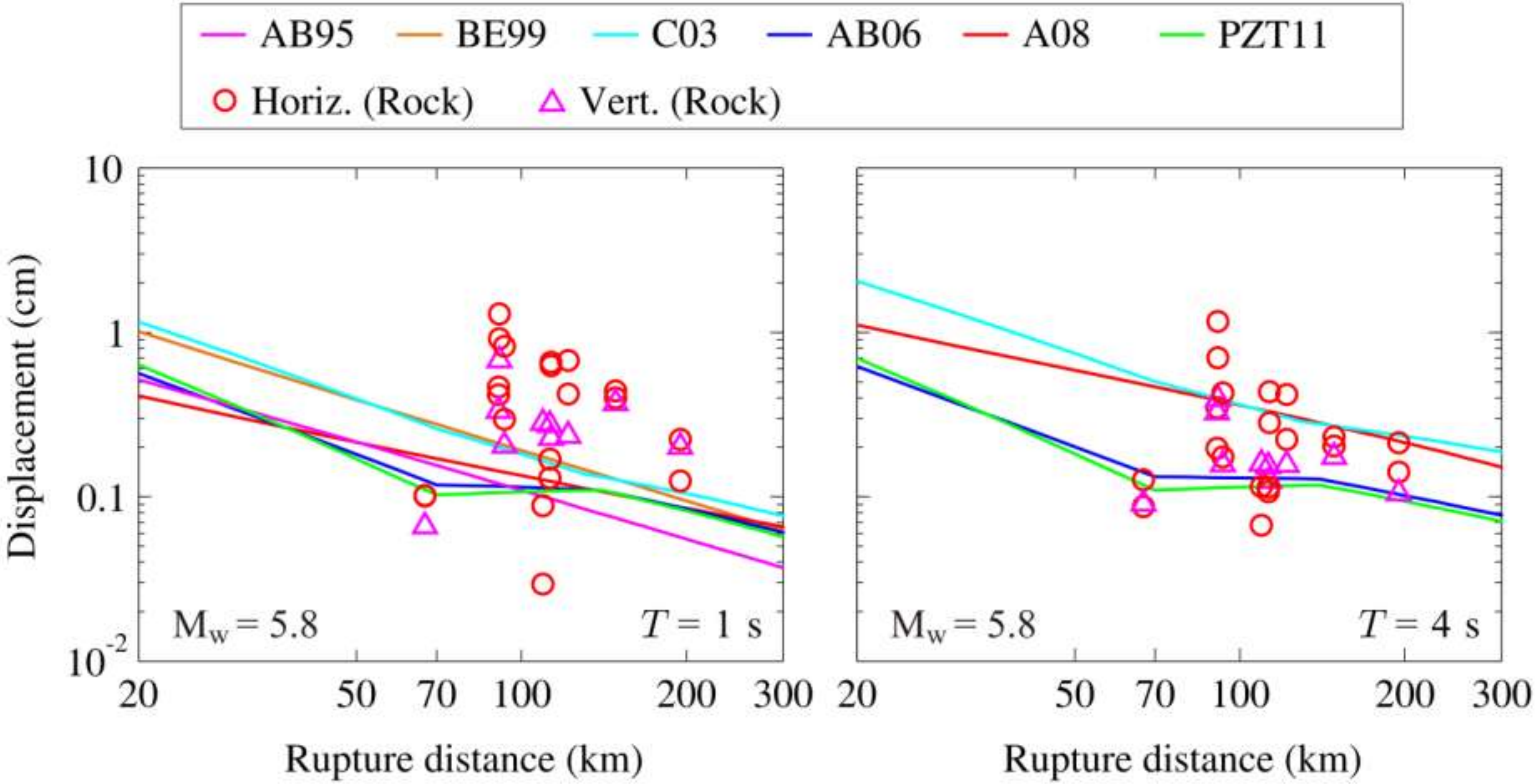

EARTHQUAKES IN CANADA
TREMBLEMENTS DE TERRE AU CANADA
www.seismo.nrcan.gc.ca
Magnitude
2.5 - 3.9
4.0 - 4.9
5.0 - 6.4
≥ 6.5
Canada


EARTHQUAKES IN CANADA
TREMBLEMENTS DE TERRE AU CANADA
www.seismo.nrcan.gc.ca
Magnitude
2.5 - 3.9
4.0 - 4.9
5.0 - 6.4
≥ 6.5
Canada
Arctic Hazard
Western Hazard
- Small frequency content
- Short attenuation
Eastern Hazard
- High frequency content
- Long attenuation

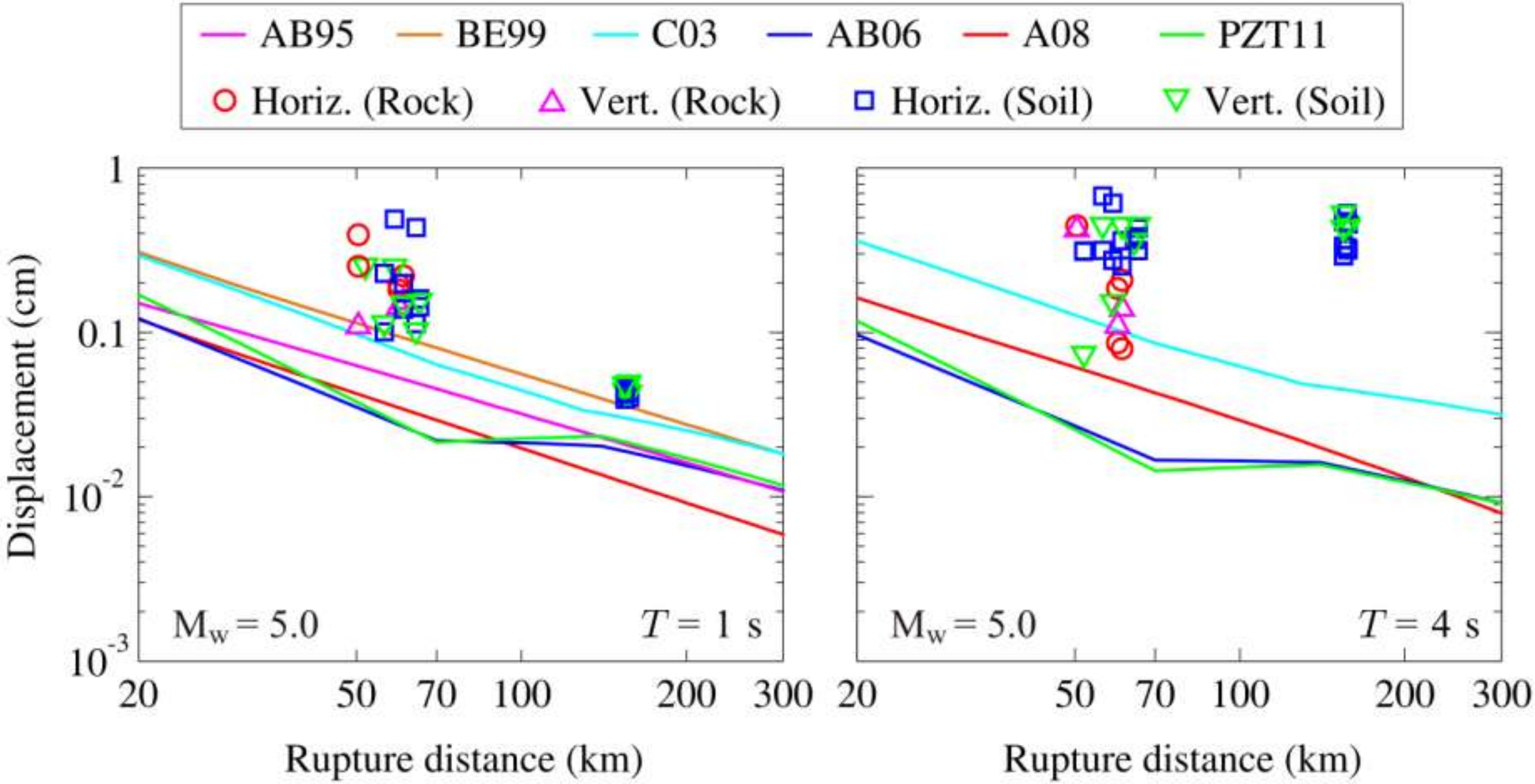
Elastic spectral displacements

- Most ground motion prediction equations focused on spectral accelerations
- Evaluation of spectral displacements is important for the design and evaluation of seismically isolated/damped bridges
- Such evaluation is even more important in Eastern Canada because of the scarcity of recorded earthquake events
- Assessment of available ground motion prediction equations
  - Atkinson and Boore (1995) [AB95]
  - Bommer and Elnashai (1999) [BE99]
  - Campbell (2003) [C03]
  - Atkinson and Boore (2006) [AB06]
  - Atkinson (2008) [A08]
  - Pezeshk et al. (2011) [PZT11]

- Saguenay event (1988)



- Val des Bois event (2010)

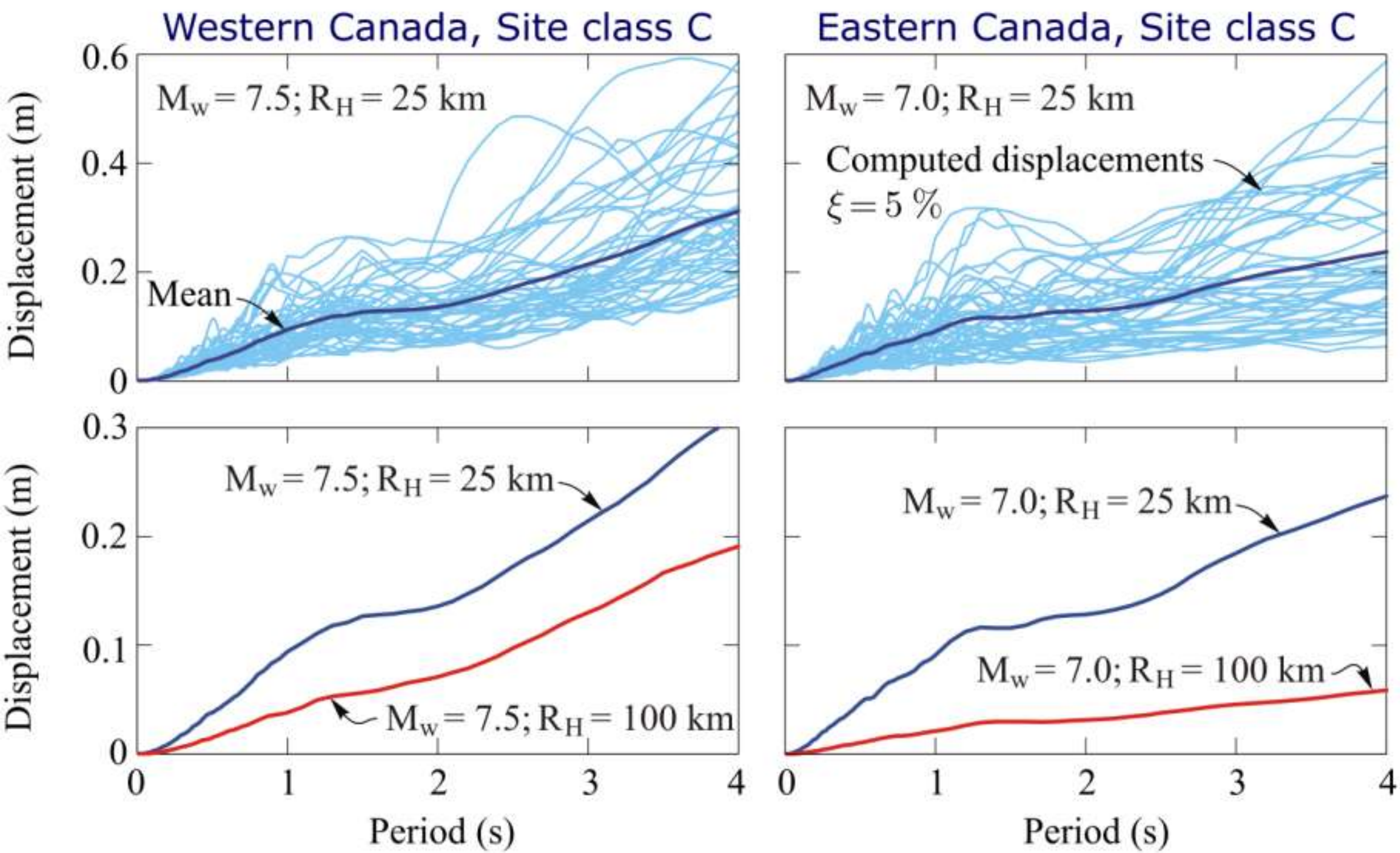


Elastic spectral displacements

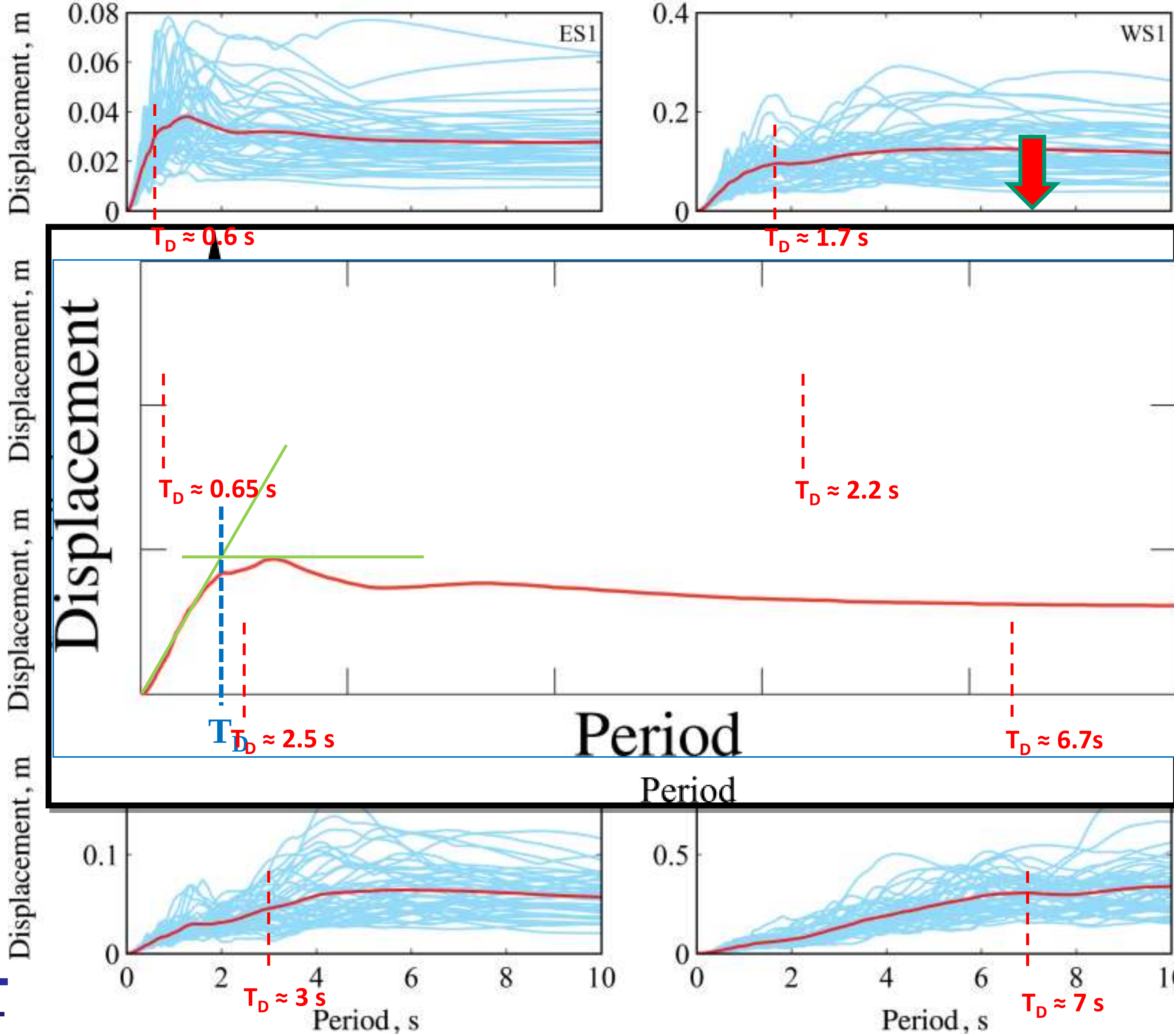
- Database of simulated earthquakes (Atkinson 2009)

	Western Canada				Eastern Canada			
	WS1	WS2	WS3	WS4	ES1	ES2	ES3	ES4
No. of Records	45	45	45	45	45	45	45	45
Magnitude (M_W)	6.5	6.5	7.5	7.5	6.0	6.0	7.0	7.0
R_H (km)	12	30	25	100	15	30	25	100
Min. R_F (km)	8.4	13.2	10.2	30.2	10.7	16.9	13.8	41.6
Max. R_F (km)	13	31.1	26.3	100.4	17.0	30.7	25.8	100.2
	Short periods		Long periods		Short periods		Long periods	

Elastic spectral displacements



ents



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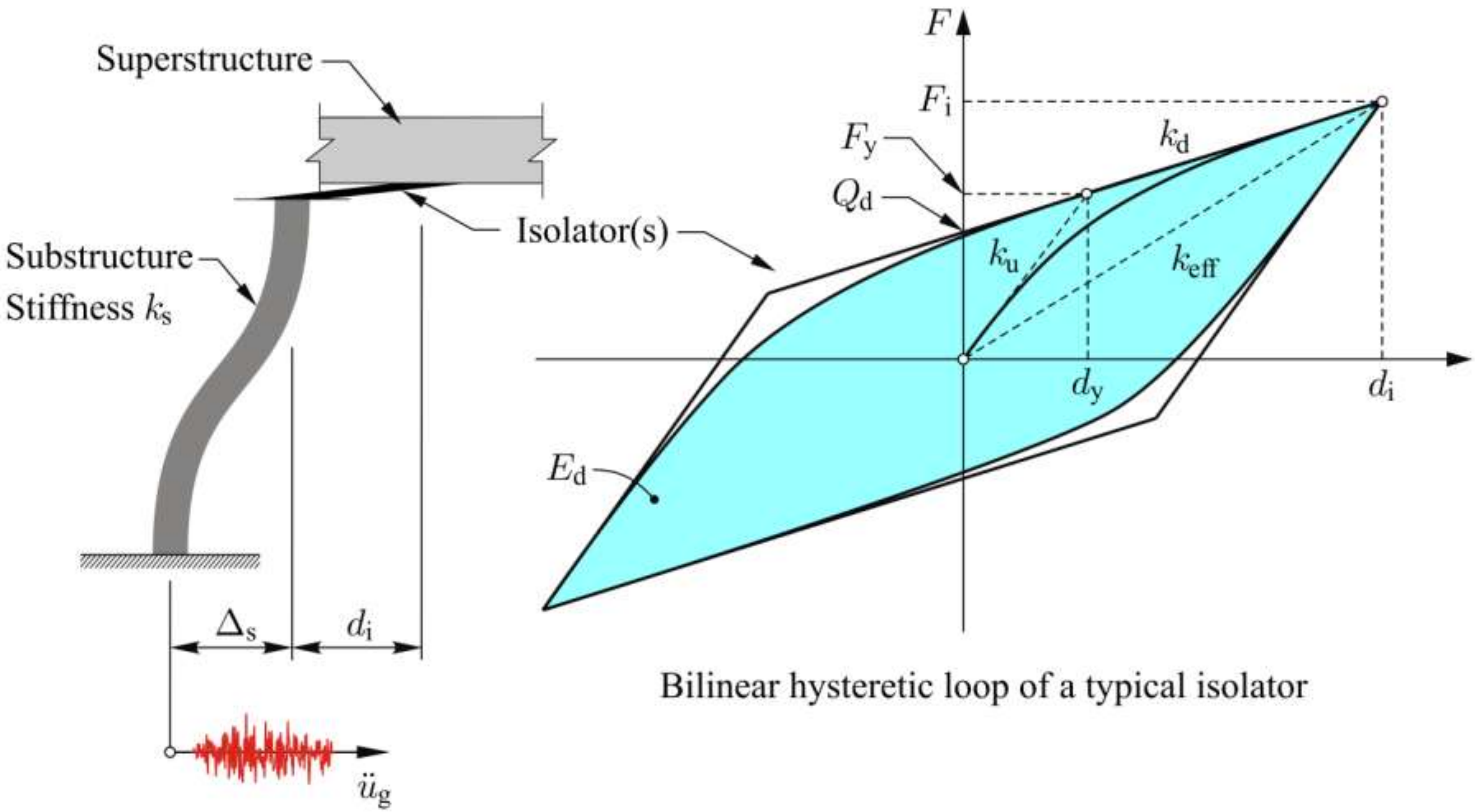
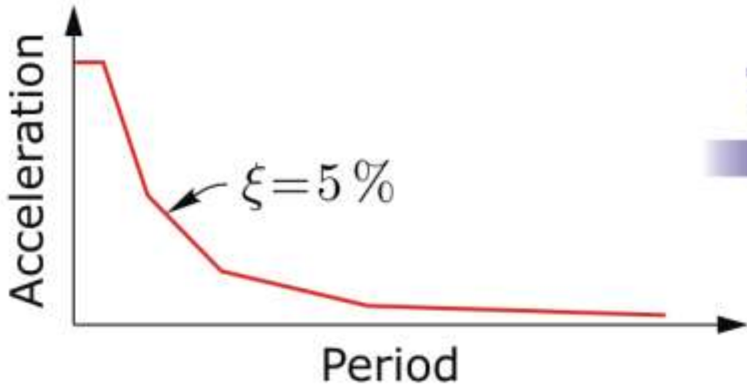
- Typical seismically-isolated bridge



Bilinear hysteretic loop of a typical isolator

Inelastic spectral displacements

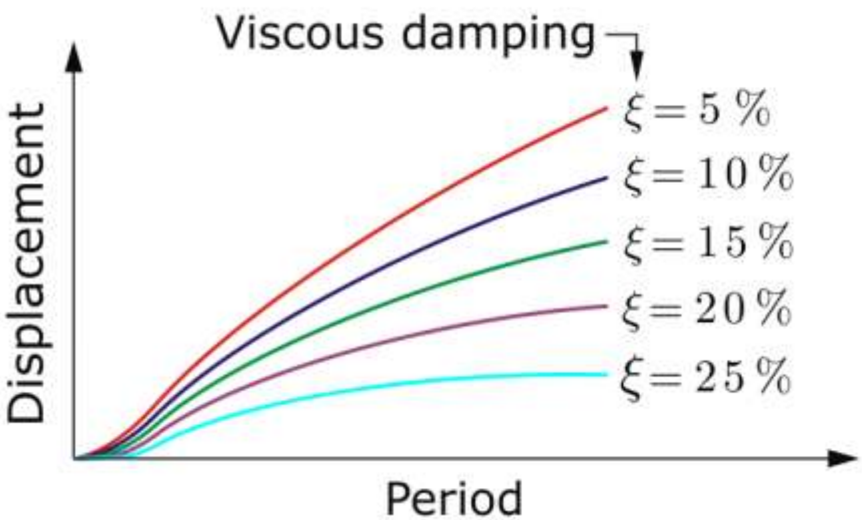
- Input: Elastic demand

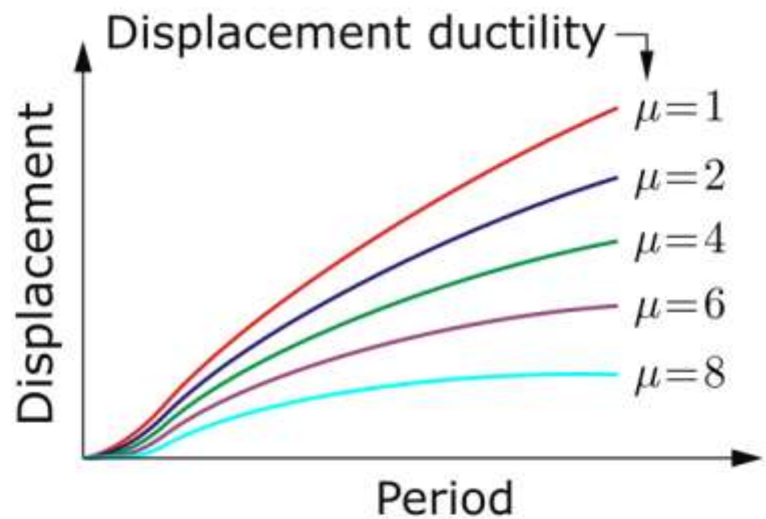


?

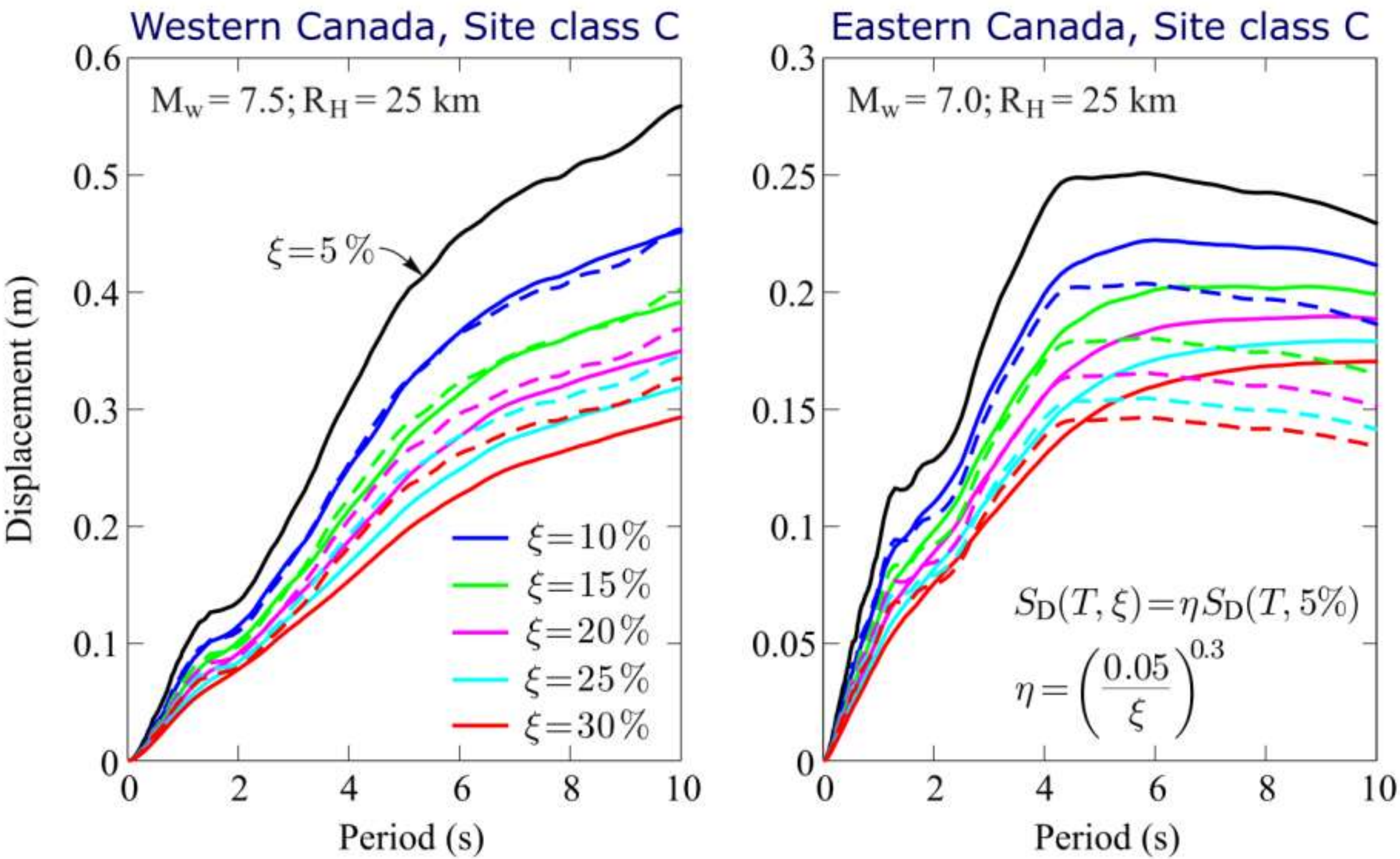
- Output: Inelastic demand
  - In terms of <u>inelastic displacement</u> design spectra
  - For given <u>performance objectives</u> expressed in terms of <u>target displacement ductilities</u> μ

- Solutions: Modified displacement spectra

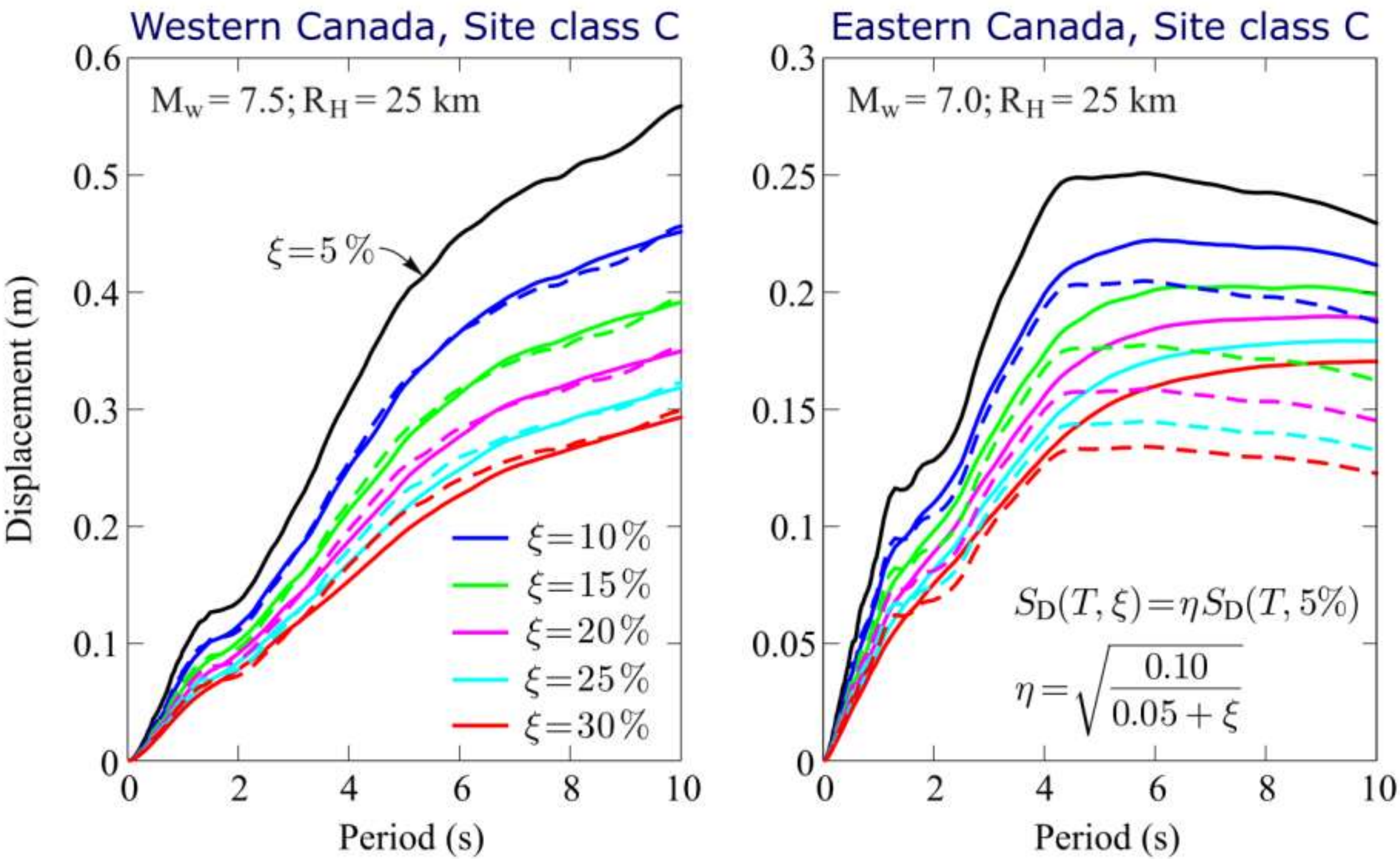




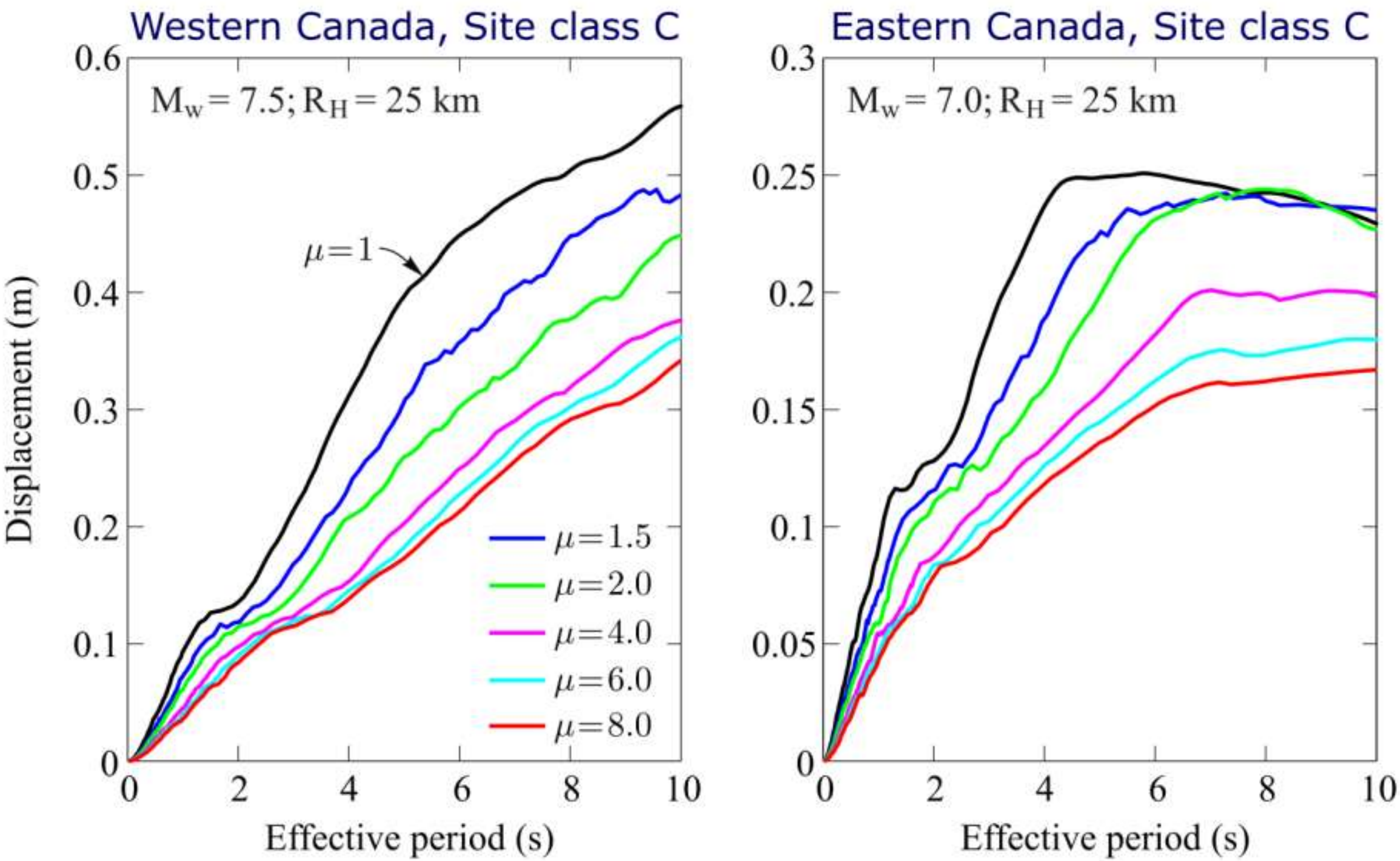
Effect of viscous damping



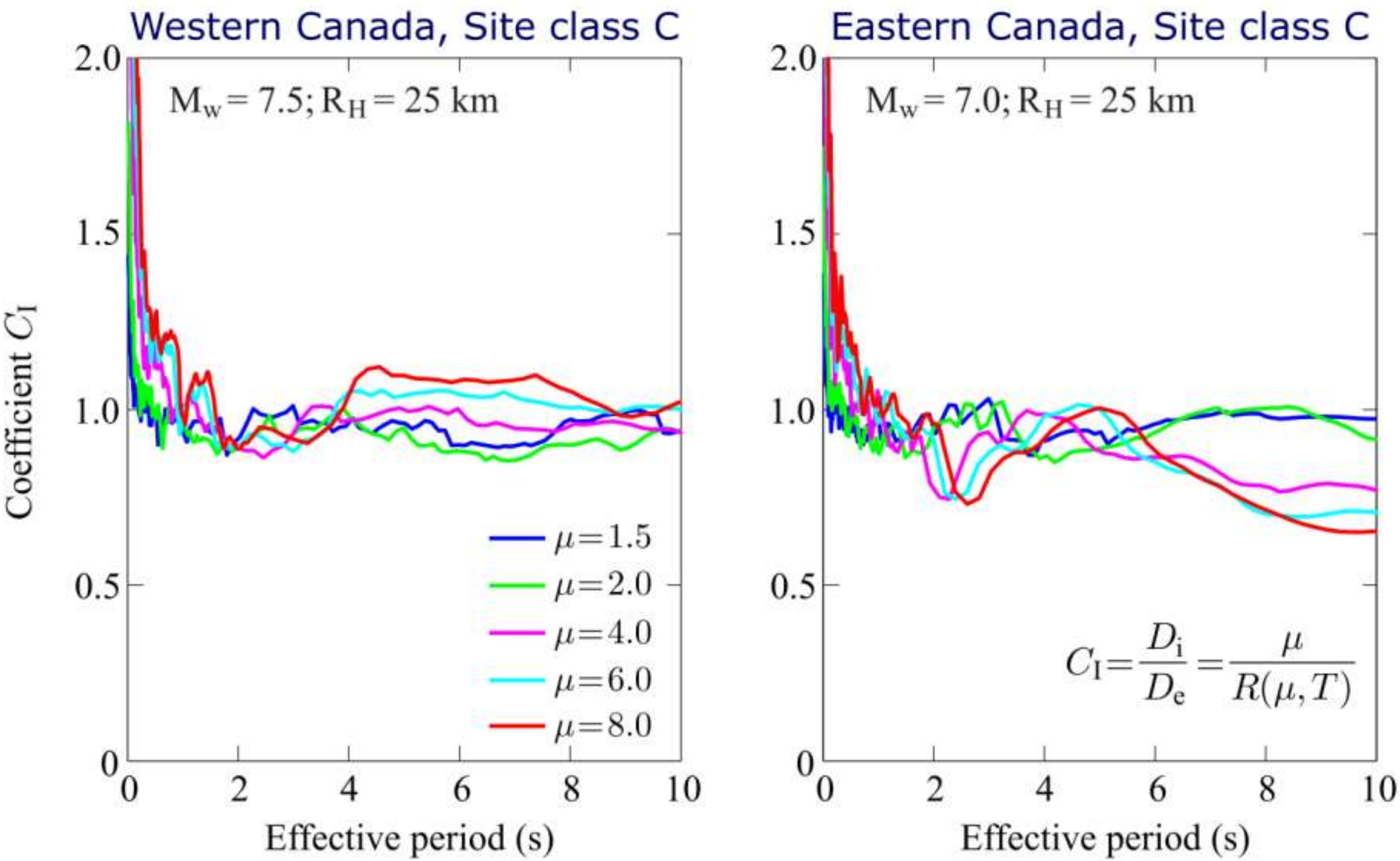
Effect of viscous damping

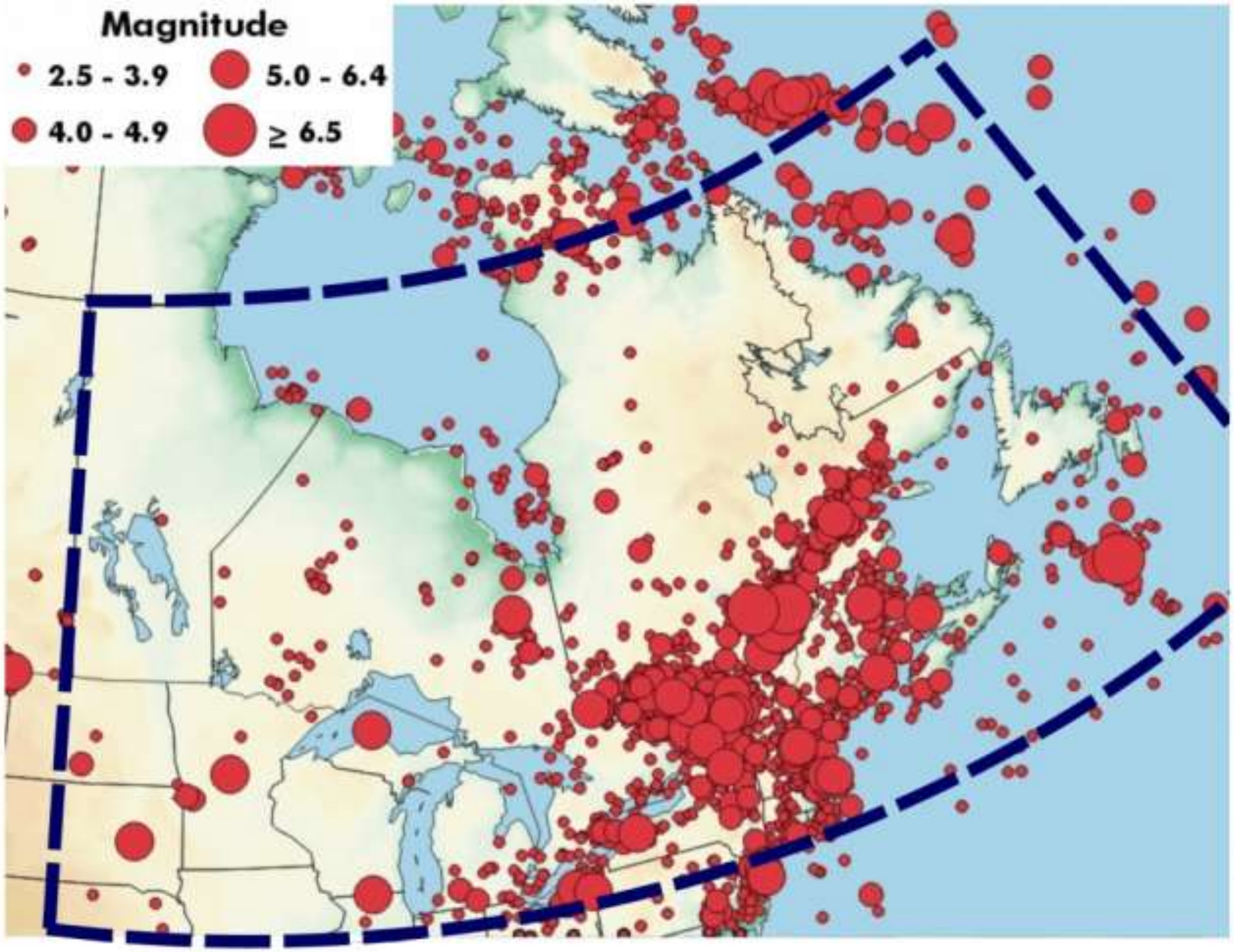


Effect of ductility



Effect of ductility





1982: Miramichi (4.8 M_N)
15 GM components

1985: Nahanni (6.4 M_b)
25 GM components

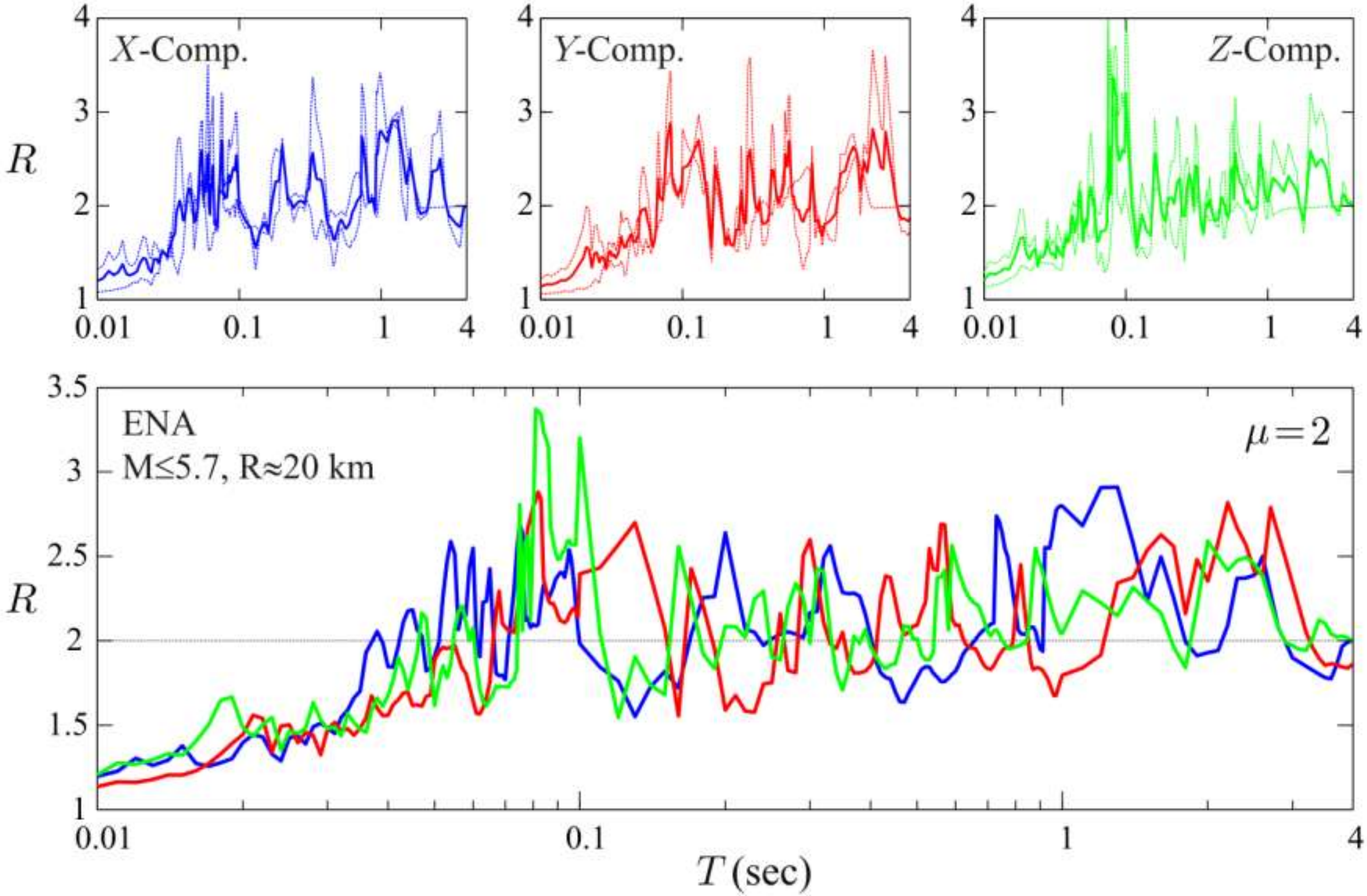
1988: Saguenay (5.9 M_W)
30 GM components

1997: Cap Rouge (5.1 M_N)
9 GM components

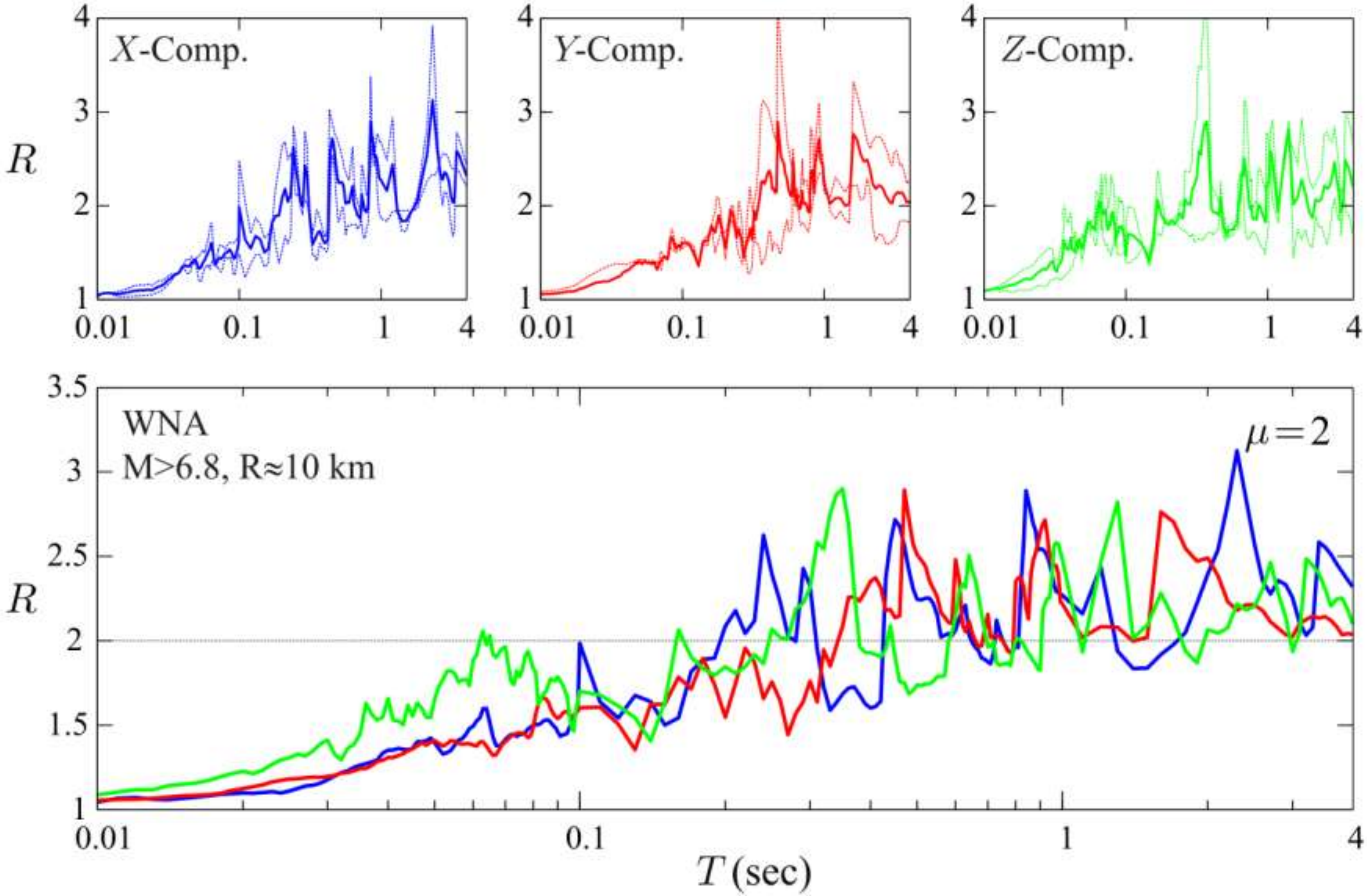
2005: Riv. du Loup (5.4 M_N)
21 GM components

2010: Val des Bois (5.0 M_W)
39 GM components

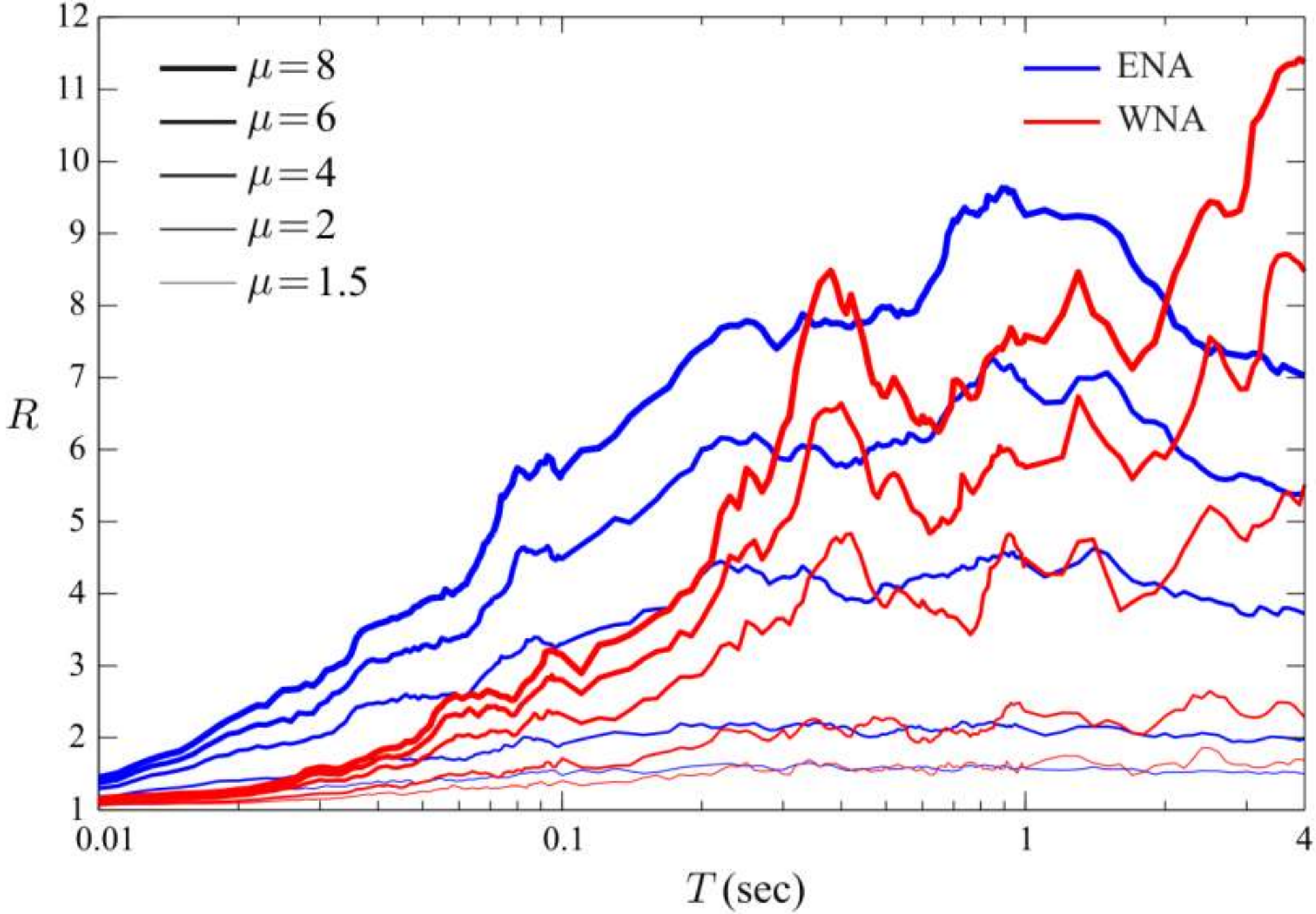
R-μ-T curves – ENA historical data



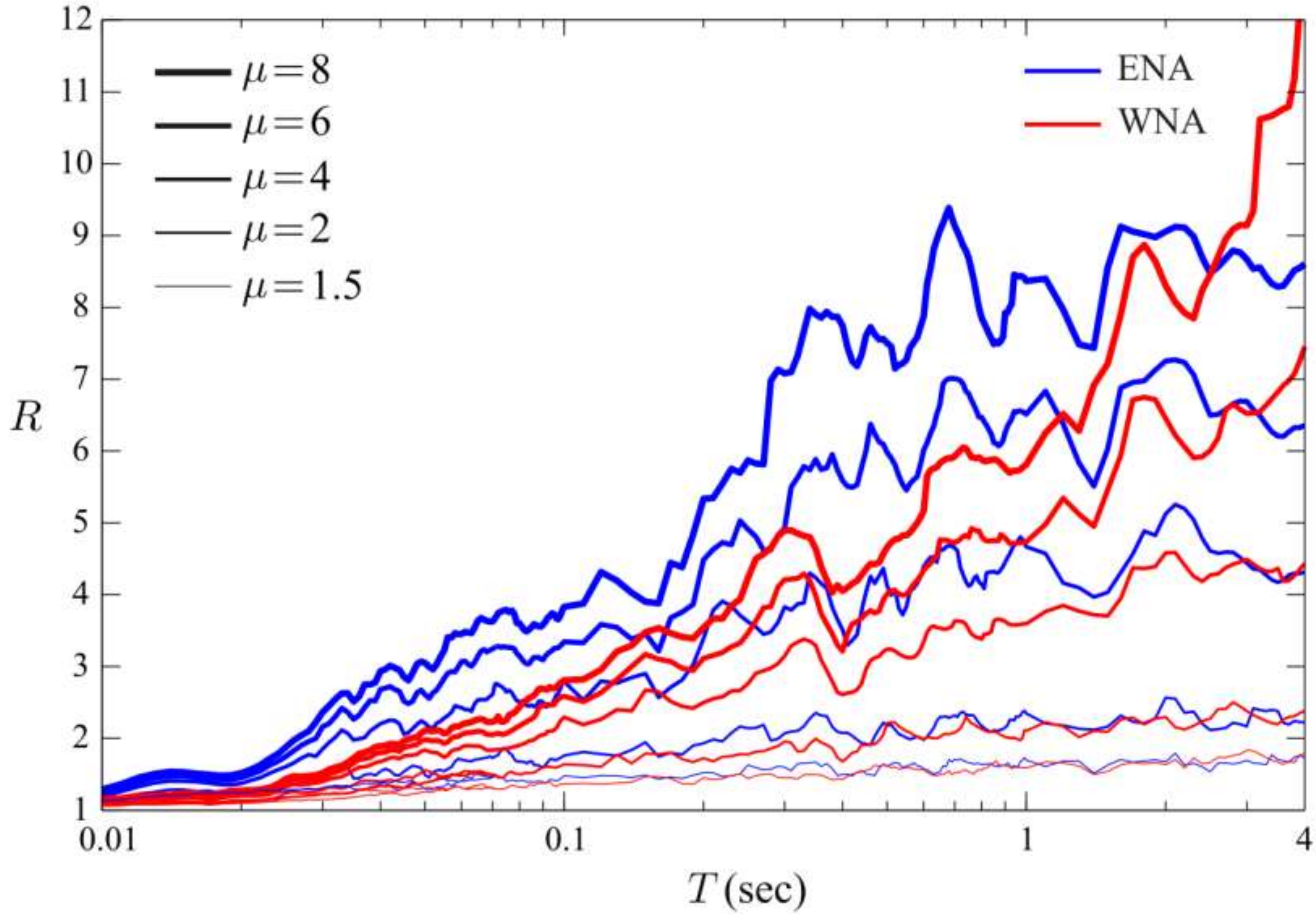
R-μ-T curves – WNA historical data



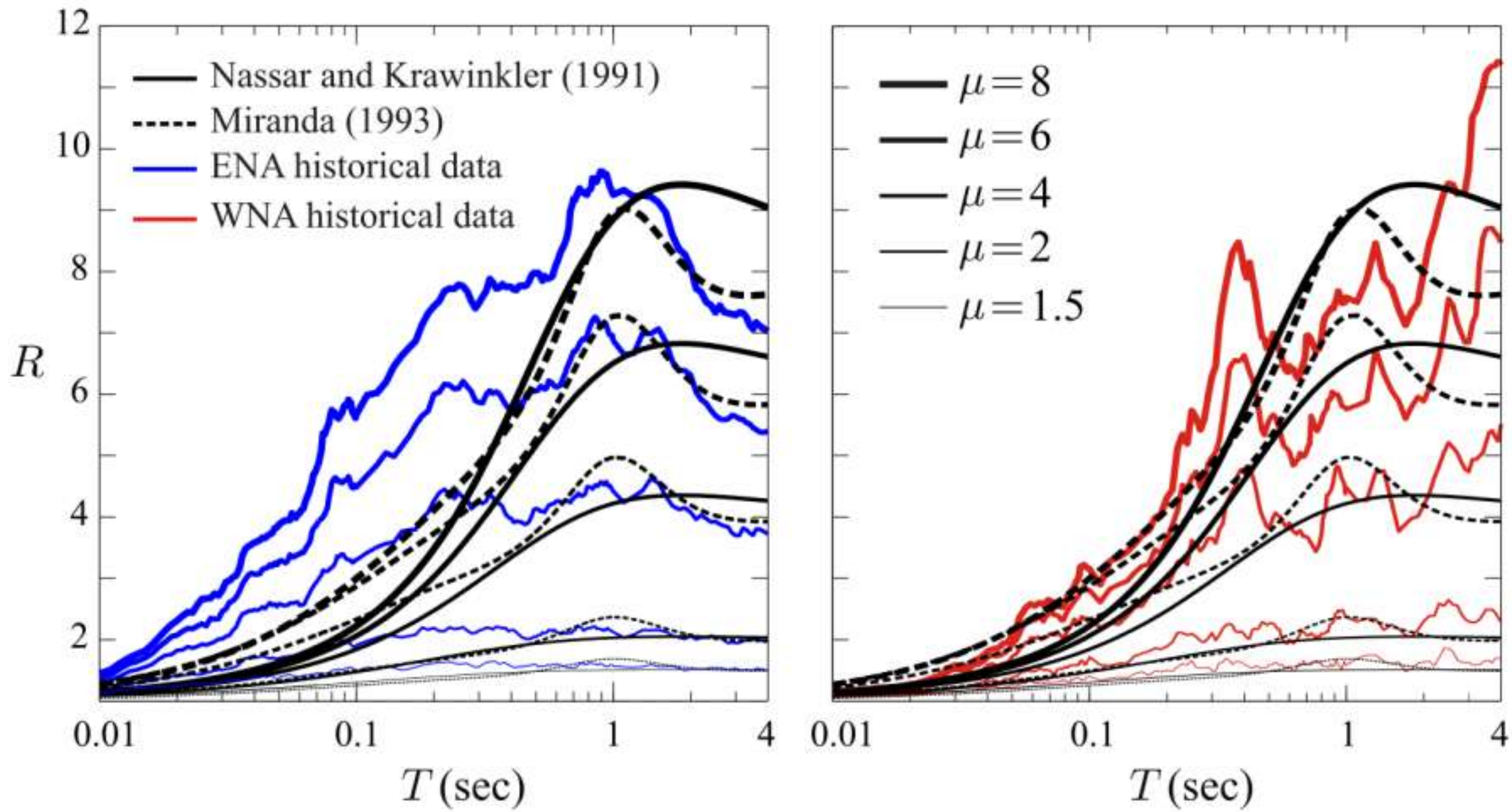
R-μ-T curves – ENA vs WNA historical data



R-μ-T curves – Simulated records

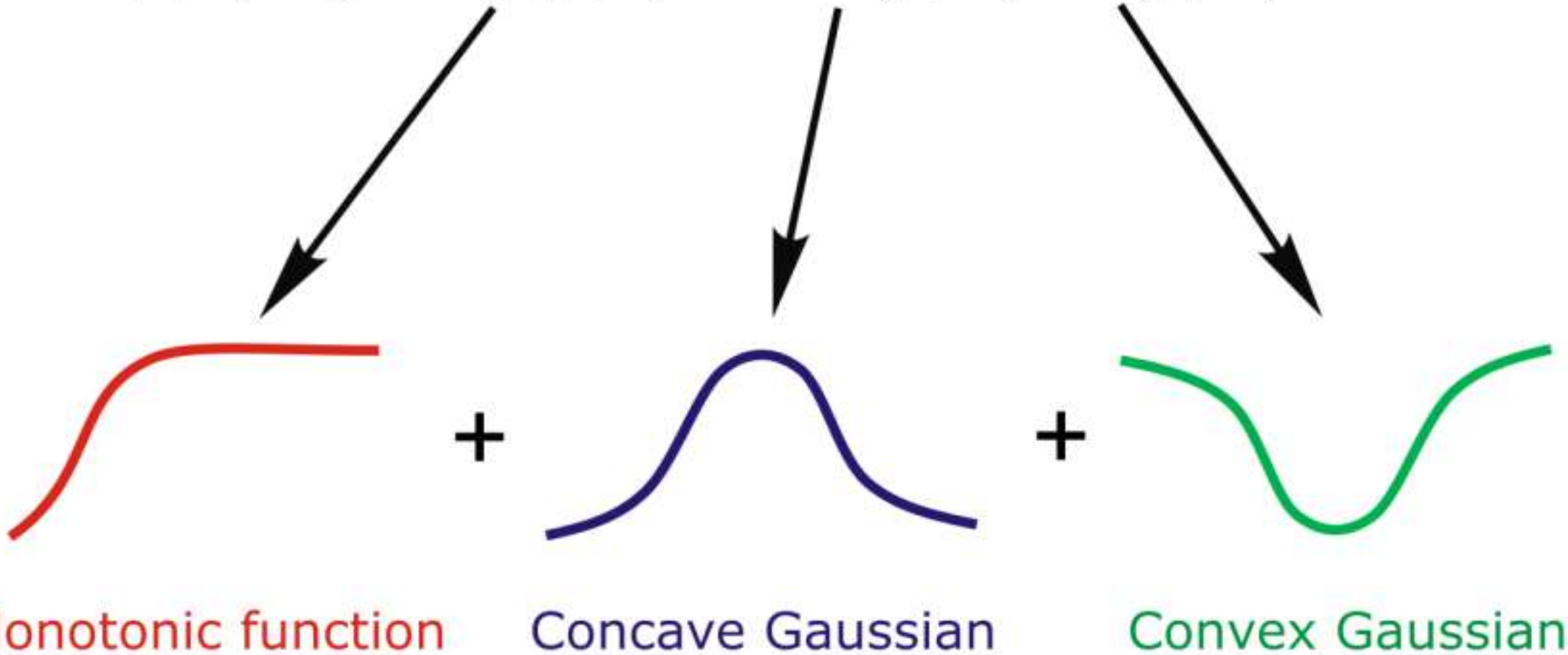


Available R-μ-T predictions vs historical data



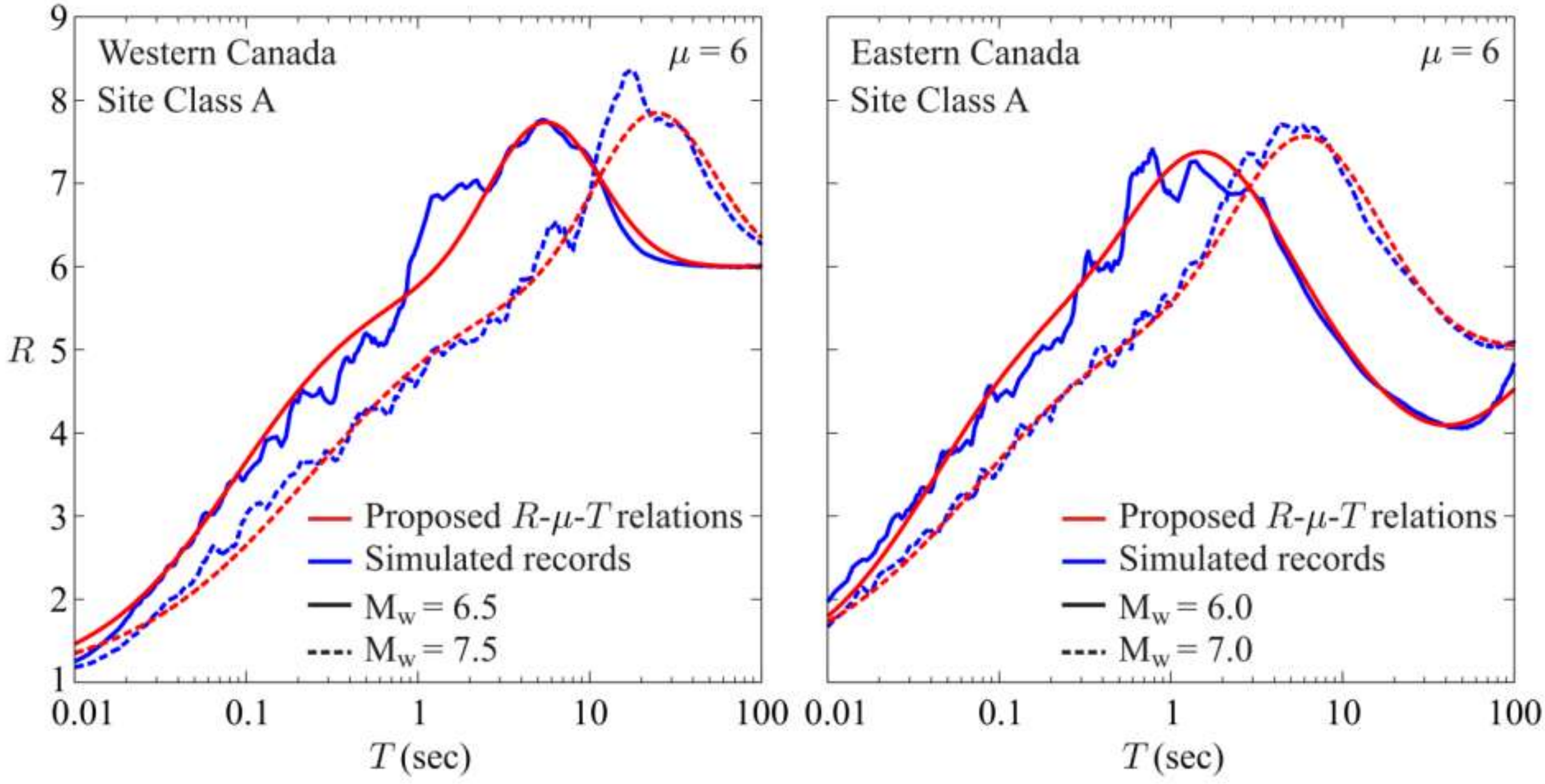
New R-μ-T relations - Canadian seismic hazard

$$R(\mu, T) = (\mu - 1) f(\mu, T) + 1 + P(\mu, T) + H(\mu, T)$$

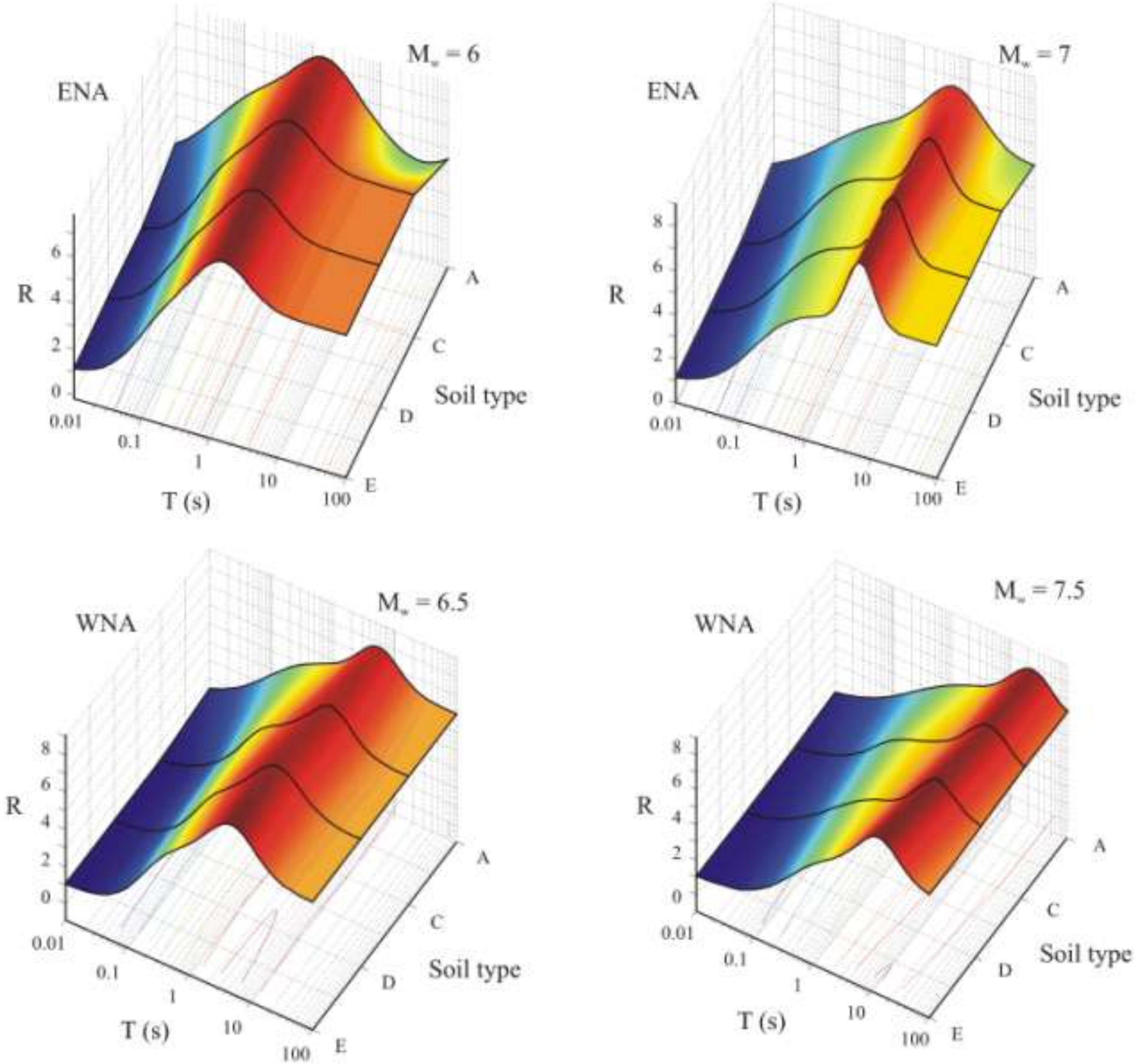


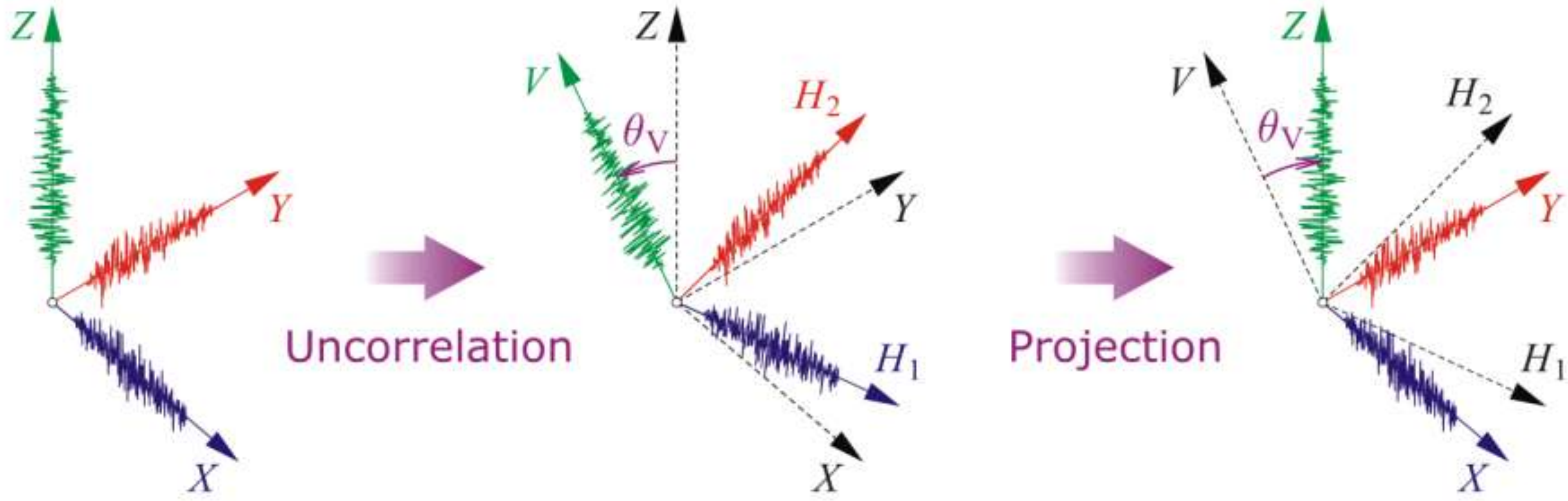
$$f(\mu, T) = \frac{10^{\kappa(\mu, T)}}{1 + 10^{\kappa(\mu, T)}} \quad \text{with} \quad \kappa(\mu, T) = p(\mu) \log_{10}(T) + q(\mu)$$

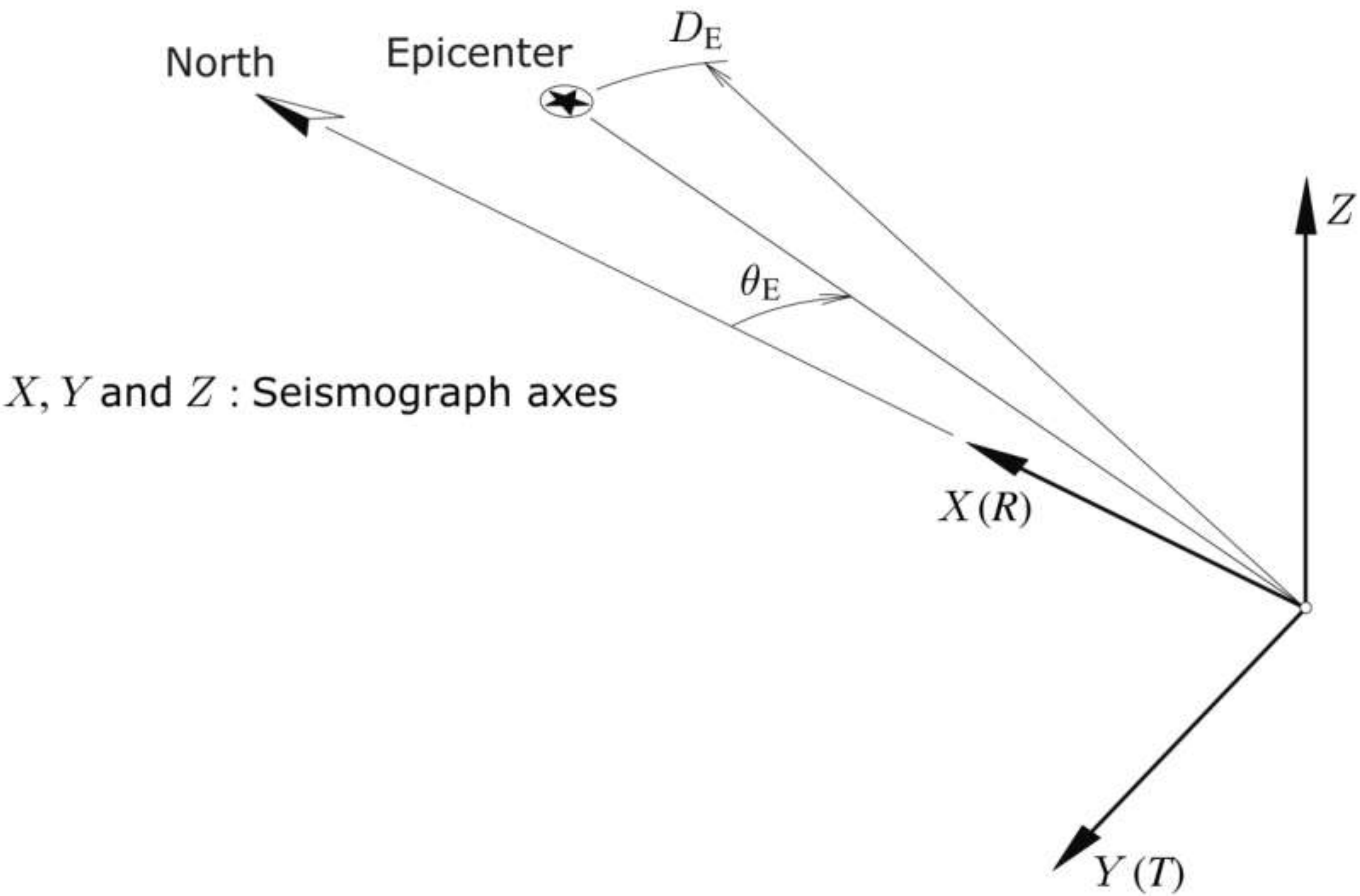
New R-μ-T relations - Canadian seismic hazard



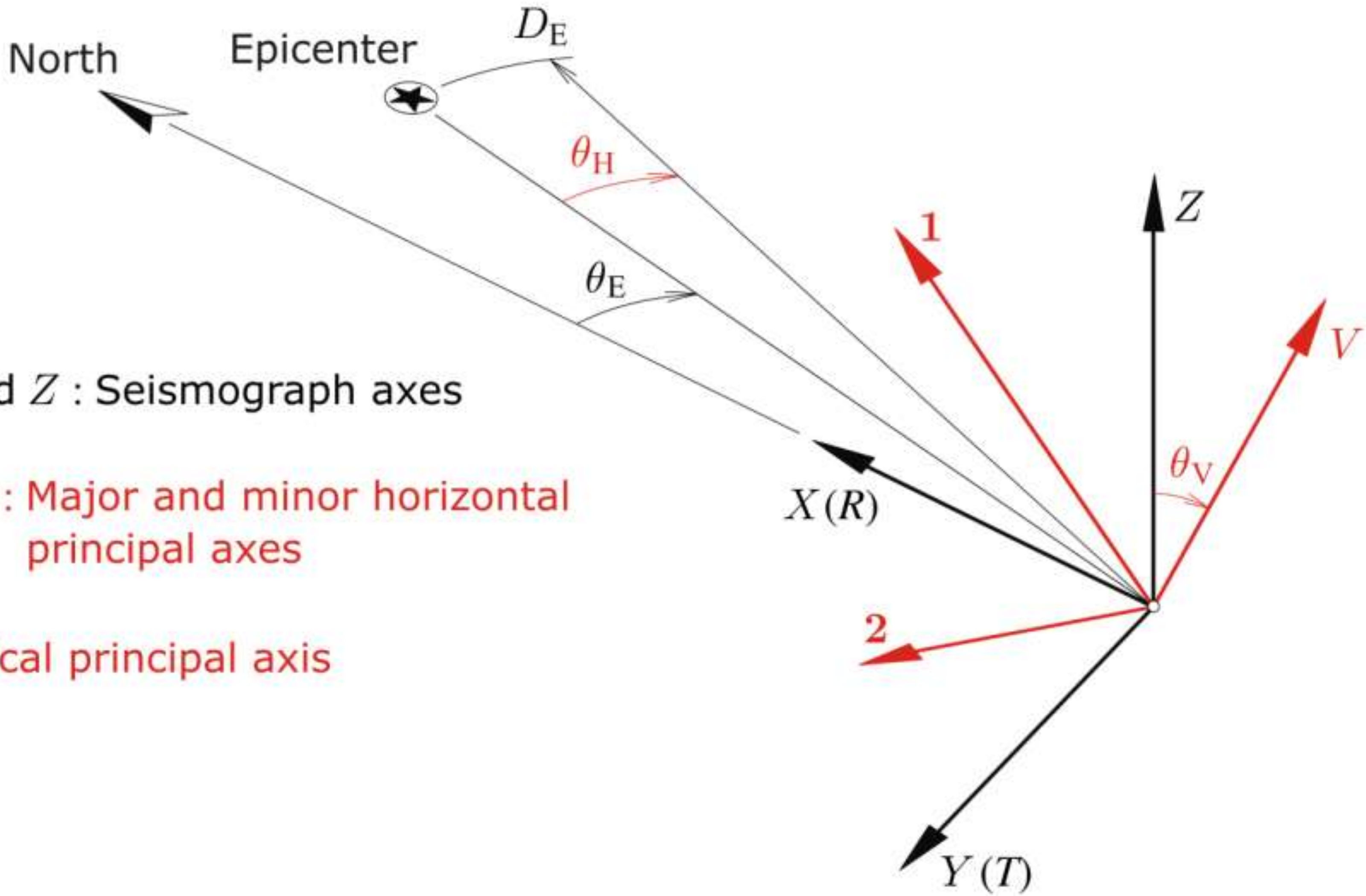
New R-μ-T relations - Canadian seismic hazard




Z
Y
X
Uncorrelation
Z
V
θ_V
H_2
Y
H_1
X
Projection
Z
V
θ_V
H_2
Y
H_1
X



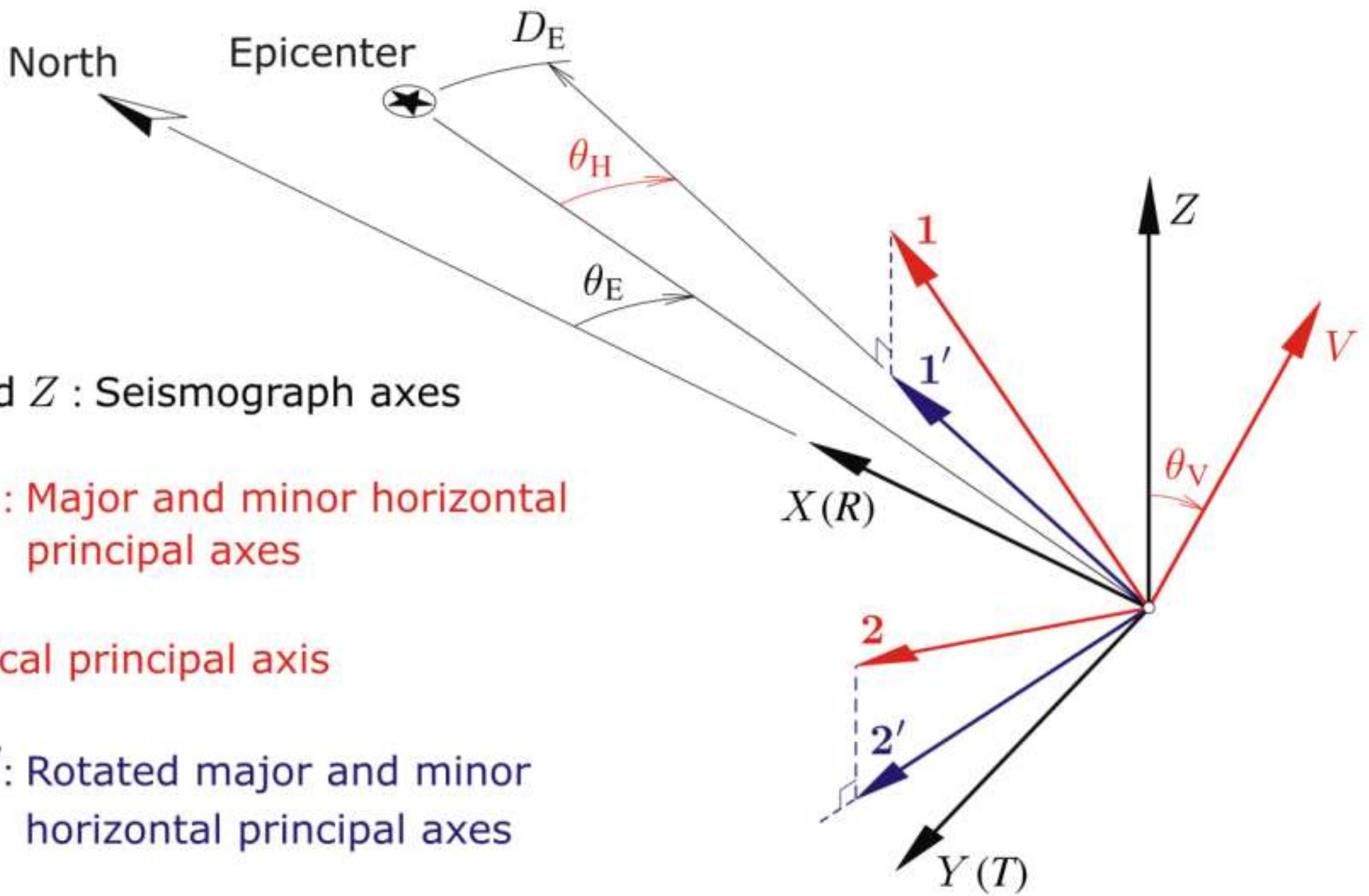
North
Epicenter
D_E
θ_E
Z
X, Y and Z : Seismograph axes
$X\,(R)$
$Y\,(T)$



X, Y and Z : Seismograph axes

1 and **2** : Major and minor horizontal principal axes

V : Vertical principal axis



X, Y and Z : Seismograph axes

1 and **2** : Major and minor horizontal principal axes

V : Vertical principal axis

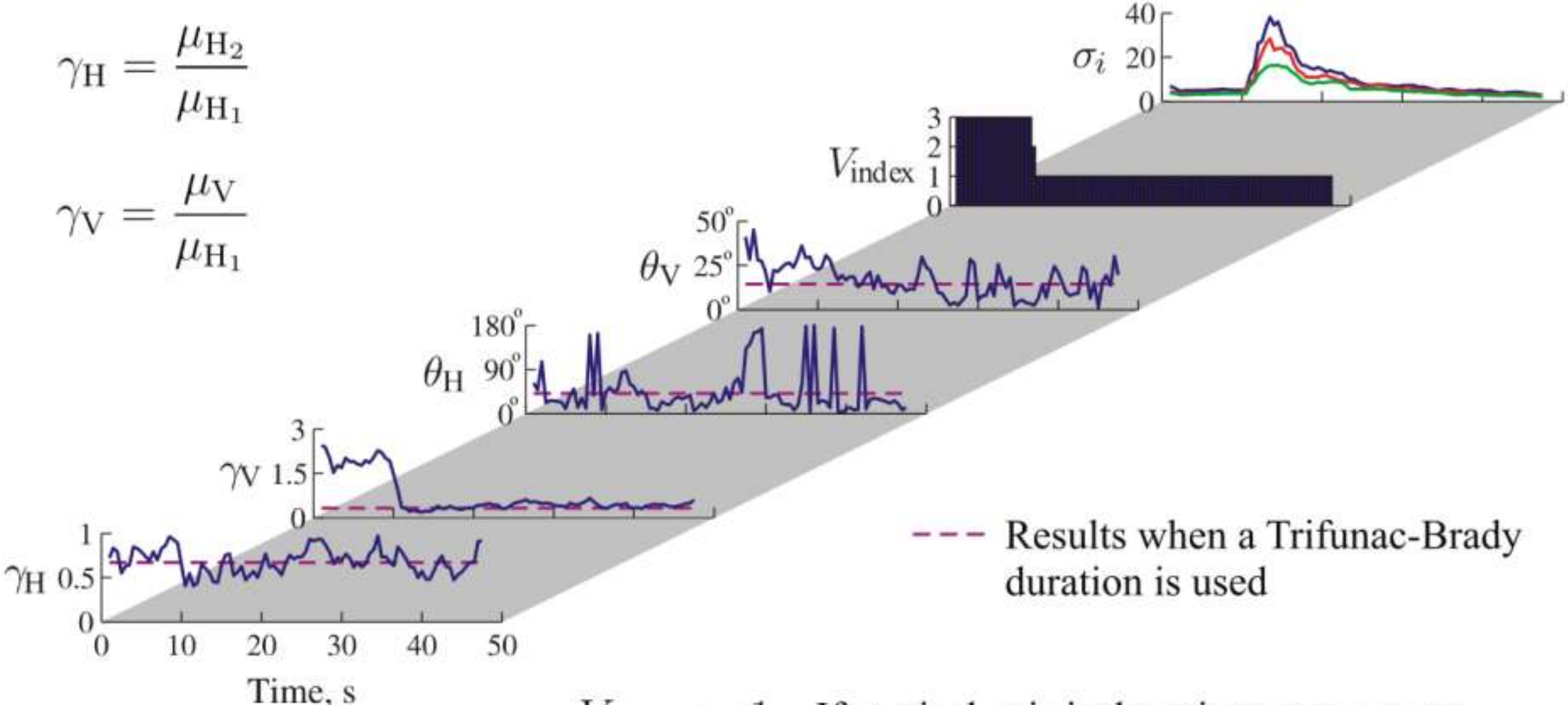
1′ and **2′**: Rotated major and minor horizontal principal axes

Time-dependent component correlations

- Example: 1988 Saguenay earthquake - Station St-Ferreol

$$\gamma_{\mathrm{H}} = \frac{\mu_{\mathrm{H}_2}}{\mu_{\mathrm{H}_1}}$$

$$\gamma_{\mathrm{V}} = \frac{\mu_{\mathrm{V}}}{\mu_{\mathrm{H}_1}}$$



$V_{\mathrm{index}} = 1$ If vertical axis is the minor component

$V_{\mathrm{index}} = 2$ If vertical axis is the intermediate component

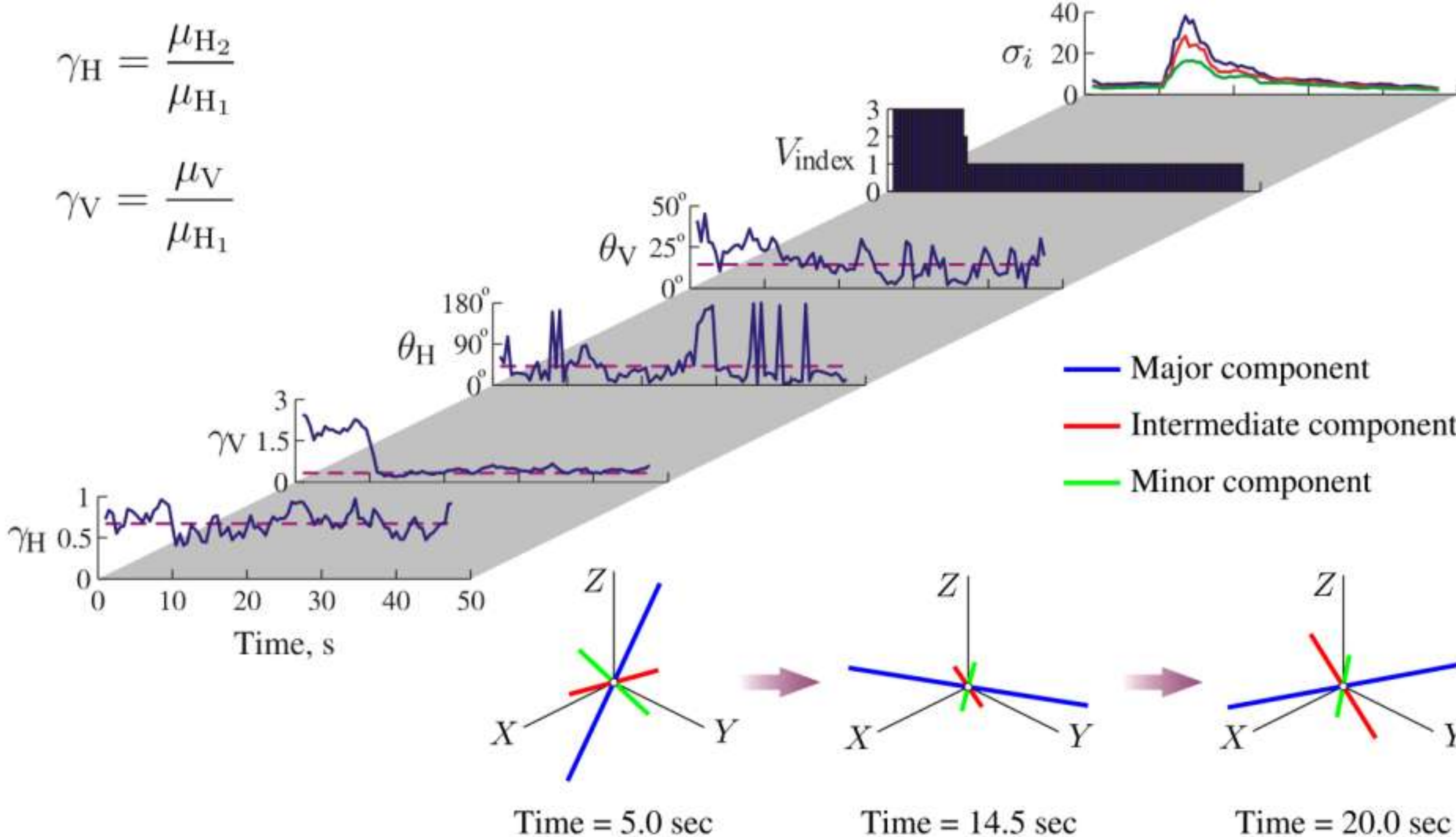
$V_{\mathrm{index}} = 3$ If vertical axis is the major component

Time-dependent component correlations

- Example: 1988 Saguenay earthquake - Station St-Ferreol

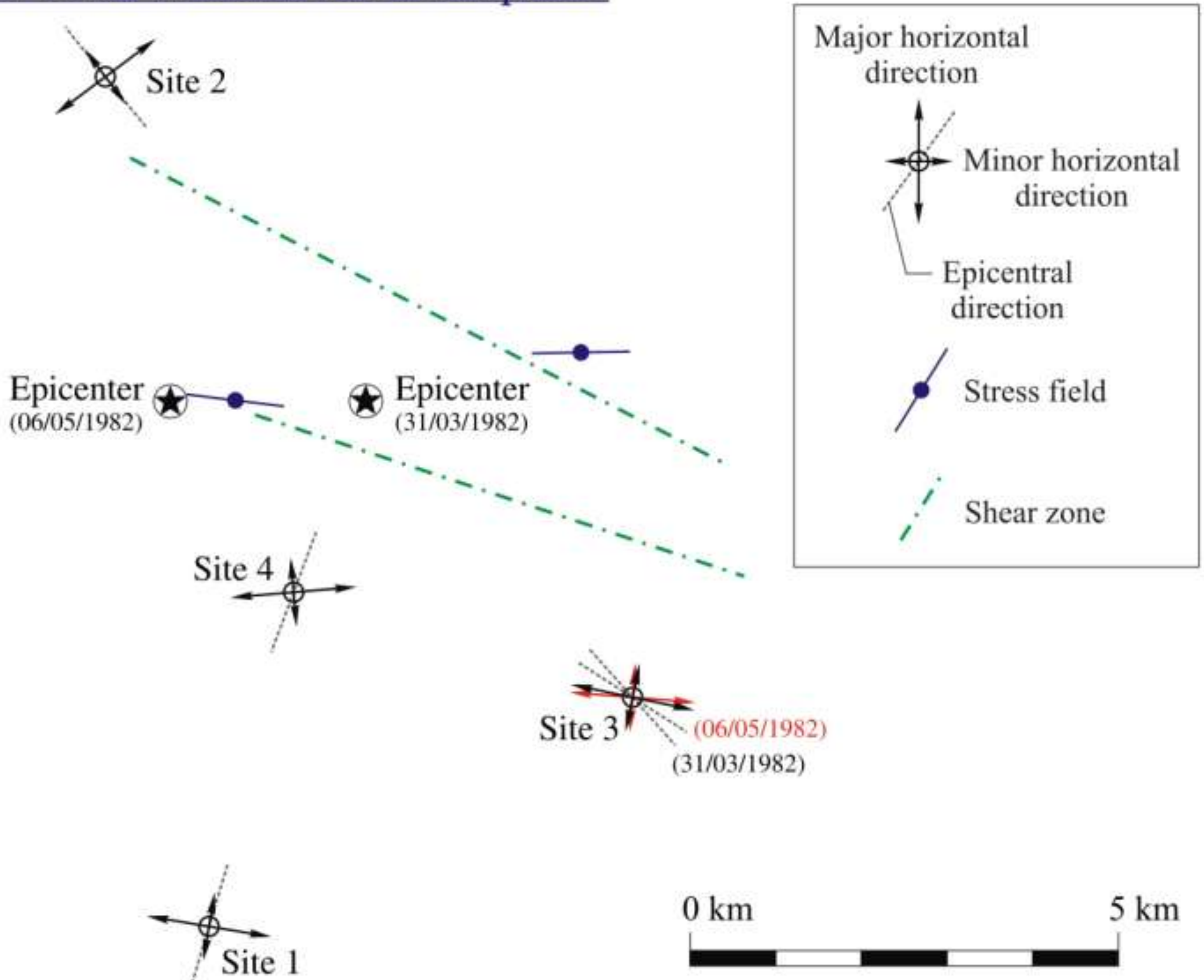
$$\gamma_H = \frac{\mu_{H_2}}{\mu_{H_1}}$$

$$\gamma_V = \frac{\mu_V}{\mu_{H_1}}$$



Correlation with geological features

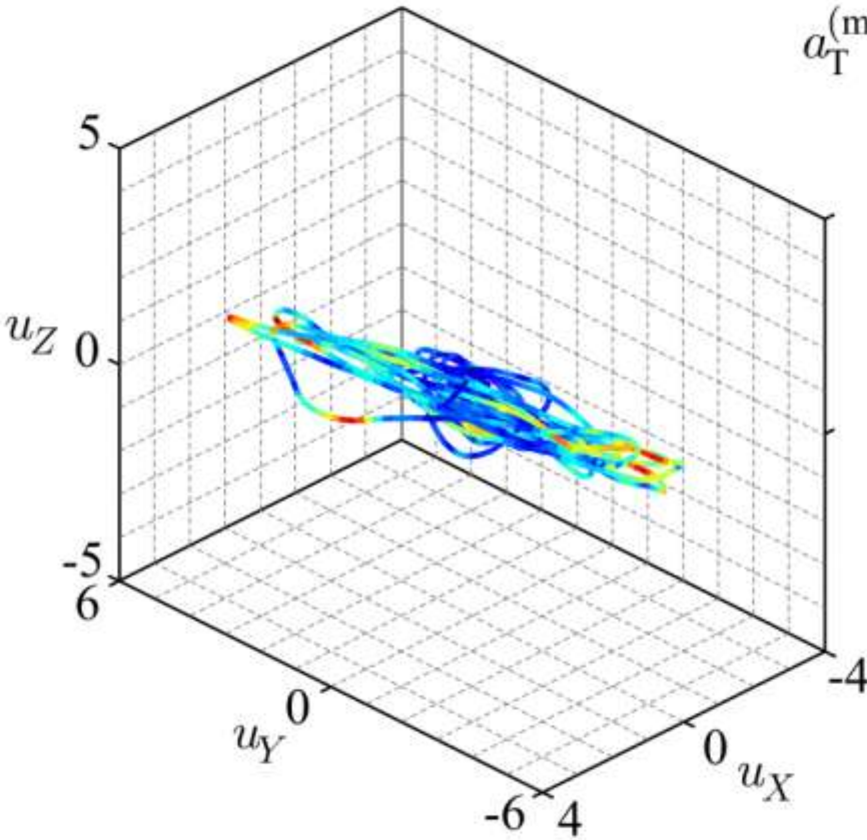
- Example: 1982 Miramichi earthquake



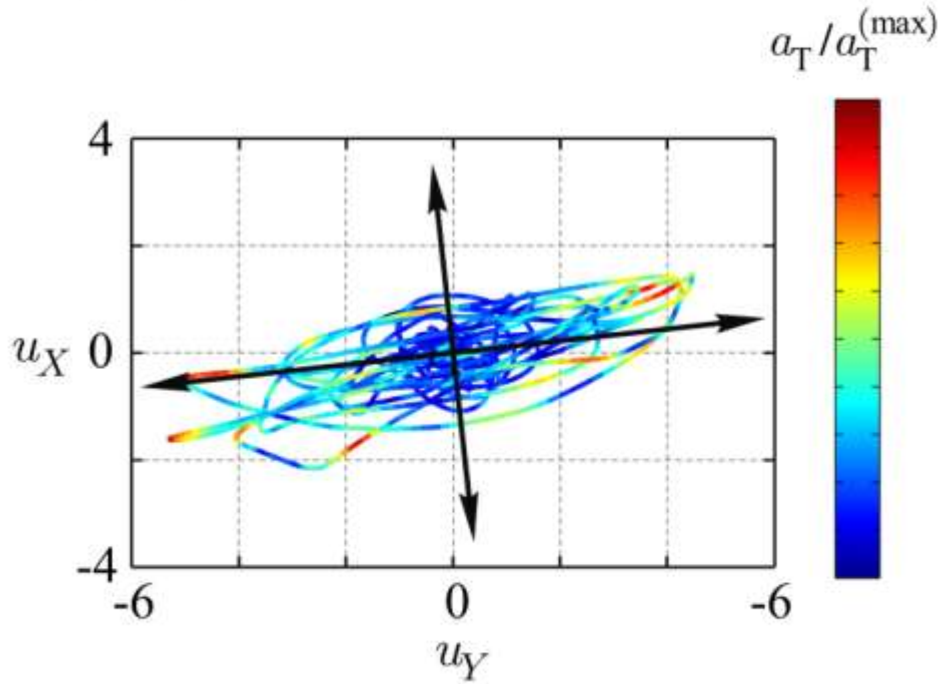
Correlation with displacement trajectories

- Example: 2010 Val des Bois earthquake - Station OTNM

$a_T^{(max)} = 0.114\,g$

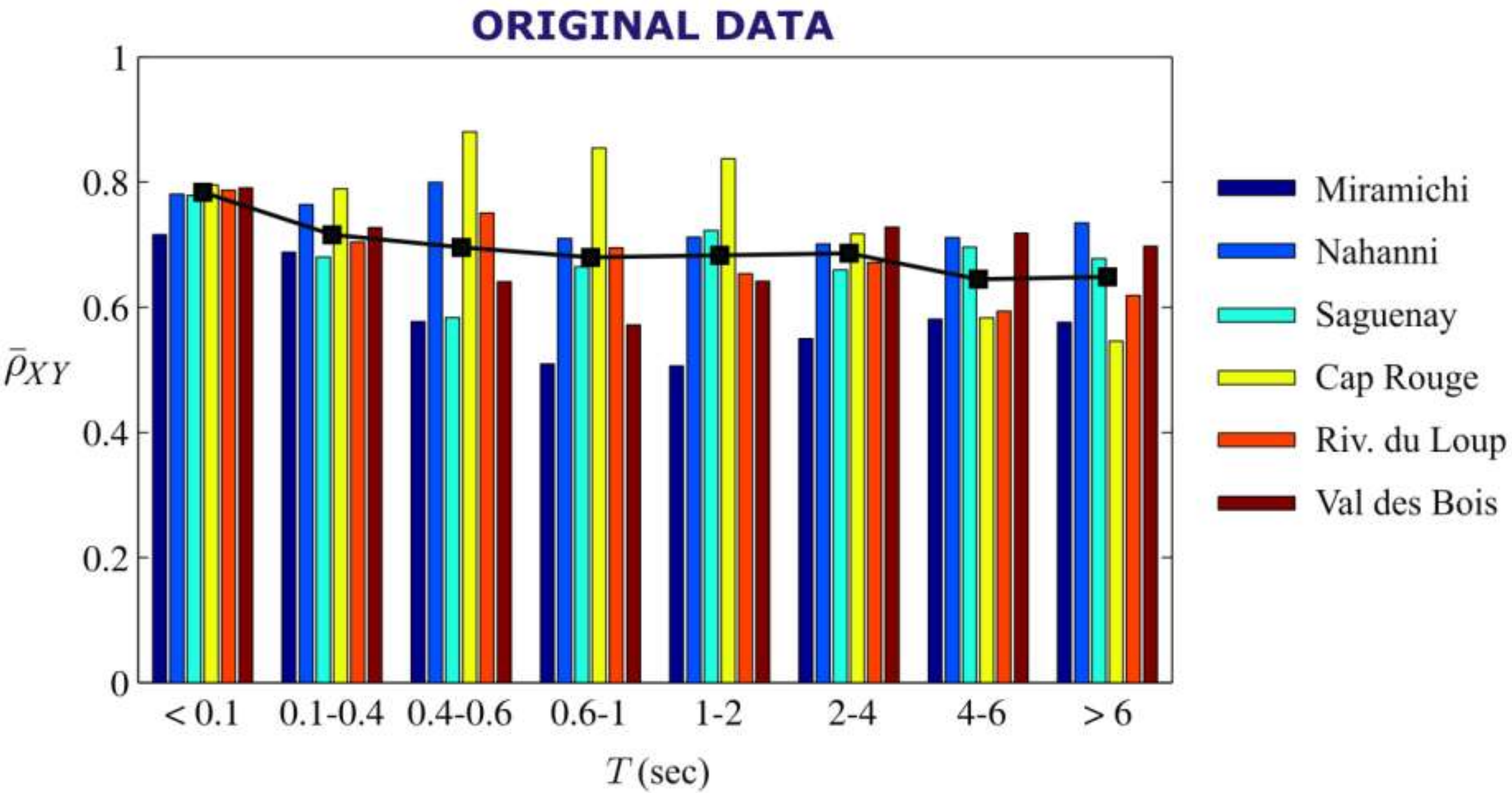


3D particle trajectory

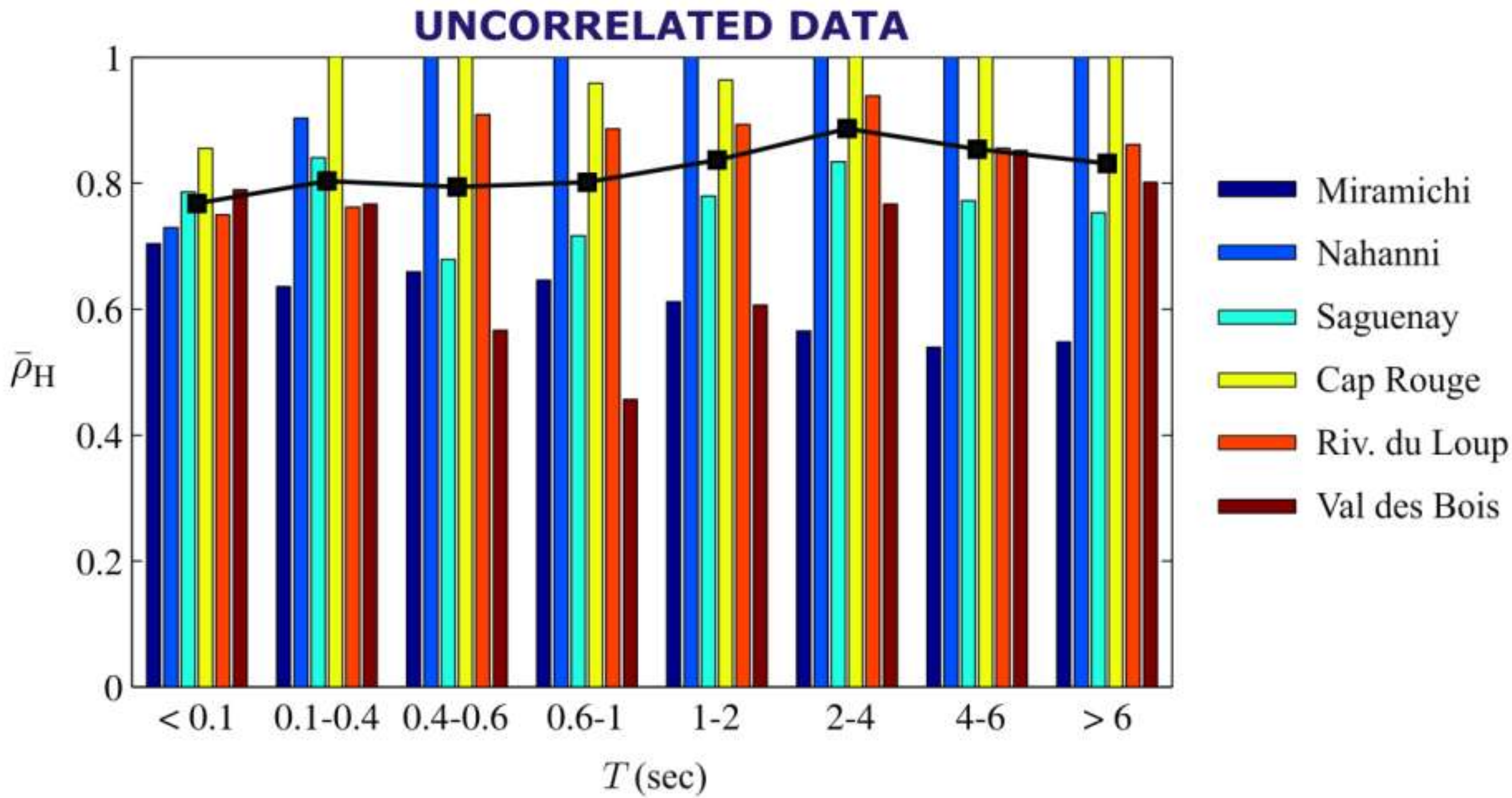


2D XY projection particle trajectory

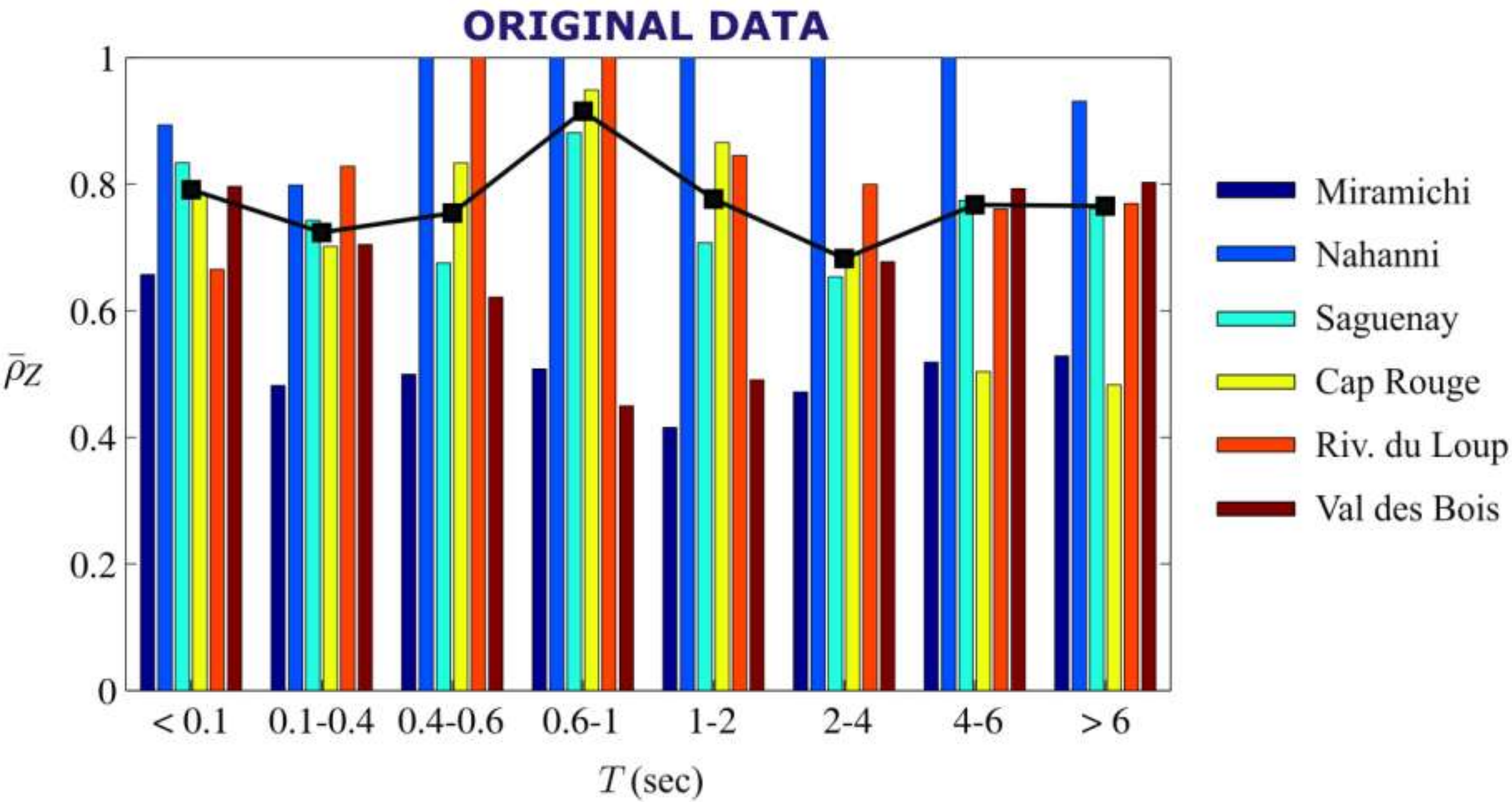
ENA GMs spectral mean values



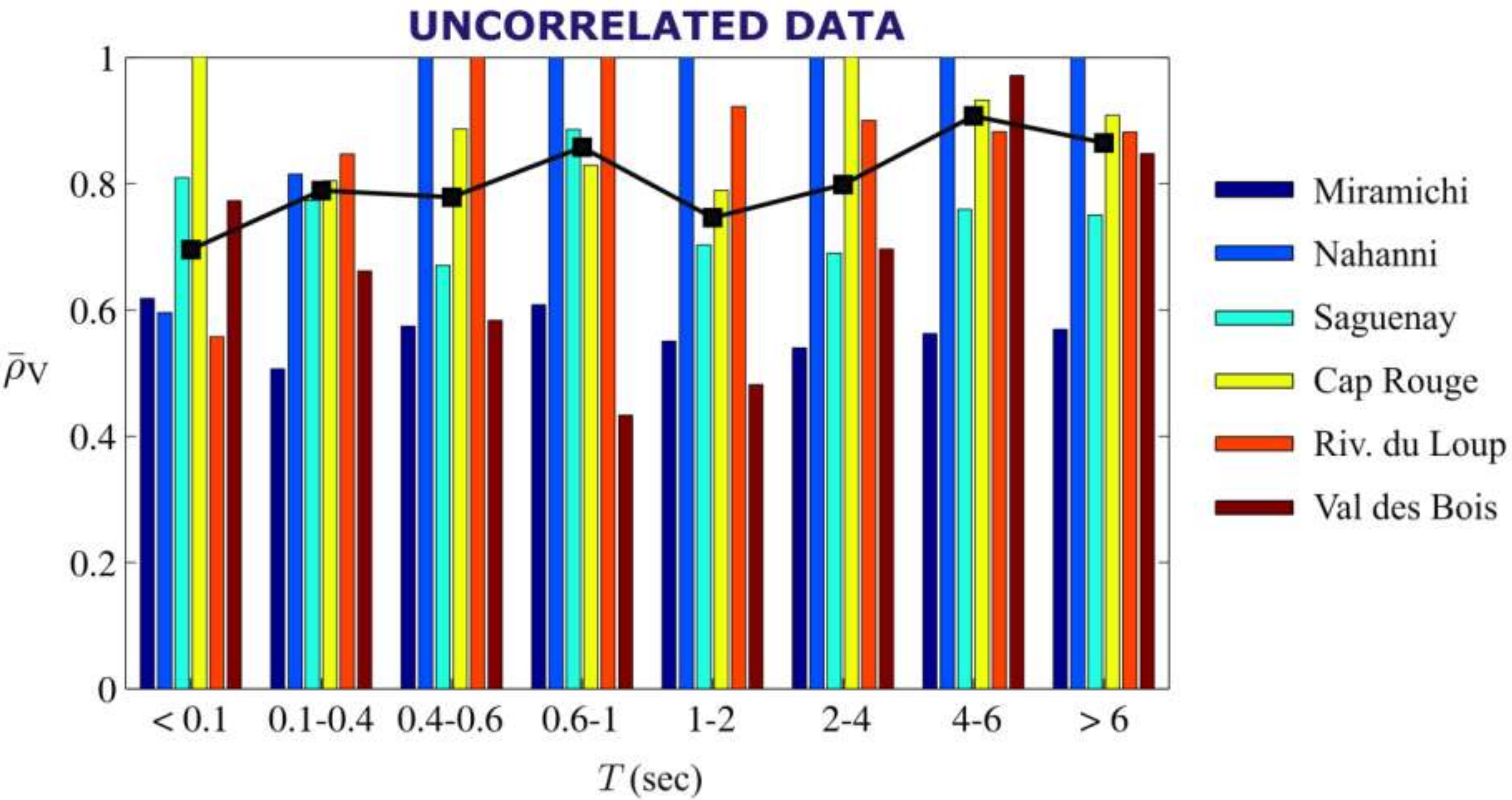
ENA GMs spectral mean values



ENA GMs spectral mean values



ENA GMs spectral mean values



- Conclusions
  - The trends of displacement demands in eastern and western Canada were identified using simulated and historical data
  - The differences between inelastic seismic demands become predominant in the longer period range, more important for seismic isolation
  - The inelastic spectral displacements in eastern Canada fail to obey to available reduction rules, either based on viscous damping or displacement ductility
- Work in progress
  - Refinement and simplification of the new ductility and damping relations, namely for eastern Canada
  - Implementation of the developed relations into simplified performance-based assessment of isolated/damped bridges in Canada

- Acknowledgements
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  - Ilona Bartosh, M.Sc. (Current)
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Thank you for your attention

Unified LRFD-based Analysis and Design Procedures for Bridge Bearings and Seismic Isolators

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SCOPE OF PRESENTATION

- Present a general description of project to develop unified LRFD-based procedures for bridge bearings and seismic isolators.
- Present some details of the approach followed in the development of LRFD-based analysis and design procedures for elastomeric isolators.
- Present some limited results in the design of sliding bearings/isolators and elastomeric bearings .
- Present summary results on effects of hysteretic and frictional heating on isolator behavior.

ACKNOWLEDGEMENTS

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- Pacific Earthquake Engineering Research Center (PEER) through a contract with the California Department of Transportation.
- MCEER, University at Buffalo, State University of New York through a contract with the Federal Highway Administration, Project 020, Title V.
- Professor A. Filiatrault and Dr. Y. Kalpakidis participated in the LRFD project.
- Reviewers for the LRFD project were Dr. T. Delis, Dr. A. Kartoum and Mr. T. Shantz on behalf of Caltrans and Professors J. Kelly and S. Mahin and Dr. F. Naeim on behalf of PEER.
- Dr. E. D. Wolff, Dr. D. M. Fenz and Dr. I. Kalpakidis participated in the work on hysteretic and frictional heating.

LRFD PROJECT OBJECTIVES

- Develop analysis and design specifications for bridge bearings and seismic isolators
  - Based on LRFD framework.
  - Based on the same fundamental principles, which include the latest developments and understanding of behavior.
  - Applied by the same principles regardless of whether the application is for seismic-isolated or conventional bridges.
  - Consider service, design earthquake and maximum earthquake effects.

REFERENCES

- "*Guide Specifications for Seismic Isolation Design*", AASHTO, 1999 and recent 2010 revision.
- "Design of Structures for Earthquake Resistance. Part 2: Bridges," Eurocode 8, EN1998-2, 2005.
- "*LRFD Bridge Design Specifications*", AASHTO, 4th Edition, 2007 and 2010, 5th Edition.
- "*Rotation Limits for Elastomeric Bearings*", Report 12-68, University of Washington, 2006 (published as report NCHRP 596, 2008).
- "*Performance of Seismic Isolation Hardware under Service and Seismic Loading*", Report MCEER-07-0012, 2007.
- "*Experimental Investigation on the Seismic Response of Bridge Bearings*", Univ. of California, Berkeley, EERC-2008-02, 2008.
- "LRFD-based Analysis and Design Procedures for Bridge Bearings and Seismic Isolators", Report MCEER-11-0004, 2011.

SUMMARY OF WORK

- Concentration on sliding and elastomeric bearings.
- Sliding bearings are either flat or spherically shaped. Key characteristic is that they have spherical rotational part (flat or Friction Pendulum). Pot bearings and disc bearings are not considered (and not typically used in California).
- Elastomeric bearings are rectangular, square, circular, hollow circular (central hole) and circular with one central core of lead. Shape factors are in the range of 5 to 30. Bearings are bolted or kept by keeper plates. Regular bridge bearings may be kept by friction. Only steel reinforced bearings are considered.

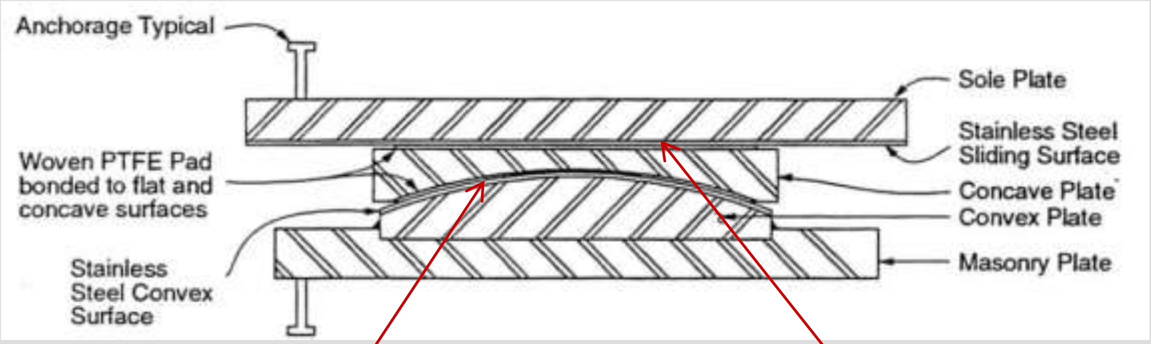
LOADINGS FOR ANALYSIS AND DESIGN OF SEISMICALLY ISOLATED BRIDGES

- Service loadings per AASHTO LRFD Specifications, 2010.

- Design earthquake (DE) per AASHTO LRFD Specifications, 2010 (probabilistic response spectrum having 7% probability of being exceeded in 75 years-return period ~1000years)
  - Design earthquake in California based on Caltrans ARS Website. Defined as the largest of (a) probabilistic response spectrum having 5% probability of being exceeded in 50 years-return period 1000years, and (b) deterministic median spectrum calculated based on the NGA project of PEER.

- Maximum earthquake not explicitly defined.
  - For isolators, the effects of maximum earthquake defined as those of the DE multiplied by a factor. For California the factor on isolator displacement is 1.5. The factor for force to be determined by analysis-range of 1.0 to 1.5. Use 1.5 default value.
  - For elastomeric bridge bearings, the effects of maximum earthquake are not considered. Bearings may overturn but sufficient bearing seat width is provided (1.5 times the DE displacement).
  - For spherical sliding bridge bearings, the maximum earthquake effects are defined as those of the DE multiplied by factor 1.5.

SLIDING BEARINGS

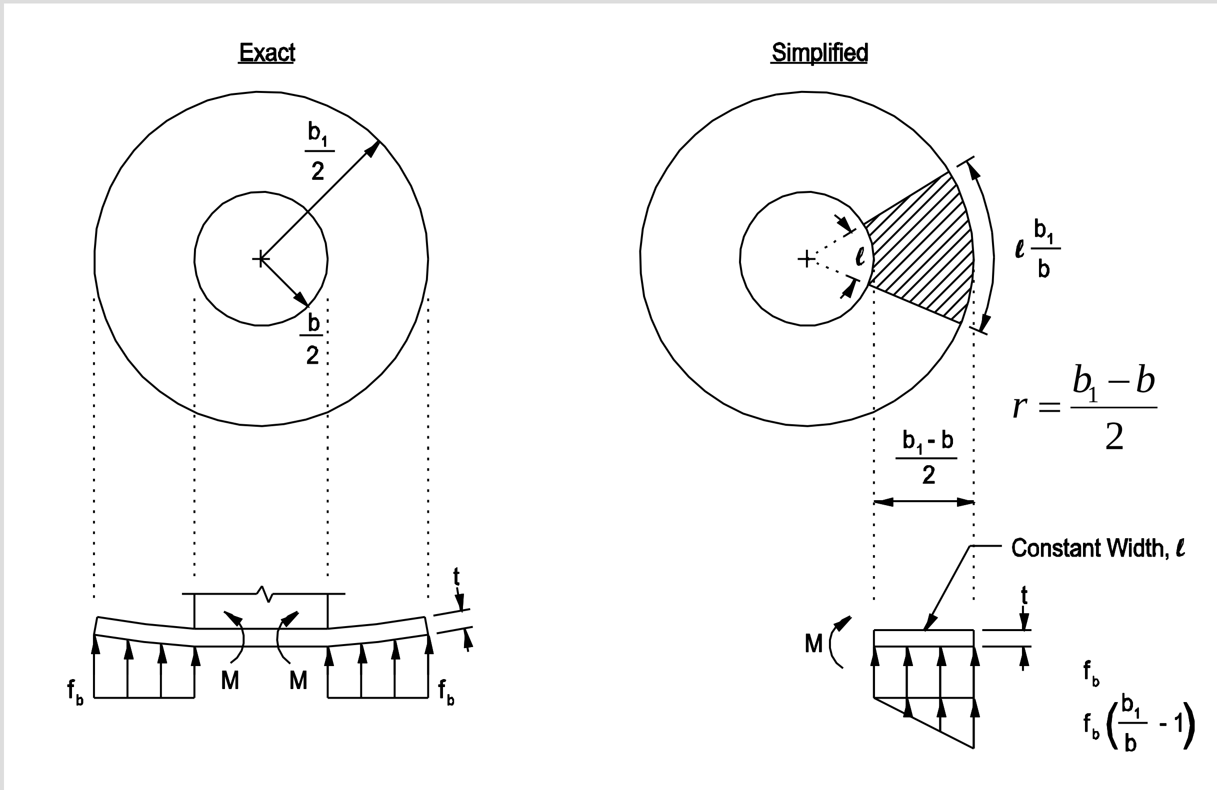


SPHERICAL SLIDING SURFACE

FLAT SLIDING SURFACE

SLIDING BEARINGS

- **Design of End Plates of Sliding Bearings**

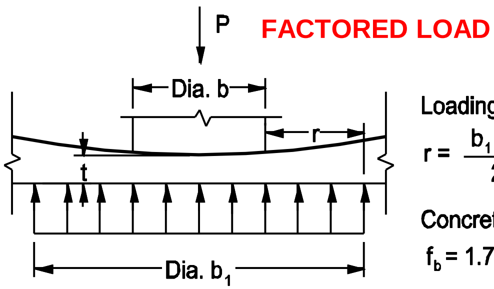


$$M_{u,e} = f_b b_1^2 \left(\frac{(1+\nu)\ln\frac{b_1}{b} - \frac{1+3\nu}{4} + \frac{1-\nu}{4}\left(\frac{b}{b_1}\right)^4 + \nu\left(\frac{b}{b_1}\right)^2}{8(1+\nu)+8(1-\nu)\left(\frac{b}{b_1}\right)^2} \right)$$

$$M_{u,s} = \left[f_b \frac{r^2}{2} + f_b \left(\frac{b_1}{b} - 1\right)\frac{r^2}{3} \right] \cdot factor$$

SLIDING BEARINGS

- Design of end plates using plastic analysis



Loading Arm

$r = \frac{b_1 - b}{2}$

Concrete Bearing Strength

$f_b = 1.7\phi_c f'_c$

$b_1 = \sqrt{\frac{4P}{\pi f_b}}$

$$\underbrace{W_i = 2\pi M_p\left(1+\frac{b}{b_1-b}\right)\left(\frac{b_1-b}{b_1}\right)}_{\text{INTERNAL WORK ALONG RADIAL LINES}} + \underbrace{\pi M_p b\cdot\frac{1}{\frac{b_1-b}{2}}}_{\text{INTERNAL WORK ON PERIMETER}} = 2\pi M_p\frac{b_1}{b_1-b}$$

INTERNAL WORK ALONG RADIAL LINES

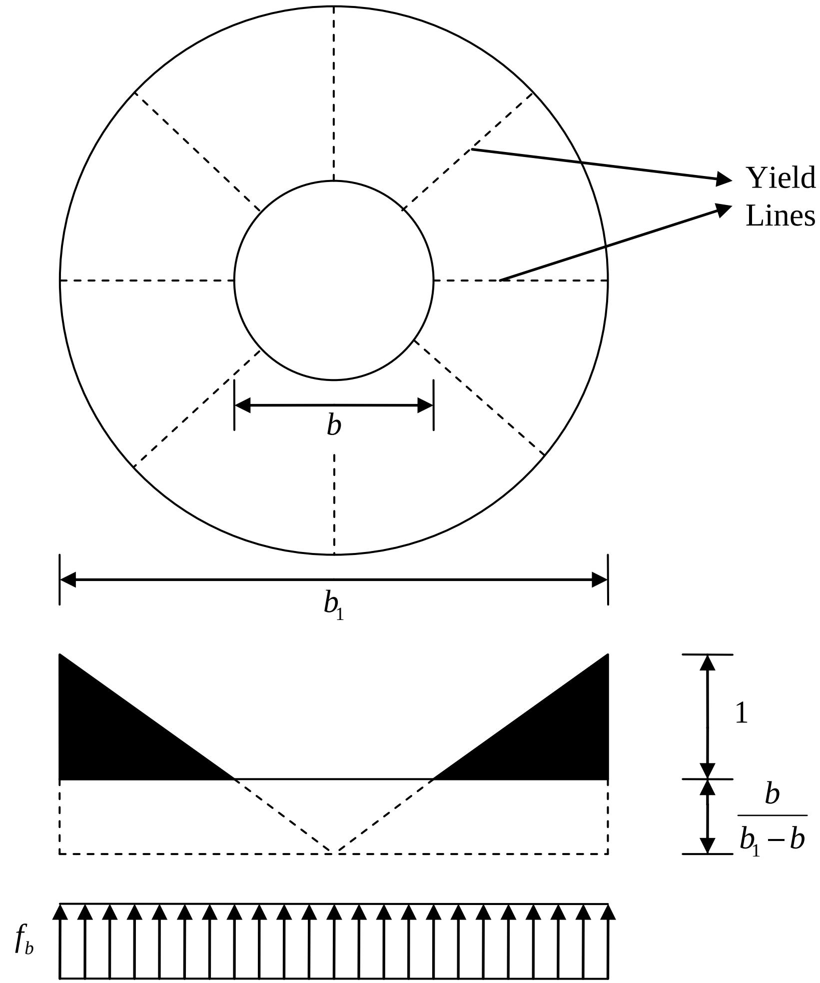
INTERNAL WORK ON PERIMETER

EXTERNAL WORK

$$W_e = f_b\left(\frac{\pi b_1^2}{4}\right)\cdot\left(\frac{b_1}{b_1-b}\right)\left(\frac{2}{3}-\left(\frac{b}{b_1}\right)+\frac{1}{3}\left(\frac{b}{b_1}\right)^3\right)$$

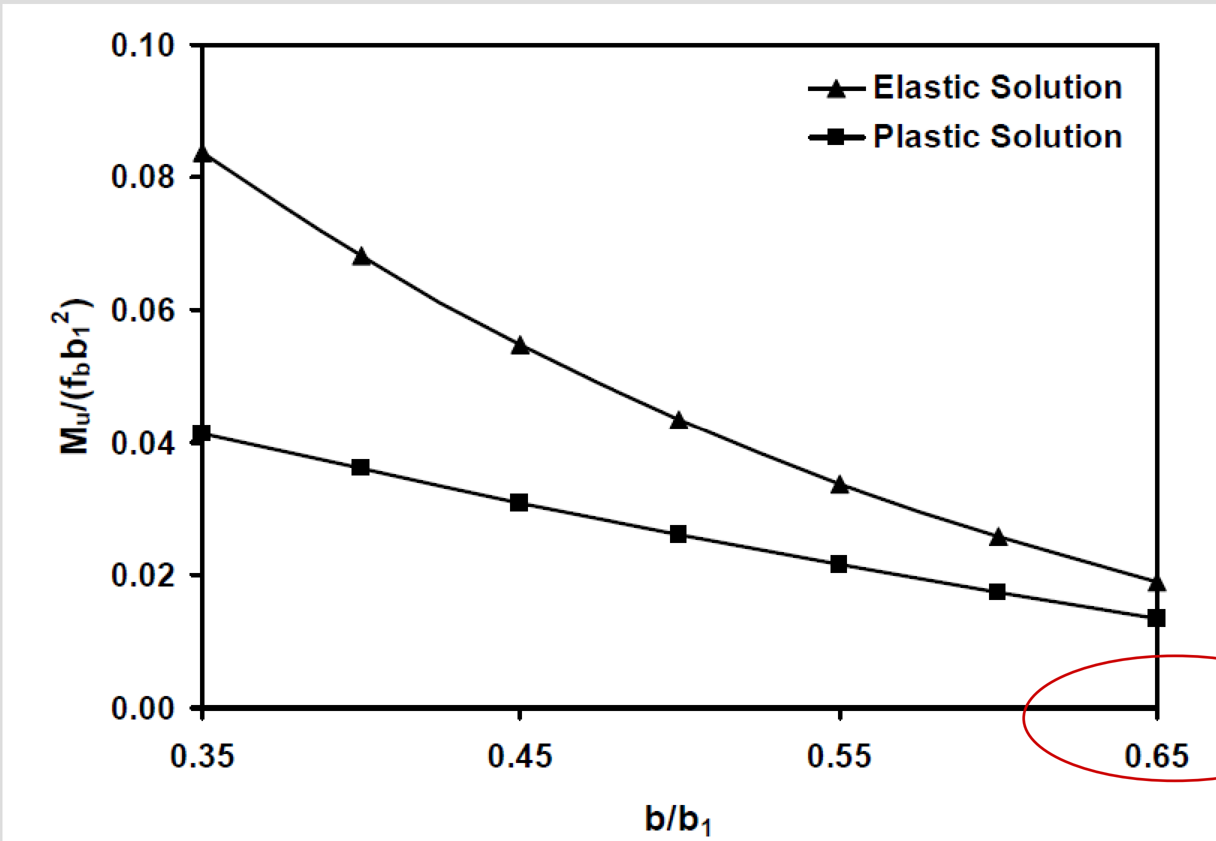
$$f_b = \frac{M_p}{b_1^2\left\{\left(\frac{1}{12}\right)-\left(\frac{1}{8}\right)\left(\frac{b}{b_1}\right)+\left(\frac{1}{24}\right)\left(\frac{b}{b_1}\right)^3\right\}}$$

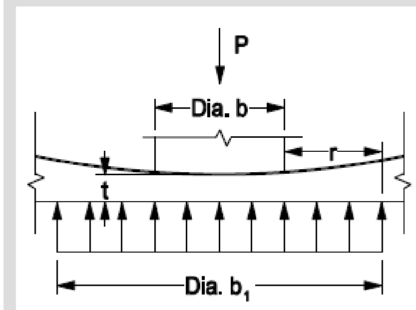
VALUE OF PRESSURE TO CAUSE COLLAPSE OF THE END PLATE



SLIDING BEARINGS

- Prediction of ultimate moment by plastic and elastic solutions
- Elastic solution is conservative and preferred





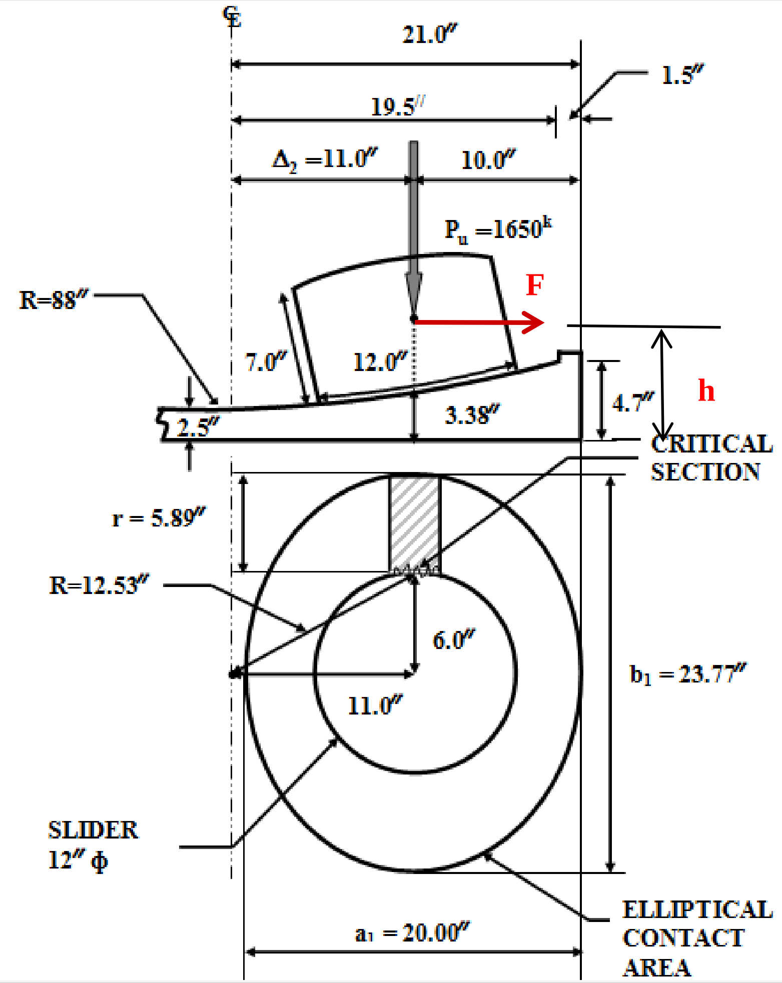
TYPICAL RANGE OF VALUES

SLIDING BEARINGS

- Design example using centrally loaded area approach
- Case where concrete to steel contact area cannot be circular

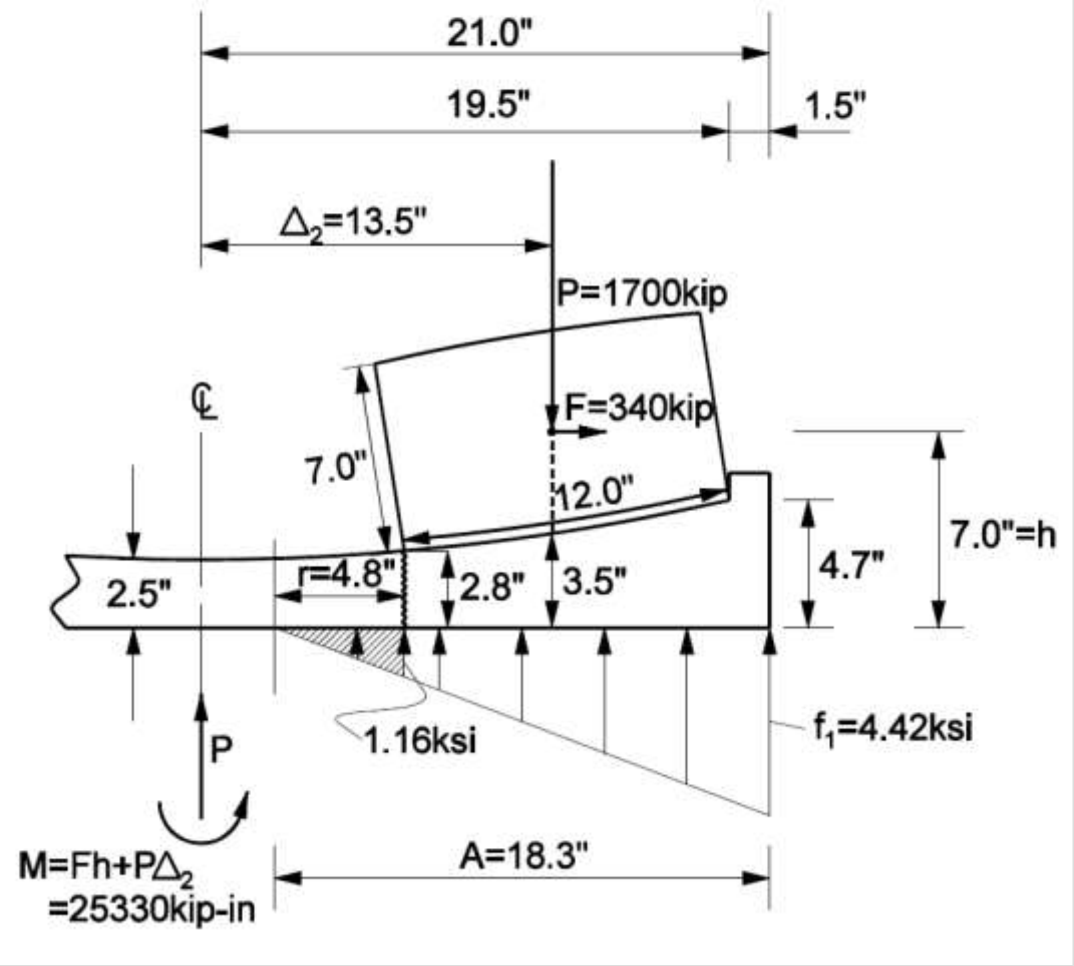
APPROACH IN WHICH THE AXIAL LOAD IS ASSUMED CONCENTRICALLY TRANSFERRED AT THE LOCATION OF THE SLIDER

MOMENT F.h IS NEGLECTED



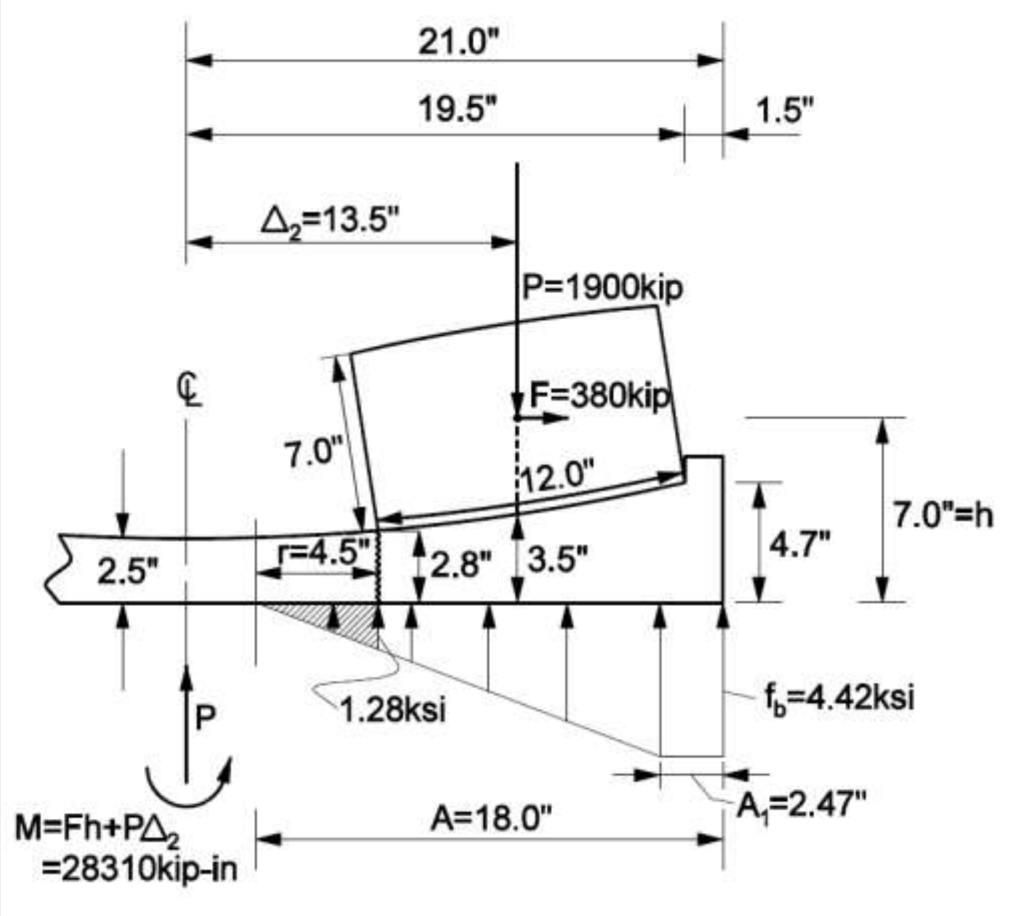
SLIDING BEARINGS

- Design example using load-moment approach
- Case where concrete to steel contact pressure is linear

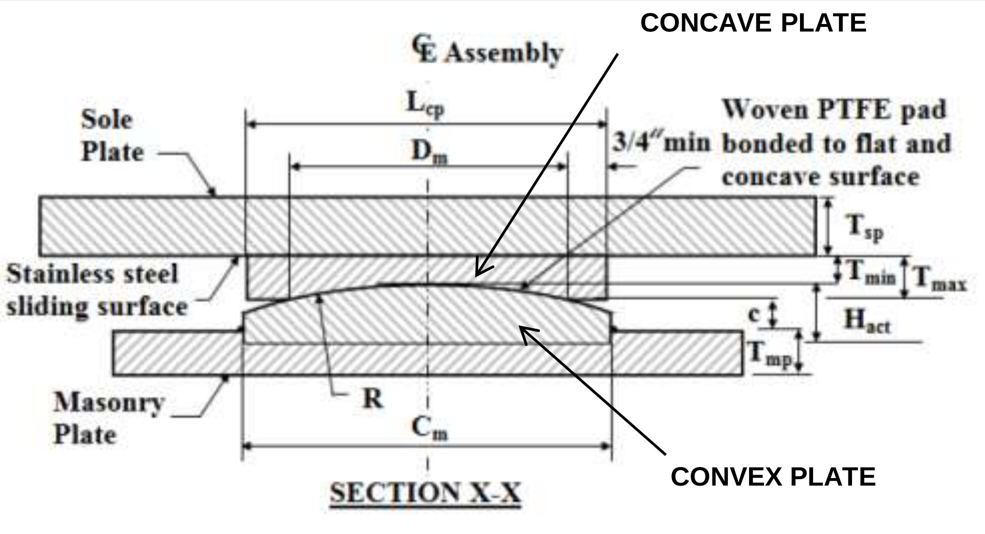


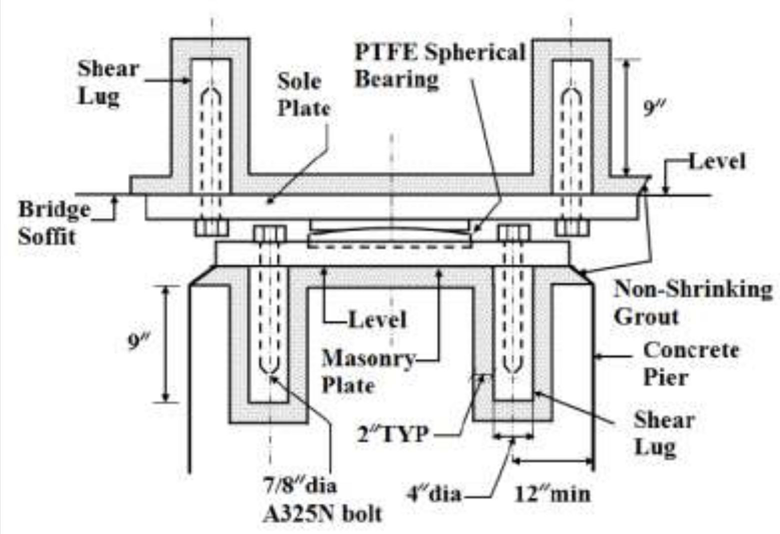
SLIDING BEARINGS

- Design example using load-moment approach
- Case where concrete to steel contact pressure is nonlinear-larger axial load



SLIDING BEARINGS



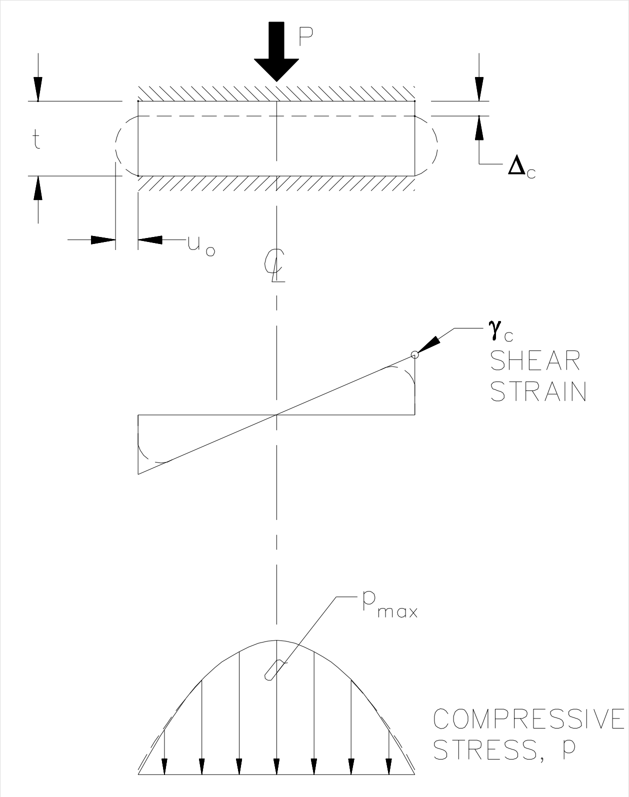


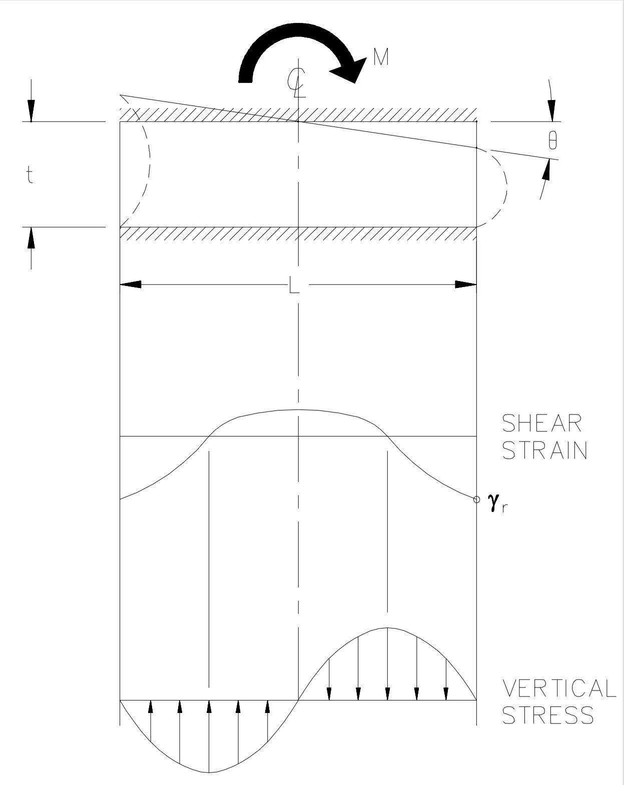
Convex			Concave						Sole		Masonry		
C_m	H_{act}	c	R	D_m	T_{max}	T_{min}	L_{cp}	L_{sp}	T_{sp}	W_{sp}	T_{mp}	L_{mp}	W_{mp}
12.50	2.00	0.56	18.00	11.00	1.75	0.75	11.75	44.75	1.50	22.75	1.00	20.50	20.50

PTFE Square Side B	Stainless Steel Plate Length L_{SS}	Stainless Steel Plate Width W_{SS}
9.50	30.50	18.50

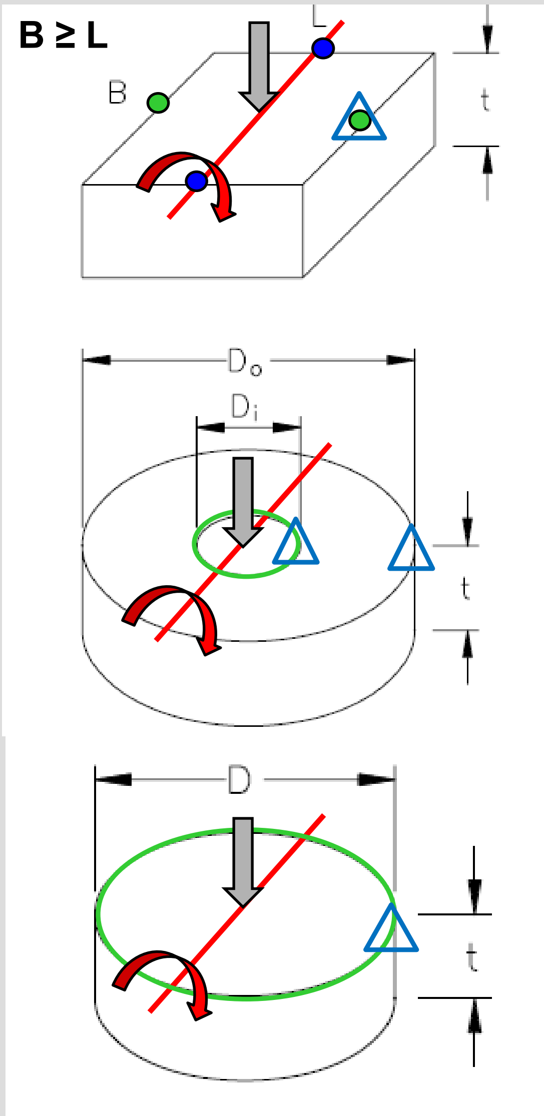
ELASTOMERIC BEARINGS

- Developed and verified simplified expressions for strains in rubber due to compression and rotation





ELASTOMERIC BEARINGS



$$S = \frac{BL}{2(B+L)t}$$

$$S = \frac{D}{4t}$$

$$S = \frac{D_o - D_i}{4t}$$

SHAPE FACTOR

△ **LOCATION OF LARGE SHEAR STRAIN DUE TO ROTATION**

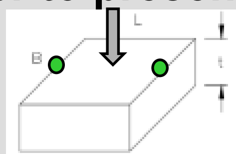
LOCATION OF MAXIMUM SHEAR STRAIN DUE TO COMPRESSION

ADDITIONAL LOCATION OF MAXIMUM SHEAR STRAIN DUE TO COMPRESSION IN SQUARE BEARING

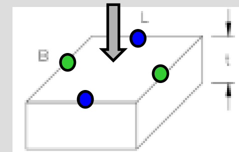
ELASTOMERIC BEARINGS

- **Compression of Elastomeric Bearings**
  - Rectangular bearings. Complex solutions (infinite series).
  - Prefer to present results in graphical or tabular form.

STRIP



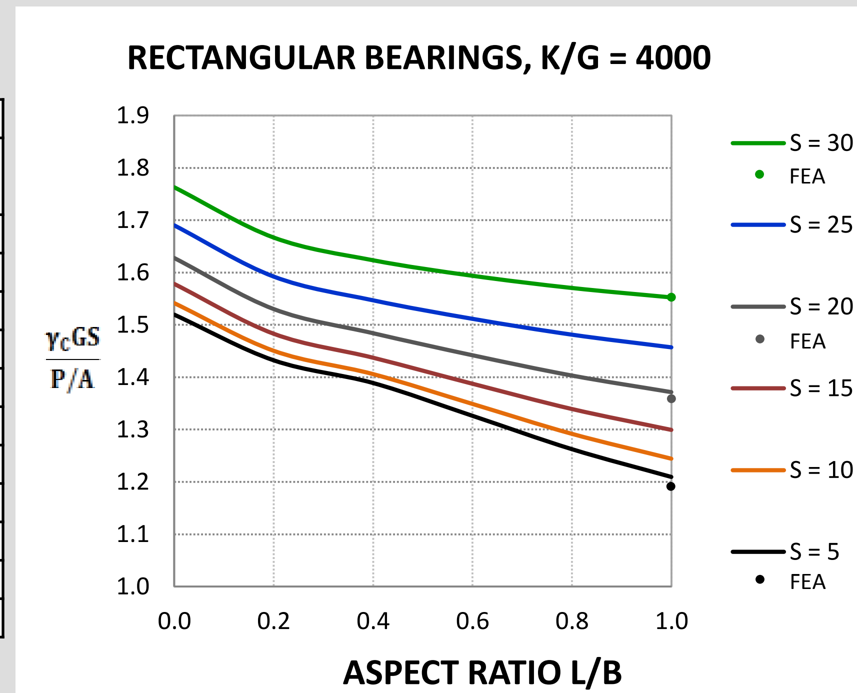
SQUARE



$$\gamma_c = \frac{P}{AGS} \cdot f_1$$

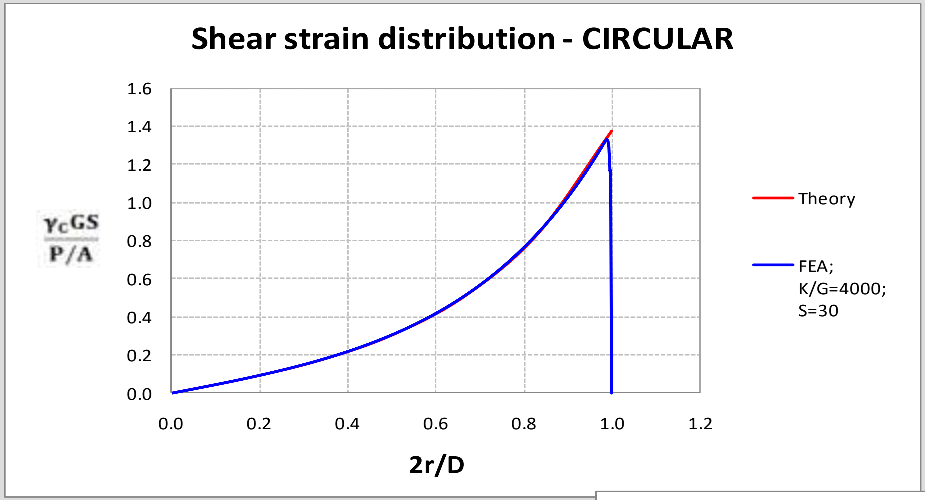
$FACTOR\_f_1$

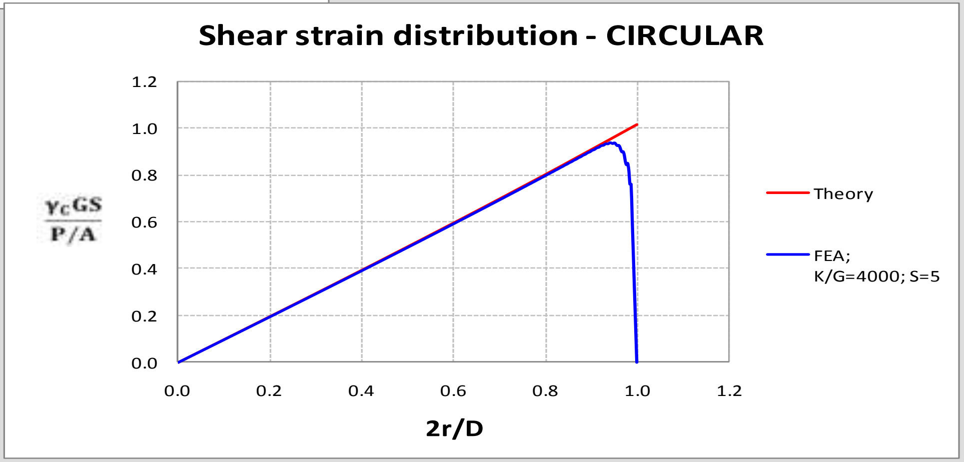
L/B S	0	0.2	0.4	0.6	0.8	1
K/G = 4000						
5	1.52	1.43	1.39	1.33	1.26	1.21
7.5	1.53	1.44	1.40	1.34	1.27	1.22
10	1.54	1.45	1.41	1.35	1.29	1.24
12.5	1.56	1.47	1.42	1.37	1.31	1.27
15	1.58	1.48	1.44	1.39	1.34	1.30
17.5	1.60	1.50	1.46	1.41	1.37	1.33
20	1.63	1.53	1.48	1.44	1.40	1.37
22.5	1.66	1.56	1.51	1.48	1.44	1.41
25	1.69	1.59	1.55	1.51	1.48	1.46
27.5	1.72	1.63	1.58	1.55	1.52	1.50
30	1.76	1.67	1.62	1.59	1.57	1.55



ELASTOMERIC BEARINGS

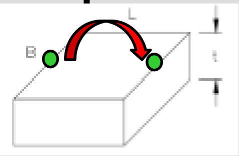
- Compression of Circular Elastomeric Bearings
  - Example of results of Finite Element Analysis





ELASTOMERIC BEARINGS

- Rotation of Elastomeric Bearings
  - Rectangular bearings. Complex solutions (infinite series)
  - Prefer to present results in graphical or tabular form



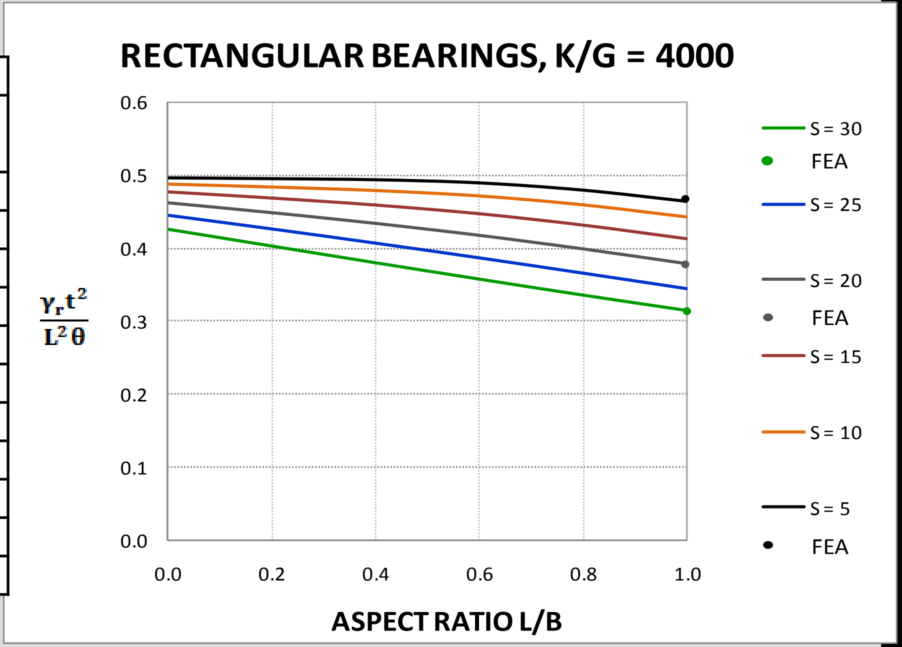
$$\gamma_r = \frac{L^2\theta}{tT_r}\cdot f_2$$

STRIP

SQUARE

$FACTOR\_f_2$

K/G = 4000						
L/B S	0	0.2	0.4	0.6	0.8	1
5	0.50	0.49	0.49	0.49	0.48	0.46
7.5	0.49	0.49	0.49	0.48	0.47	0.45
10	0.49	0.48	0.48	0.47	0.46	0.44
12.5	0.48	0.48	0.47	0.46	0.45	0.43
15	0.48	0.47	0.46	0.45	0.43	0.41
17.5	0.47	0.46	0.45	0.43	0.42	0.40
20	0.46	0.45	0.43	0.42	0.40	0.38
22.5	0.45	0.44	0.42	0.40	0.38	0.36
25	0.45	0.43	0.41	0.39	0.37	0.35
27.5	0.44	0.42	0.39	0.37	0.35	0.33
30	0.43	0.40	0.38	0.36	0.34	0.31



LRFD FORMULATION FOR ELASTOMERIC ISOLATORS

- **SERVICE LOAD CHECKING**

Dead load: P_D

Live load: P_{Lst} (static component), P_{Lcy} (cyclic component)

Factored axial load: $P_u = \gamma_D P_D + \gamma_L P_{Lst} + 1.75\gamma_L P_{Lcy}$

Load factors γ_D and γ_L are given by the appropriate code or guideline

Non-seismic lateral displacement: Δ_{Sst} (static), Δ_{Scy} (cyclic)

Non-seismic bearing rotation: θ_{Sst} (static), θ_{Scy} (cyclic)

SHEAR STRAIN DUE TO COMPRESSION

$$\gamma_{Cs}^{u} = \frac{P_u}{A_r G S} \cdot f_1$$

SHEAR STRAIN DUE TO ROTATION

L=dimension L for rectangular bearings (B>L)

L=D for circular ; L=Do for hollow circular bearings

$$\gamma_{r_s}^{u} = \frac{L^2(\theta_{Sst} + 1.75\theta_{Scy})}{tT_r} \cdot f_2$$

SHEAR STRAIN DUE TO LATERAL DISPLACEMENT

$$\gamma_{S_s}^{u} = \frac{\Delta_{Sst} + 1.75\Delta_{Scy}}{T_r}$$

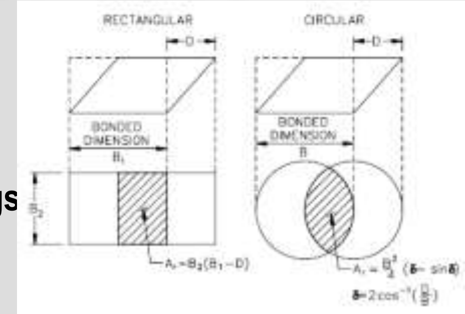
CRITERIA

$$\frac{\gamma_D P_D + \gamma_L P_{Lst}}{A_r G S} \cdot f_1 \le 3.5 \qquad \gamma_{C_s}^{u} + \gamma_{S_s}^{u} + \gamma_{r_s}^{u} \le 6$$

AASHTO 2010 HAS LIMITS OF 3.0 AND 5.0, RESPECTIVELLY. DIFFERENCE JUSTIFIED ON BASIS OF HIGHER QUALITY OF CONSTRUCTION AND PROTOTYPE AND PRODUCTION TESTING . EVEN HIGHER LIMITS COULD BE JUSTIFIED AS AASHTO IS NOT TRULY LRFD

LRFD FORMULATION FOR ELASTOMERIC ISOLATORS

- **SERVICE LOAD CHECKING CRITERIA CONTINUED**



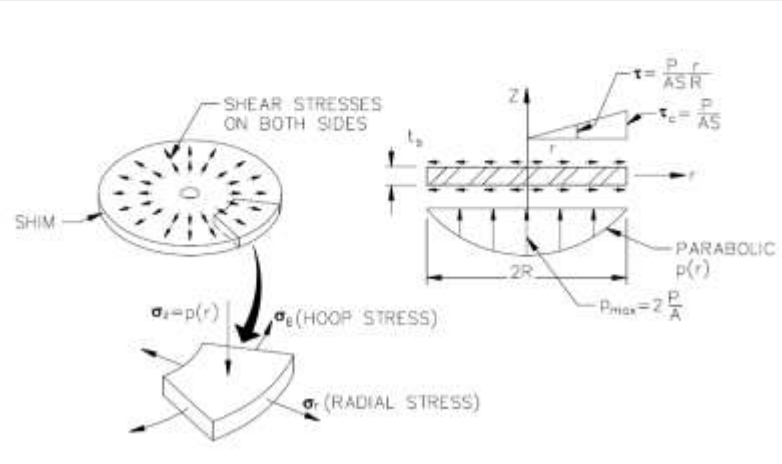
BUCKLING $$\frac{P'_{cr_s}}{\gamma_D P_D + \gamma_L (P_{Lst} + P_{Lcy})} \geq 2.0 \qquad P'_{cr_s} = P_{cr} \frac{A_{rs}}{A}$$ **for bolted bearings**

EQUIVALENT TO USING A Φ FACTOR OF 0.5

A_{rs} **reduced bonded area for displacement** $\Delta_{Sst} + \Delta_{Scy}$

SHIMS $$t_s \geq \frac{\alpha t}{1.08 F_y \frac{A_{rs}}{P_u} - 2} \geq 1.9 \text{ mm}$$

α=1.65 for shims without holes, α= 3 otherwise

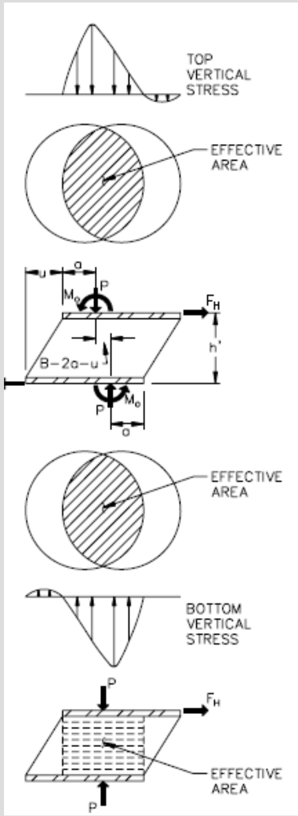


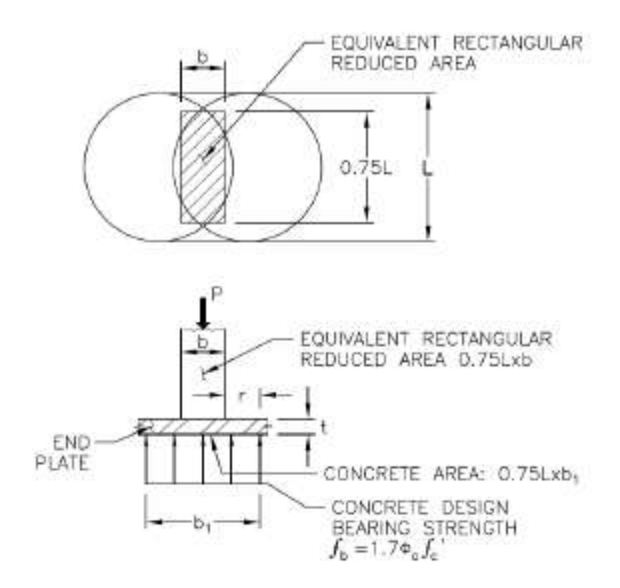
LRFD FORMULATION FOR ELASTOMERIC ISOLATORS

- SERVICE LOAD CHECKING CRITERIA CONTINUED

END PLATES

PROCEDURE UTILIZES MINIMUM MATERIAL STRENGTHS AND APPROPRIATE Φ FACTORS





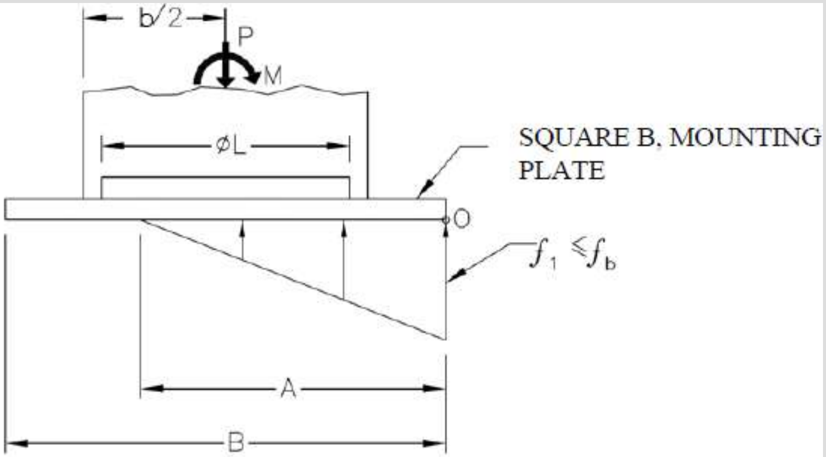
REDUCED AREA PROCEDURE

(End plate treated as column base plate subjected to concentric load from above over equivalent reduced area)

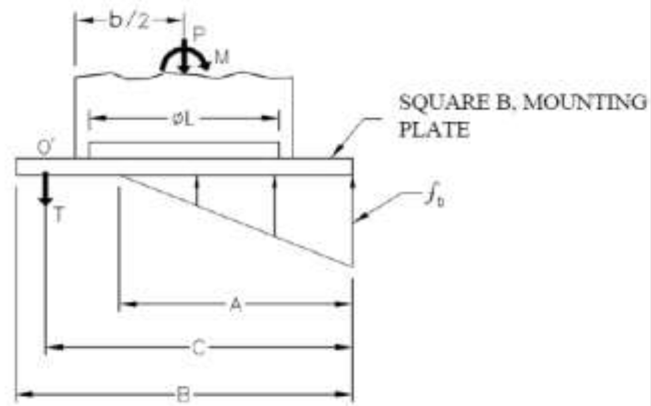
LRFD FORMULATION FOR ELASTOMERIC ISOLATORS

- **SERVICE LOAD CHECKING CRITERIA CONTINUED**

END PLATES **PROCEDURE UTILIZES MINIMUM MATERIAL STRENGTHS AND APPROPRIATE φ FACTORS**



$$\left.\begin{aligned} &P - B\cdot A\cdot f_1/2 = 0 \\ &M - \frac{PB}{2} + \frac{PA}{3} = 0 \end{aligned}\right\} \quad \begin{aligned} &A = \frac{3}{2}B - 3\frac{M}{P} \\ &f_1 = \frac{2P}{AB} \leq f_b \end{aligned}$$



$$\left.\begin{aligned} &P + T - f_b\frac{A\cdot B}{2} = 0 \\ &f_b\frac{A\cdot B}{2}\left(C - \frac{A}{3}\right) - M - P\left(C - \frac{B}{2}\right) = 0 \end{aligned}\right\}$$

$$A^2\left(\frac{Bf_b}{6}\right) - A\left(\frac{f_b B\cdot C}{2}\right) + \left(M - PC - \frac{PB}{2}\right) = 0$$

$$T = \frac{f_b AB}{2} - P$$

WITHOUT BOLT TENSION **WITH BOLT TENSION**

LOAD-MOMENT PROCEDURE

(End plate treated as column base plate subjected to axial force and moment)

LRFD FORMULATION FOR ELASTOMERIC ISOLATORS

- **DESIGN EARTHQUAKE CHECKING (5% in 50 years or 1000 year return period)**

Dead load: P_D

Seismic live load (considered static component): P_{SL}

Earthquake axial load due to DE shaking: $P_{E_{DE}}$, where earthquake-induced axial loads can result from both overturning moments in the superstructure and vertical earthquake shaking

Factored axial load: $P_u = \gamma_D P_D + \gamma_L P_{SL} + P_{E_{DE}}$

Load factors γ_D and γ_L are given by the appropriate code or guideline

Non-seismic bearing rotation: θ_{Sst} (static), θ_{Scy} (cyclic)

Seismic lateral displacement: $\Delta_{E_{DE}}$.

Non-seismic lateral displacement: $\gamma\Delta_S = \gamma(\Delta_{Sst} + \Delta_{Scy})$

AASHTO GUIDE SPECIFICATIONS FOR SEISMIC ISOLATION DESIGN (1999/2010) HAVE LIMIT OF 5.5 FOR UNFACTORED LOADS AND WITHOUT CONSIDERATION OF NON-SEISMIC DISPLACEMENTS. CONVERTING TO FACTORED LOADS, THE LIMIT SHOULD BE ABOUT 7.0.

SHEAR STRAIN DUE TO COMPRESSION

$$\gamma^u_{C_{DE}} = \frac{P_u}{A_r G S} \cdot f_1$$

SHEAR STRAIN DUE TO ROTATION

$$\gamma^u_{r_s} = \frac{L^2(\theta_{Sst} + 1.75\theta_{Scy})}{tT_r} \cdot f_2$$

SHEAR STRAIN DUE TO LATERAL DISPLACEMENT

$$\gamma^u_{S_{DE}} = \frac{\gamma\Delta_S + \Delta_{E_{DE}}}{T_r} \qquad \gamma = 0.5$$

CRITERIA

$$\gamma^u_{C_{DE}} + \gamma^u_{S_{DE}} + 0.5\gamma^u_{r_s} \le 7.0$$

LRFD FORMULATION FOR ELASTOMERIC ISOLATORS

- **DESIGN EARTHQUAKE CHECKING CRITERIA CONTINUED**

BUCKLING no stability criteria in DE

SHIMS

$$t_s \geq \frac{1.65t}{1.08F_y \dfrac{A_{rDE}}{P_u} - 2} \geq 1.9 \text{ mm}$$

A_{rDE} reduced bonded area for displacement $D = \gamma\Delta_S + \Delta_{E_{DE}}$

$F_y =$ minimum yield strength

END PLATES SAME PROCEDURE AS FOR SERVICE LOADS AND USING MINIMUM MATERIAL STRENGTHS AND APPROPRIATE Φ FACTORS

LRFD FORMULATION FOR ELASTOMERIC ISOLATORS

- **MAXIMUM CONSIDERED EARTHQUAKE (MCE) CHECKING**

Dead load: P_D

Seismic live load (considered static component): P_{SL}

Earthquake axial load due to MCE shaking: $P_{E_{MCE}}$, where earthquake-induced axial loads can result from both overturning moments in the superstructure and vertical earthquake shaking

Factored axial load: $P_u = \gamma_D P_D + \gamma_L P_{SL} + P_{E_{MCE}}$

Load factors γ_D and γ_L are given by the appropriate code or guideline

Non-seismic bearing rotation: θ_{Sst} (static), θ_{Scy} (cyclic)

Seismic lateral displacement: $\Delta_{E_{MCE}}$.

Non-seismic lateral displacement: $0.5\gamma\Delta_S = 0.5\gamma(\Delta_{Sst} + \Delta_{Scy})$

$\Delta_{E_{MCE}} = factor_D \cdot \Delta_{E_{DE}}$ $P_{E_{MCE}} = factor_P \cdot P_{E_{DE}}$ $factor_D = 1.5$ $factor_P = 1.0 - 1.5$

FACTOR OF 1.5 APPLIES FOR CALIFORNIA (SAME FACTOR IN AASHTO GUIDE SPECIFICATIONS FOR SEISMIC ISOLATION DESIGN). FACTOR EXPECTED LARGER THAN 1.5 FOR EASTERN US (2010 AASHTO GUIDE SPECS HAVE 2.0)

LRFD FORMULATION FOR ELASTOMERIC ISOLATORS

- **MAXIMUM CONSIDERED EARTHQUAKE CHECKING**

SHEAR STRAIN DUE TO COMPRESSION
(factor accounts for shape and location of strain)

$$\gamma^{u}_{C_{MCE}} = \frac{P_u}{A_r G S} \square f_1$$

SHEAR STRAIN DUE TO ROTATION
(factor accounts for location of strain)

$$\gamma^{u}_{r_s} = \frac{L^2(\theta_{Sst} + 1.75\theta_{Scy})}{tT_r} \square f_2$$

SHEAR STRAIN DUE TO LATERAL DISPLACEMENT

$$\gamma^{u}_{S_{MCE}} = \frac{0.5\gamma\Delta_S + \Delta_{E_{MCE}}}{T_r}$$

CRITERIA $\gamma^{u}_{C_{MCE}} + \gamma^{u}_{S_{MCE}} + 0.25\gamma^{u}_{r_s} \leq 9.0$

LIMIT OF 9.0 IS ABOUT A 30% INCREASE OVER THE DE LIMIT FOR STRAIN TO ACCOUNT FOR THE INCREASED SEISMIC LOADS AND DISPLACEMENT BY FACTOR 1.5

BUCKLING $\dfrac{P'_{cr_{MCE}}}{P_u} \geq 1.1$ $\quad P'_{cr_{MCE}} = P_{cr}\dfrac{A_{rMCE}}{A}$ for bolted bearings

A_{rMCE} reduced bonded area for displacement

$$D = 0.5\gamma\Delta_S + \Delta_{E_{MCE}}$$

CONSISTENT WITH 1999/2010 AASHTO GUIDE SPECS WHERE BEARING NEEDS TO BE STABLE AT 1.5 TIMES THE DE DISPLACEMENT AND 1.2 TIMES THE DEAD PLUS LIVE LOAD PLUS SEISMIC DE LOAD WITH Φ=0.9

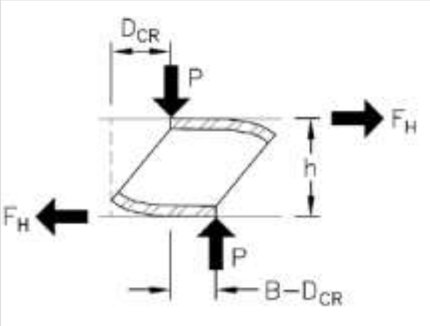
LRFD FORMULATION FOR ELASTOMERIC ISOLATORS

- MAXIMUM CONSIDERED EARTHQUAKE CHECKING CRITERIA CONTINUED

OVERTURNING

$$\frac{D_{cr}^{u}}{0.5\gamma\Delta_{S}+\Delta_{E_{MCE}}} \geq 1.1$$

D_{cr}^{u} CALCULATED USING 0.9 TIMES DEAD LOAD, LOWER BOUND ISOLATOR PROPERTIES, BONDED DIAMETER AND BEARING HEIGHT INCLUDING END AND MASONRY PLATES



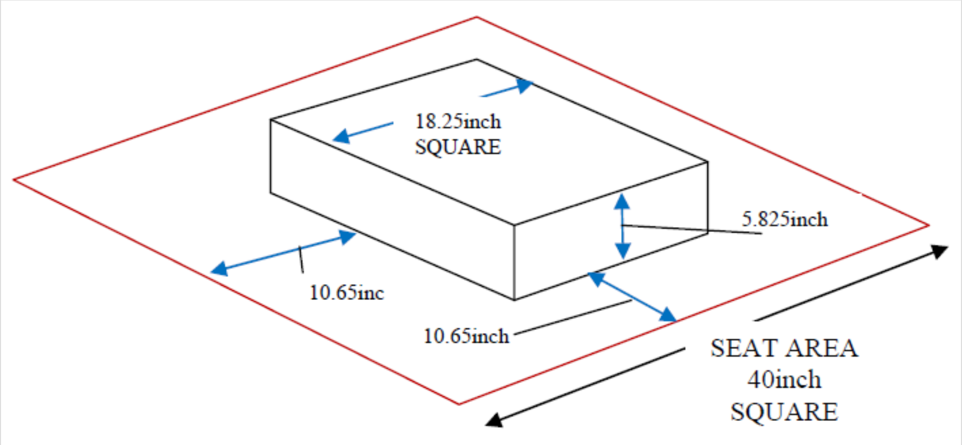
SHIMS

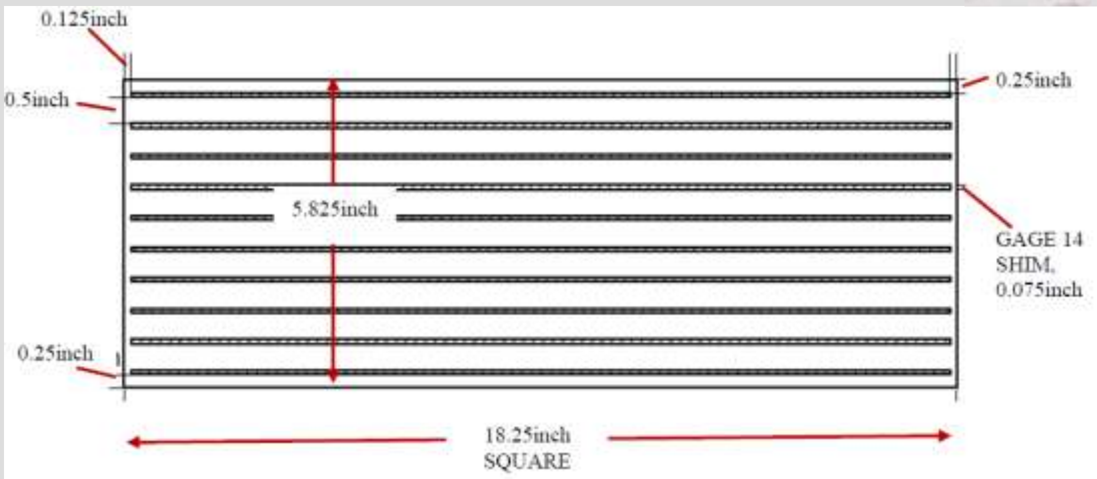
$$t_s \geq \frac{1.65t}{1.08F_{ye}\frac{A_{rMCE}}{P_u}-2} \geq 1.9 \text{ mm}$$

F_{ye} = expected yield strength

END PLATES SAME PROCEDURE AS FOR SERVICE LOADS AND USING EXPECTED MATERIAL STRENGTHS AND Φ FACTORS EQUAL TO UNITY

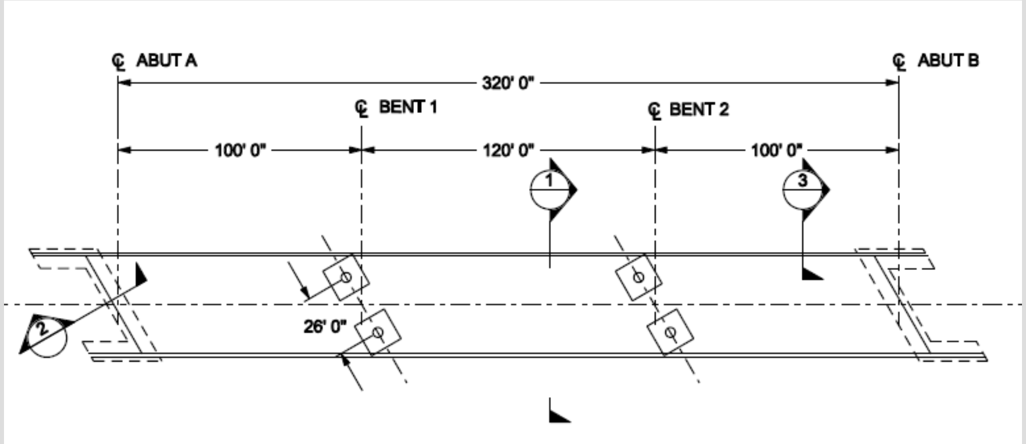
LRFD FORMULATION FOR ELASTOMERIC BRIDGE BEARINGS

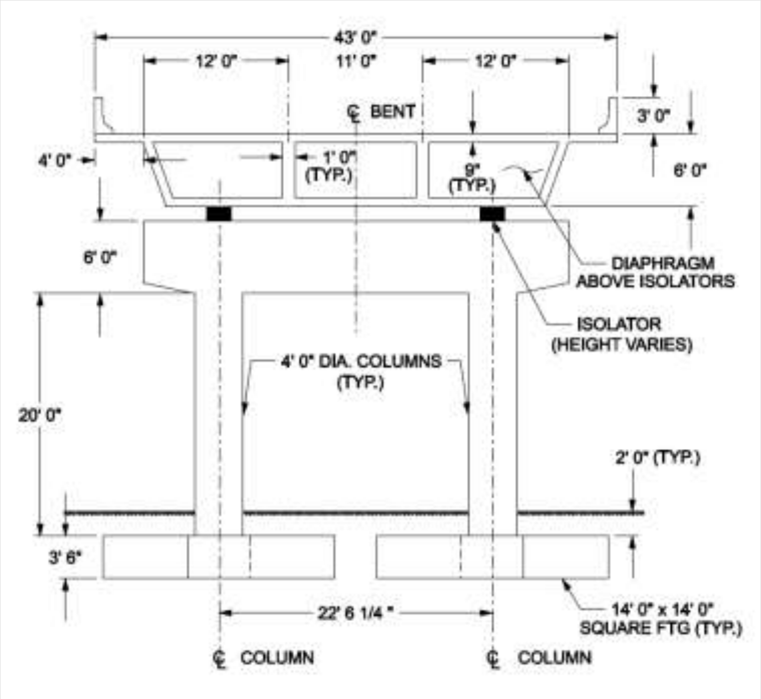


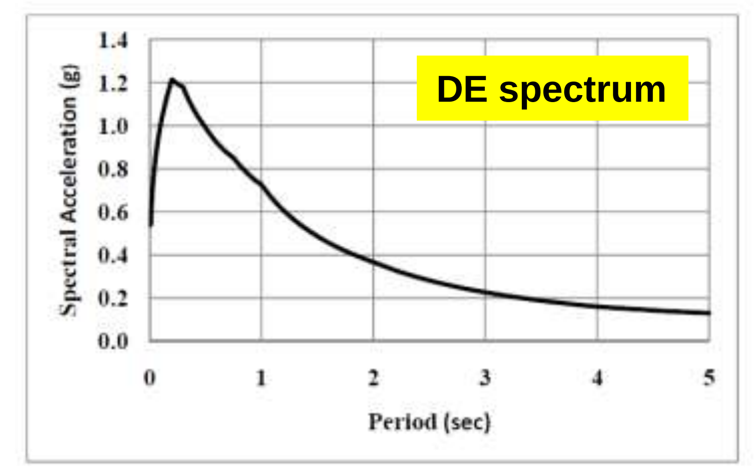


SEISMIC ISOLATION ANALYSIS AND DESIGN EXAMPLES

- Three examples presented in detail. One with Triple FP, one with Lead-rubber and one with single FP isolators







TRIPLE FP ANALYSIS AND DESIGN EXAMPLE

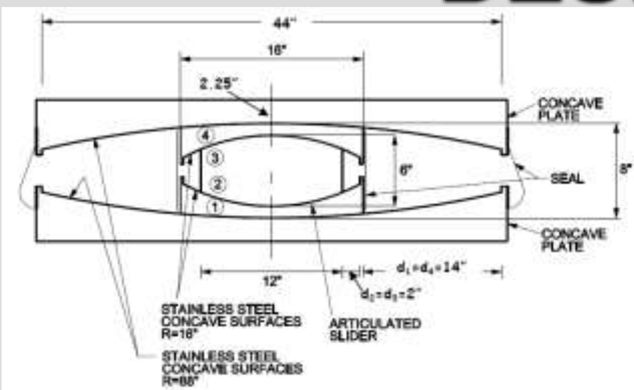
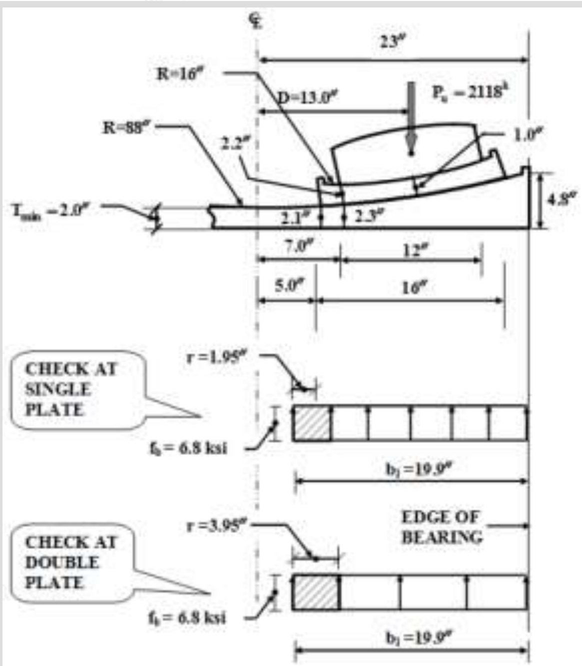
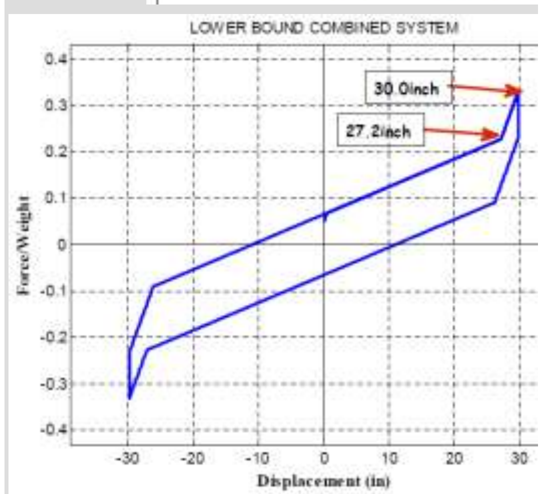
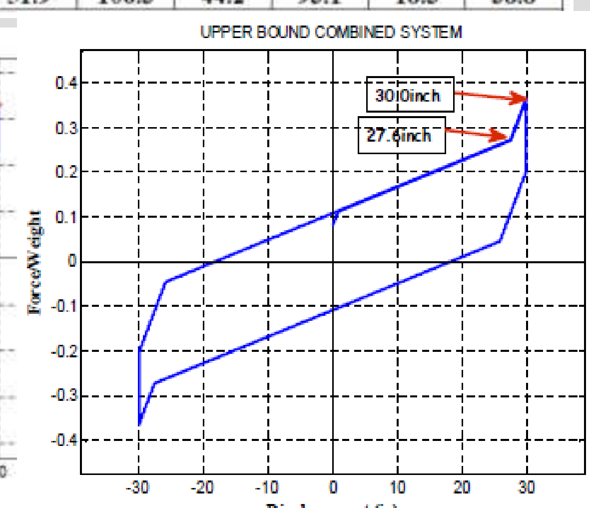


TABLE 11-6 Response History Analysis Results for Lower Bound Analysis of the Triple FP System in the Design Earthquake

Earthquake	Resultant Displacement (inch)		Longitudinal Shear (kip)		Transverse Shear (kip)		Additional Axial Force (kip)	
	Abut.	Pier	Abut.	Pier	Abut.	Pier	Abut.	Pier
01 NP	22.2	20.7	54.7	103.1	54.2	116.7	24.7	50.3
02 NP	33.6	32.5	74.3	162.7	54.3	115.2	21.5	44.6
03 NP	18.0	17.3	50.8	99.1	48.3	97.0	20.4	46.4
04 NP	18.5	16.9	57.7	131.5	36.2	73.0	16.3	28.1
05 NP	13.2	12.8	52.9	109.9	43.5	94.1	15.0	34.2
06 NP	11.0	10.6	36.3	69.8	37.1	80.4	16.2	37.7
07 NP	6.9	7.0	36.4	68.2	36.0	75.6	14.0	30.2
Average	**17.6**	**16.8**	**51.9**	**106.3**	**44.2**	**93.1**	**18.3**	**38.8**





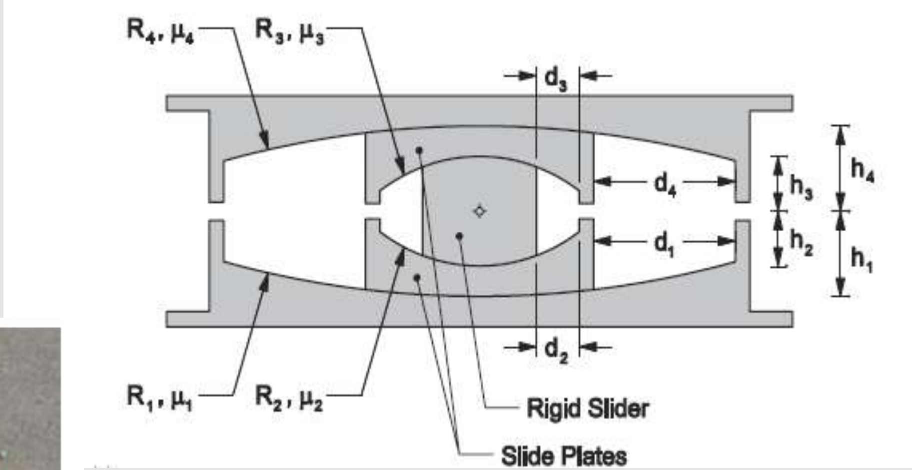


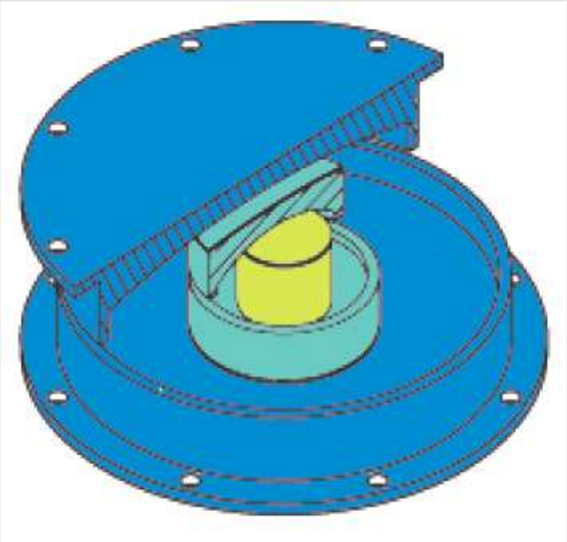
TRIPLE FP ISOLATOR

NEAR DOUBLE THE DISPLACEMENT CAPACITY OF SINGLE FP ISOLATOR FOR THE SAME SIZE

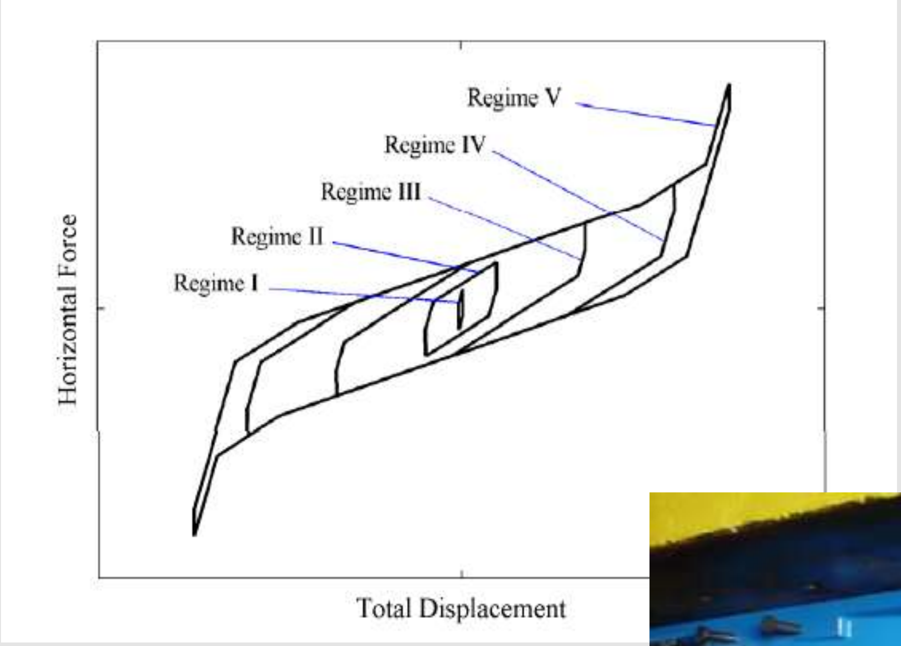
REDUCED SLIDING VELOCITY

SEVERAL STAGES OF ADAPTIVE BEHAVIOR





ADAPTIVE ISOLATORS
TRIPLE FP BEARING

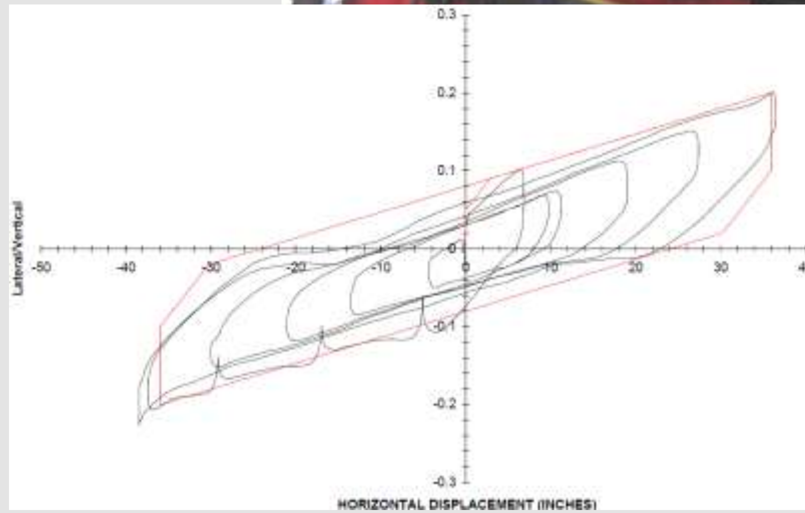


SAN BERNARDINO COURTHOUSE
PROTOTYPE TESTING
NOVEMBER 2010

LARGE SCALE TESTING

SAN BERNARDINO COURTHOUSE
PROTOTYPE TESTING
LARGE BEARING
NOVEMBER 2010

EPS LARGE-SCALE TESTING MACHINE
LOAD=6535kN
AMPLITUDE=940mm
PEAK VELOCITY=1300mm/sec



LEAD-RUBBER ANALYSIS AND DESIGN EXAMPLE

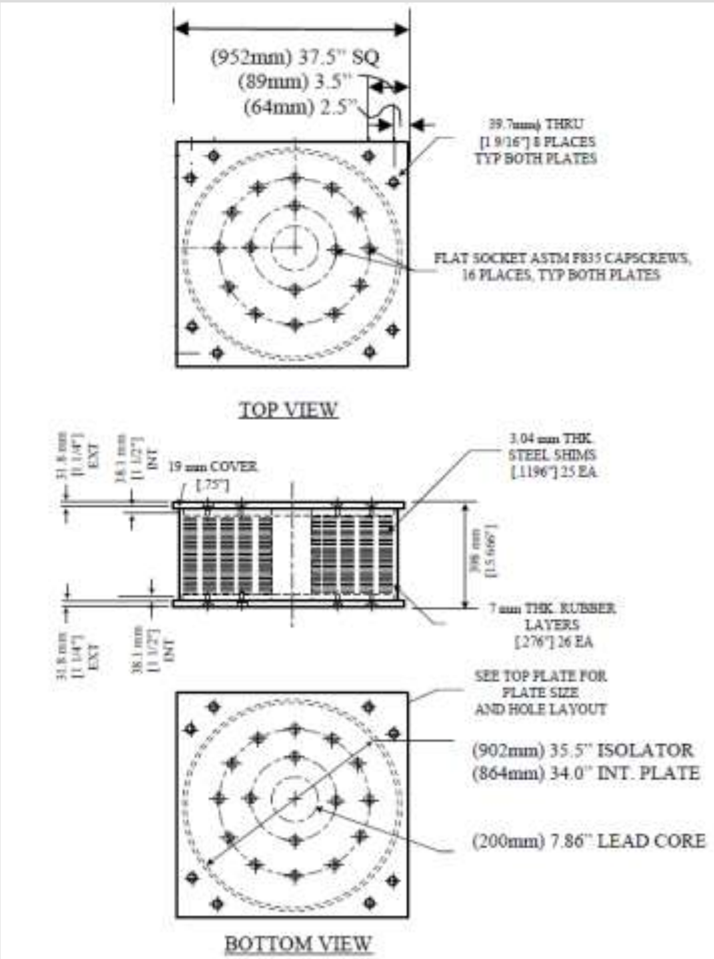
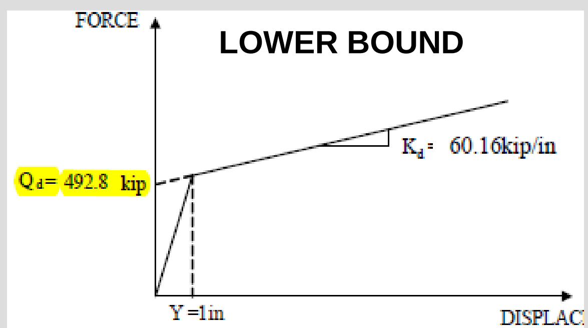
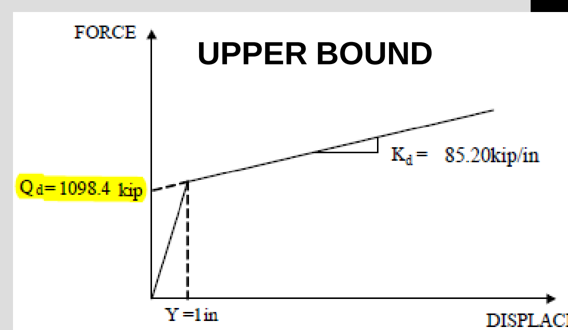


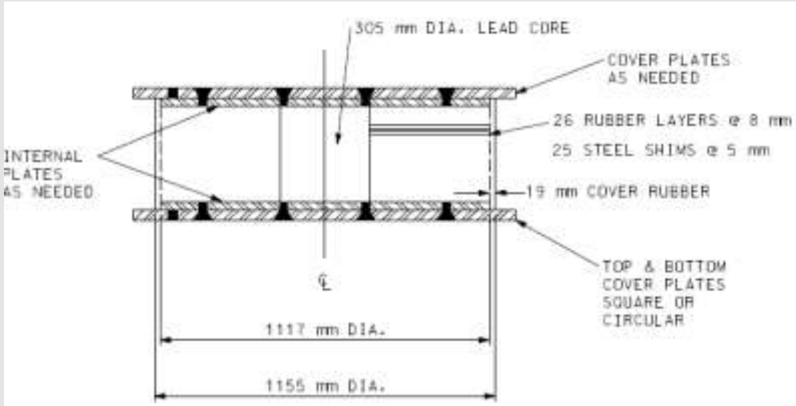
TABLE 12-5 Response History Analysis Results for Lower Bound Analysis of the Lead-Rubber System in the Design Earthquake

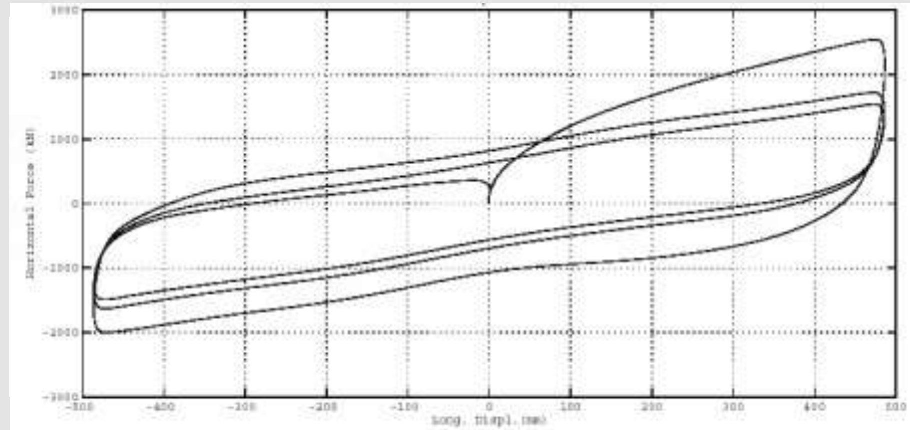
Earthquake	Resultant Displacement (inch)		Longitudinal Shear (kip)		Transverse Shear (kip)		Additional Axial Force (kip)	
	Abut.	Pier	Abut.	Pier	Abut.	Pier	Abut.	Pier
01 NP	13.9	13.8	112.0	116.0	141.7	160.4	60.9	65.9
02 NP	21.6	20.6	196.1	196.7	96.9	111.2	44.9	36.6
03 NP	12.5	11.8	131.5	142.5	135.2	151.1	62.3	67.9
04 NP	14.5	13.2	150.6	158.8	98.2	112.6	54.0	64.4
05 NP	10.5	10.2	131.4	141.0	117.4	133.7	57.0	67.2
06 NP	11.0	10.5	80.5	96.1	123.2	136.1	57.2	61.3
07 NP	7.5	7.5	91.8	103.1	85.2	109.4	40.5	46.2
Average	**13.1**	**12.5**	**127.7**	**136.3**	**114.0**	**130.6**	**53.8**	**58.5**

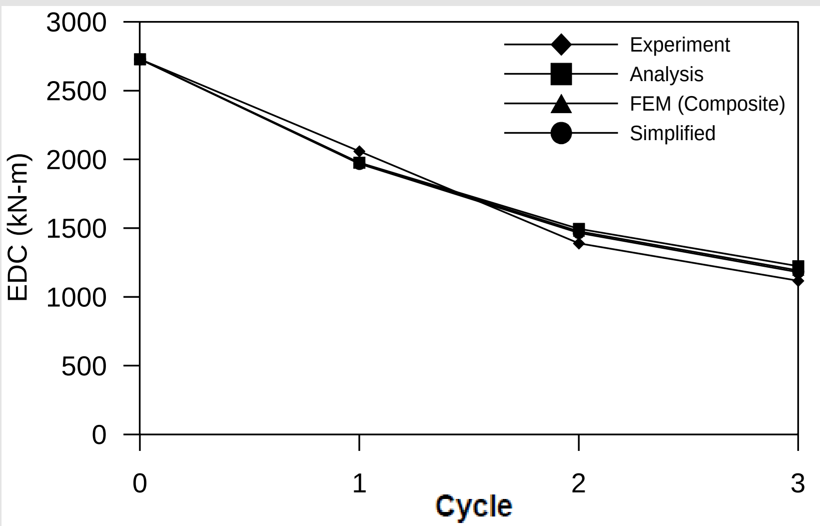


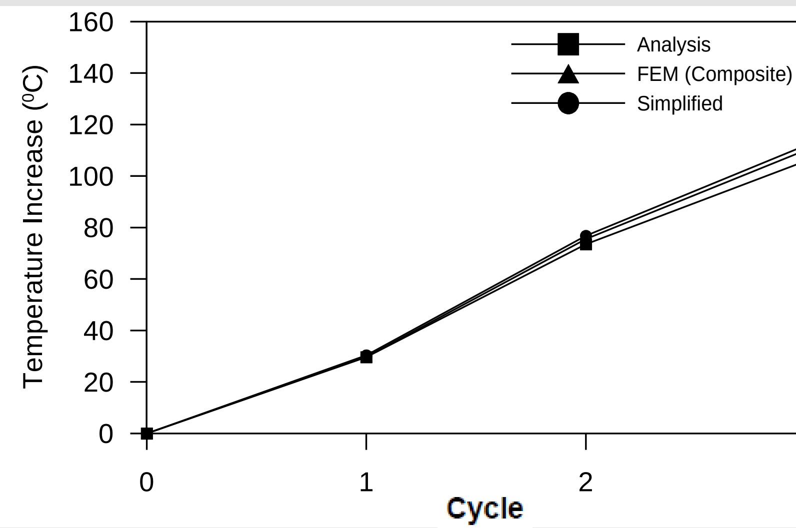


LEAD RUBBER CORE HEATING









ERZURUM HOSPITAL, TURKEY, 2007
LOAD=10260kN, DISPLACEMENT=480mm,
VELOCITY=1m/sec

LEAD RUBBER CORE HEATING

- Complex heat conduction problem

$$\rho_L \cdot c_L \cdot V_L \frac{dT_L}{dt} = q'''(t) \cdot V_L - 2 \cdot q_1(t) - q_2(t)$$

$$q'''(t) = \frac{\left|\sigma_{YL} \cdot A_L \cdot \frac{du}{dt}\right|}{V_L} = \frac{\sigma_{YL} \cdot \left|\frac{du}{dt}\right|}{h_L}$$

$$\sigma_{YL} = \sigma_{YL0} \cdot \exp(-E_2 \cdot T_L)$$

ρ_L **lead density**

c_L **lead specific heat**

q_1 heat flux to top or bottom end plate

q_2 heat flux to shim plates

σ_{YL} **effective yield stress of lead (function of T_L)**

q''' heat production rate (energy per volume per time)

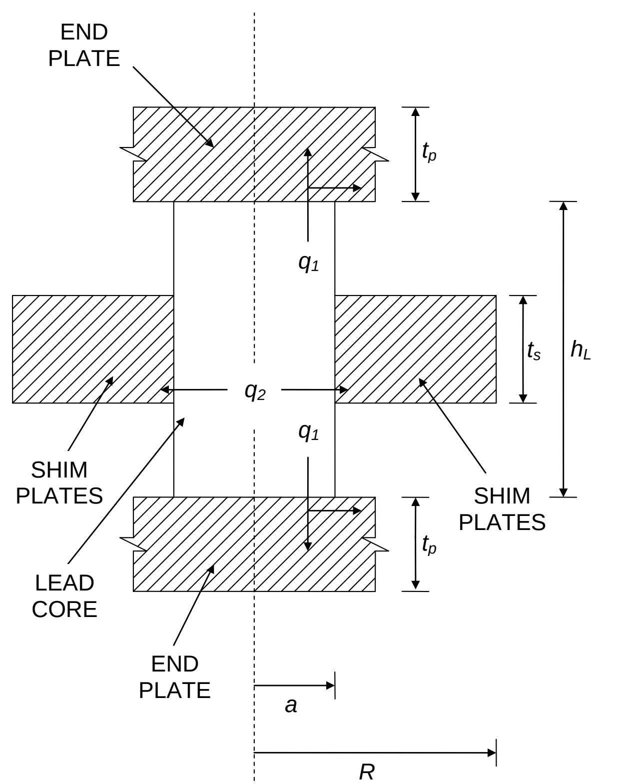
V_L **volume of lead core**

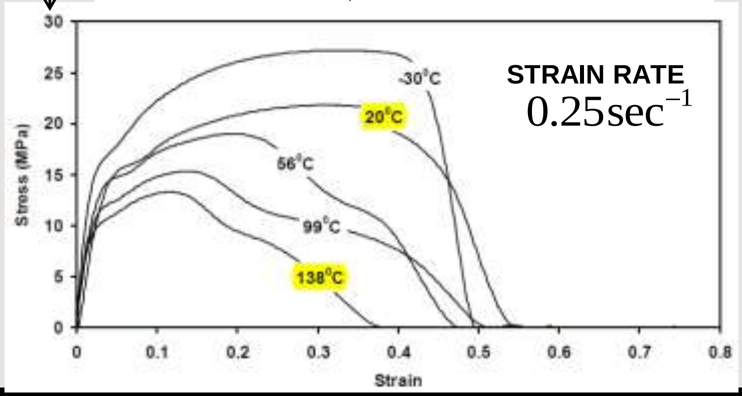
h_L **height of lead core**

A_L **area of lead core**

$\frac{du}{dt}$ **velocity of top of bearing wrt bottom**

T_L **lead core temperature rise**





LEAD RUBBER CORE HEATING

$$\frac{dT_L}{dt} = \frac{\sigma_{YL}\left(T_L\right)\cdot v(t)}{\rho_L c_L h_L} - \frac{k_S \cdot T_L}{a\cdot \rho_L c_L h_L}\cdot\left(\frac{1}{F} + 1.274\cdot\left(\frac{t_s}{a}\right)\cdot\left(t^+\right)^{-1/3}\right)$$

Effective Yield Stress

Lead Core Height

Velocity

Lead Core Radius

Total Shim Thickness

Dimensionless Time

$$F = \begin{cases} 2\cdot\left(\frac{t^+}{\pi}\right)^{1/2} - \frac{t^+}{\pi}\cdot\left[2-\left(\frac{t^+}{4}\right)-\left(\frac{t^+}{4}\right)^2-\frac{15}{4}\left(\frac{t^+}{4}\right)^3\right], & t^+ \le 0.6 \\ \frac{8}{3\pi} - \frac{1}{2\left(\pi\cdot t^+\right)^{1/2}}\cdot\left[1-\frac{1}{3\cdot\left(4t^+\right)}+\frac{1}{6\cdot\left(4t^+\right)^2}-\frac{1}{12\cdot\left(4t^+\right)^3}\right], & t^+ > 0.6 \end{cases}$$

$$t^+ = \frac{\alpha_S \cdot t}{a^2}$$

Thermal Diffusivity of Steel

$$\sigma_{YL} = \sigma_{YL0}\cdot\exp(-E_2\cdot T_L)$$

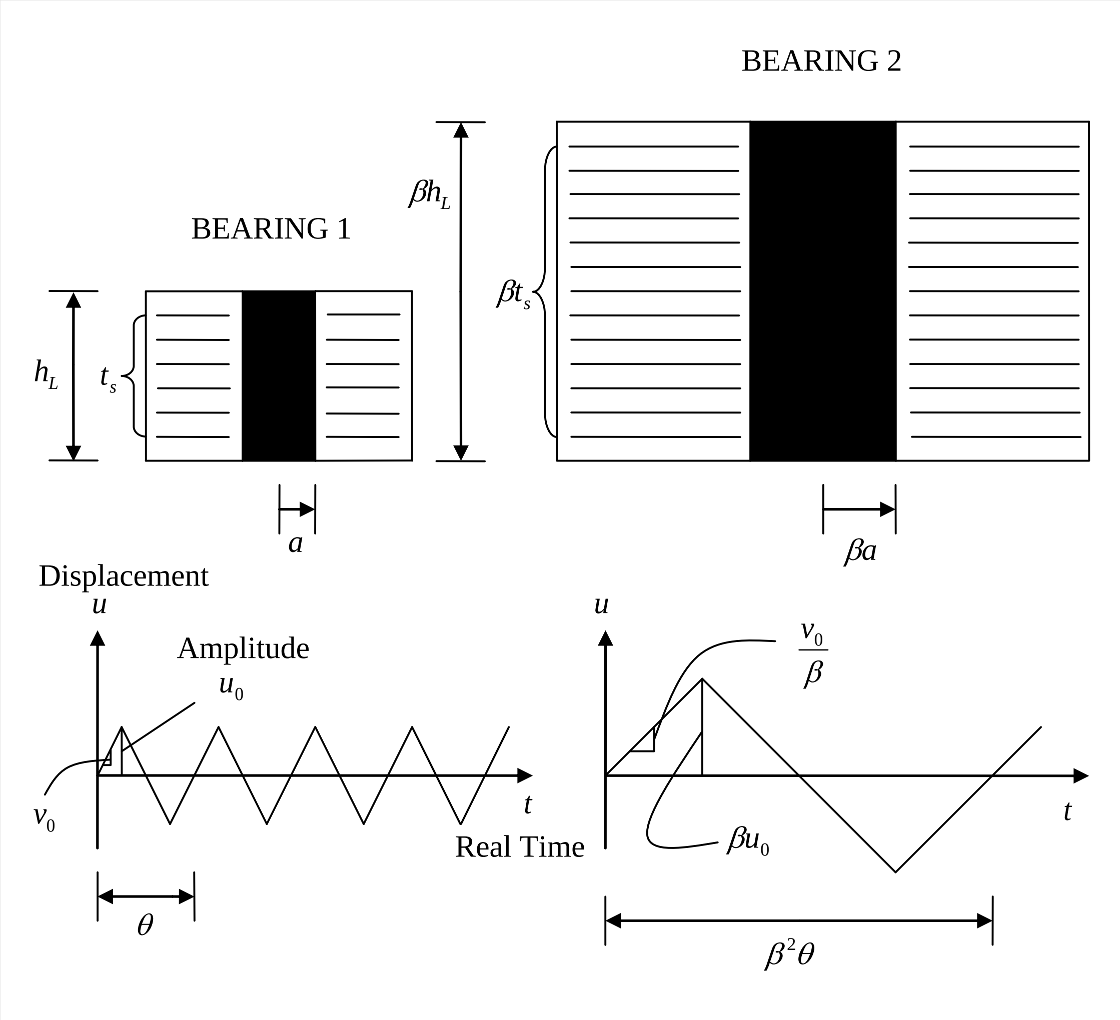
SIMPLIFIED SOLUTION VALID WHEN HEAT CONDUCTION IN STEEL PLATES AND SHIMS IS NEGLECTED

$$T_L' = \frac{1}{E_2}\cdot\ln\left(1+\frac{E_2\sigma_{YL0}S}{\rho_L c_L h_L}\right)$$

$$S = \int v(t)dt$$

LEAD RUBBER CORE HEATING

- Scaling of Lead Rubber Bearings for Testing



$$Q = E_2 T_L$$ **Temperature**

$$\left(\frac{\alpha_S t}{a^2}\right)$$ **Time**

$$\left(\frac{t_s}{a}\right)$$ **Shim Thickness**

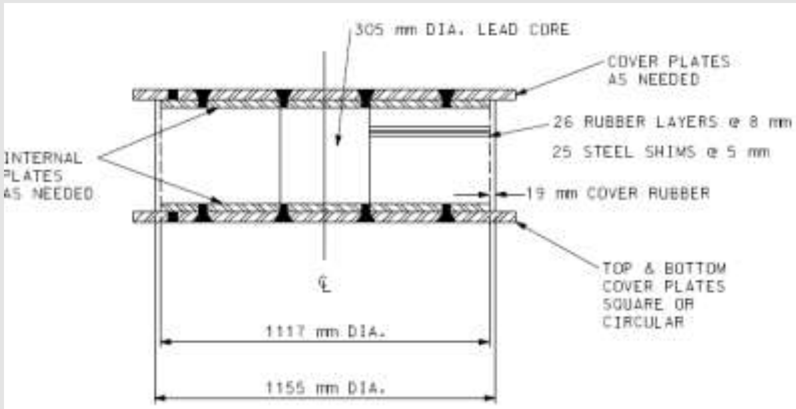
$$\left(\frac{a \rho_S c_S}{\rho_L c_L h_L}\right)$$ **Lead Core Shape**

$$\left(\frac{E_2 \sigma_{YL0} v_0 a^2}{\alpha_S \rho_L c_L h_L}\right)$$ **Strain Rate**

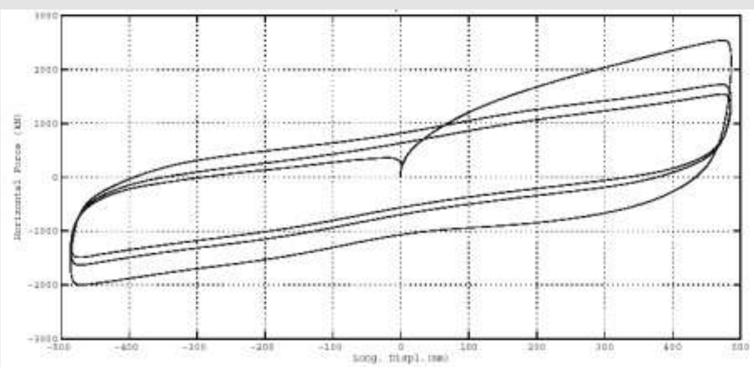
$$S_T = S_L{}^2 = \beta^2$$ **Thermodynamic Similarity**

$$S_T = \sqrt{S_L} = \sqrt{\beta} \ldots ?$$ **Gravity Scaling**

LEAD RUBBER CORE HEATING

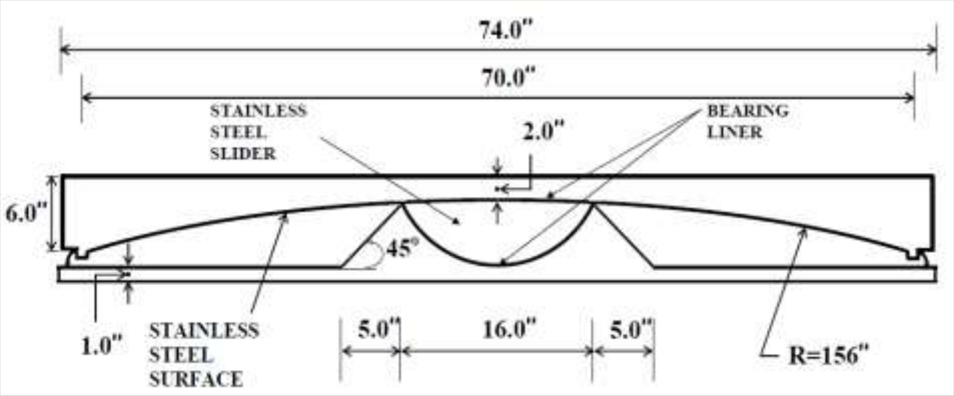


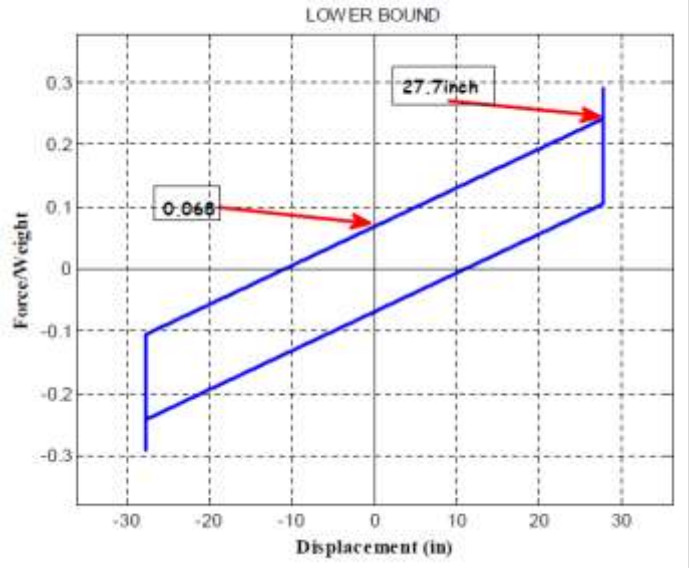
ERZURUM HOSPITAL, TURKEY, 2007
LOAD=10260kN, DISPLACEMENT=480mm, VELOCITY=1m/sec

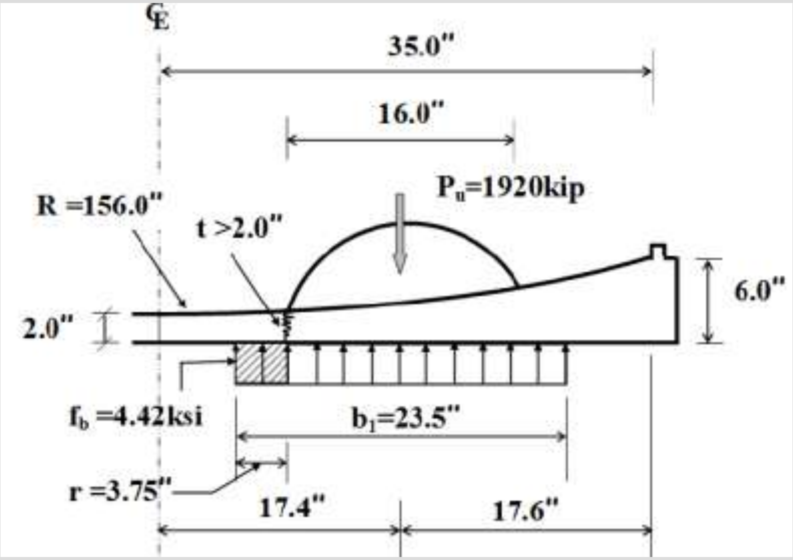


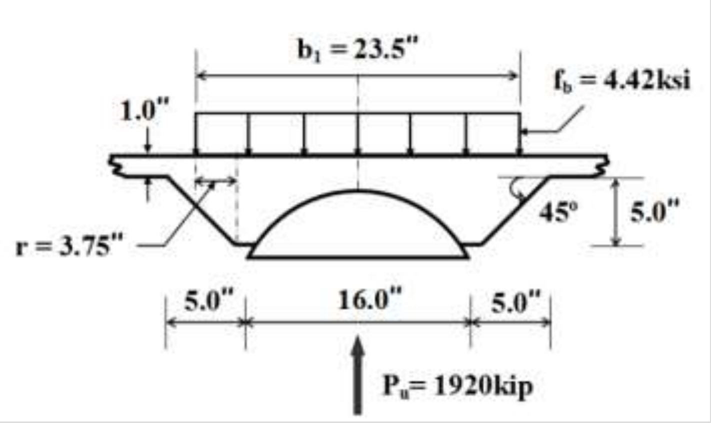
- **REDUCED SCALE BEARING TESTING**
  - LENGTH SCALE 2 (HALF SIZE)
  - LOAD 2565kN
  - DISPLACEMENT AMPLITUDE 240mm
  - VELOCITY 2m/sec
  - CAN CORRECTLY CAPTURE REDUCTION OF STRENGTH PER CYCLE
  - CANNOT PROVIDE RELIABLE INFORMATION ON STARTING VALUE OF STRENGTH
- **SHAKE TABLE TESTING**
  - LENGTH SCALE 4 (QUARTER SIZE)
  - LOAD PER BEARING 640kN
  - FOR REALISTIC TEST WITH AT LEAST 8 BEARINGS CAN ONLY BE DONE AT E-DEFENSE IN JAPAN
  - TO PROPERLY MODEL GRAVITY (UNSCALED), TIME NEEDS TO BE COMPRESSED BY FACTOR $\sqrt{4}=2$. VELOCITY IN THE EXPERIMENTS IS THEN REDUCED BY FACTOR 4/2=2.
  - THERMODYNAMIC SIMILARITY REQUIRES THAT VELOCITY IS INCREASED BY FACTOR 4.
  - IMPOSSIBLE TO CORRECTLY OBSERVE HEATING EFFECTS.

SINGLE FP ANALYSIS AND DESIGN EXAMPLE

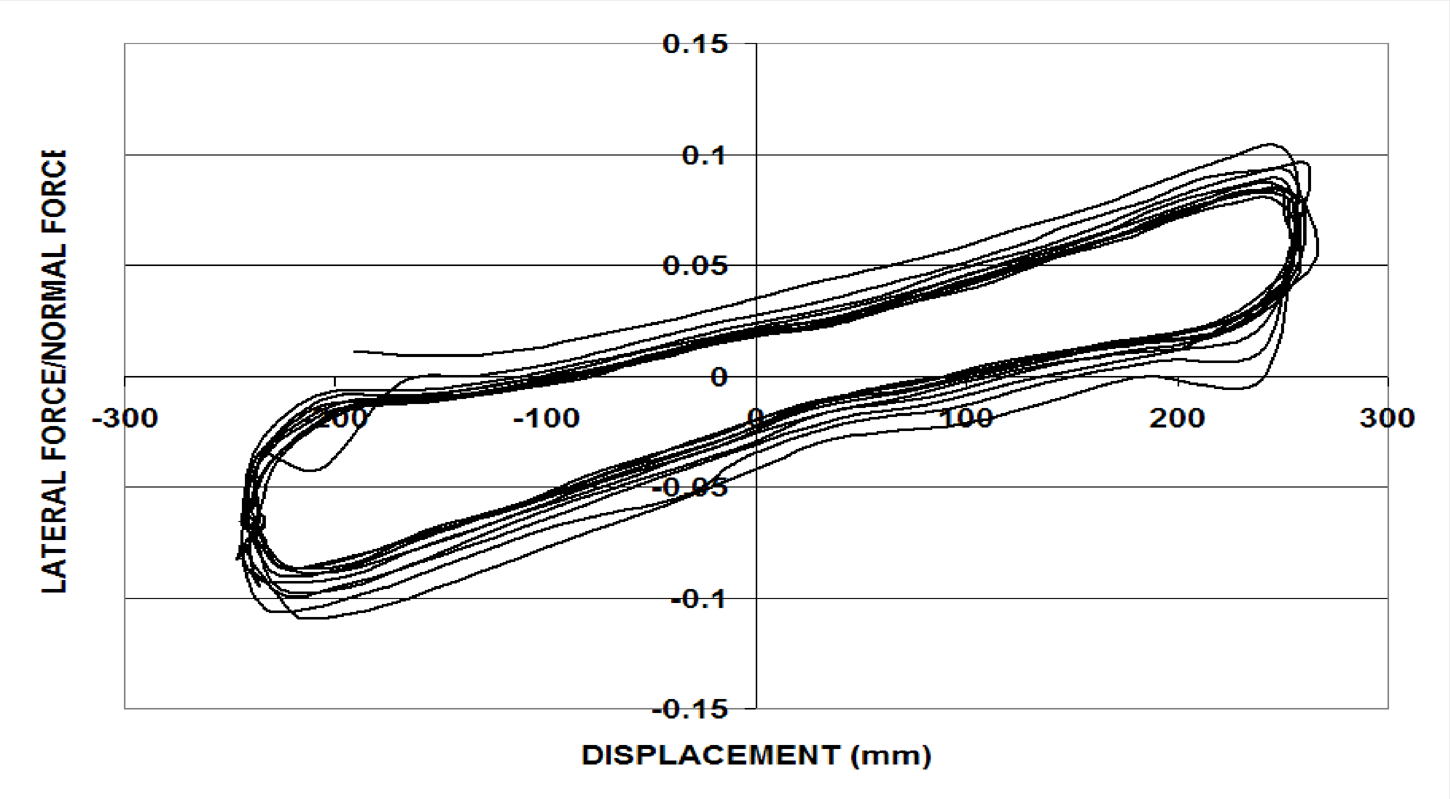








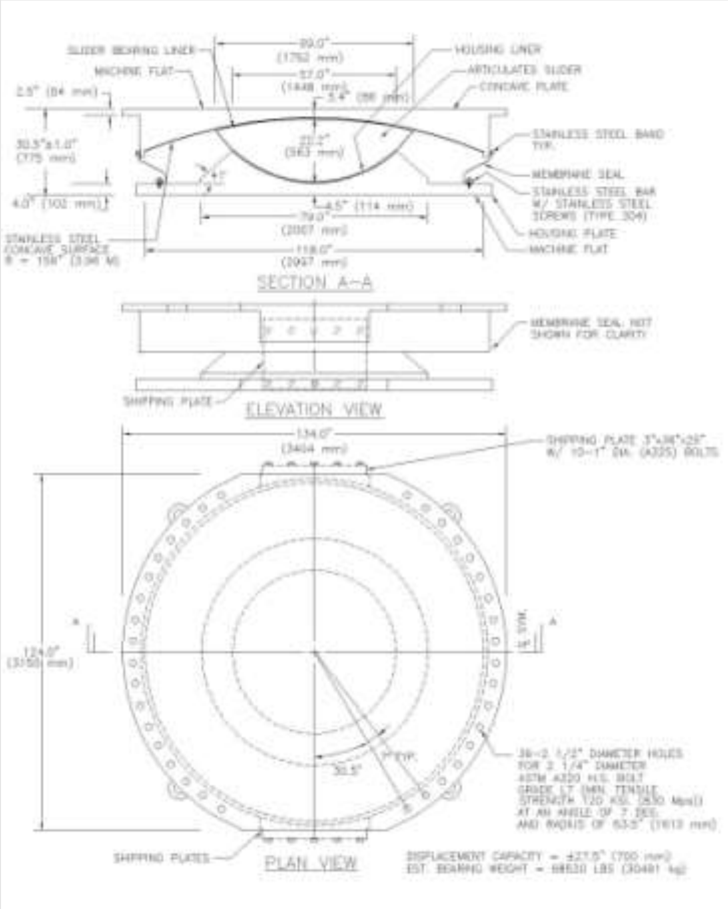
FP BEARING HEATING EFFECTS

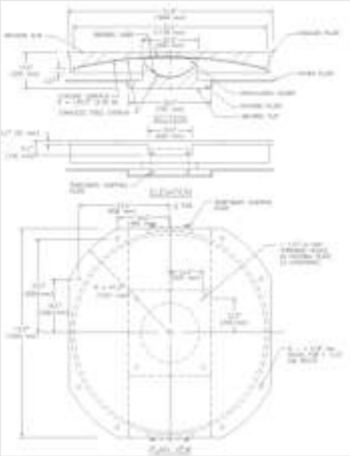


SAKHALIN II PLATFORMS PROTOTYPE BEARING PR1,
LOAD=6925kN, DISPLACEMENT=240mm, VELOCITY=0.9 m/sec
EPS BEARING TESTING MACHINE, OCTOBER 2005

LUNSKOYE/PILTUN PLATFORMS

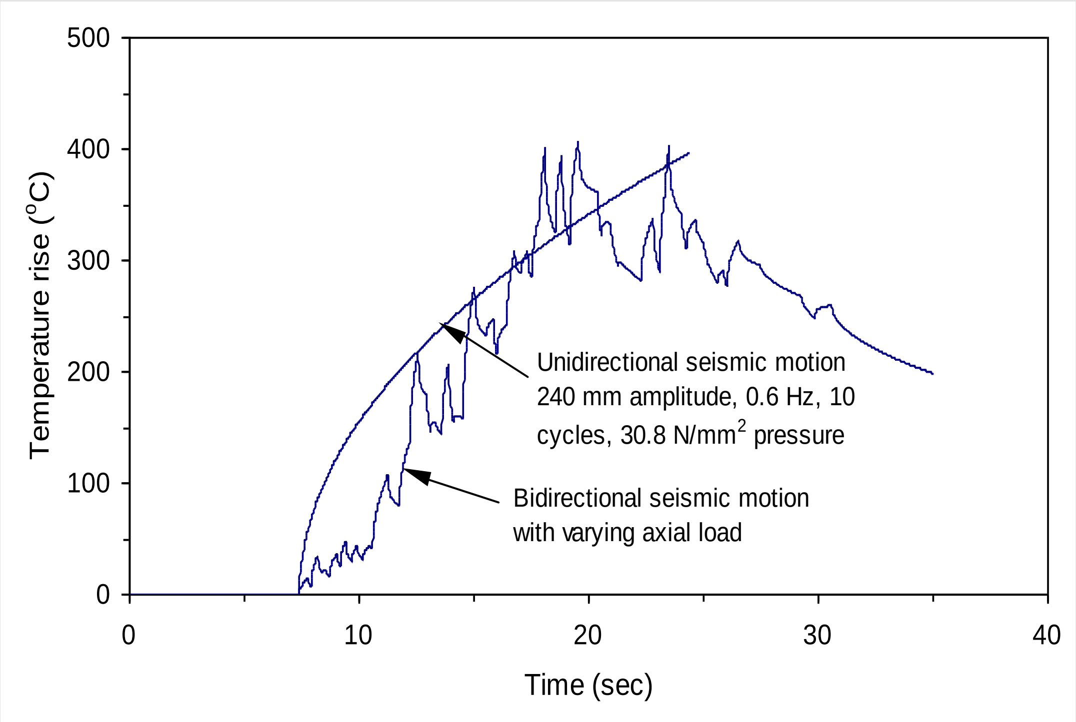
FULL SIZE PRDUCTION BEARING





REDUCED SIZE PROTOTYPE BEARING

LUNSKOYE/PILTUN PLATFORMS



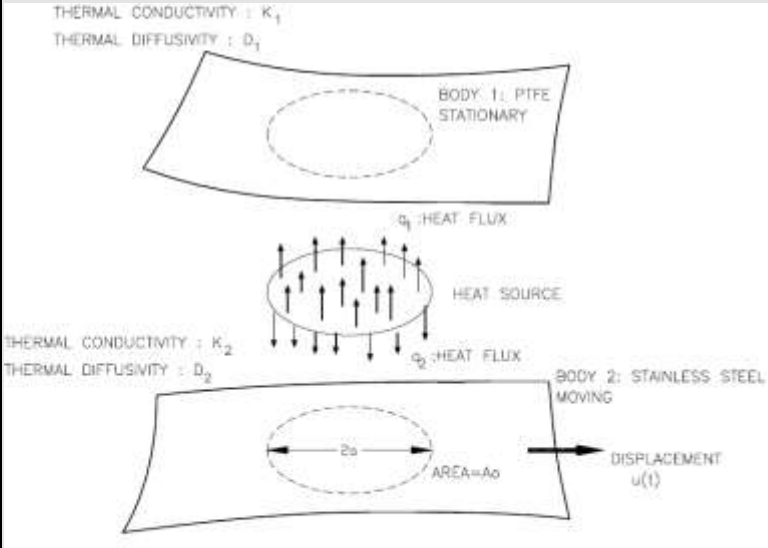
$$T(x,t)=\frac{D^{1/2}}{\pi^{1/2}k}\int_{o}^{t} q(t-\tau)\exp\left(-\frac{x^2}{4D\tau}\right)\frac{d\tau}{\tau^{1/2}}$$

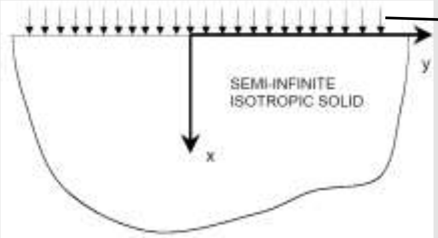
k **is the thermal conductivity of stainless steel**
D is the thermal diffusivity of stainless steel
q is the heat flux=μ·p·v

SCALING PROCESS

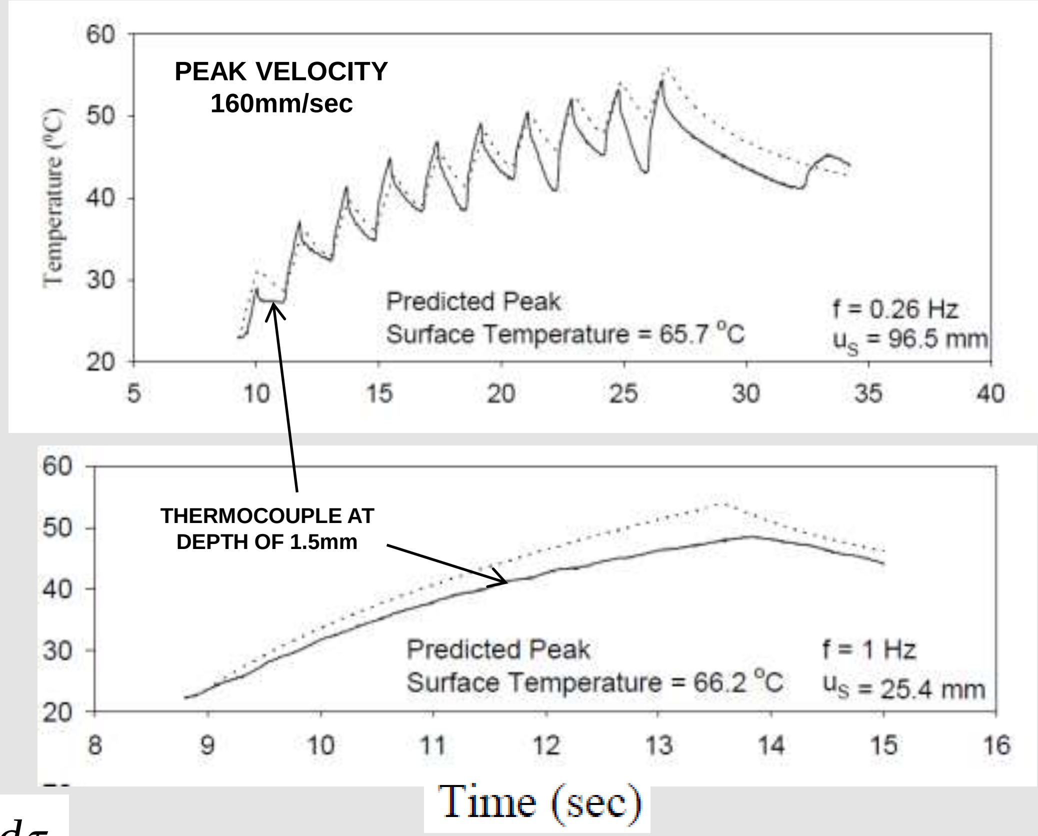
1. MAINTAIN AVERAGE PRESSURE
2. MAINTAIN EDGE PRESSURE
3. MAINTAIN THICKNESS OF LINER
4. MAINTAIN FREE LENGTH AND THICKNESS OF STAINLESS STEEL OVERLAY
5. SELECT BEARING THICKNESSES TO MAINTAIN THERMODYNAMIC CONDITIONS
6. SELECT TESTING PROCEDURE TO SIMULATE TEMPERATURE RISE DUE TO FRICTIONAL HEATING AT SLIDING INTERFACE IN MOST CRITICAL LOADING CASE (RELATED TO WEAR OF LINER)

FRICTIONAL HEATING





INTERMITTENT HEAT FLUX



$$T(x,t) = \frac{D^{1/2}}{\pi^{1/2}k} \int_{o}^{t} q(t-\tau)\exp\left(-\frac{x^2}{4D\tau}\right)\frac{d\tau}{\tau^{1/2}}$$

k **is the thermal conductivity of stainless steel**

D is the thermal diffusivity of stainless steel

q is the heat flux=μ·p·v

CONCLUSIONS

- LRFD-based analysis and design procedures for seismic isolators and bridge bearings have been developed and documented in extensive report.
- Detailed examples of application (isolator design, simplified and dynamic analysis, assessment of adequacy of isolators and connections) developed.
- If California proceeds with the use of these procedures, it is inevitable that other states in the US will follow.
- While procedures specialize for bridges, adaptation and application for buildings and other structures is direct (and simpler given that service load effects are less complex).
- Maximum earthquake effects in bridges are assessed indirectly by use of multiplying factor on the design earthquake effects-value of factor depends on design earthquake definition, on maximum earthquake definition, on site conditions and on isolation system properties.
- Heating effects important in modeling behavior, in selecting bounding values of properties for analysis and in testing of isolators.

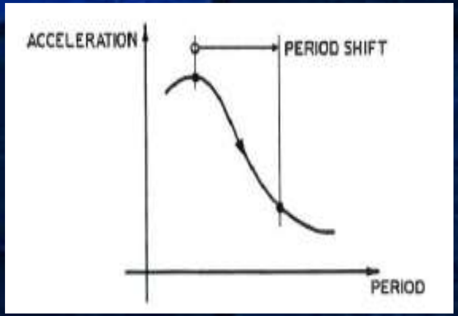
Seismic Isolation and Energy Dissipation Devices in California

Tim Delis Ph.D., P.E.
Senior Bridge Engineer
Joint Seals & Bearings Specialist
Caltrans

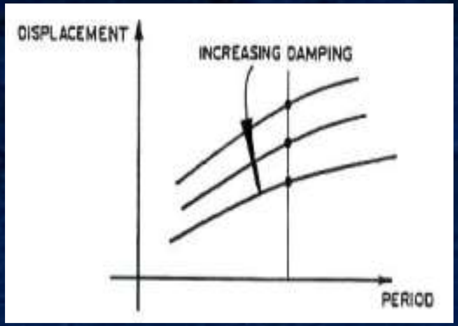
CSRN-NEES
Vancouver, BC
April 30, 2012

Seismic Isolation and Energy Dissipation Devices

- Seismic Isolation:
  - shifts the period of the structure and this results in smaller earthquake forces:
  - increases the relative displacements
    - across the flexible bearing (isolation devices) and
    - at the expansion joints (seismic joints)



- Energy Dissipation:
  - reduces relative displacements by adding damping into the structure:
    - Viscous damping, velocity dependent, fluid viscous dampers
    - Hysteretic damping, displacement dependent, yielding devices



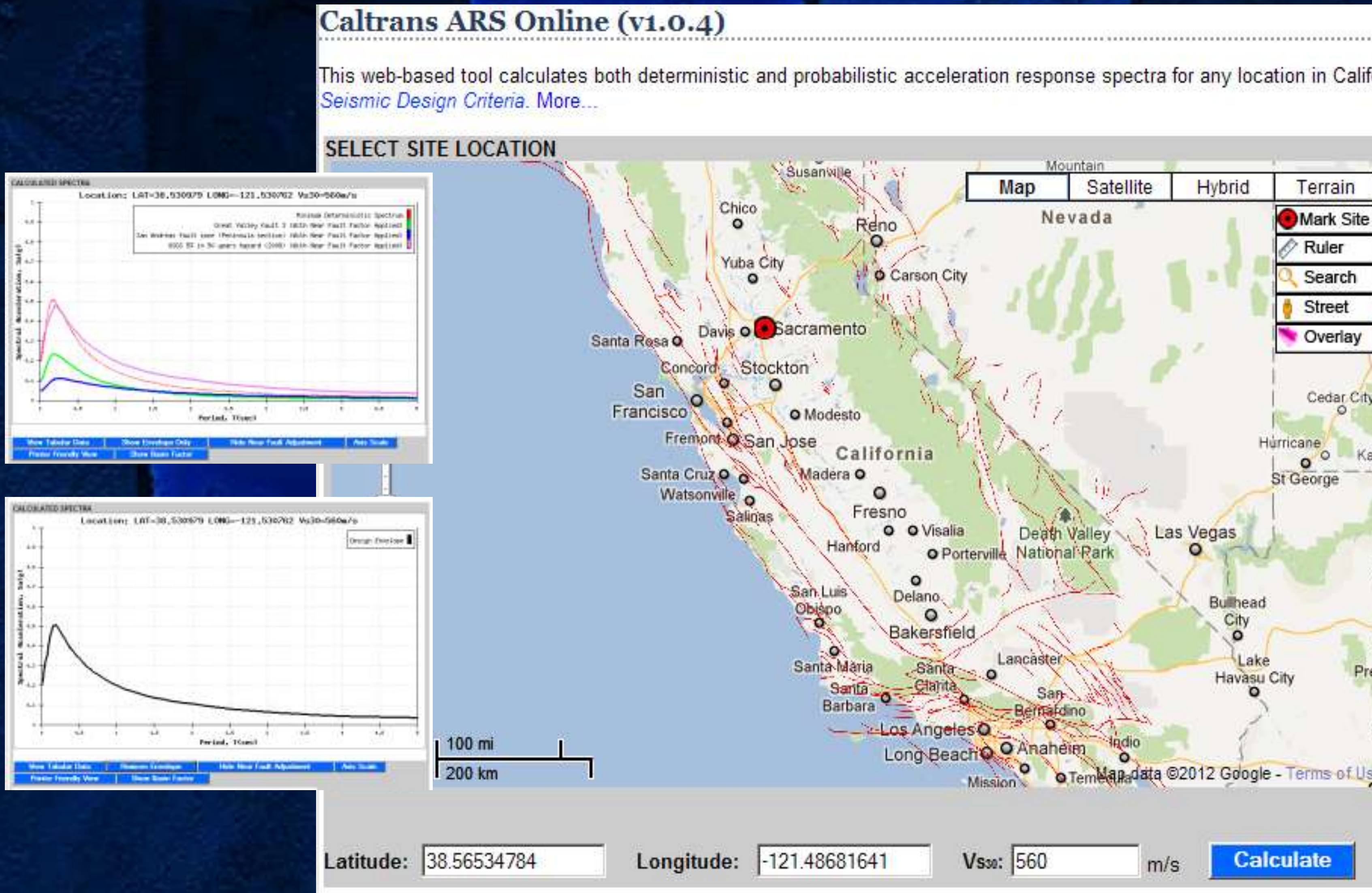
- For each seismic isolation project Caltrans requires that:
  - the isolation device design conforms to AASHTO Guide Specifications for Seismic Isolation Design (3rd Edition, July 2010)

  - the requirements stated in Caltrans Seismic Design Criteria (SDC, Version 1.6, Nov. 2010)

CALTRANS
SEISMIC DESIGN CRITERIA
VERSION 1.6
NOVEMBER 2010

  - the project Special Provisions
  - Design spectra are obtained using Appendix B of the Caltrans SDC. Caltrans ARS website (http://dap3.dot.ca.gov/shake/index.php)

Caltrans ARS Online



- For seismic isolation, Caltrans requires that:
  - Isolated bridges shall meet the service load requirements per AASHTO LRFD with CA Amendments in addition to SDC req.
  - Bent stiffness and mass balance requirements shall be maintained per SDC regardless of the isolation
  - All service horizontal forces are transmitted to substructure by effective sizing the isolators or by external shear keys
  - Isolation bearings & superstructure support shall be designed for min. 125% of the bearing design displacement demand
  - The hazard level for isolated bridges shall be the same as for non-isolated bridges specific to the project
  - The lateral force causing plastic hinging in substructure shall be greater than
    - > 1.2 times the non-seismic lateral force
    - > the lateral force resulting from isolator reaching 125% displacement
    - > 0.20g, when g is the DL reaction on the substructure

- Cont.
  - Bearings shall not experience uplift under service and seismic demands.
  - Superstructure unseating shall be prevented if the displacement demand exceeds the 125% brg. dislp. capacity.
  - Jacking locations shall be provided for bearing replacement
  - Minimum ductility demand for columns and shafts is 3.0
  - P-Delta check for columns, piles, shafts when superstructure is at 125% displacements
  - Foundations shall be capacity protected with the exception plastic hinging in Type I shafts could be below ground.

Seismic Isolation Devices

LRB/FPS

- Prequalified list of suppliers.
- Prototype Test:
  - For every type of isolator, two full scale prototype isolation bearings are manufactured and tested.
  - Tests include a series of dynamic (fully reversed cyclic) and static tests to verify the design requirements for the particular size and configuration used in the design.
- Proof Test:
  - Every production bearing is tested and evaluated for
    - its compression capacity, 1.5 (DL+LL), 5-min
    - Combined compression and shear: 3 fully reversed cycles of loading at max seismic displacement. The compressive load is 1.0(DL +LL) shown on the plans.

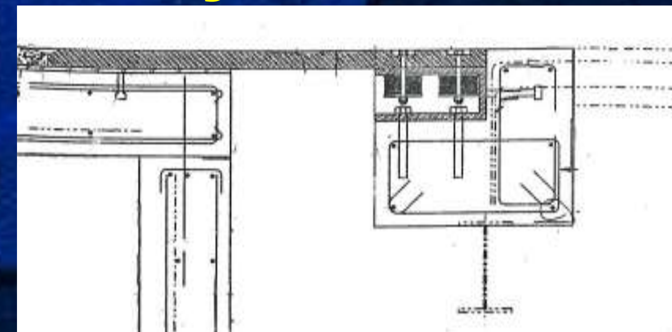
Seismic Isolated Bridges in CA

Partial list

- Benicia Martinez Br.
- Richmond San Rafael Br.
- Antioch Br.
- Golden Gate Br. Approach Viaducts
- Rio Hondo Busway Br.
- Coronado Br.
- Feather River Br.
- Sierra Point Overhead
- Santa Anna River Br.
- Eel River Br.
- Feather River Br.

SIERRA POINT UNDERCROSSING 1985 SEISMIC RETROFIT

Rio Hondo Busway



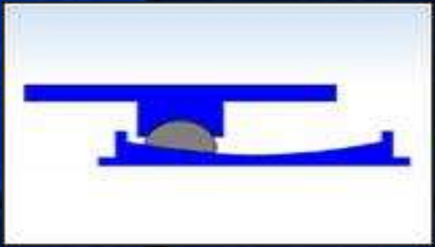
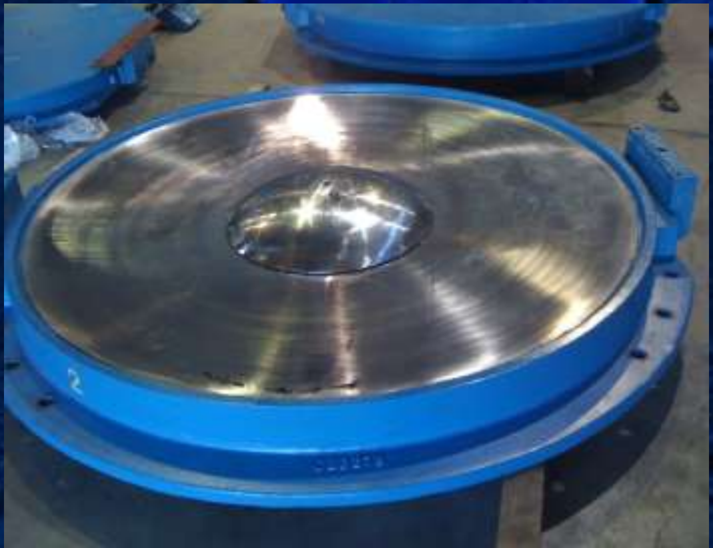
Plan dim: 50”x50”

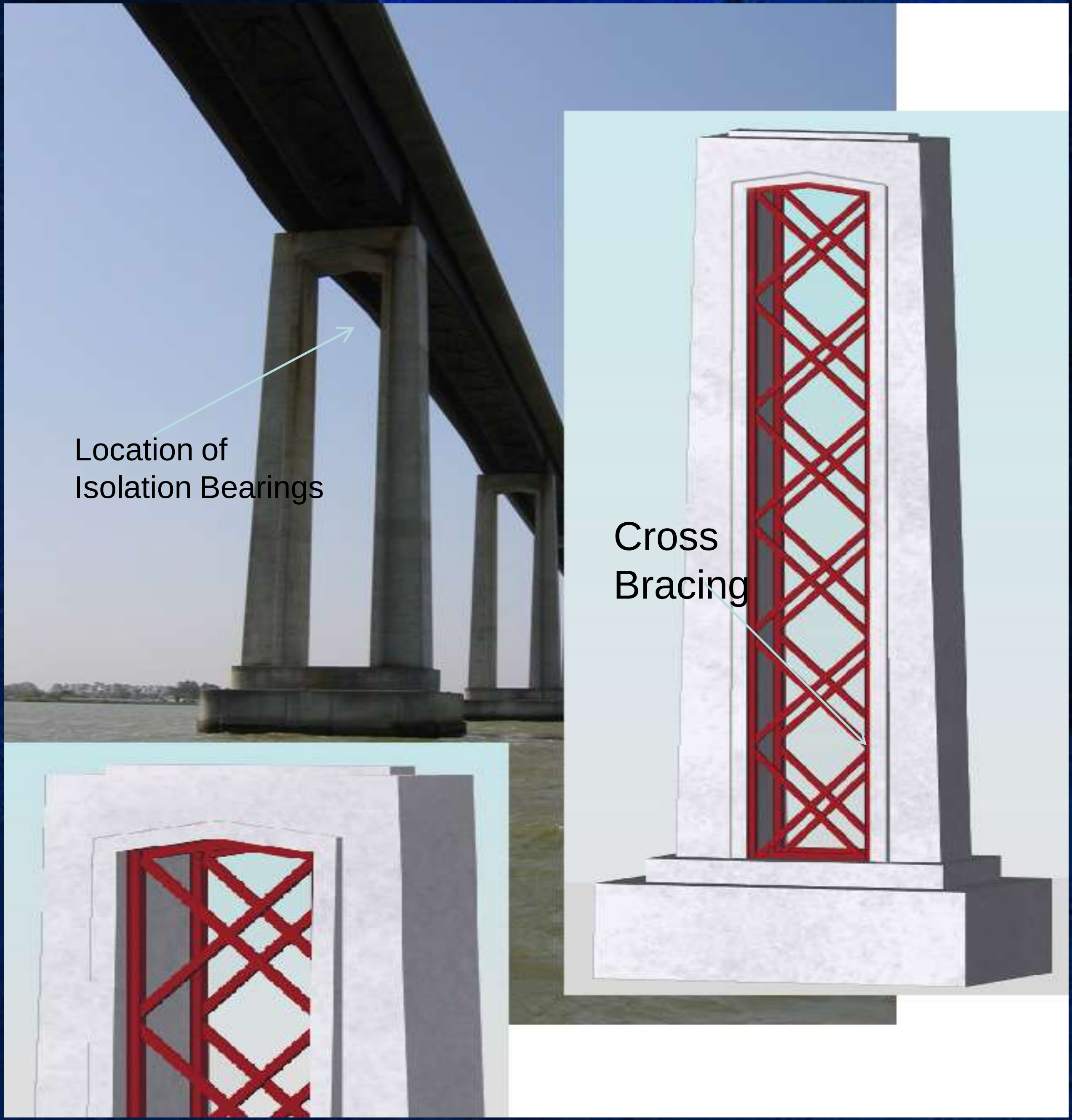
DL= 510 kips

Te=2.33 sec

Lateral Displ.=12”

ANTIOCH BRIDGE



Location of
Isolation Bearings
Cross
Bracing

Antioch Bridge

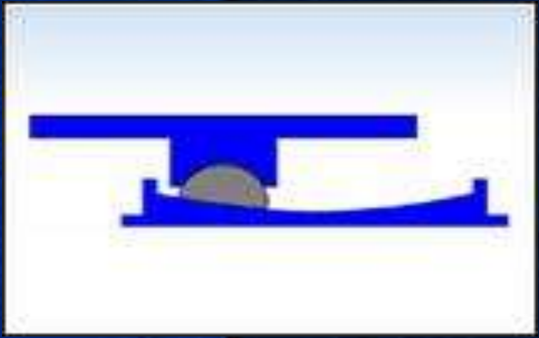
Main Span Retrofit

- **Seismic isolation was used to reduce base shear.** The fundamental as-built period in the transverse direction was **2.6** seconds and **1.8** seconds in the longitudinal direction.

- The isolated structures resulted in a shift of the fundamental period to 6.7 sec in the transverse and **3.2** seconds in the longitudinal direction.

- The transverse base shear reduction in each of the five structural frames was:
  - **Frame 1 – 79% reduction in base shear**
  - **Frame 2 – 49% reduction in base shear**
  - **Frame 3 – 23% reduction in base shear**
  - **Frame 4 – 49% reduction in base shear**
  - **Frame 5 – 74% reduction in base shear**

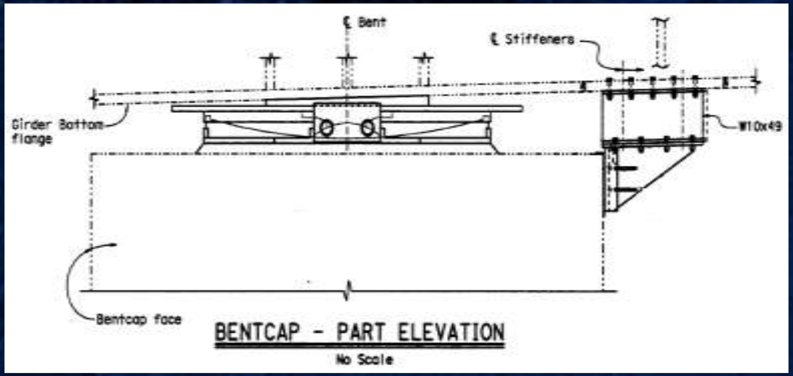
Antioch Bridge FPS Bearing Type I



*- Plan Dimensions= 91'' x 84.4''

*- Height = 9.0''

*- DL = 2160 kips



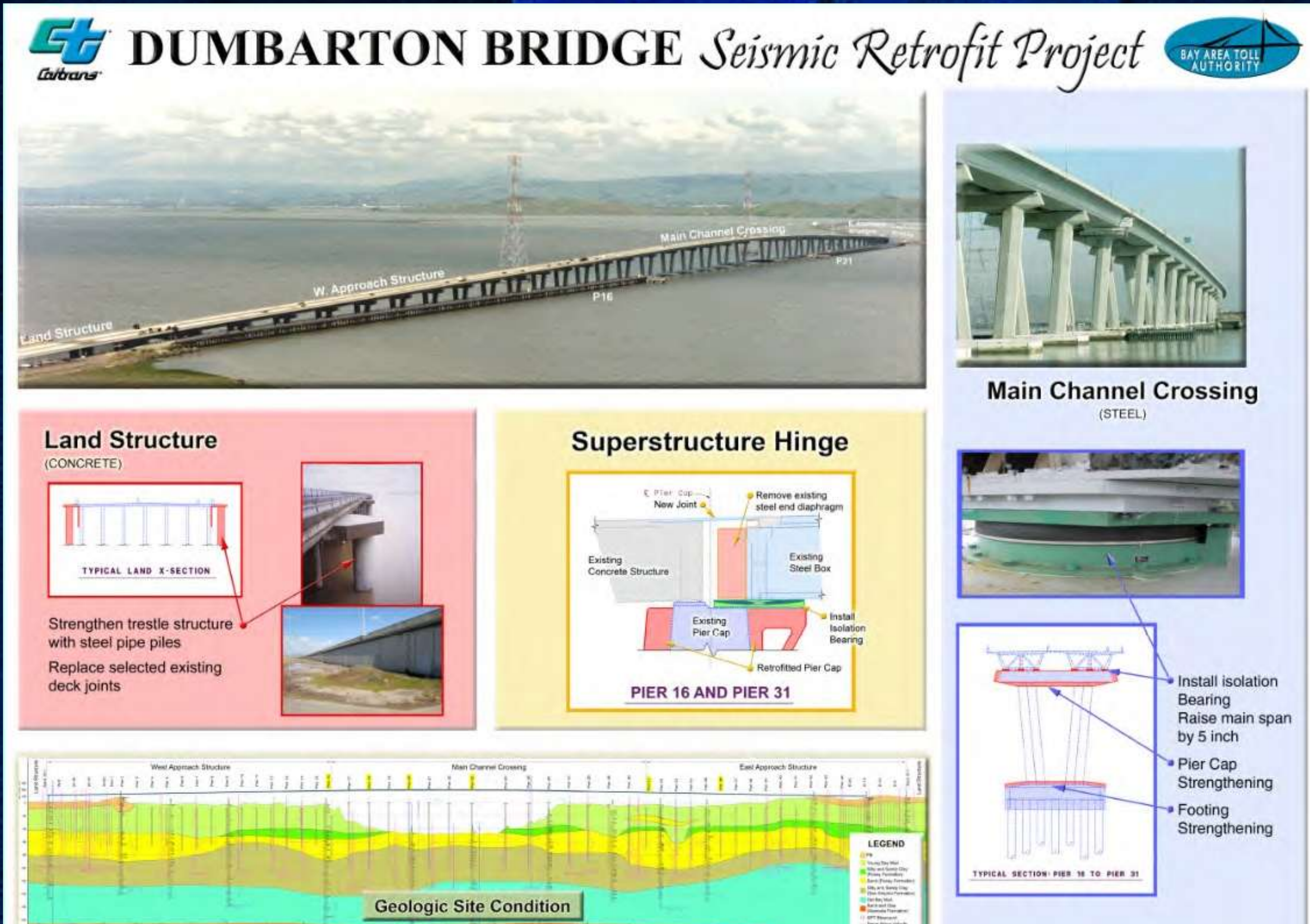
*- Lateral Design Displ.= 23 ''

*- Yield force/DL = 0.06W

*- Max. Lateral Force/DL = 0.16W

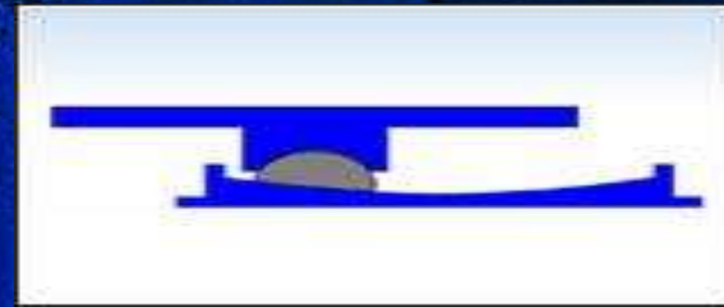
ANTIOCH BRIDGE SEISMIC RETROFIT

Staging Construction (Bearing Replacement)



DUMBARTON BRIDGE Seismic Retrofit Project
Caltrans
BAY AREA TOLL AUTHORITY
Land Structure
W. Approach Structure
Main Channel Crossing
P16
P21
Main Channel Crossing
(STEEL)
Land Structure
(CONCRETE)
TYPICAL LAND X-SECTION
Strengthen trestle structure with steel pipe piles
Replace selected existing deck joints
Superstructure Hinge
℄ Pier Cap
New Joint
Remove existing steel end diaphragm
Existing Concrete Structure
Existing Steel Box
Existing Pier Cap
Install Isolation Bearing
Retrofitted Pier Cap
PIER 16 AND PIER 31
Install isolation Bearing
Raise main span by 5 inch
Pier Cap Strengthening
Footing Strengthening
TYPICAL SECTION: PIER 16 TO PIER 31
West Approach Structure
Main Channel Crossing
East Approach Structure
LEGEND
Geologic Site Condition

Dumbarton Bridge FPS Bearing



*- Plan Dimensions= 100'' x 100''

*- Height = 9.5''

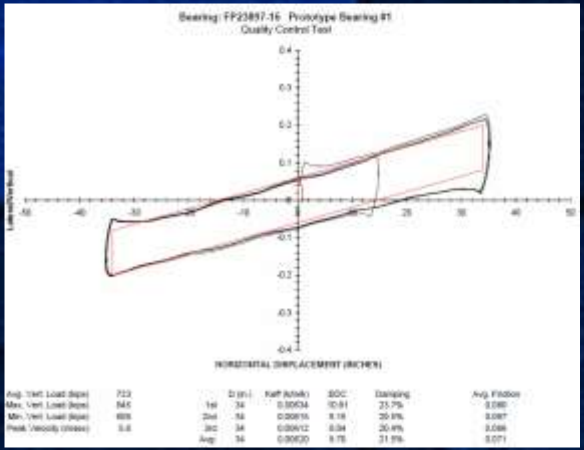
*- DL = 720 kips

*- Lateral Design Displ.= 34 ''

*- Yield force/DL = 0.06W

*- Max. Lateral Force/DL = 0.23W

Dumbarton Bridge Bearing, Testing



Dumbarton Bridge Low Height Friction Isolator Bearing

Shear Ring Failure to check ring capacity > bolts Capacity
if EQ displ. > Design Displ.

Minimal Visible Sheathing of
Bearing Liner Material

Benicia Martinez Bridge Approach Structutre

*- Plan Dimensions= 45'' x 45''

*- Height = 14''

*- DL = 620 kips

*- Lateral Design Displ.= 12 ''

*- Yield force/DL = 0.09W

*- Max. Lateral Force/DL = 0.33W

Benicia Martinez Bridge Main Span

*- Plan Dimensions= 139" x 139" one of the largest Single FPS bearing in sizes (12 ft x 12 ft) made in the World

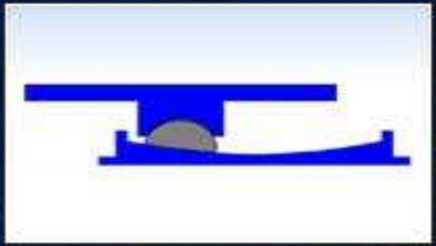
*- Height = 20.5"

*- DL = 3029 kips

*- Lateral Design Displ.= +-48.6 "

*- Yield force/DL = 0.06W

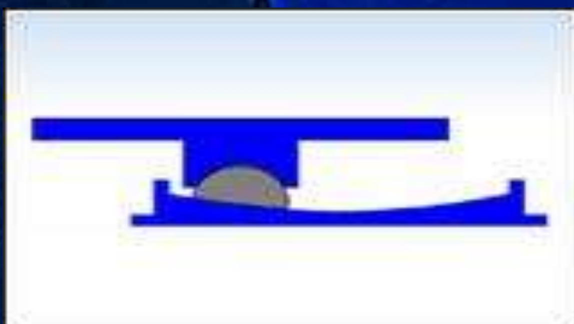
*- Max. Lateral Force/DL = 0.24W



Caltrans

Folsom Bridge Natomas Crossings

FPS bearings used to accommodate large pre-stressed shortening of the superstructure



(Courtesy of HDR, Inc.)

GOLDEN GATE BRIDGE NORTH AND SOUTH VIADUCT

*- Plan Dimensions= 35'' x 35''

*- Height = 18''

*- DL = 563 kips

*- Lateral Design Displ.= 15 ''

*- Yield force/DL = 0.09W

*- Max. Lateral Force/DL = 0.29W

Richmond San Rafael Bridge

RICHMOND SAN RAFAEL BRIDGE

Triple Lead Rubber Isolation Bearings

*- Plan Dimensions= 63'' x 58''

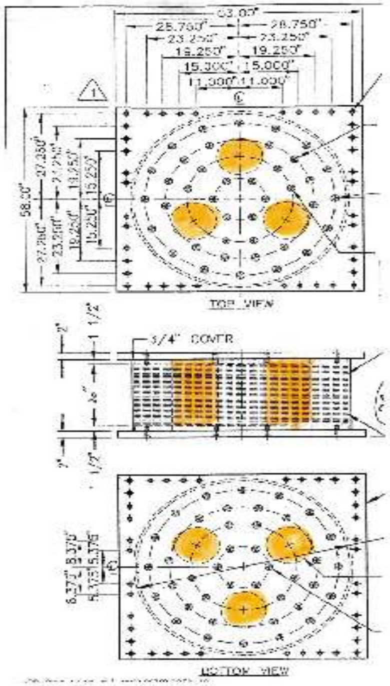
*- Height = 23.7''

*- DL = 1330 kips

*- Lateral Design Displ.= 18 ''

*- Yield force/DL = 0.22W

*- Max. Lateral Force/DL = 0.40W



WEST SPAN SAN FRANCISCO OAKLAND BAY BRIDGE

*- Unidirectional Friction Pendulum Sliding Bearing

*- Plan Dimensions= 96'' x 52''

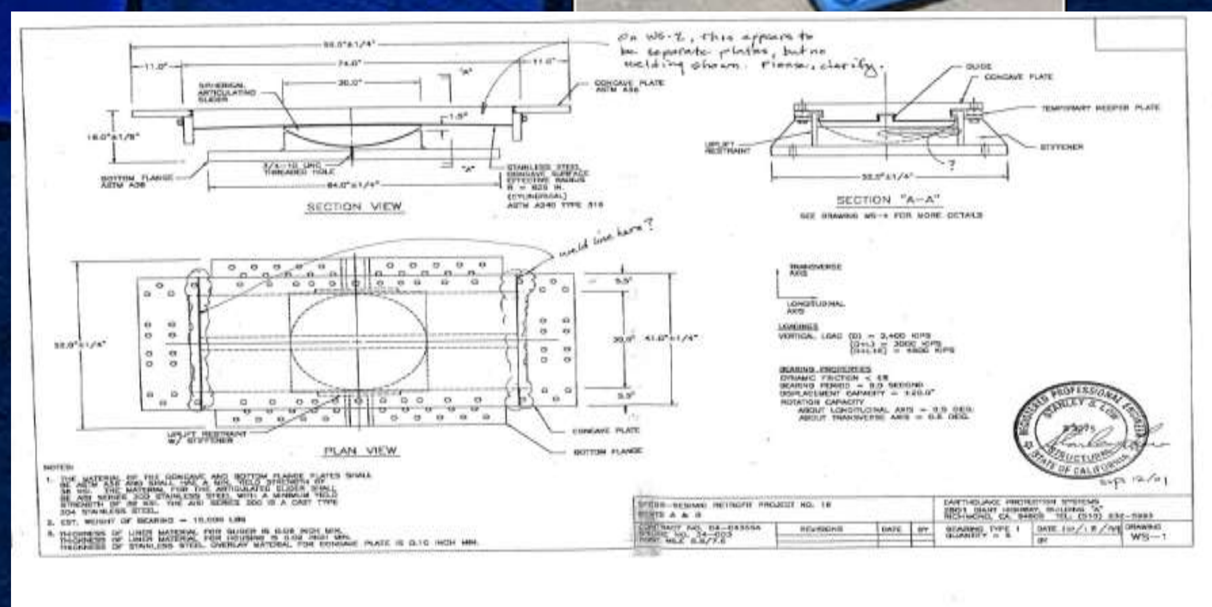
*- Height = 16''

*- DL = 2400kips

*- Lateral Design Displ.= 18

*- Yield force/DL = 0.06W

*- Max. Lateral Force/DL = 0.09W



Big Bear Bridge

Triple Friction Pendulum Sliding Bearing on top of Arch

*- Plan Dimensions= 66'' x 66''

*- Height = 22.5''

*- DL = 4600 kips

*- Lateral Design Displ.= 18 ''

*- Yield force/DL = 0.06W

*- Max. Lateral Force/DL = 0.12W

What is a Seismic Joint?

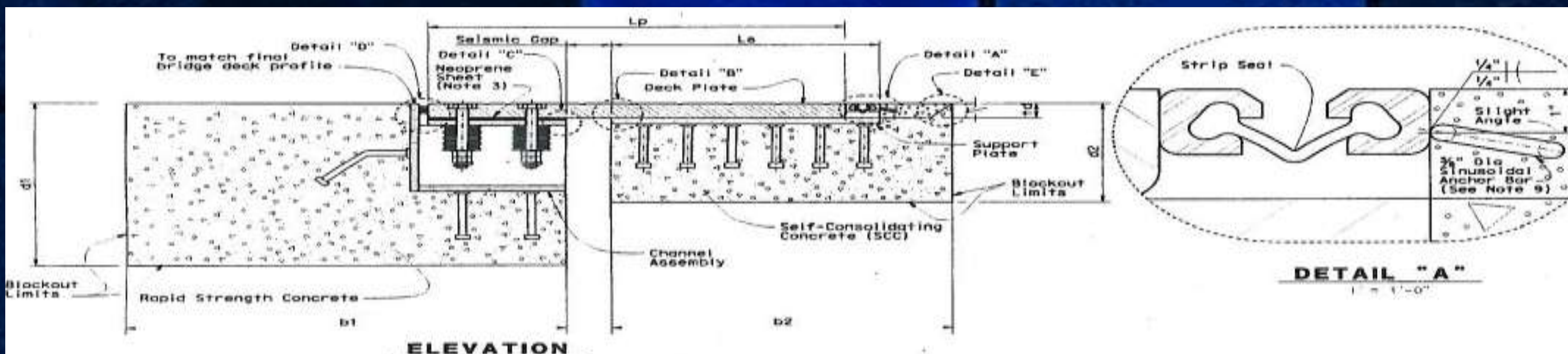
A Seismic Joint is an expansion joint that accommodates large movements in both service and seismic conditions and maintains its full functionality with no or minor damage right after a major seismic event

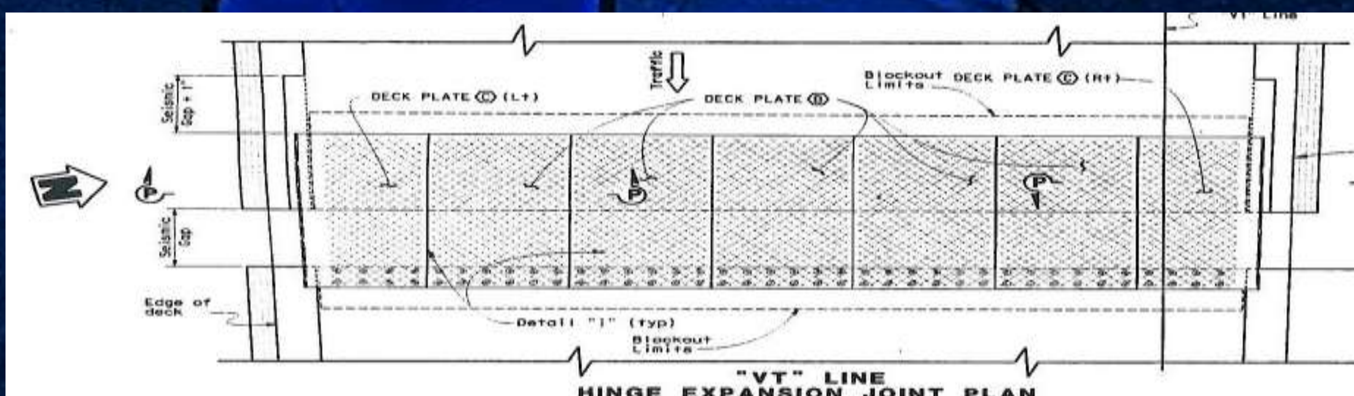
Where can Seismic Joints be used?

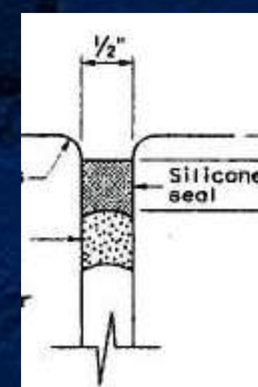
- At joints locations with large differential long. and/or transverse displacements
- In highly skewed or curved bridges
- At locations where traffic disruption due to joint damage is not acceptable
- When bridge frames must be structurally independent in seismic conditions
- On “important” bridges
- In seismic isolated bridges

Seismic Joint

Caltrans Seismic Joint Design



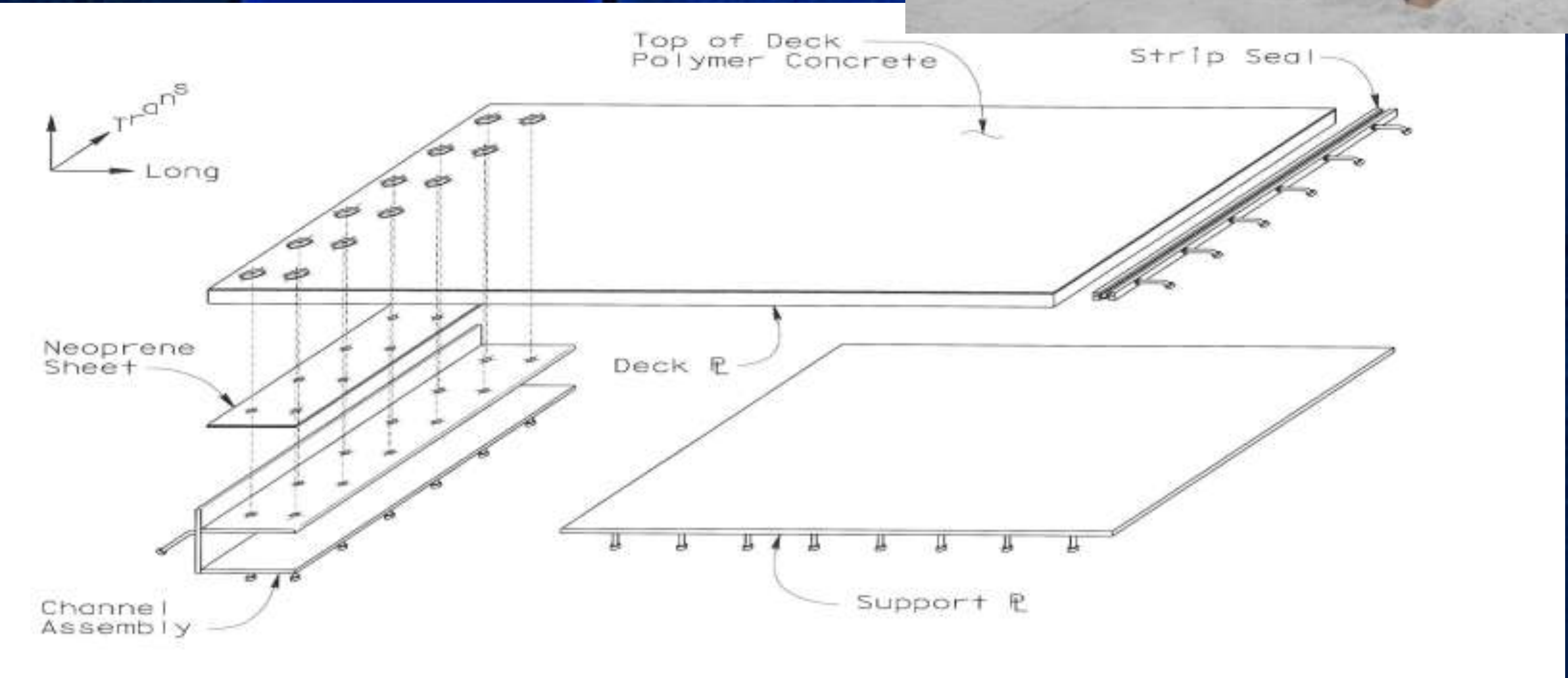




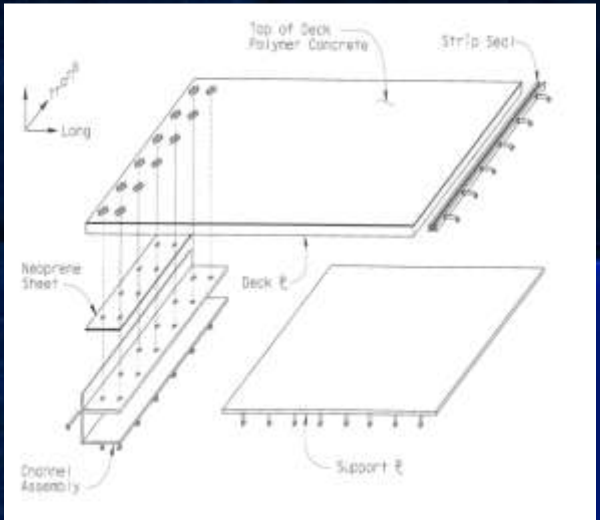
Caltrans Seismic Joint Type I

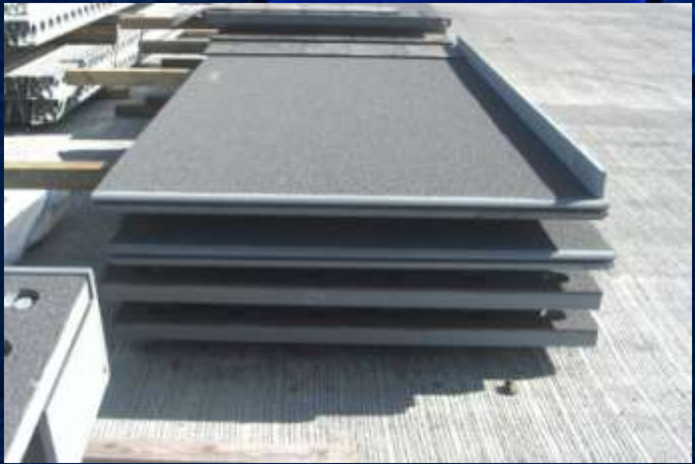
Half Channel

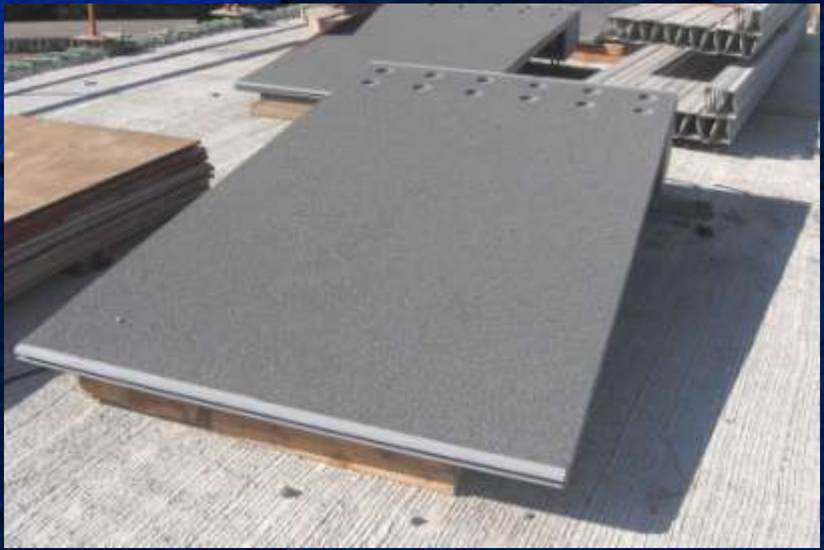
Max. Service MR=4”



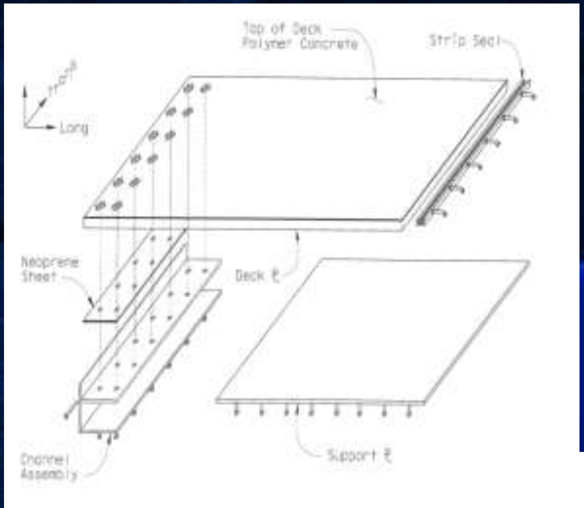
Deck Plate

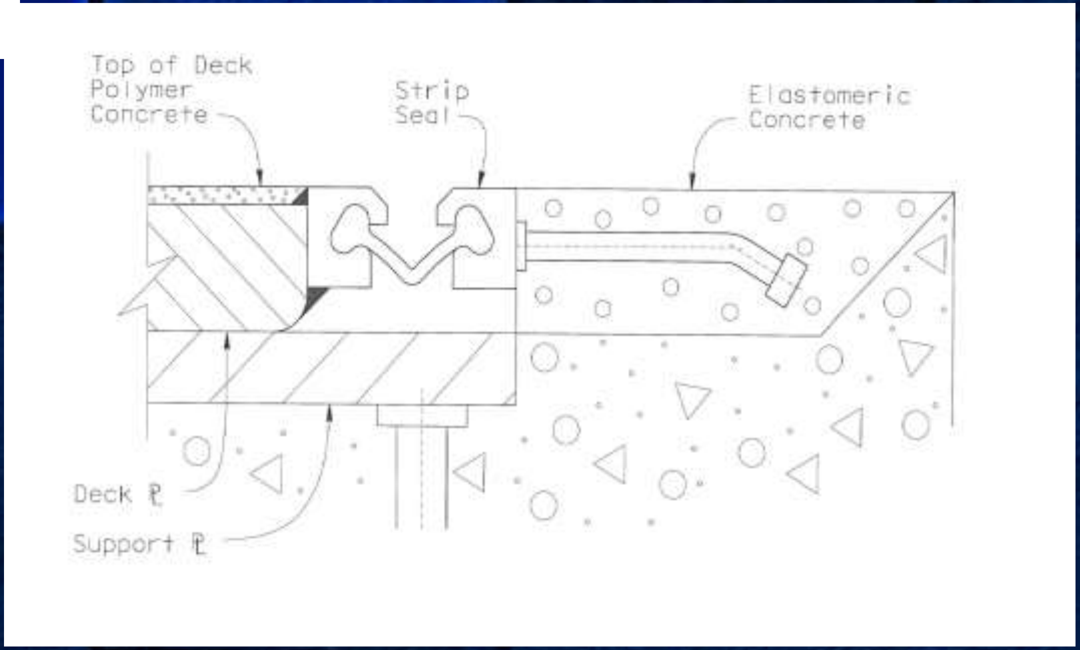

Top of Deck
Polymer Concrete
Strip Seal
Long
Neoprene Sheet
Deck ℄
Channel Assembly
Support ℄



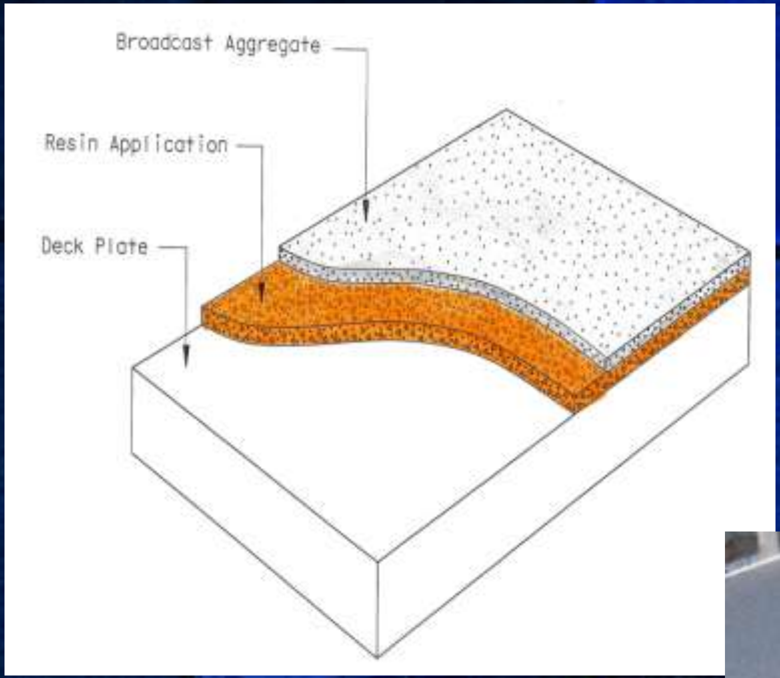


Strip Seal Max. Service MR=4 “

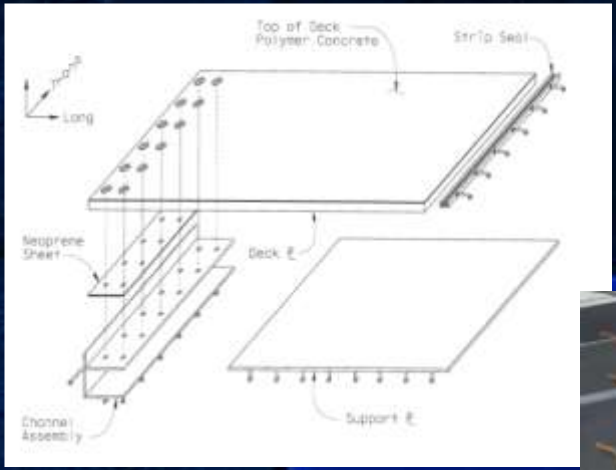


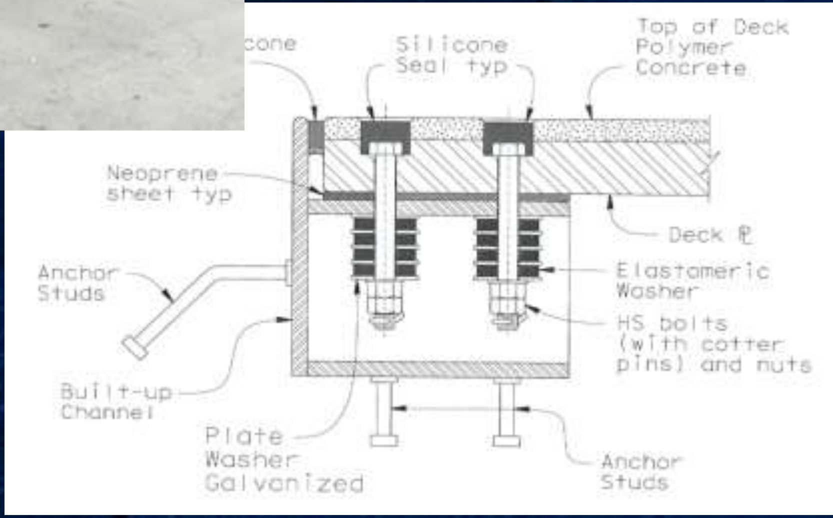


Polymer Concrete Overlay



Channel Assembly





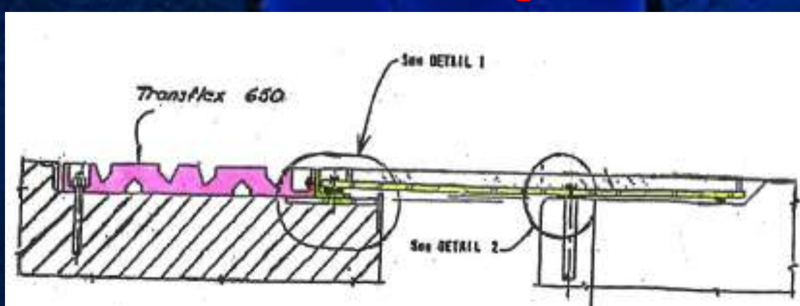
Bridges with Seismic Joints

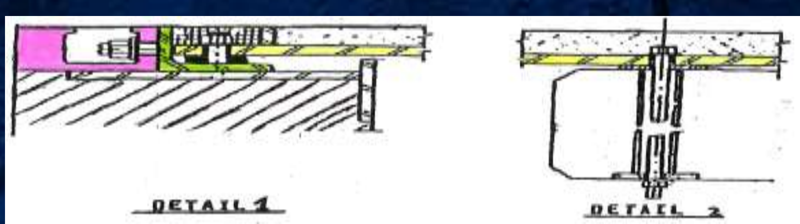
Several versions of this joint system has been installed or is under construction in various California bridges:

- Benicia-Martinez
- Rio Hondo Busway
- West Approach SFOBB
- Oakland Approach SFOBB
- Dumbarton Br.
- Presidio Viaduct
- San Mateo-Hayward Br.
- Schuyler Heim Br.

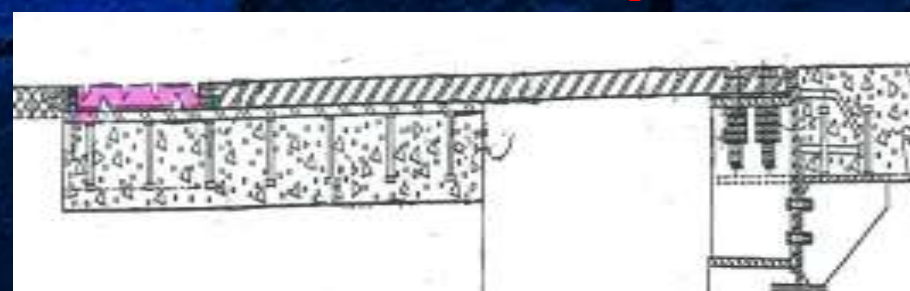
Dumbarton Bridge

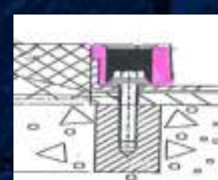
1978 Design

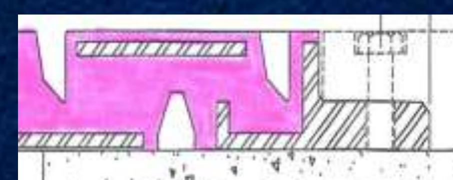




2010 Design



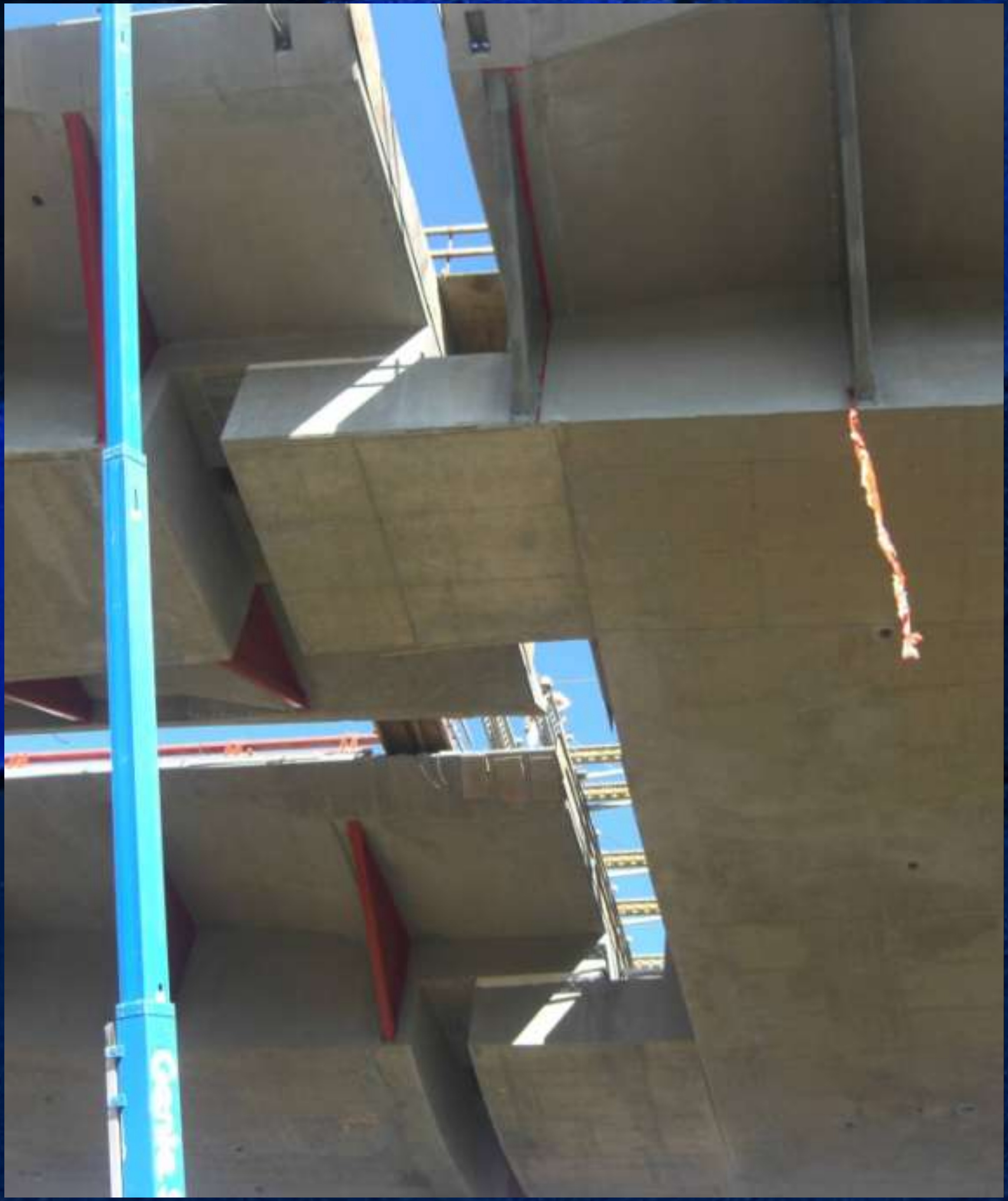




Presidio Viaduct

Seismic Joint Information

	THERMAL (in)	SEISMIC (in)	DECK PL Length (in)	DECK PL Thickness (in)	SEISMIC GAP @ 70ºF
ABUT 1	±1	±11	51	2.5	12
ABUT 7	±1	±5	56	2.5	6
HINGE	±2	±40	112	2.5	42



Energy Dissipation Devices
Viscous Dampers

- Prequalified list of suppliers.

- Prototype Test:
  - For every type of viscous dampers two full scale prototype dampers are manufactured and tested.
  - Stroke verification
  - Dynamic tests include: wind loads, five fully reversed cycles of sinusoidal loading at various increments (0.25, 0.5, 0.75, 1.0, 1.5) of the peak design velocity.
    - Measured peak forces shall be within 15% of the target values.

Caltrans

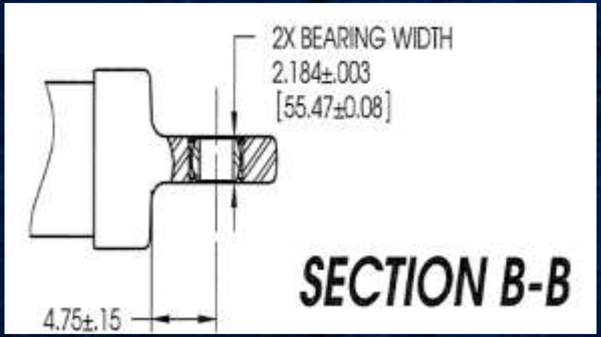
Energy Dissipation Devices
Viscous Dampers

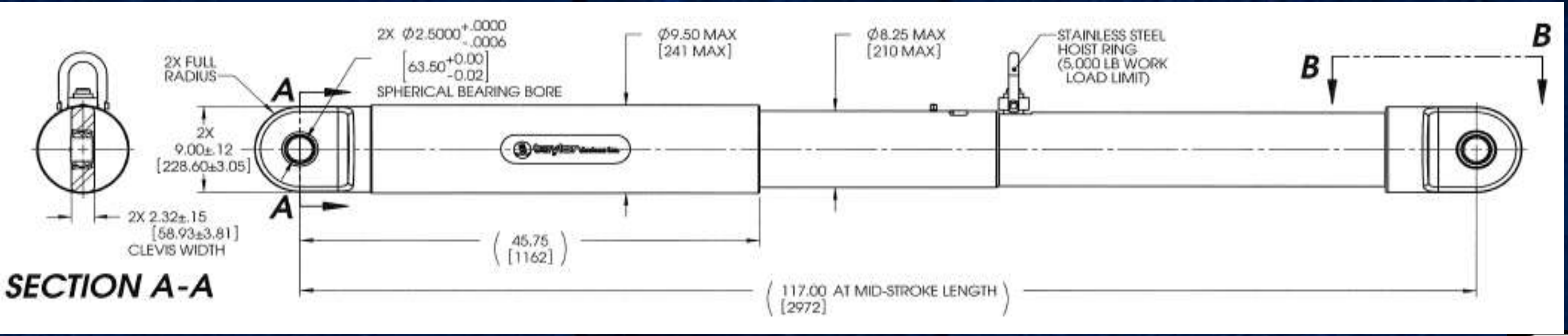
- Proof Test: All viscous dampers to be placed on the bridge are proof tested.
  - Proof Cyclic Test: three fully reversed cycles of sinusoidal loading at velocity increments of 0.20, 0.50 & 0.75 times the peak design velocity shown on the plans.
  - Proof Pressure test.
  - Stroke Verification test.
  - Full Velocity and Stroke test: five fully reversed cycles of sinusoidal loading through the total design stroke and achieve the peak design velocity shown on the plans.
    - Measured peak forces shall be within 15% of the target values.

Viscous Dampers

To minimize seal wear:

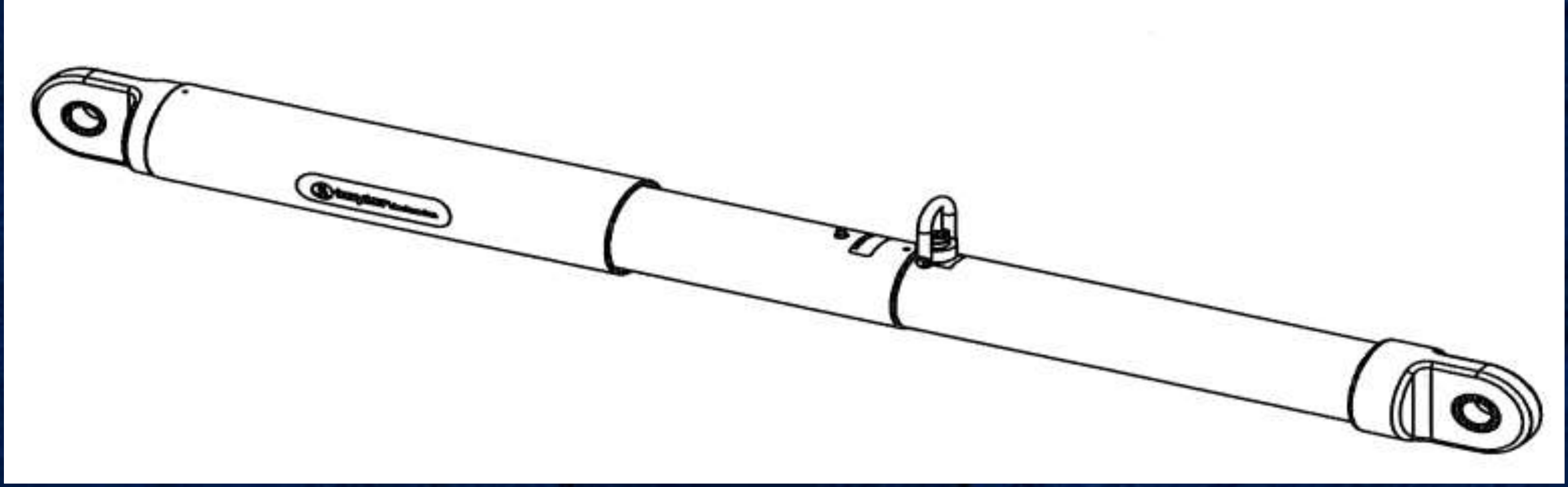
I) reduce total piston travel, by disengaging dampers from service live load movements. Provide adequate clearance (±1/4” to ±1/2”) at the clevis-to-pin connection





Viscous Dampers

ii) reduce pressure on the seals by providing central damper support



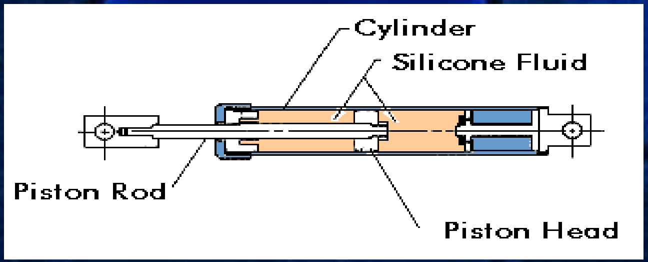
Viscous Dampers

- For long corrosion resistance, use a 3-part paint system
  - Zinc-rich primer
  - Epoxy intermediate coat
  - Paint shield
- Maintenance crews shall be able to measure internal fluid pressure from side valve and refill device if needed

Bridges w/ Viscous Dampers

- SFOBB/ Bay Br.
- Richmond San Rafael Br.
- Vincent Thomas Br.
- Coronado Br.
- Rio Vista Br.
- 91-5 HOV Connector Separation Br.
- Santiago Br.

91/5 HOV Connector Separation



Stroke = 16"
Max. Force =250 kips @ 42 in/sec
F=CVexp(0.3)
Total 8 units

Santiago Bridge

Stroke = 30”
Max. Force =160 kips
F=CVexp(0.4)

WEST SPAN SAN FRANCISCO OACKLAND BAY BRIDGE

Three types of Viscous Dampers

Stroke
***-Type A = 18”**
***-Type B = 6”**
***-Type C = 16”**

Axial Capacity
***-Force = 450 kips**

***-Force = 650 kips**

***-Force = 550 kips**

RICHMOND SAN RAFAEL BRIDGE

Two type of Viscous Dampers were used

Stroke

*-Type A = 20’’

*-Type B = 38’’

Axial Capacity

*Type A = 500 kips tower

*-Type B = 225kips Side Span at tower

Caltrans

VINCENT THOMAS BRIDGE

Dampers installed at West and East Towers deck connections

Stroke

*- Type A =52'' Main Span at tower

*- Type B = 44'' Side Span at tower

*- Type C = 16'' Side Span at truss

Axial Capacity

*-Type A = 200 kips Main Span at tower

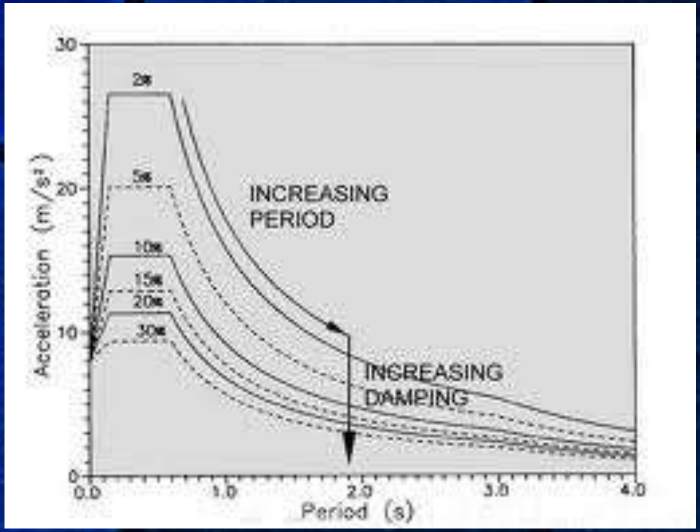
*-Type B = 80 kips Side Span at tower

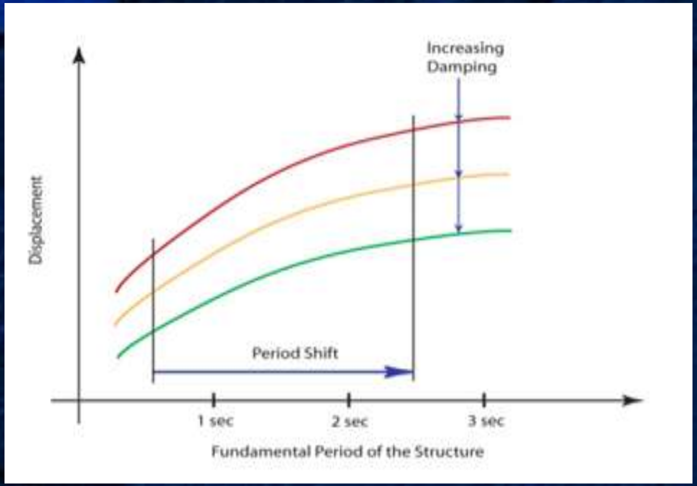
*- Type C= 250 kips Side Span at truss

Thank you for your time

AASHTO SEISMIC ISOLATION DESIGN CRITERIA

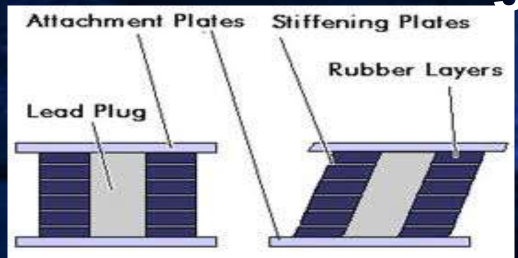
- The Basic elements in seismic isolation systems are:
- 1- Vertical-load carrying device
- 2- lateral flexibility so that the period of vibration of the total system is lengthened sufficiently to reduce the force response
- 3- A damper or energy dissipator so that relative deflections across the flexible mounting can be limited to a practical design level.
- 4- A mean of providing rigidity under low service load level such as wind and braking forces.
- 5- Lateral Restoring Force

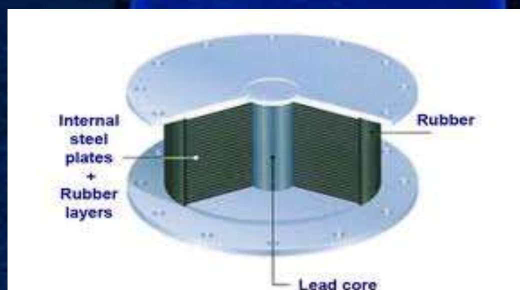




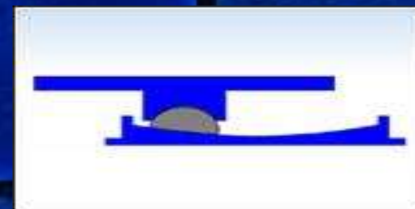
Seismic Isolation Types used by Caltrans

Lead Rubber Bearings



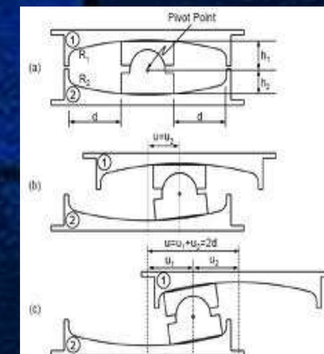


Friction Pendulum Sliding Bearing



Single

Triple



Doubl
e

LRB: *-Yielding of Lead Plug provides energy dissipation (Damping)
*- Rubber Layers provide lateral flexibility and restoring force
*- Stiffening plates increase vertical rigidity

FPS: *-Friction provides energy dissipation (Damping)
*-Sliding provides flexibility
*-Dish curvature promotes restoring force
*-Steel plates provide vertical rigidity

Caltrans

Applications & Performance of Full-Scale Devices

Ian Aiken, Ph.D., P.E.
Principal, Seismic Isolation Engineering, Inc.
Emeryville, California

Joint CSRN-NEES Workshop on the Seismic Isolation and Damping of Bridge Structures
University of British Columbia
April 30, 2012

CSRN RCRP Canadian Seismic Research Network
Réseau canadien pour la recherche parasismique

Funded by NSERC / Subventionné par le CRSNG

NEES

Outline

- San Francisco-Oakland Bay Bridge, Detour Tie-In
- Recent full-scale dynamic testing
  - Test facilities capabilities/limitations
  - Defining demand
  - Analytical modeling
- Foresthill Road Bridge Seismic Retrofit
- Great East Japan Earthquake

2

San Francisco-Oakland Bay Bridge Detour Tie-In

- Known as the RORI (roll-out, roll-in) Project
- Move traffic for 4-5 years to allow final construction of the SAS span West end alignment into the Yerba Buena tunnel
- Tie-in completed over Labor Weekend, Sept. 2009
- Existing segment, double-deck, 300-ft long, 3200t
- New segment, 3600t
- All work 150+ ft above ground

3

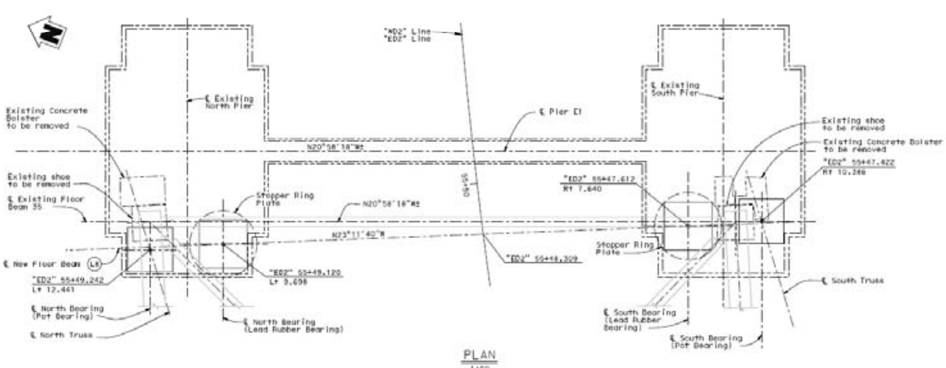
San Francisco-Oakland Bay Bridge Detour Tie-In

4

Detour Tie-In Structure

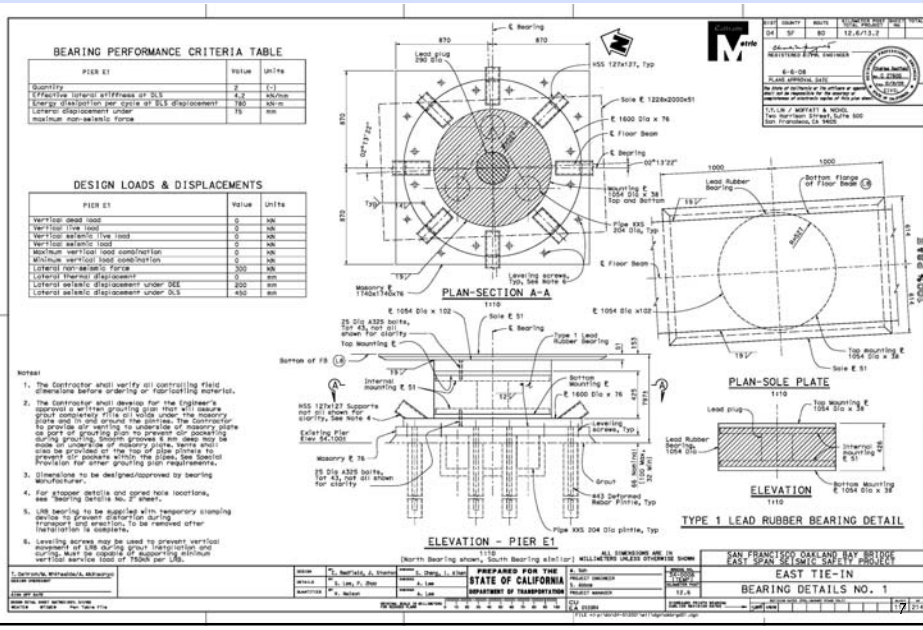
5

Pier E1 Plan, Bearing Locations

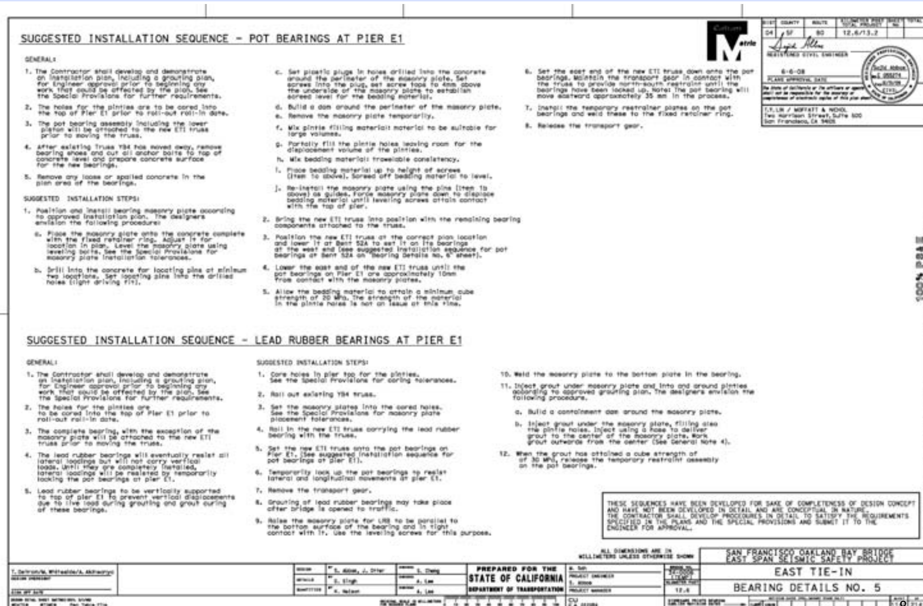


6

Bearing Details



Bearing Details, Suggested Installation Sequence



San Francisco-Oakland Bay Bridge Detour Tie-In

9

San Francisco-Oakland Bay Bridge Detour Tie-In

Pier E1 North Bearing Locations

11

Lead-Rubber and Pot-Slider Bearings, Pier E1 North

12

Elastomeric Bearing Back-Up System

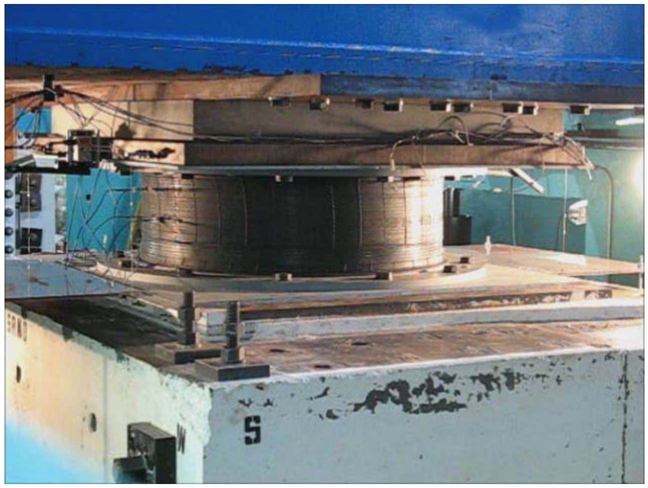
13

Large Displacement, Multi-Axis Dynamic Testing

40 in. dia lead-rubber bearing, 3150 kips, max displacement 21.3 in.
Input: calculated structure response for scaled 1999 Kocaeli (Izmet EQ) records

14

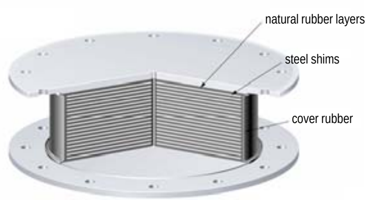
Large Displacement, Multi-Axis Dynamic Testing

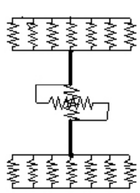


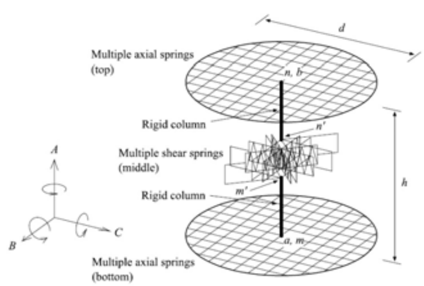
1.3m dia HDR bearing, 51 MN (11,500 kips), max X and Y displacements approx 70 cm
Input: calculated structure response for 2003 Tomakomai (Tokachi-oki, EQ) records

15

Advanced 3-Dimensional Modeling of Bearings



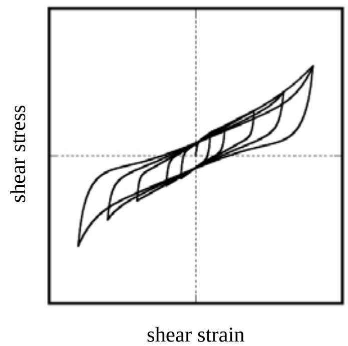




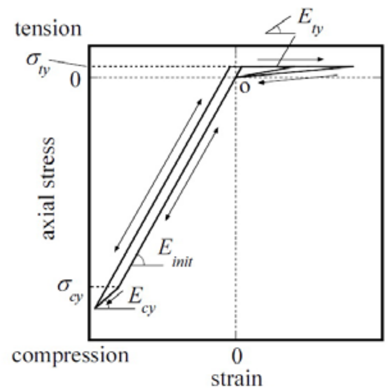
16

Advanced 3-Dimensional Modeling of Bearings

Shear Force-Deformation



Axial Force-Deformation



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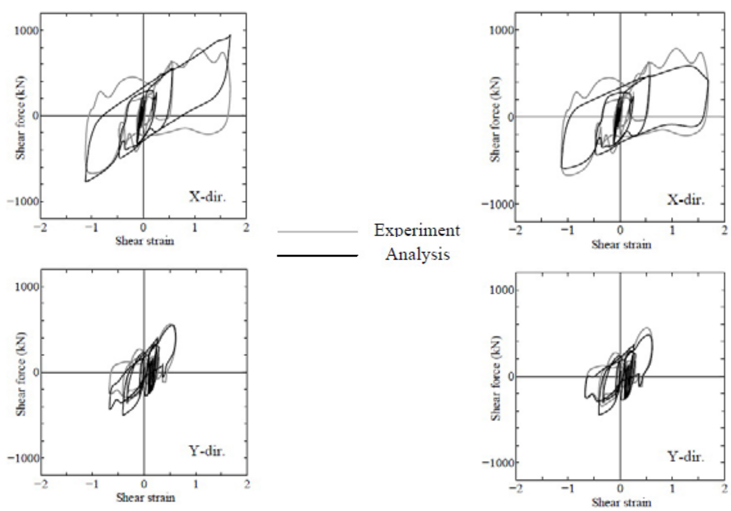
Advanced 3-Dimensional Modeling of Bearings

Case 1
(no P-Δ, uniform E_c)

Case 2
(P-Δ, Bessel function E_c)



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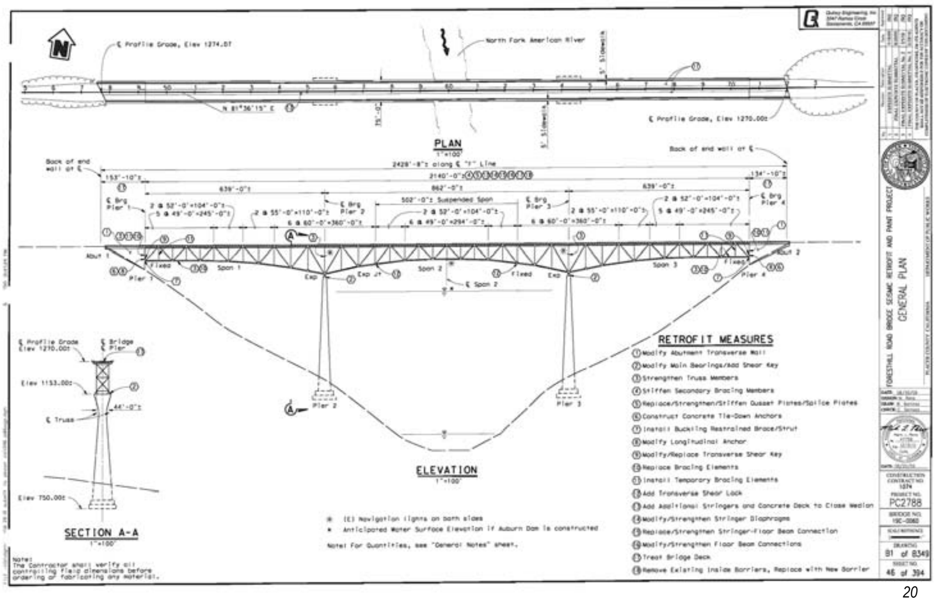
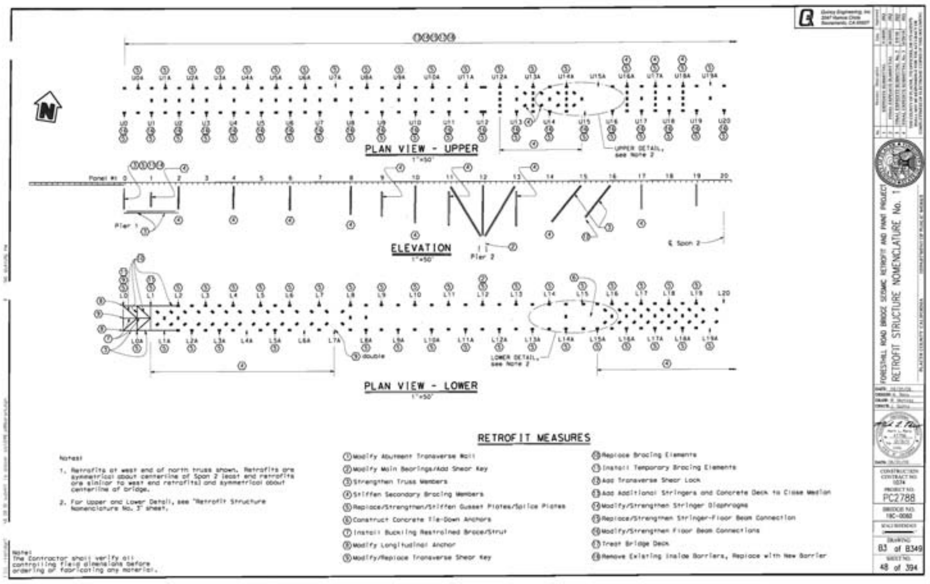
Foresthill Road Bridge Seismic Retrofit

Foresthill Road Bridge Seismic Retrofit



Foresthill Road Bridge Seismic Retrofit



21

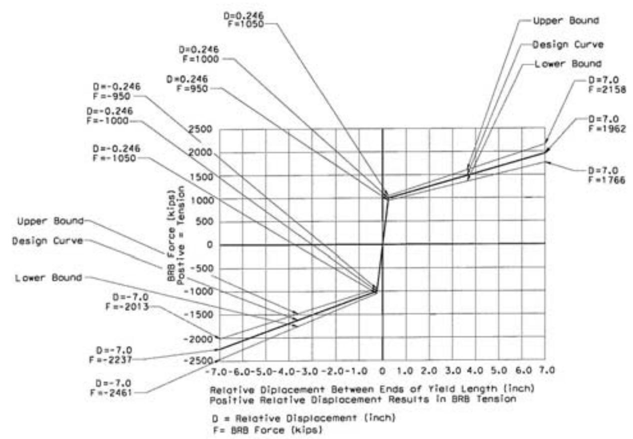
North Abutment

22

Buckling-Restrained Brace

23

Brace Performance Criteria



General Notes:

1. Buckling Restrained Braces (BRB) shall be constructed with the following maximum dimensions:
   Length = L = 276 inch
   Width = W =16 inch
   Height in Elevation = H = 22 inch

2. BRB shall connect to the plates shown in sheets "New Horizontal Bracing Members (Details No.7)", "New Horizontal Bracing Members (Details No. 8)" and "Buckling Restrained Brace (Anchor Connection Details) sheets. Connections may be bolted or pinned and shall be provided with the BRB. Connections shall be designed by the manufacturer and shall be submitted for review by the Engineer.

3. BRB Force-Displacement characteristics of the brace within the yield length shall fit within the envelope of the BRB Characteristic Curve. The yield length of the BRB shall be 171 inch.

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Test Plan

Table 4.1: Foresthill Displacement Loading Protocol

Test	Cycles	Target Deformation	Target Strain	Elongation (in)	Horizontal Rotation	Vertical Rotation	X (in)	Y (in)	Z (in)	CPD
a	2	0.5 Δ_{by}	0.07%	0.12	0.02%	0.01%	0.12	0.05	0.03	0
b	2	0.75 Δ_{by}	0.11%	0.19	0.03%	0.02%	0.19	0.07	0.04	0
1	2	Δ_{by}	0.14%	0.25	0.04%	0.02%	0.25	0.10	0.06	0
2	2	0.5 Δ_{bm}	1.00%	1.71	0.25%	0.15%	1.71	0.69	0.41	49
3	2	1.0 Λ_{bm}	2.00%	3.42	0.50%	0.30%	3.42	1.38	0.83	156
4	2	1.25 Δ_{bm}	2.50%	4.28	0.63%	0.38%	4.28	1.73	1.04	291
5	2	1.5 Δ_{bm}	3.00%	5.13	0.75%	0.45%	5.14	2.07	1.24	454
6	--	1.0 Δ_{bm}	2.00%	3.42	0.50%	0.30%	3.42	1.38	0.83	TBD

Notes: Tests 'a' and 'b' are preliminary tests prior to conducting specified loading
Horizontal rotation is brace end rotation about the vertical axis
Vertical rotation is brace end rotation about the horizontal axis

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Brace Testing, UC San Diego

Results, Tests 1 – 5

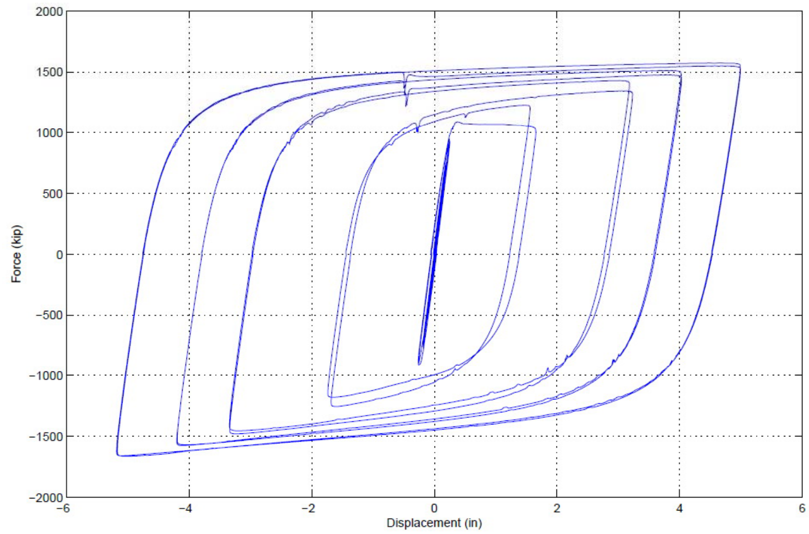

Figure 5.3 Force-Displacement Loops for Specimen UBB-T1-2, Test 1-5

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Results, Test 6, Additional Cycles

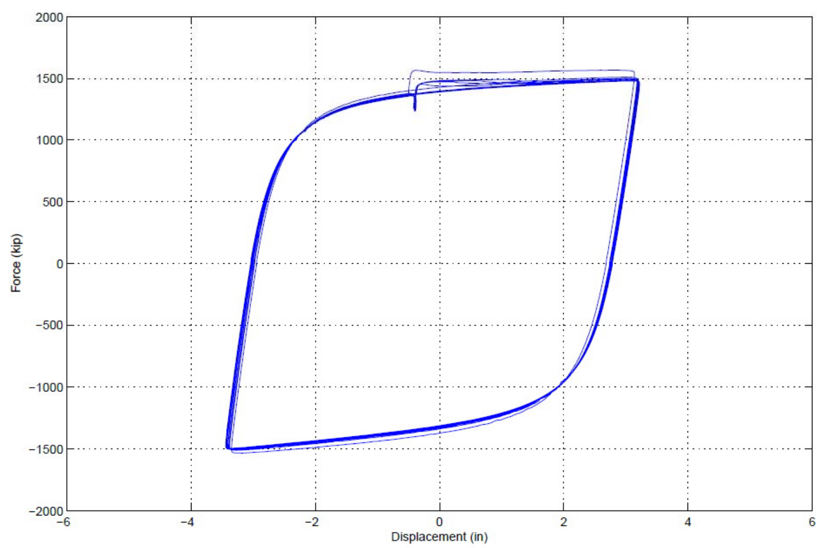

Figure 5.4 Force-Displacement Loops for Specimen UBB-T1-2, Test 6, 10 cycles

28

Test Results

Table 5.2: Maximum Forces for Specimen UBB-T1-1, Tests 1-5

Test	Cycles	Target	Target Strain	T max (kip)	C max (kip)	β	ω	βω
1	2	Δ_{by}	0.14%	951	897	--	--	--
2	2	0.5 Δ_{bm}	1.00%	1240	1268	1.02	1.16	1.19
3	2	1.0 Δ_{bm}	2.00%	1441	1489	1.03	1.35	1.40
4	2	1.25 Δ_{bm}	2.50%	1516	1575	1.04	1.42	1.48
5	2	1.5 Δ_{bm}	3.00%	1576	1661	1.05	1.48	1.56

Table 5.3: Maximum Forces for Specimen UBB-T1-2, Tests 1-5

Test	Cycles	Target	Target Strain	T max (kip)	C max (kip)	β	ω	βω
1	2	Δ_{by}	0.14%	952	897	--	--	--
2	2	0.5 Δ_{bm}	1.00%	1227	1257	1.02	1.15	1.18
3	2	1.0 Δ_{bm}	2.00%	1428	1481	1.04	1.34	1.39
4	2	1.25 Δ_{bm}	2.50%	1512	1577	1.04	1.42	1.48
5	2	1.5 Δ_{bm}	3.00%	1573	1661	1.06	1.48	1.56

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Test Acceptance Criteria – AISC 341

6.1 AISC 341-2005 Criteria

2. The plot showing the applied load vs. displacement history shall exhibit stable, repeatable behavior with positive incremental stiffness.

   Satisfied. From Figures 5.1 through 5.4, it is seen that both braces exhibited stable, repeatable behavior with positive incremental stiffness.

3. There shall be no fracture, brace instability or brace end connection failure.

   Satisfied. No fracture, brace instability or connection failure was observed in either specimen, for Tests 1–5 or Test 6.

4. In each cycle to a deformation greater than the yield deformation, Δ_{by}, the maximum tension and compression forces shall not be less than the yield force of the brace core plate, P_y.

   Satisfied. See Figures 5.1 through 5.4.

5. In each cycle to a deformation greater than the yield displacement, Δ_{by}, the ratio of the maximum compression force to the maximum tension force shall not exceed 1.3.

   Satisfied. See Tables 5.2 and 5.3.

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Test Acceptance Criteria – Project-Specific

6.6 Foresthill Road Bridge Seismic Retrofit and Paint Project Special Provisions

7. The two prototype brace test specimens shall display characteristics within 5% of each other.

   Satisified: See Table 5.3. The maximum difference in the peak tension or compression force values between the two test specimens for Tests 1–5 is 1.0%.

8. Test results shall display force-displacement characteristics within the upper and lower bound envelope as shown on the plans.

   Satisfied: See Figure 6.1 which shows the complete set of hysteresis loops for Specimen UBB-T1-1 and superimposes (in black) the lines of the upper and lower bound force envelopes (the loops for only one specimen are shown, since the results for the two specimens are essentially identical). It can be seen that the maximum brace force for each test amplitude is within the force bound range.

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Foresthill Road Bridge Seismic Retrofit

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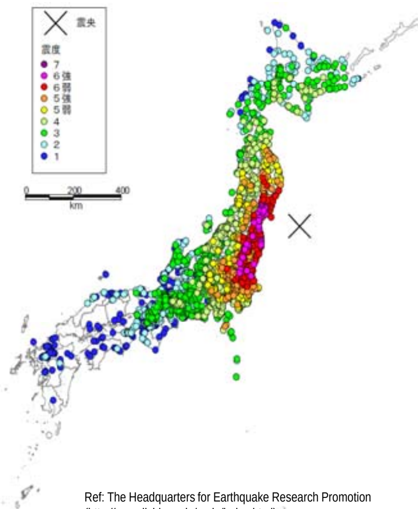
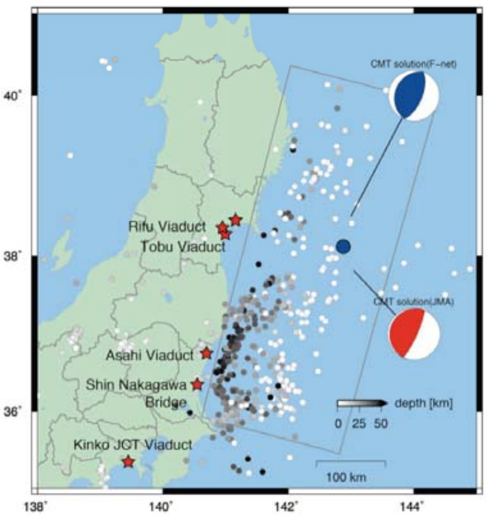
Great East Japan Earthquake, M9.0, 3/11/11



Ref: The Headquarters for Earthquake Research Promotion
(http://www.jishin.go.jp/main/index.html)



33

Great East Japan Earthquake, M9.0, 3/11/11

- Immediately after the main shock, MLIT conducted safety inspections of 1,350 bridges (JMA 5+)
- 815 suffered some form of damage; no bridges severely damaged by earthquake ground motions
- 140 bridges affected by tsunami, 5 washed away
- Typical damage: span unseating, foundation scour, failure of bearings, column shear failures, approach fill settlement
- Some failures of elastomeric bearings

Refs. Takahashi; Buckle et al., JAEE Conference, March 2012

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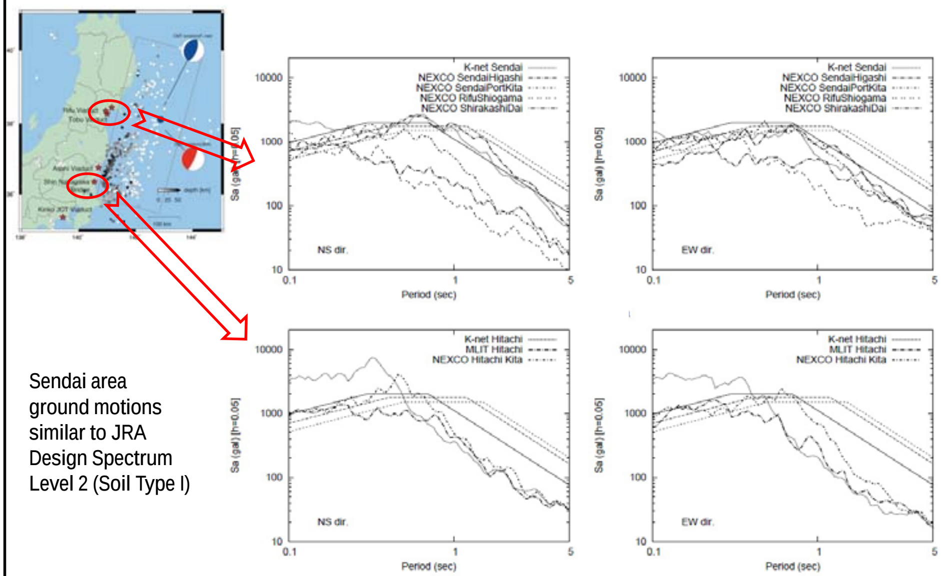
Great East Japan Earthquake, M9.0, 3/11/11



Sendai area ground motions similar to JRA Design Spectrum Level 2 (Soil Type I)

Ref. Takahashi, JAEE Conference, March 2012

35

Tobu Viaduct, Sendai

- 4.4 km long viaduct, damage mostly confined to a 10-span section
- New on- and off-ramps under construction, piers from single-column to two-column (between piers 54 and 56)
- Structure changed from 3/4/5 steel box girders to 8 steel plate girders
- Elastomeric bearings with external stoppers to restrain transverse movement

Ref. Buckle et al., JAEE Conference, March 2012

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Tobu Viaduct, Sendai

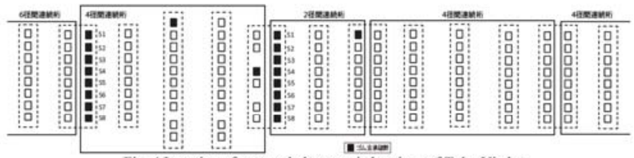


Fig. 4 Location of ruptured elastomeric bearings of Tobu Viaduct

Ref. Takahashi, JAEE Conference, Mar/2012

37

Bridges with Dampers

38

Thank You

Questions?

CSRN RCRP Canadian Seismic Research Network
Réseau canadien pour la recherche parasismique

Funded by NSERC / Subventionné par le CRSNG

NEES

Bridge Seismic Isolation

Steve Zhu, Ph.D., P.Eng., P.E.

Executive Engineer

Buckland & Taylor Ltd.

BUCKLAND & TAYLOR LTD.
a COWI company

Seismic Isolation Benefits

- Reduction in Seismic Force Effects on Substructures
- Savings in Bridge Initial Construction Costs
- Reduction in Seismic Damage to Substructures
- Quick Restoration of Post-Earthquake Service
- Savings in Post-Earthquake Repair Costs
- Optimum Distributions of Seismic Force Effects to Different Piers and Abutments
- Robust Response Behaviour to Larger Seismic Events

Seismic Isolation Principals

Period Elongation (Flexibility)

- Reduce Seismic Force Effects
- Increase Relative Displacements
- Elastomeric Bearings
- Sliding Bearings

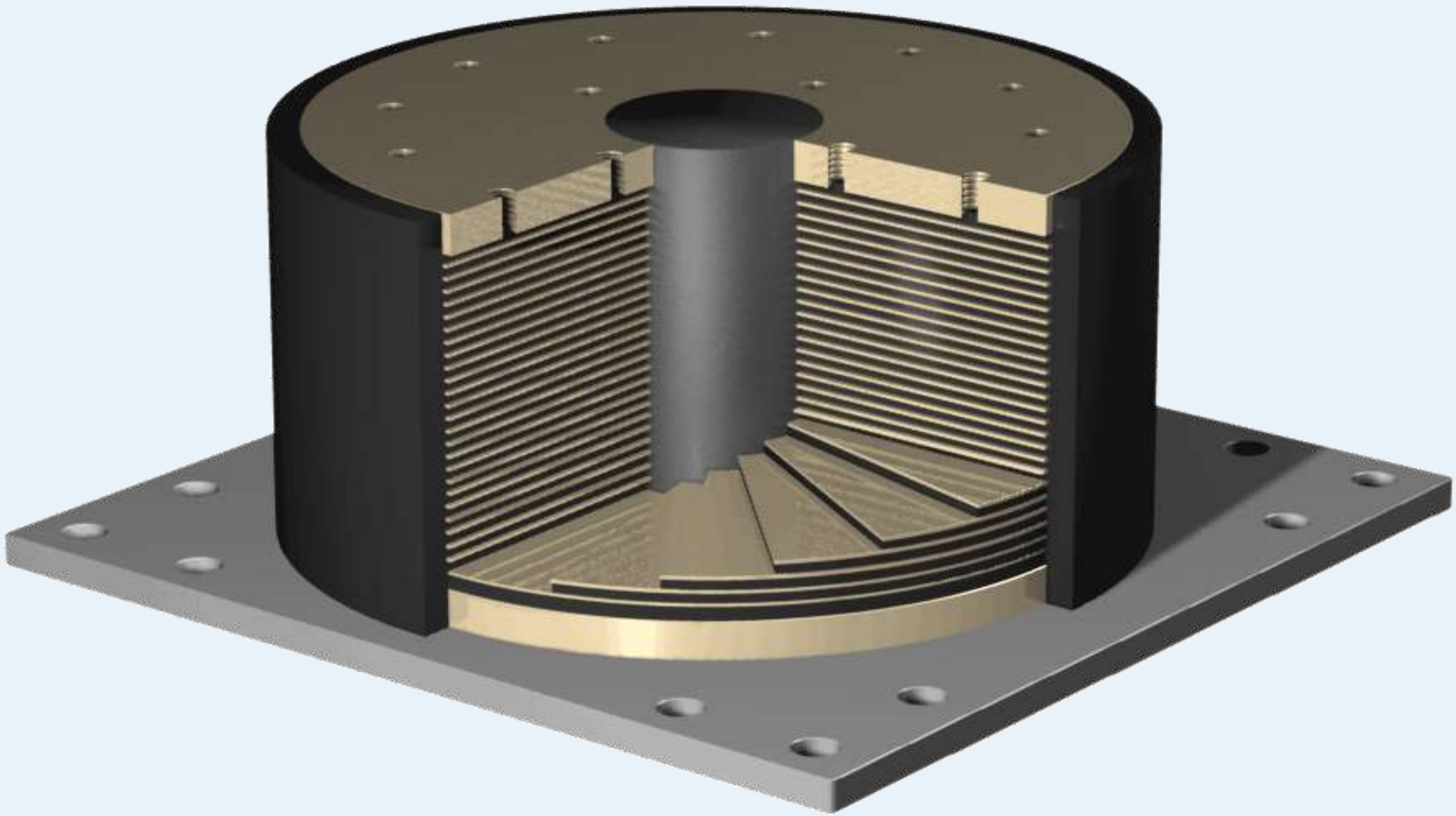
Energy Dissipation

- Reduce Relative Displacements
- Hysteretic Energy Dissipation
- Viscous Energy Dissipation
- Friction Energy Dissipation

Seismic Isolation Systems

- Elastomeric Bearings with a Lead Core
- High Damping Rubber Bearings
- Friction Pendulum Bearings
- Sliding Bearings with Supplementary Dampers
- Viscous Dampers
- Hysteretic Dampers
- Friction Dampers
- Viscoelastic Dampers
- Shape Memory Alloy Dampers

Elastomeric Bearing with a Lead Core



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a COWI company

Friction Pendulum Bearings

Friction Pendulum Bearings

Fluid Viscous Dampers

Main Considerations

- **Provide Lateral Rigidity under Service Lateral Loads (e.g. Wind, Braking, Centrifugal Loads)**

  - Lead Core

  - Friction

  - Dampers

  - Fuses

- **Provide Lateral Restoring Force during Seismic Response**

  - Elastomer

  - Polyurethane Spring

  - Pendulum Principal

- **Ensure System Stability during Seismic Response**

  - Analysis

  - Testing

Design Standards

- 1991 AASHTO Guide Specifications
- 1999 AASHTO Guide Specifications
- 2010 AASHTO Guide Specifications
- CAN/CSA-S6-06 CHBDC (Similar to 1991 AASHTO Guide Specifications)
- BC MoT Supplement to CAN/CSA-S6-06 CHBDC

Design Standards

CAN/CSA-S6-06 CHBDC

- More Stringent Shear Strain Requirements for Elastomeric Bearings
- Less Stringent Testing Requirements

2010 AASHTO Guide Specifications

- Less Stringent Shear Strain Requirements for Elastomeric Bearings
- More Stringent Testing Requirements
- Some Guidelines on Sliding Bearings
- Some Guidelines on System Modification Factors

BC MoT Supplement to CAN/CSA-S6-06 CHBDC

- Refer to 1999 AASHTO Guide Specifications for Testing Requirements
- Seismic Displacement plus 40% of Thermal Displacement
- Site Specific Study for Seismic Isolation on Type IV Soils

Design Earthquakes

- 475 Year Design Earthquake
- 975 Year Design Earthquake
- 2475 Year Design Earthquake
- Subduction Event (Long Duration of Strong Shaking)
- Elastic Seismic Response Coefficient in Code vs. Uniform Hazard Spectra

Analysis Methods

- **Multi-Mode Response Spectral Analysis**

  - Reduced Effective Stiffness
  - Increased Equivalent Viscous Damping
  - Iterative
  - Most Applications

- **Nonlinear Time History Analysis**

  - Nonlinear Behaviour of Isolation System
  - Important Structures
  - Equivalent Viscous Damping Exceeding 30% of Critical
  - Very Soft Soil Condition (Type IV Soils)
  - Close to an Active Fault

Testing

- **System Characterization Tests**

  - Product Specific

  - Establish Fundamental Properties of an Isolation System

  - Development of a New Isolation System

  - Substantially Different Version of an Existing System

- **Prototype Tests**

  - Project Specific

  - Verify Deformation and Damping Parameters Used in Design and Analysis

  - Two Full-Size Specimens of Each Type and Size

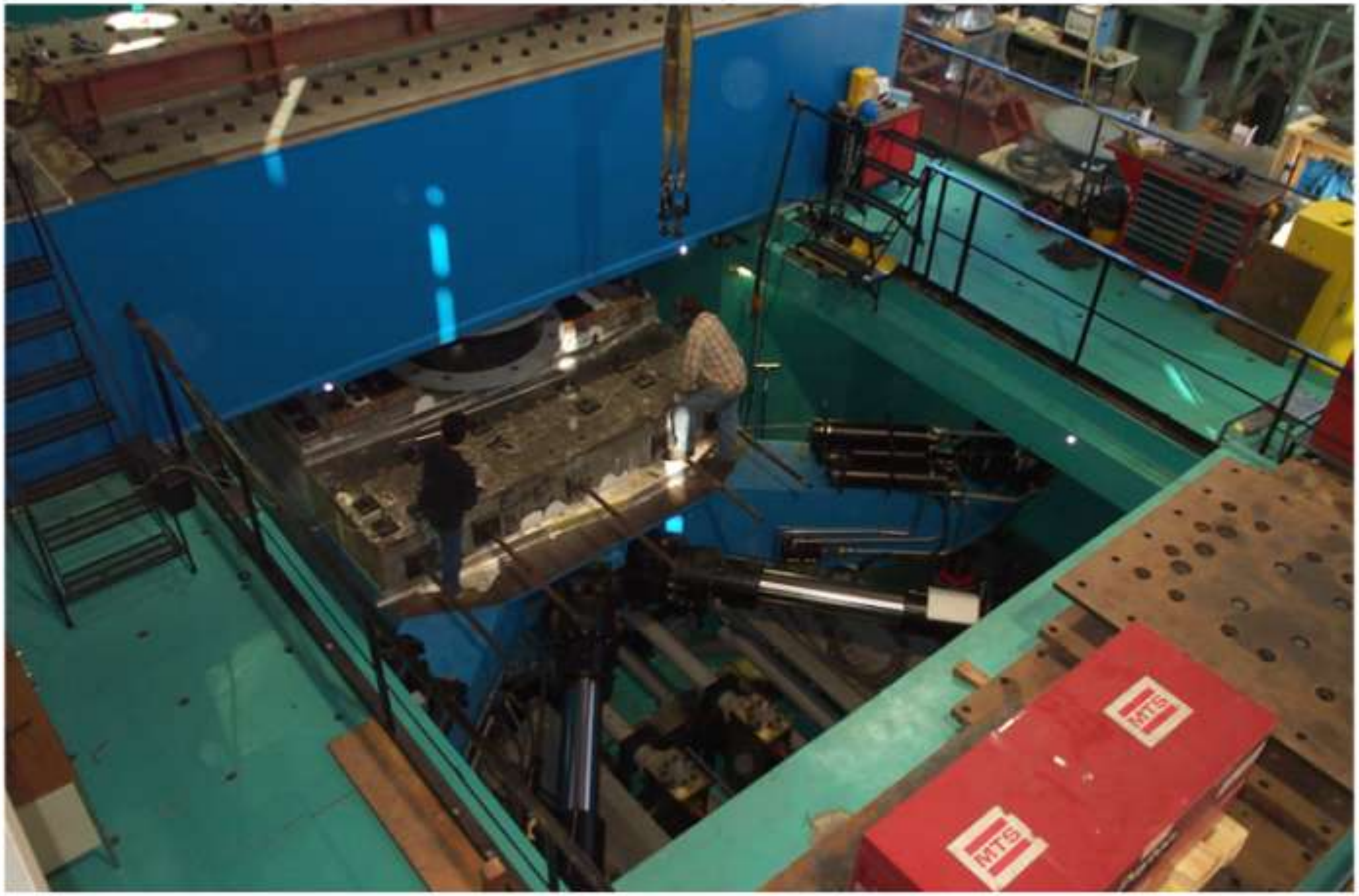
- **Quality Control Tests**

  - Project Specific

  - AASHTO 2010 Requires Proof Load & Combined Compression/Shear Tests on All Bearings

  - CAN/CSA-S6-06 CHBDC Requires Proof Load Tests on All Bearings and Combined Compression/Shear Tests on 20% of Bearings

Testing of Rubber Lead Core Bearing



Combined Compression and Shear Testing

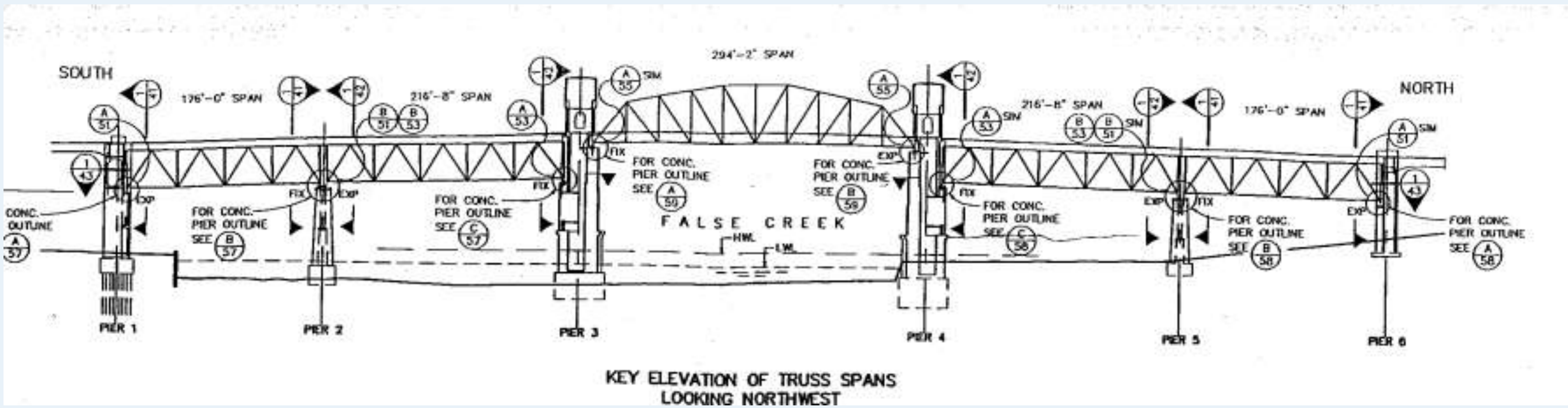
Testing of Friction Pendulum Bearing

Design

- Substructures to Remain Essentially Elastic
  - R = 1.0 for Lifeline and Emergency-Route Bridges
  - R = 1.5 for Other Bridges
- Ductile Detailing for Potential Plastic Hinge Regions
- Adequate Seat Width for Seismic Isolation Bearings

Burrard Bridge, Vancouver, BC

Burrard Bridge, Vancouver, BC



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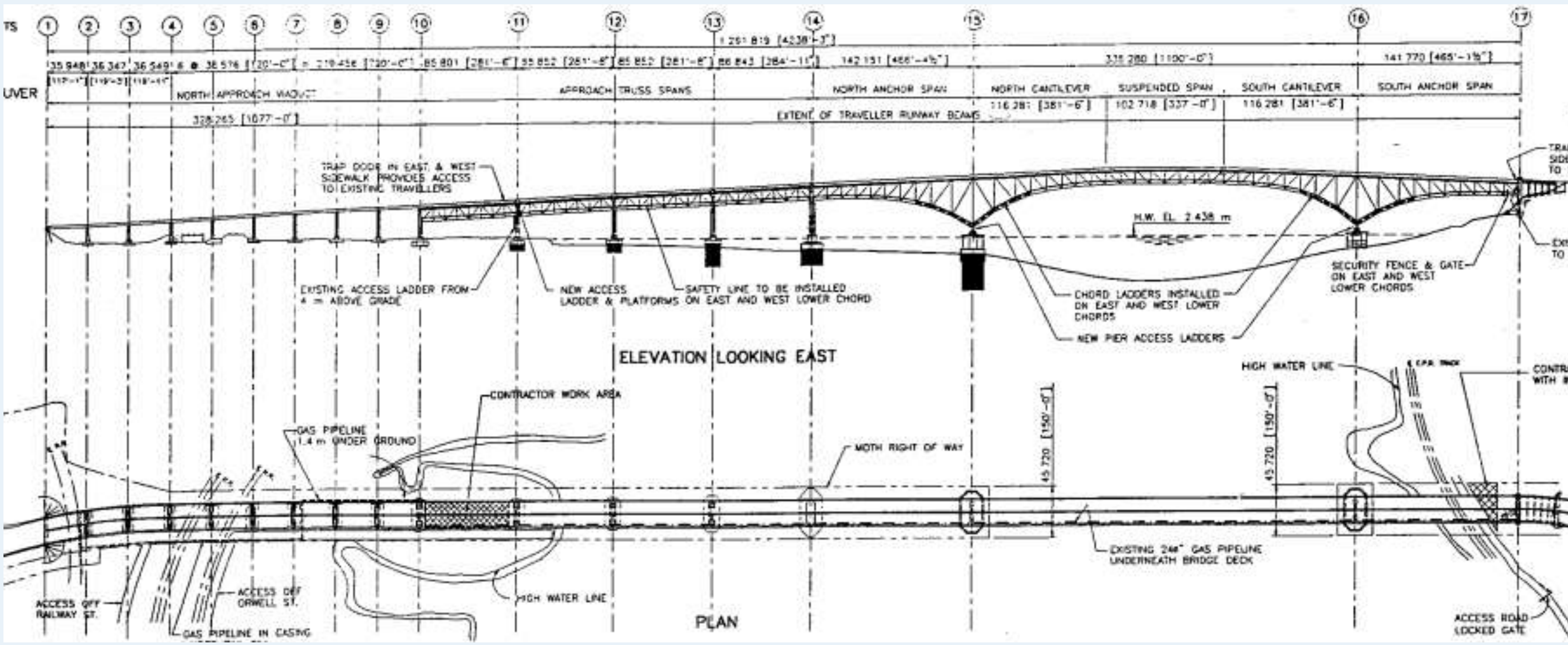
Burrard Bridge, Vancouver, BC

Burrard Bridge, Vancouver, BC

Second Narrows Bridge, Vancouver, BC



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Second Narrows Bridge, Vancouver, BC

Granville Bridge, Vancouver, BC

South Approach to Granville Bridge, Vancouver, BC

South Approach to Granville Bridge, Vancouver, BC

South Approach to Granville Bridge, Vancouver, BC

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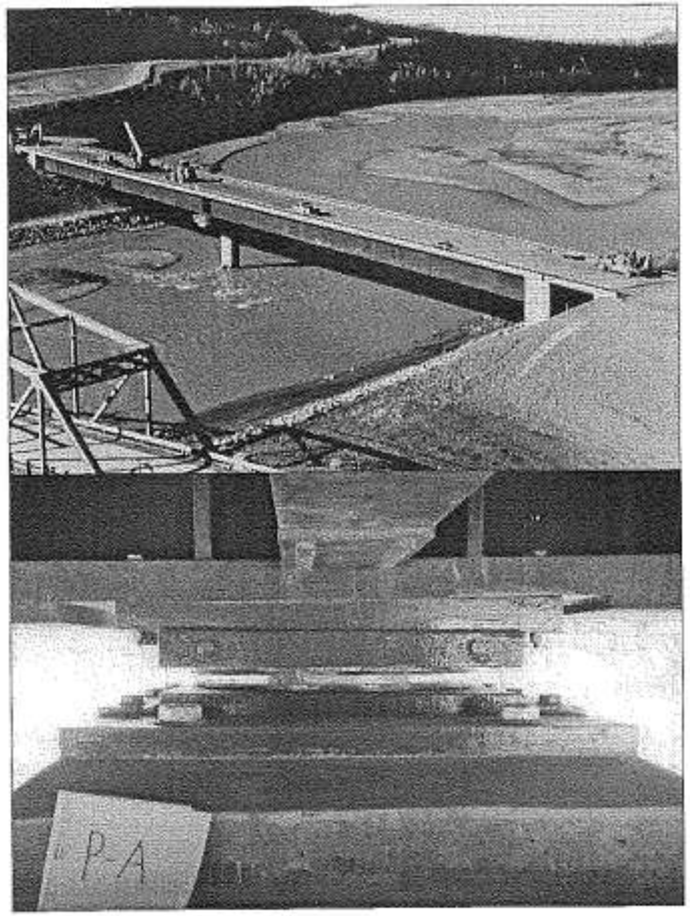
White Water Bridge, Yukon



Owner: Government of the Yukon, Canada
Engineer: Buckland & Taylor, Vancouver
Contractor: Peter Kiewit & Sons

BUCKLAND & TAYLOR LTD.
a COWI company

Golden Ears Bridge - Location

BUCKLAND & TAYLOR LTD.
a COWI company

Golden Ears Bridge – Main Spans

Golden Ears Bridge – South Approach

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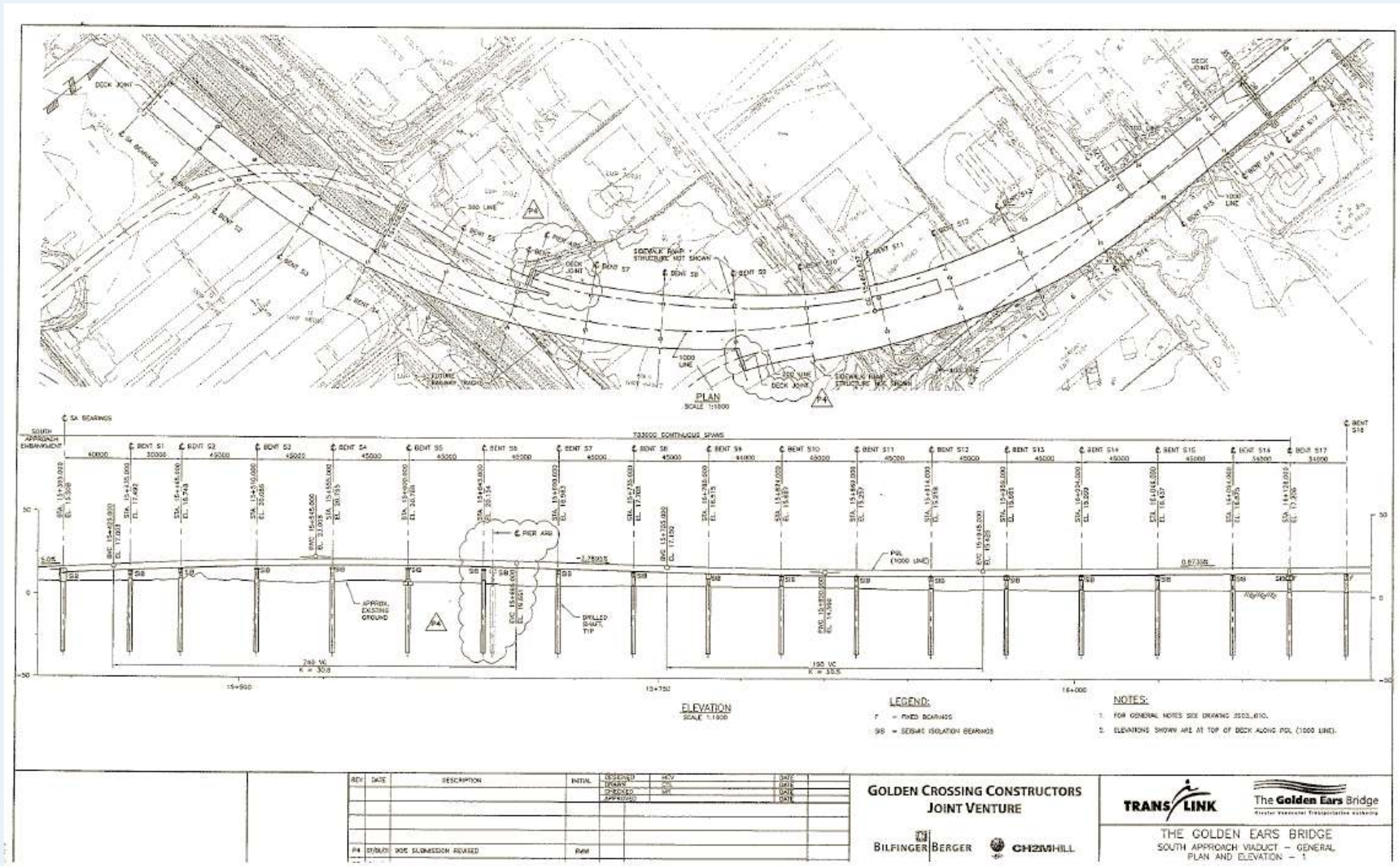
Golden Ears Bridge – South Approach

Golden Ears Bridge – South Approach



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Golden Ears Bridge – South Approach

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a COWI company

Testing of Seismic Isolation Bearings

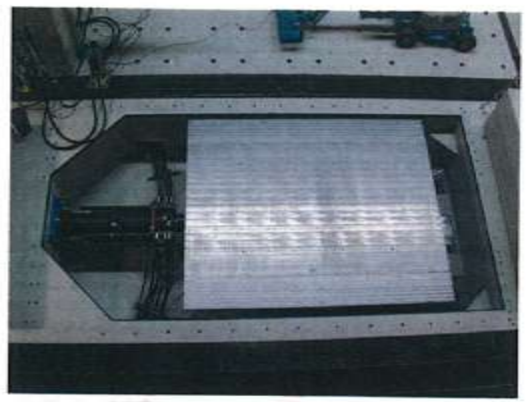


Figure 1. The TREES Lab Shaking Table

Figure 2. The TREES Lab Bearing Tester

Testing of Seismic Isolation Bearings

6.2 ALGAPEND APS 9100/1200-5 #2

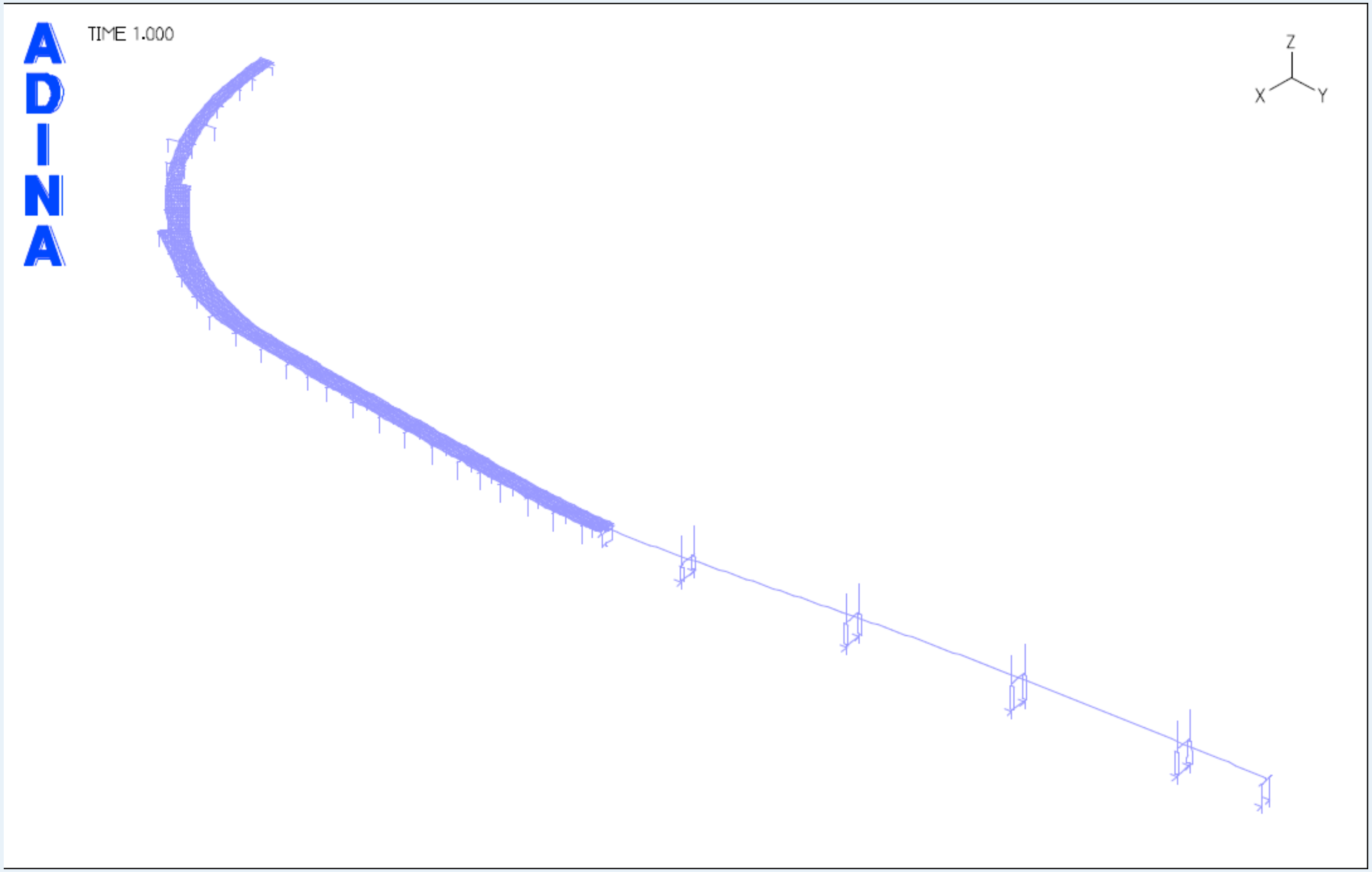
Figure 11. ALGAPEND APS 9100/1200-5 # 2 prior testing

Figure 12. ALGAPEND APS 9100/1200-5 # 2 prior testing

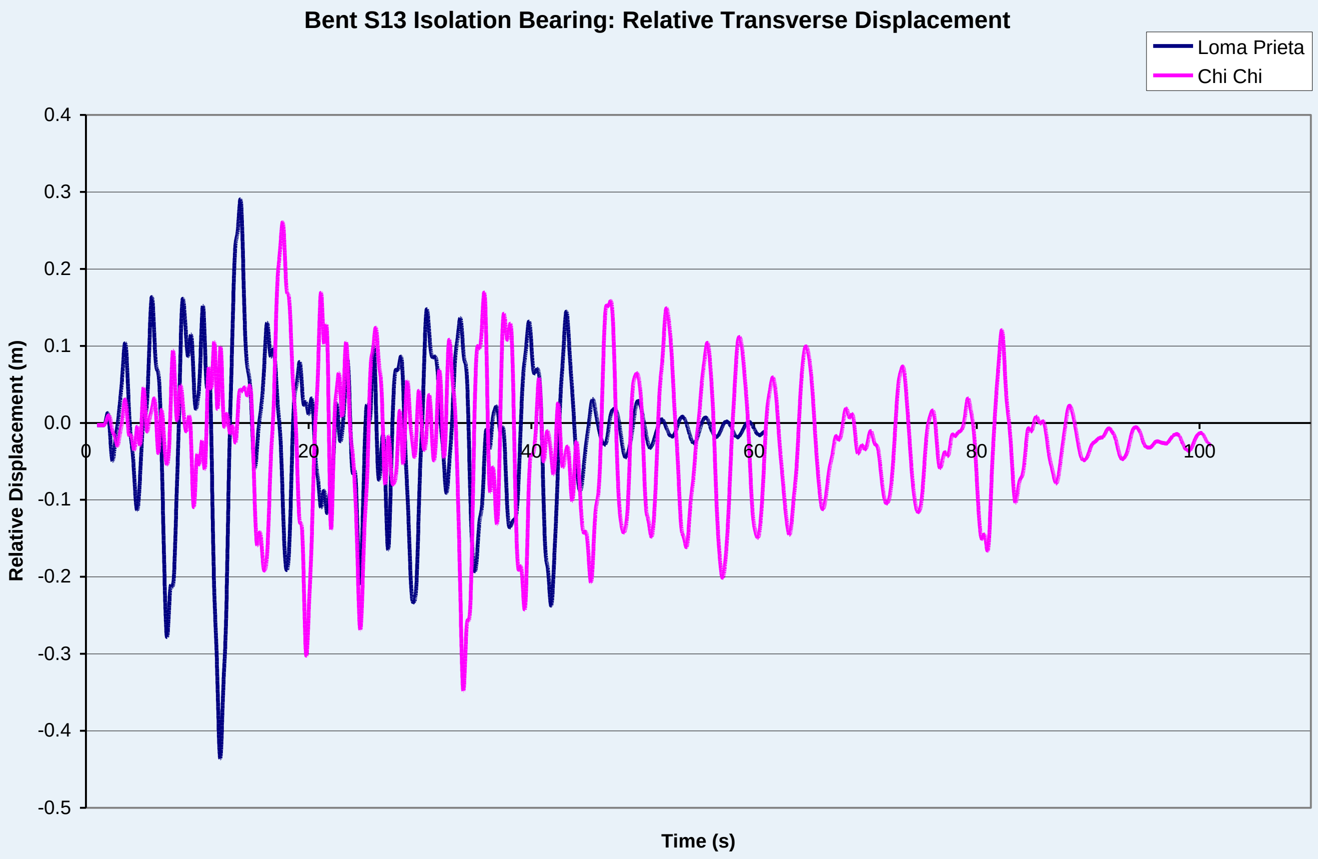
Figure 13. ALGAPEND APS 9100/1200-5 # 2 testing

Figure 14. ALGAPEND APS 9100/1200-5 # 2 testing

3D Computer Model



Nonlinear Time History Analysis



Design Displacement Combinations

- 475 Year Design Earthquake

  C/S + 0.5 T + EQ

- 2475 Year Design Earthquake

  C/S + EQ

Where

C/S = Creep and Shrinkage Effects

T = Thermal Effects

EQ = Seismic Effects

Performance Criteria for Deck Expansion Joints

- 475 Year Design Earthquake
  - Immediate access for normal traffic
  - Expansion joint structural components remain essentially elastic
  - Tear of neoprene seals permitted
- 975 Year Design Earthquake
  - Limited access for emergency vehicles
  - Structural components may become inelastic (damaged)
  - Longitudinal support bars should not pull out of boxes
- 2475 Year Design Earthquake
  - No service requirements
  - Structural components may completely fail
  - Fuses fail to limit seismic load transfer to adjacent non-isolated structures

Issues

- How to Combine with Thermal Displacements
- Cold Weather Effects
- Vertical Load Stability
- How to Deal with Uplift
- Appropriate Levels of Lateral Restoring Force
- Reliability over Time
- Maintenance
- How to Systematically Address Various Effects to Provide Level of Protection Appropriate for Bridge Importance and Design Earthquake Level Considered.

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Examples of Bridge Retrofit – Don Kennedy

CSRN / NEES Workshop Seismic Isolation & Damping, 12 / 04 / 30

GLOBAL PERSPECTIVE.
LOCAL FOCUS.

CANADA'S 50 BEST MANAGED COMPANIES

Seismic retrofit strategies

- Seismic isolation & / or added damping (devices)
- Increased strength
- Plastic behaviour (capacity design)
- Enhanced deformation capacity
- Redundancy (re-articulate, alt'v load paths, STU's)
- Locked – in superstructures (e.g. integral abutments)
- Ground improvement
- Prioritize routes; consequences of loss
- Often done with peer reviews; careful with VE reviews of "value" defined as saved first cost.

Rehabilitation / seismic retrofit benefits

- We rarely do seismic-only retrofits
- Structural rehabilitations are of immediate value
- Seismic-only retrofits are an insurance policy; hopefully no value
- Integrate rehabilitation, strengthening and retrofit
- Eliminate deck joints, re-articulate superstructures (link decks or make continuous)
- Often requires bearing replacement (LL θ, thermal δ)
- We always consider seismic isolation as a strategy
- Is now more accepted and increasingly common

Potential advantages of seismic isolation (some may not apply to dampers)

- Improved post-seismic performance
  - Less damage, less future repair cost
  - Immediate use for post-seismic recovery
- May better meet Owner or Society’s expectations
- If replacing bearings for re-articulation, incremental cost of isolation vs regular bearings is minimal
- Typically very high quality products
- May save significant sub-structure retrofit costs
- Even if sub-structure retrofit not avoidable, may still add value and resiliency.
- *May* be easier to analyze, assess and implement

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Possible limitations of seismic isolation

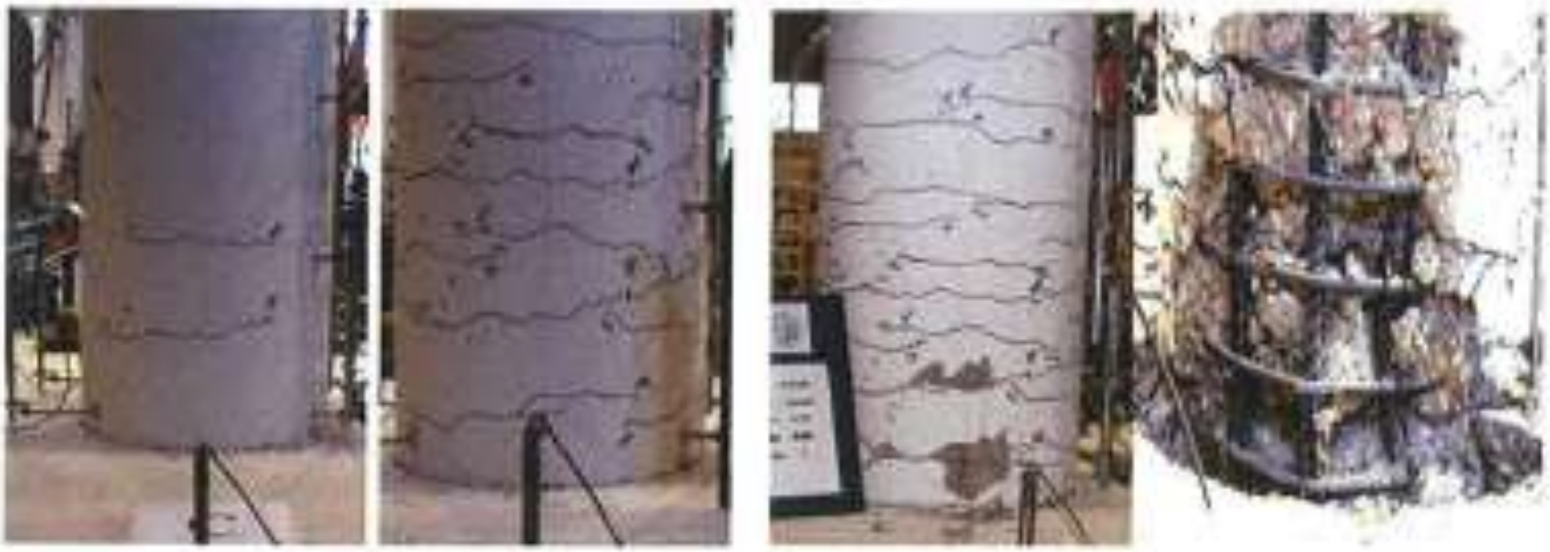
- May not be effective in some bridges (δT, δ damping)
  - Deep, soft soils (flexibility, soil amplification, site period)
  - Flexible piers; flexible foundations
  - Above factors in combination
- Liquefaction / lateral spreading
- Risk in jacking / replacement (tender, construction)
- Limited space, height, or jacking provision
- Integral or partially integral structures or components
- Relative displacement capacity limitations
- Perceptions of complexity or cost (Owner / Eng'r)

Owner and societal objectives

Figure 71: Performance-Based design Objective matrix and modification (blue line) to increase the targeted performance toward a Damage-Control level.

- Expectations / communication among Owner, Agencies, Public
- One key lesson being embraced following Christchurch earthquakes; easier to embrace once it's real?

Damage descriptions (subjective?)



No damage Minor damage Moderate damage Collapse

Fig. 13—Categorization of damage states under bending-shear.

Incorporation of Decoupled Damage Index Models in Performance-Based Evaluation of RC Circular and Square Bridge Columns under Combined Loadings, A. Belarbi, S. Prakash, and P. F. Silva, ACI SP-271—5.

Deformation and strength - based retrofit Objectives – as-built Mission

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Retrofit Objectives – as-built Pitt River Bridge Column ties

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Retrofit Objectives – as-built Pitt River Bridge Column ties

Retrofit Objectives – as-built Pitt River Bridge Column ties

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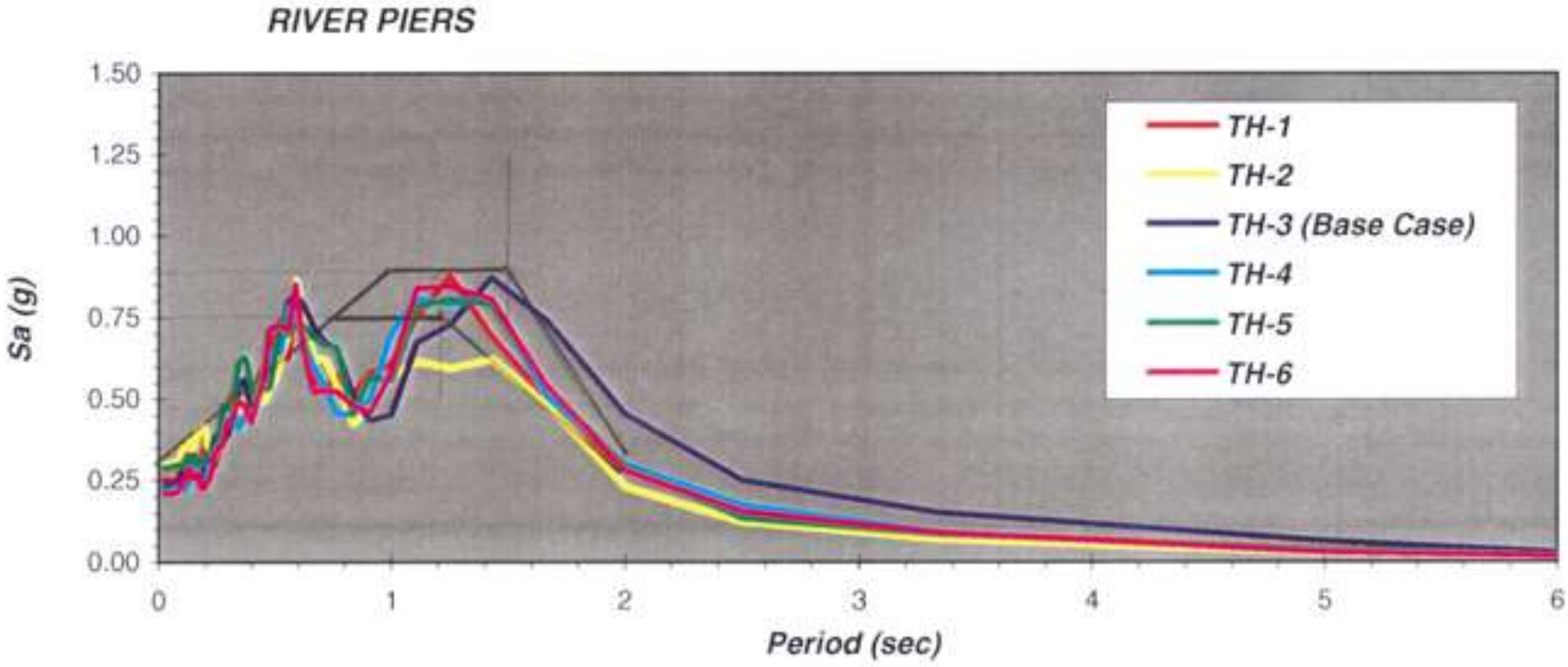
Bridge retrofit examples

Mission Bridge Seismic Rehabilitation
Mission, BC (Associated Engineering)

Mission Bridge Seismic Rehabilitation
Mission, BC



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Mission Bridge suspended spans

Mission Bridge Re-articulation

Mission Bridge Re-articulation

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Mission Bridge – N1 / S1 River Piers

Mission Bridge – N3, N4, S3, S4

Mi

Mission Bridge – Column base spalling

Mission Bridge – Seat extension

Mission Bridge – Load Path & Brg

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Mission Bridge N5 – Shear and load path

- N1 / S1 load path

Oak Street Bridge (Klohn – Crippen)

Oak Street Bridge – Half-scale testing

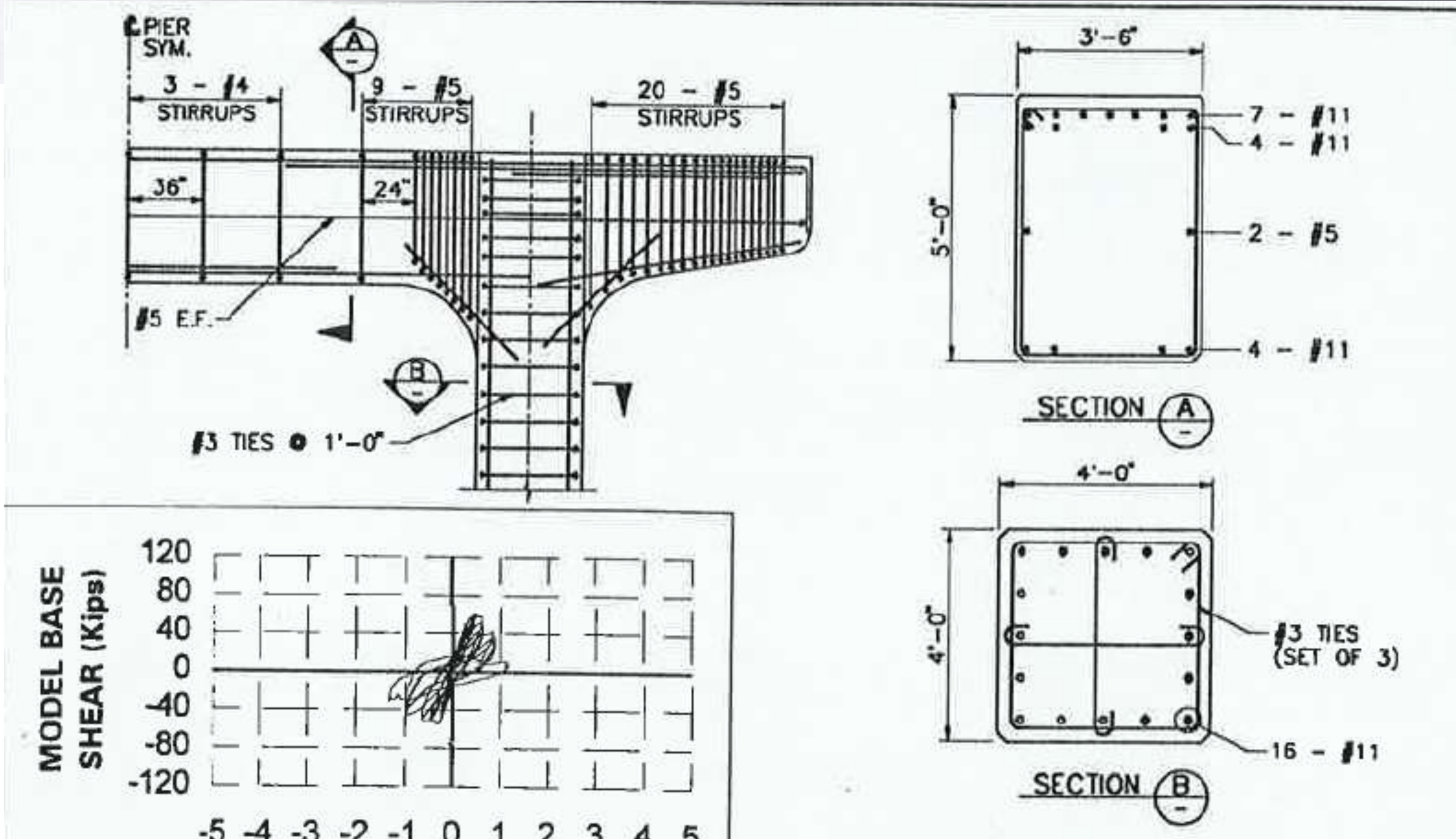
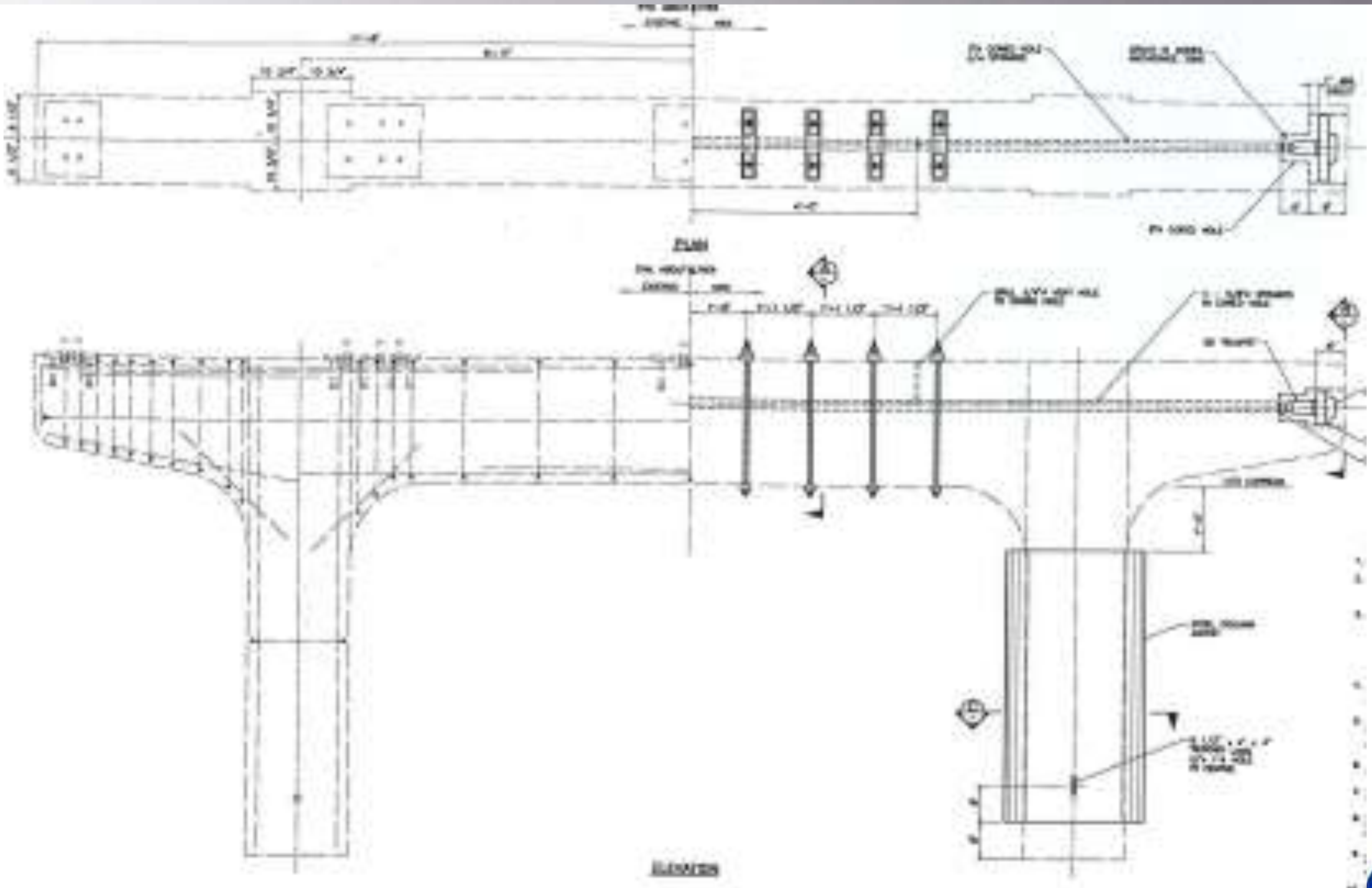


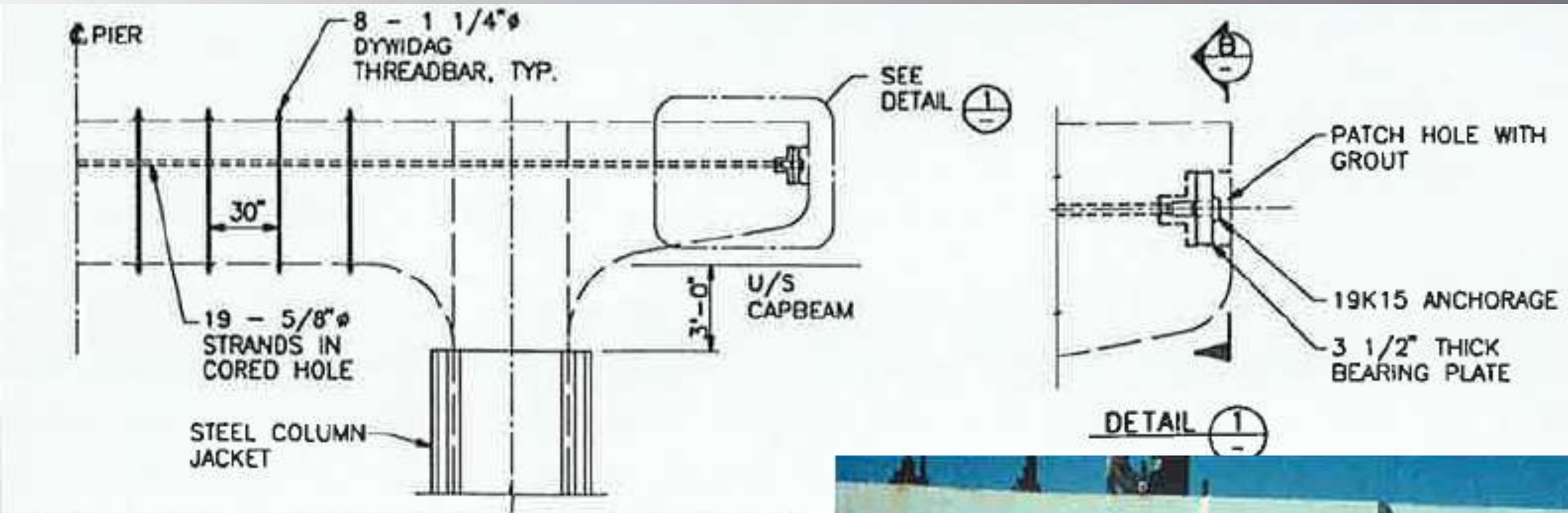
FIGURE 1 – OSB1–AS BUILT
(PROTOTYPE DIMENSIONS)

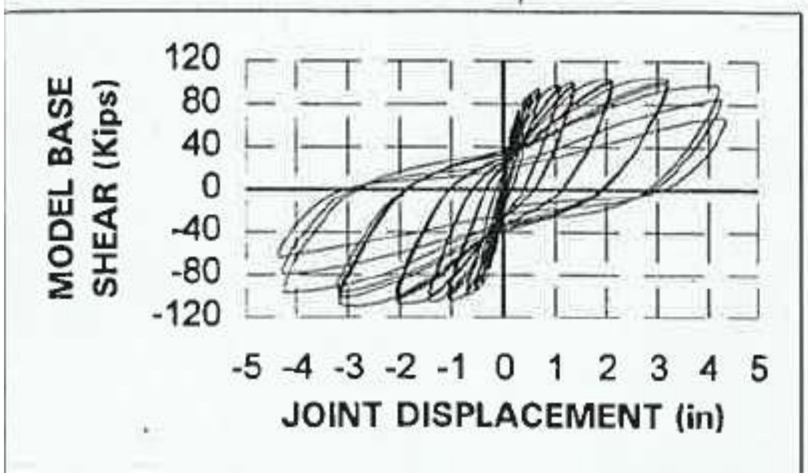
Oak Street Bridge – Half-scale testing

Oak Street Bridge – Half-scale testing



Oak Street Bridge – Half-scale testing





Oak Street Bridge – Typical Approach Pier Retrofit

Oak Street Bridge – South approach seismic settlements

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Oak Street Bridge – Strength and Rocking Piers (north approaches)

Oak Street Bridge – Other retrofits

Oak Street Bridge – Other retrofits

Oak Street Bridge – S1 pier and load

Queensborough Bridge (Sandwell)

Queensborough Bridge - S1 Isolation

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Queensborough Bridge – other retrofits

Queensborough Bridge – other retrofits

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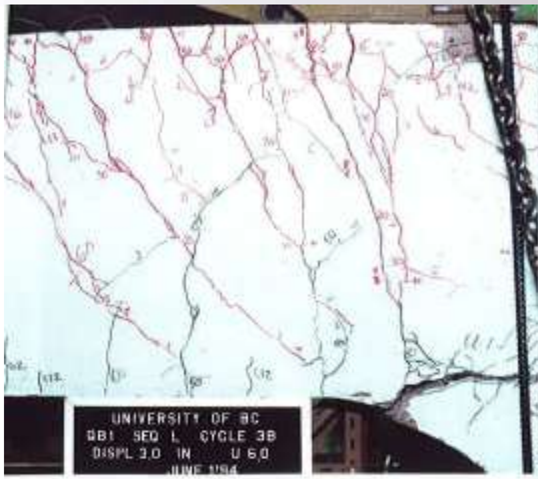
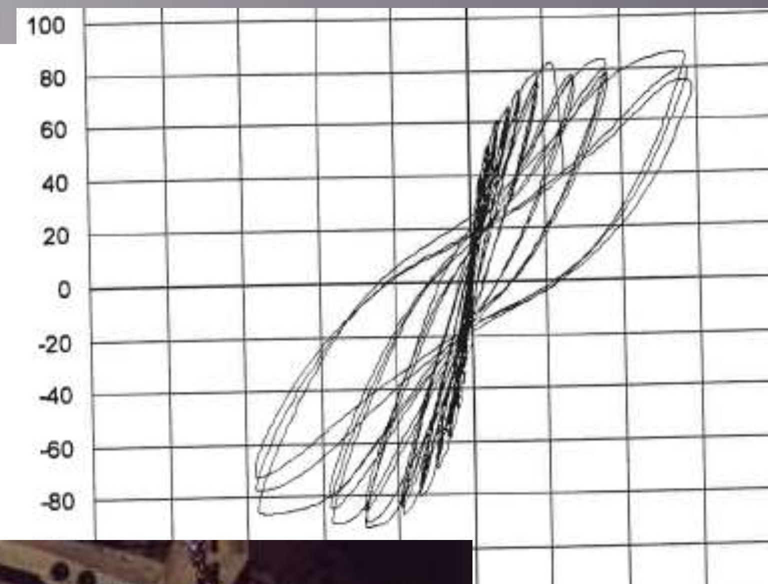
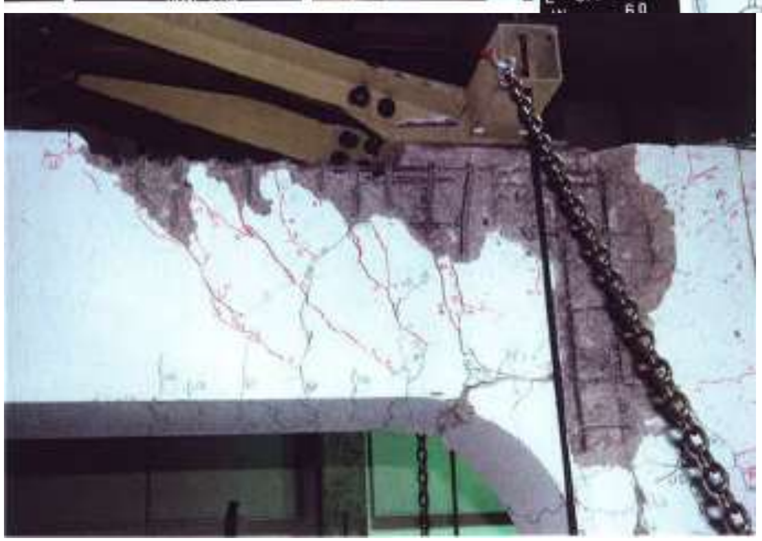
Queensborough Bridge – repair test







Queensborough Bridge – column jacketing; pier aspect ratio

Tynehead Pedestrian Bridge (AE) – isolation bearings – vibration risk and….

Tynehead Pedestrian Bridge - seismic benefits; period shift, elastic design

Fraser Heights Bridge (AE) – Thermal articulation and seismic isolation

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Fraser Heights Bridge – Thermal articulation and seismic isolation

Knight Street Bridge (Associated Engineering)

Knight Street Bridge

Knight Street Bridge

Knight Street Bridge

Knight Street Bridge

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Knight Street Bridge

Knight Street Bridge – compaction piles

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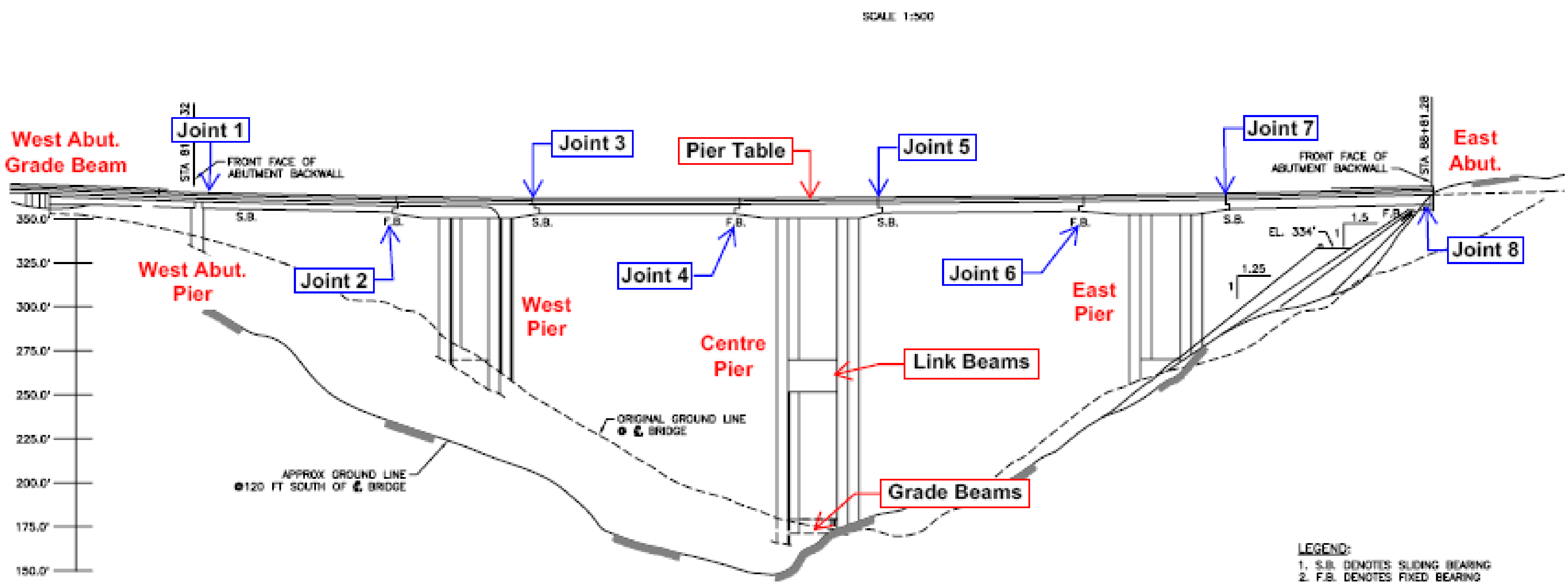
Knight Street Bridge – compaction piles (trees – carbon capture and storage?)

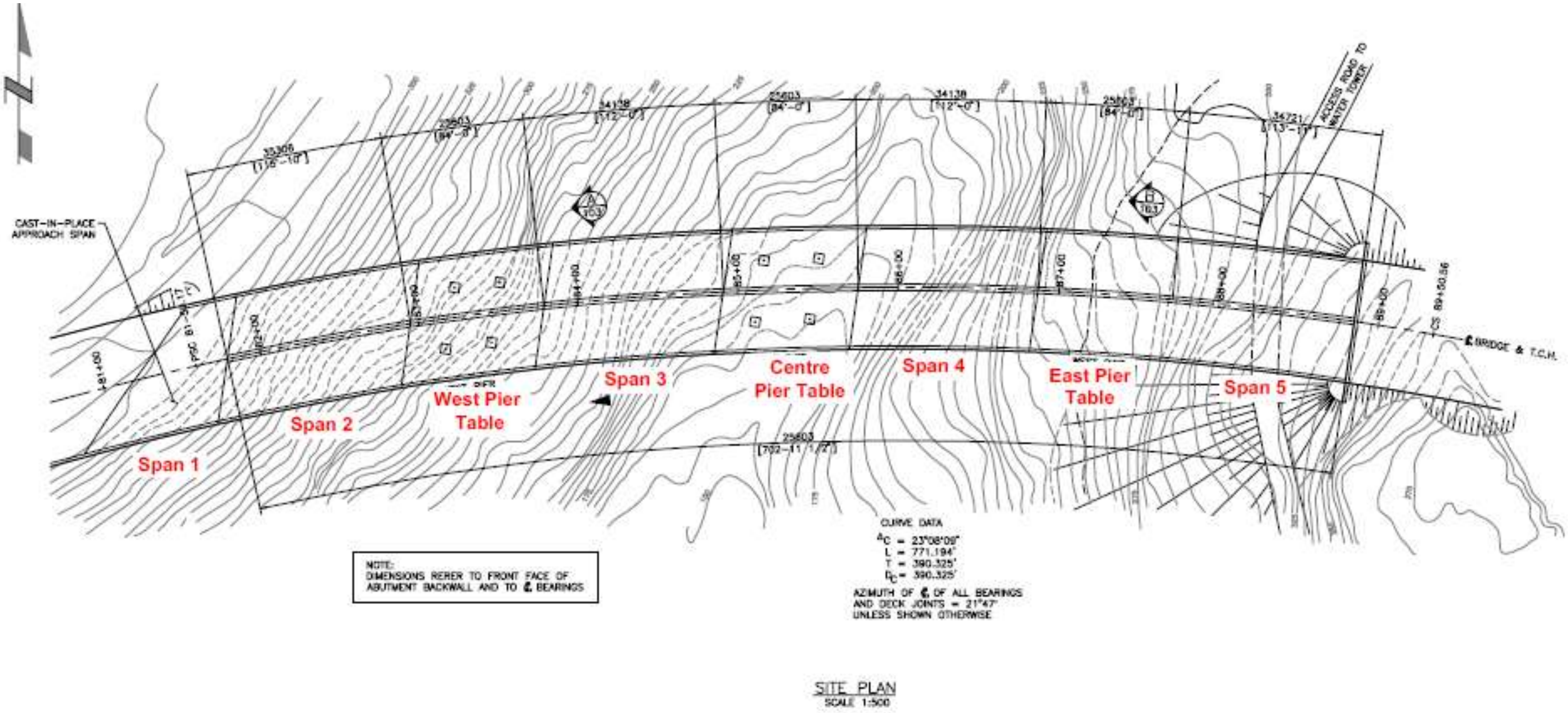
Knight Street Bridge

Nelson Creek Bridge, West Vancouver, BC
(Associated Engineering)

Nelson Creek Bridge



Nelson Creek Bridge, West Vancouver, BC
(Associated Engineering)



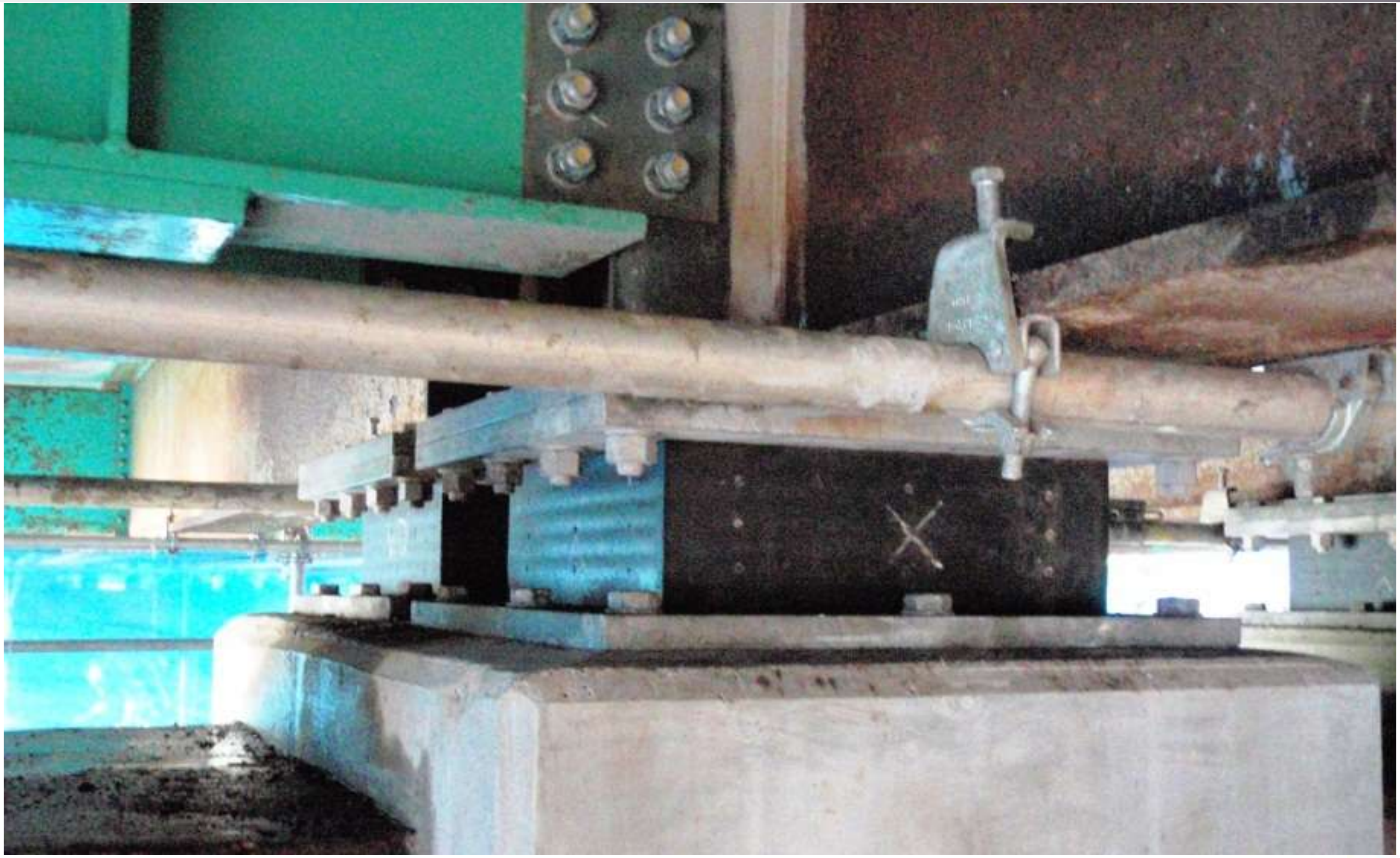
Nelson Creek Bridge

Nimpkish River Bridge, Vancouver Island, BC
(Associated Engineering)

- Existing steel through-truss, Northern Vancouver Island. Low traffic volume, but non-redundant route
- Rehabilitation primarily bearing replacement.
- Sub-structure includes heavy concrete wall piers on precast concrete battered piles through the river
- Was not a 'seismic retrofit' project, but we used tall elastomeric bearings, no added damping, gained significant seismic benefit at little cost. Limited dynamic analysis (small effort to extend the model)
- Bridge seismically is better, certainly no worse
- Could add damping later if a full seismic retrofit is sought.

Nimpkish River Bridge

Nimpkish River Bridge



Colebrook Rd Overpass (Associated Engineering)

Colebrook Rd Overpass Rehabilitation

Colebrook Rd Overpass - Rehabilitation

Colebrook Rd Overpass Rehabilitation

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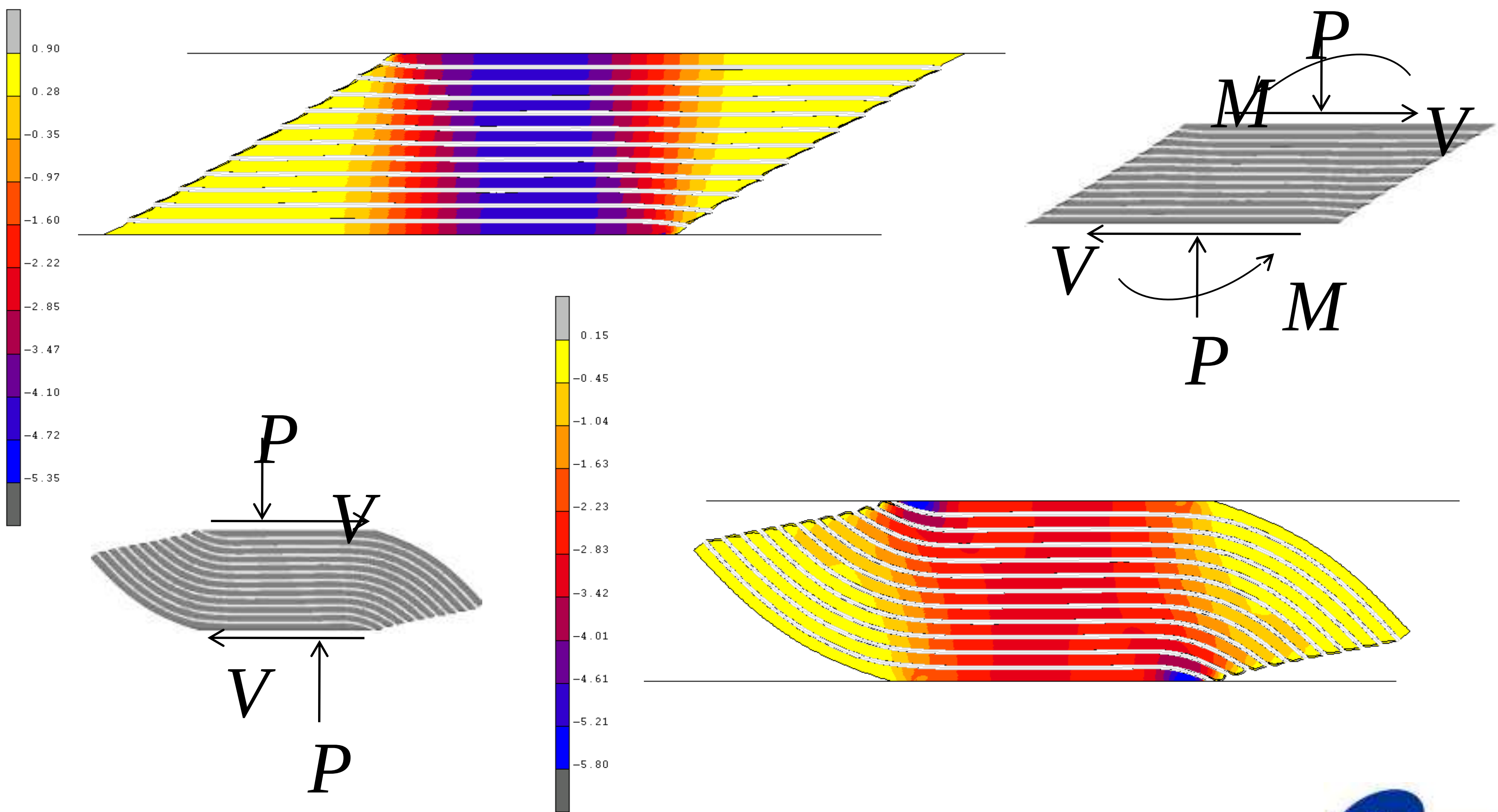
Colebrook Rd Overpass Rehabilitation

Colebrook Rd Overpass seismic retrofit

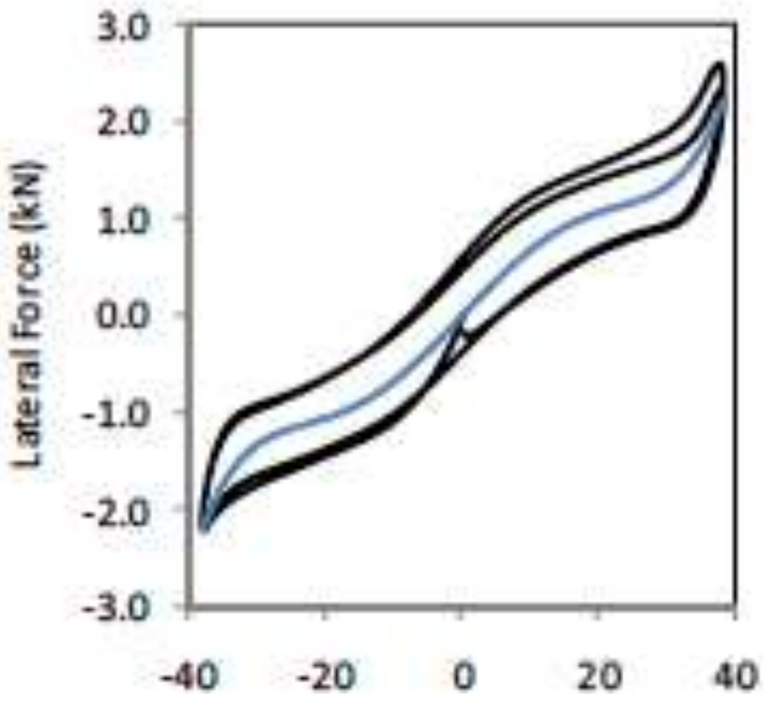
Roger Pierlet Bridge repair / retrofit

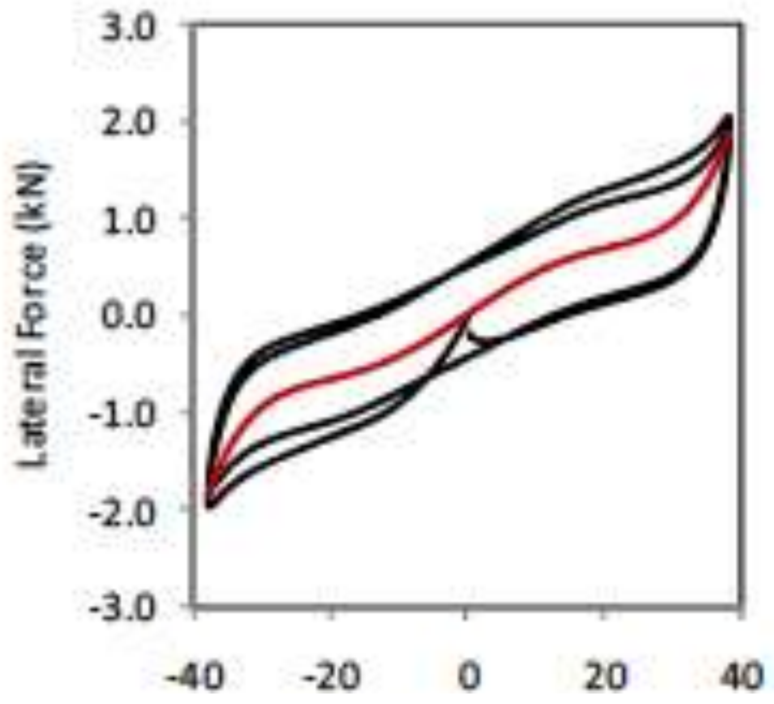
(Klohn-Crippen Berger)

Stress contour for S22 (MPa) at 200% t_r lateral displacement



Stable unbonded fibre-reinforced isolation bearings





Seismic retrofit summary

- Seismic isolation or added damping warrants serious, concerted consideration.
- High quality products, good competition & support.
- Have become common, often have improved post-seismic performance.
- Retrofit ideally done in concert with rehabilitation.
- Strength, ductility, other strategies can be very effective and reliable.
- Often more resiliency / reliability gained in added deformation capacity than increasing strength
- Strength, ductility, other strategies can be more uncertain and possibly more complex to assess

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GLOBAL PERSPECTIVE.
LOCAL FOCUS.

CSRN 12/04/30
Thank you for participating

CANADA'S 50 BEST MANAGED COMPANIES

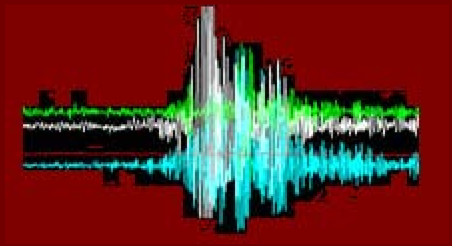
ISOLATED AND DAMPED BRIDGES

OWNER'S VIEW

CSRN –NEES Workshop on the Seismic Isolation and Damping of Bridge Structures
2012 April 30

Sharlie Huffman, P.Eng.

Sr. Seismic & Structural Health Engineer



Bridge Engineering

BRITISH COLUMBIA

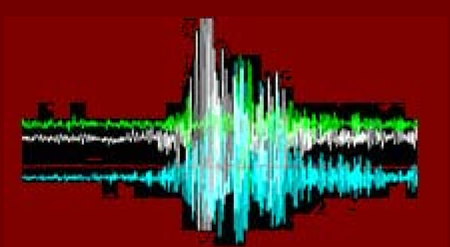
Ministry of Transportation

ISOLATED & DAMPED BRIDGES

- BRIDGE PERFORMANCE CHALLENGES
- IMPLEMENTATION CHALLENGES
- WHAT OWNER WANTS
- BENEFITS RECAP
- WHERE DO WE WANT TO BE

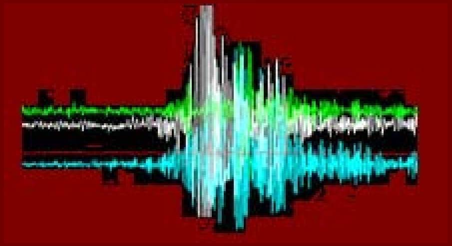
BRIDGE PERFORMANCE CHALLENGES

- High seismic area
- Aging bridge stock – deterioration & exceeded design lives
- Increased traffic, loading
- Increased understanding of seismic impacts
- Increased public expectations



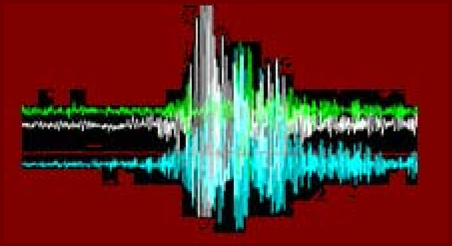
BC SEISMIC DESIGN

- Performance-based design for new and retrofit
- 2 - 3 level earthquake criteria for new, important structures (475 year & 2475 year returns)
- Seismic Retrofit Program criteria overhaul
- Requirement for explicit demonstration of met performance



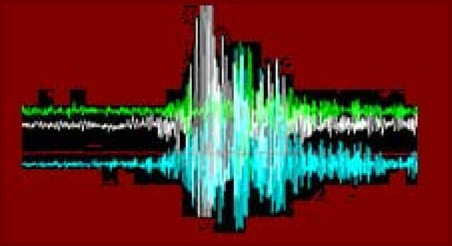
BC SEISMIC DESIGN

- Performance heavily reliant on displacement
- Definite role for isolation, damping and restraint devices (controlling displacement is what they do)



IMPLEMENTATION CHALLENGES

- **Familiarity – both on owner and designer side**
- **Perception that dampers are for buildings – not economical for most bridges**
- **Perception that dampers are too expensive for “ordinary” bridges**
- **Lack of Code specifications**
- **Extensive testing requirements**

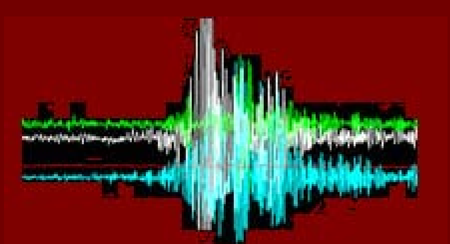


New Construction

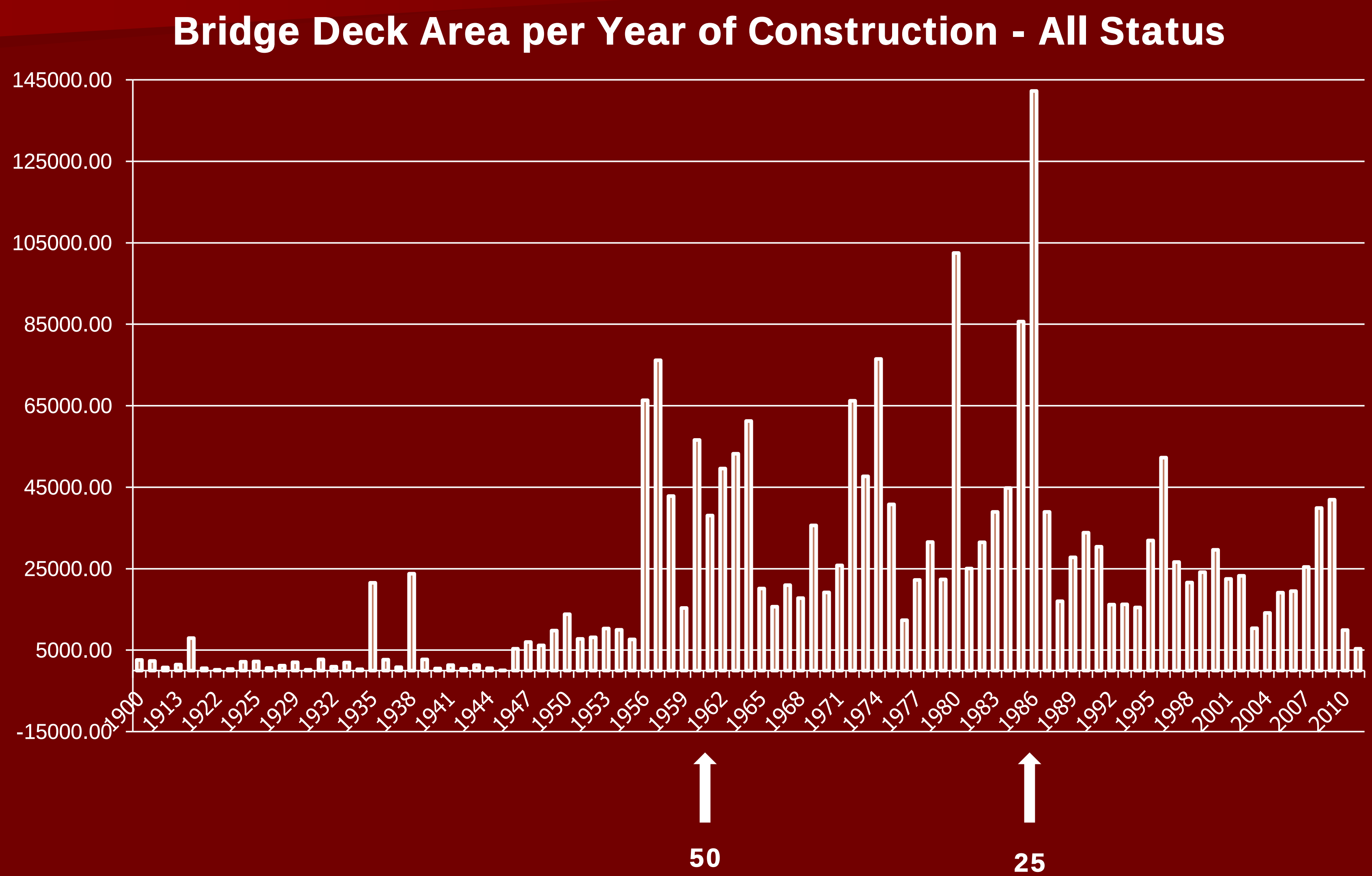
- Isolation (seismic) bearings common over the last couple of decades
- Supplemental damping rare and more recent
  - Port Mann (146)
  - Stawamus Chief Hwy 99 Ped OP #9446
- Some use of lock-up devices

Seismic Retrofit

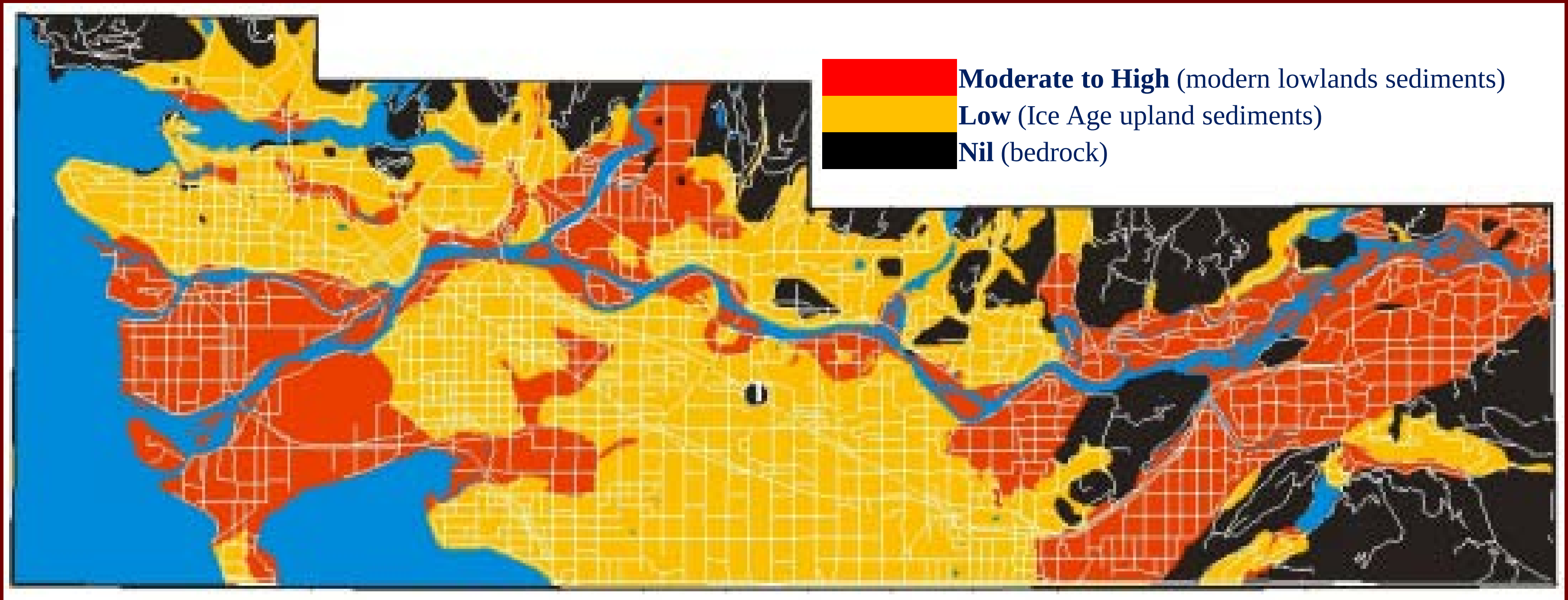
- Much of the lower mainland is susceptible to liquefaction at less than 1:475 yr event and amplification
- It can be difficult and costly to retrofit an old bridge to current specifications
- Limited funding - Ground improvements are very expensive but replacements even more so
- Isolation and damping are attractive options in bridging the gap between then and now



Seismic Retrofit



LML Soils


Moderate to High (modern lowlands sediments)
Low (Ice Age upland sediments)
Nil (bedrock)

What Does Owner Want

- Robust – survive anything
- Long life
- Cheap
- Low Maintenance
- No surprises

RECAP SOME BENEFITS OF ISOLATION AND DAMPING

- Retrofits
  - Where extensive ground improvements are otherwise needed to reduce shaking
  - Where older designs are vulnerable to superstructure displacement and unseating
  - Where connections/components cannot be economically brought up to desired performance due to displacement demands
  - Where it makes sense to achieve a higher level of retrofit

RECAP SOME BENEFITS OF ISOLATION AND DAMPING

- New design
  - With performance expectations at long return EQ’s
  - Where required ground improvements are difficult or uneconomical
  - Other economical considerations (shear keys, rocking)

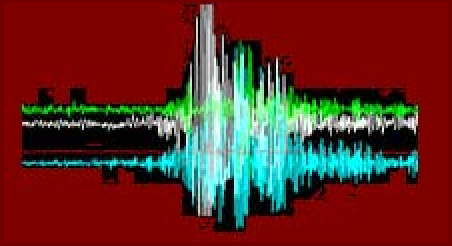
WHERE WOULD WE LIKE TO BE

- Isolating &/or damping bearings are common in BC and expected in higher seismic areas
- Broader consideration of supplemental damping for medium bridges where appropriate
- Expect that isolation and damping will be included in Benefit/Cost options
- Strategy reports on expected damage, repairs & time for return to service (any add’l cap?)
- Retrofit of older designs most probable area for economical use of supplemental damping

WHERE WOULD WE LIKE TO BE

- Future standard specifications & performance criteria with pre-tested products
- Isolation covered more broadly in S6
- Supplemental damping covered in S6
- Look to research/CSRN
  - How much deformation = how much damage (risk)
  - Defining critical performance aspects
  - Improved analysis methods
  - Device specifications/testing ?
  - Expand opportunities for isolation and damping

THANK YOU



British Columbia