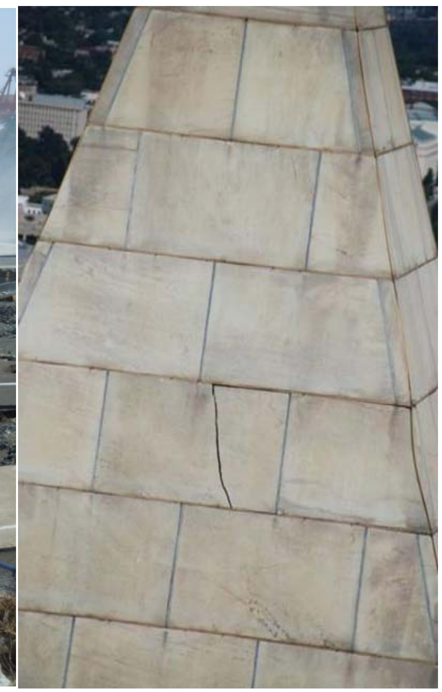
Geotechnical Engineering Implications from Recent Worldwide Earthquakes



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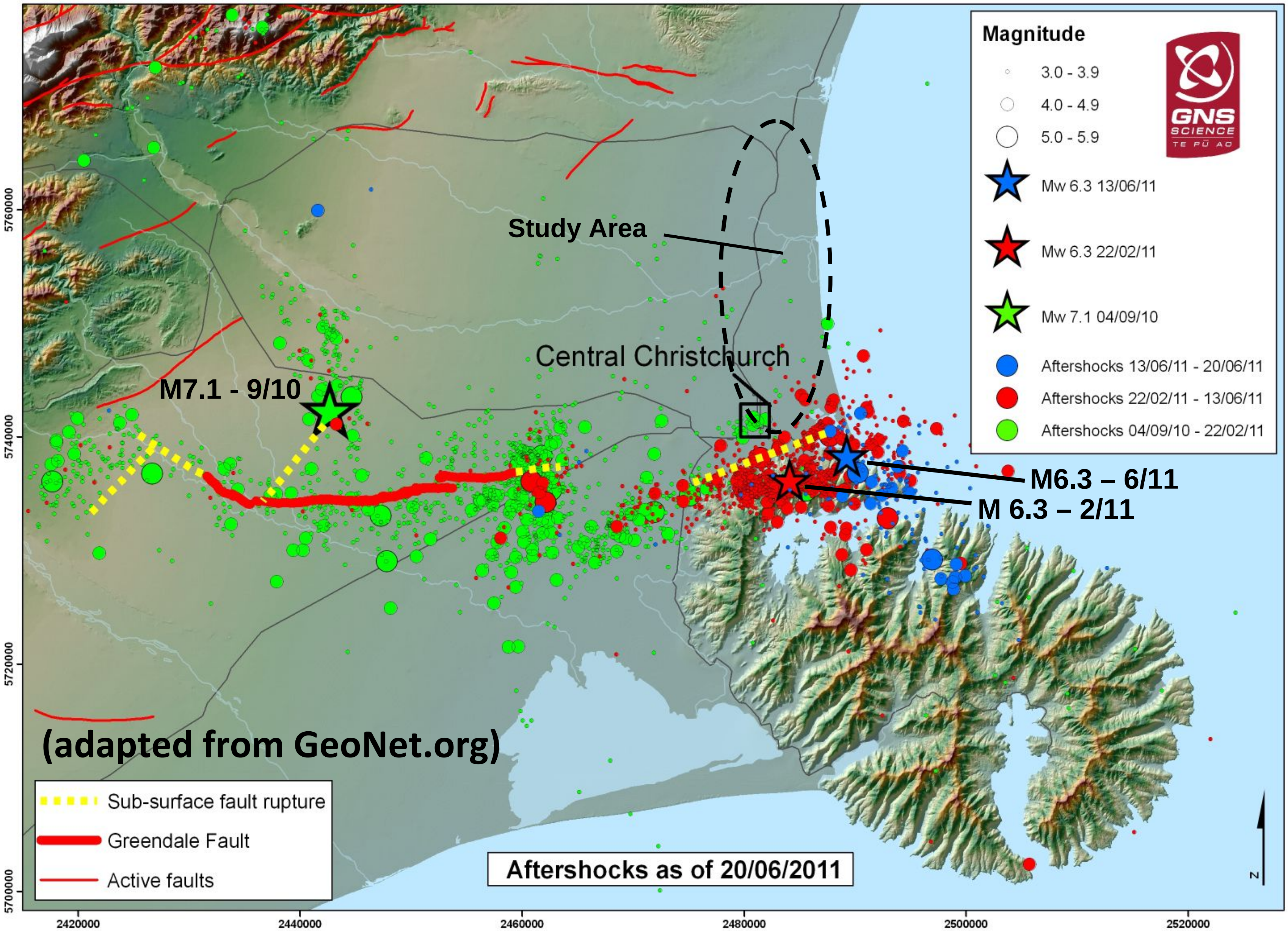
- Brett Maurer, VT PhD. Student (NZ Liquefaction/LPI)
- Dr. Guney Olgun, Research Asst. Professor, VT Civil Engr.
- Port and Airports Research Institute (PARI), Japan
- National Science Foundation - GEER Reconnaissance Teams in 2010-11 New Zealand and 2011 Virginia EQs

We'll discuss*:

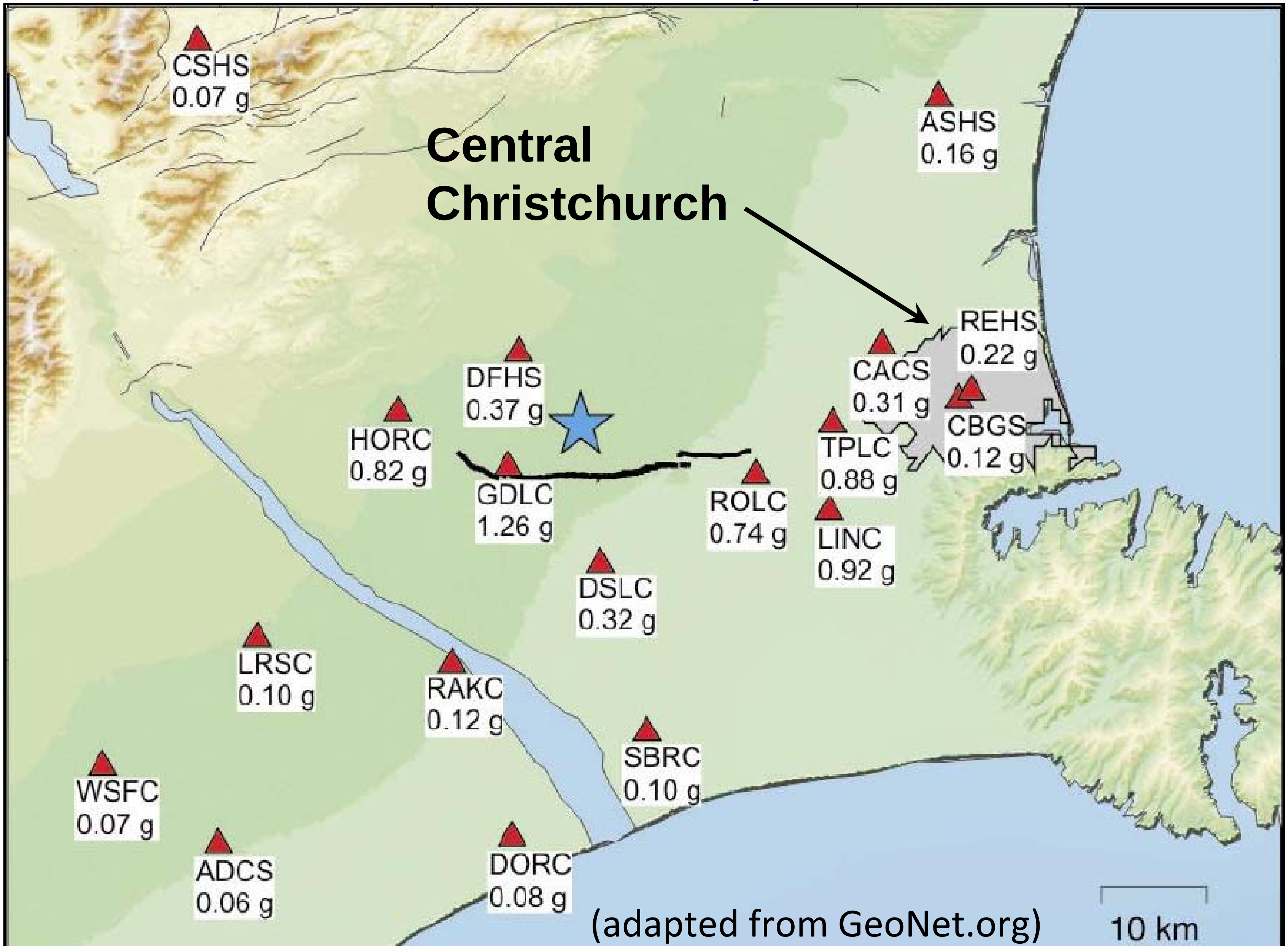
- 2010-11 M6.3-7.1 Christchurch,New Zealand
- 2011 M9 Tohuku, Japan (and aftershocks)
- 2011 M5.8 Mineral, Virginia

*Special note: Numerous slides can not yet be made available publically due to ongoing studies that are legally sensitive. Those slides have been eliminated from this pdf presentation.

2010-11 New Zealand EQs: M7.1, M6.3, M6.3



PGAs M7.1 Sept 2010 EQ



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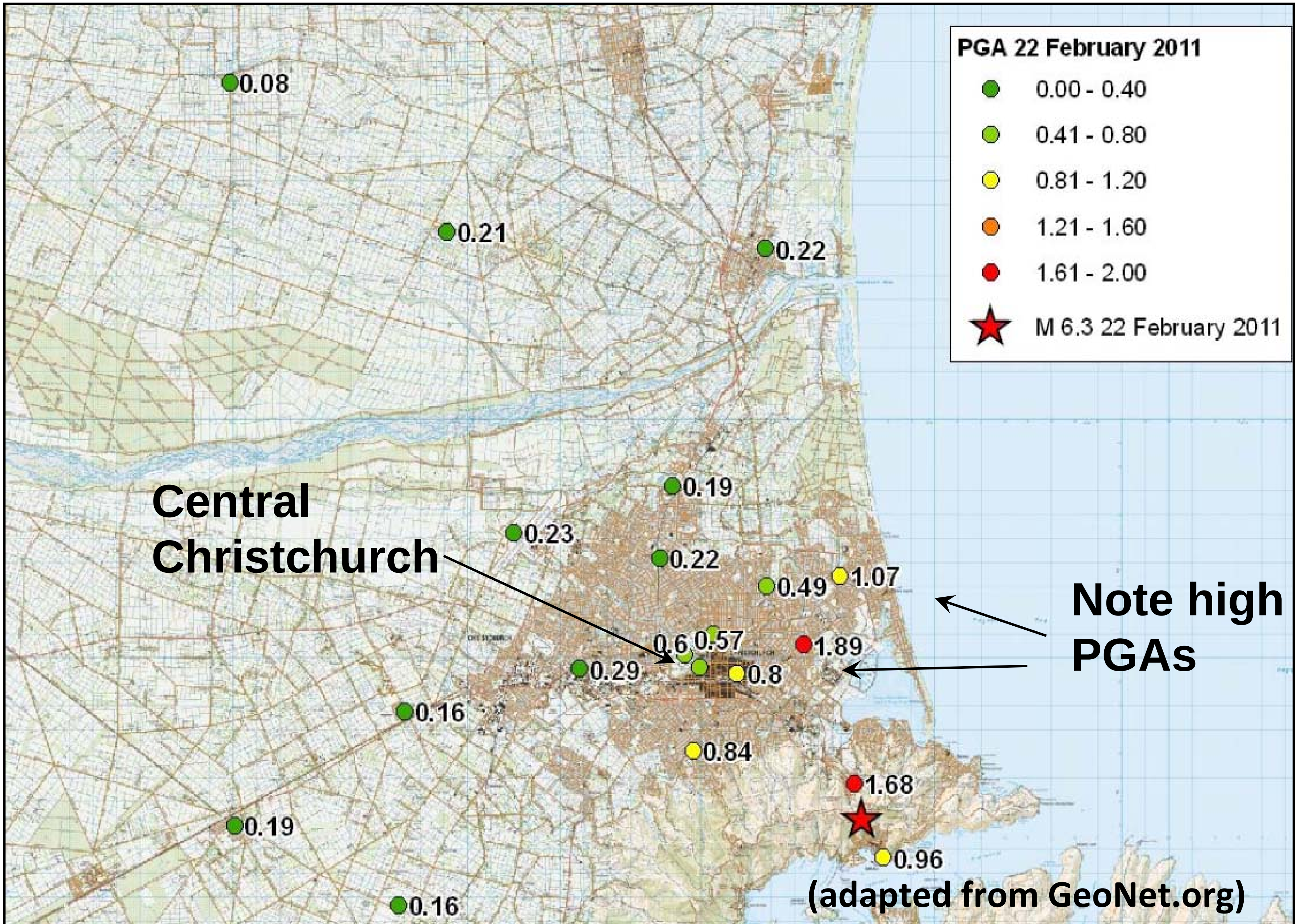
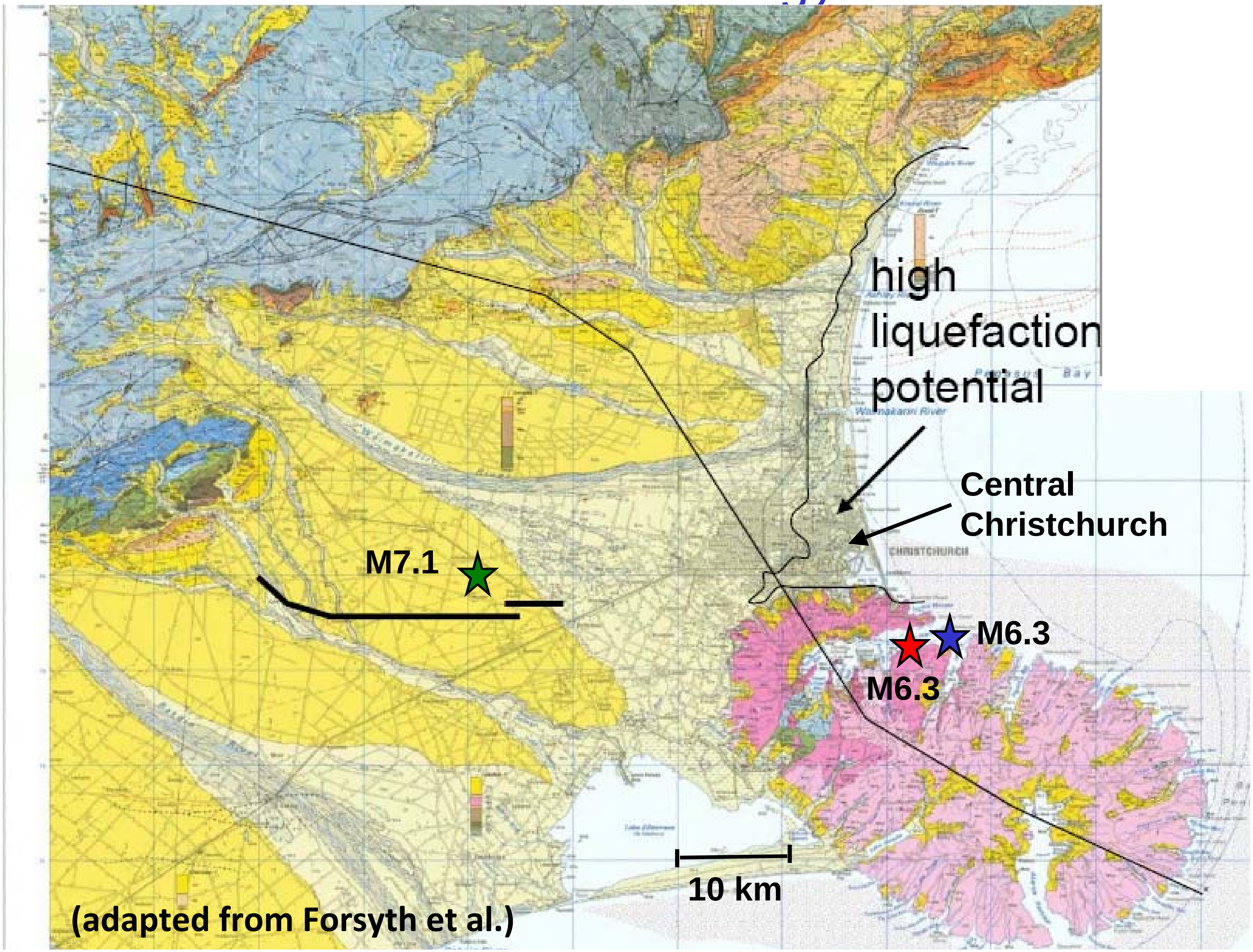
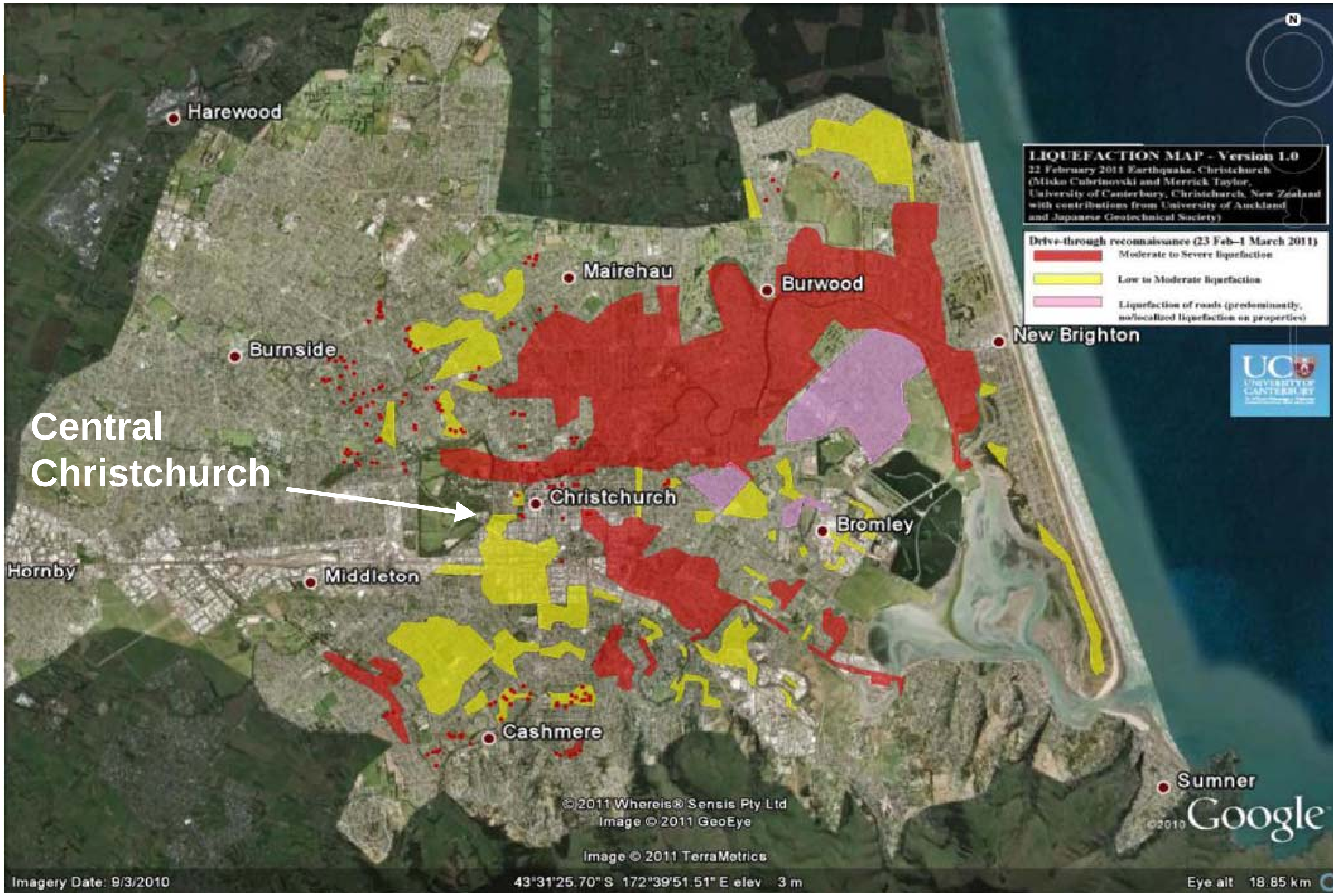
PGAs of M6.3 Feb 2011 EQ – the most damaging



Surficial Geology



Liquefaction – Feb 2011 M6.3 Christchurch



(M. Cubrinovski – U. Canterbury)

Severe liquefaction - 2011 M6.3 Christchurch

Liquefaction: 2011 M6.3 Christchurch - QEII Stadium

Lots of Liquefaction in recent natural sands and fills

(a)

(b)

(c)

- Liqn' damages were carefully documented, sites well-characterized

Photos from NSF GEER NZ Reconnaissance Team

Liquefaction Potential Index (LPI)

- LPI (0 to 100) characterizes the damage potential of soil liquefaction manifested at the ground surface (more meaningful than FS alone); LPI 5 is threshold
- LPI combines depth, cumulative thickness of liquefiable intervals, and factor of safety of liquefiable intervals into a single parameter
- LPI has been often used for hazard characterization, but calibration to observed liquefaction severity is limited and the accuracy of LPI predictions is highly uncertain.
- NZ provides unique opportunity for LPI calibration – LPIs there from 0 to 40 (about as high as imaginable)

Improved Soil Sites – (mostly stone columns)

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Water's Edge Apts. –
Stone Column Improvement Ineffective

C. Anderson (2011)

AMI Stadium – 38,000 seat stadium
Stone Column Improvement Ineffective

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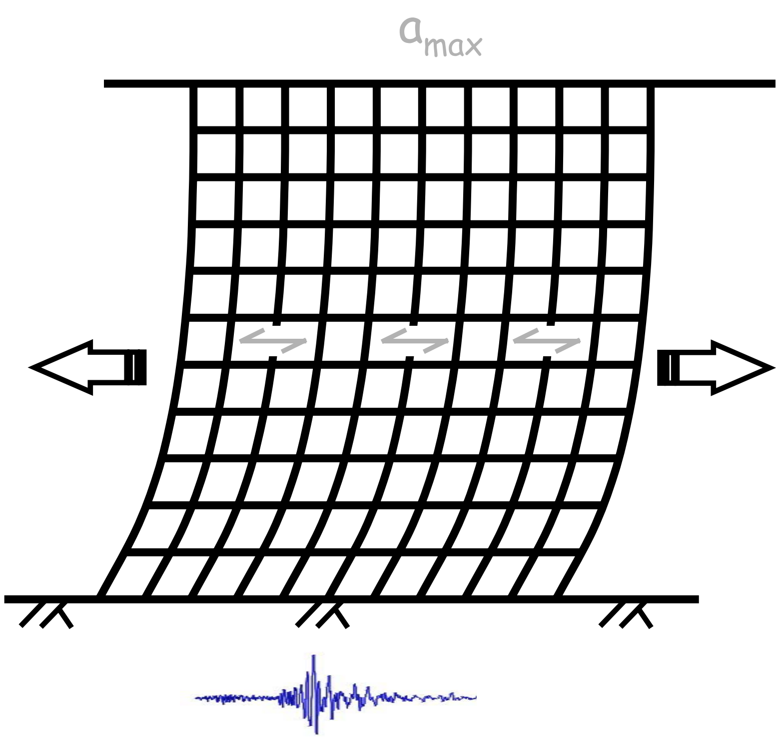
Ineffective Seismic Performance - Why?

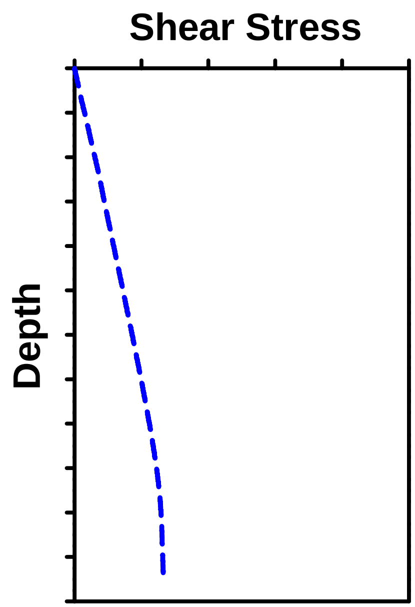
- Our detailed studies just beginning; not yet sure
- Sites were shaken harder than design levels (CBD sites designed for 0.4g; AMI and Water's Edge felt PGAs of ~ 0.7g and ~ 1g, respectively)
- Sites contained soft silty strata that were not densified during column installation
- Seismic shear stress reduction in soil mass (and therefore CSR) is less than current design approaches predict
- Columns cannot effectively reduce settlements if confinement is lost due to liquefaction and/or softening of surrounding soils

Seismic Shear Stresses

Olgun (2010)

- Horizontally layered ground – unreinforced case
- Seismic shear stresses induced as a result of shaking





τ_{soil} - unimproved

$$CSR = 0.65 \frac{\tau_{max}}{\sigma'_v} \frac{1}{MSF} r_d$$

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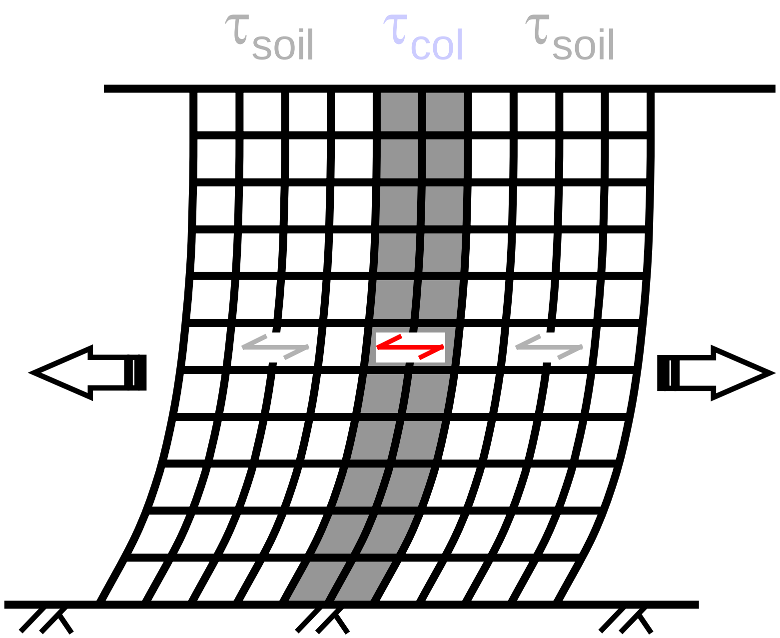
Shear Stress Reduction

- Column <u>assumed</u> to deform as a shear beam
- $\tau_{column}/\tau_{soil} = G_{column}/G_{soil}$



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Shear Stress Reduction

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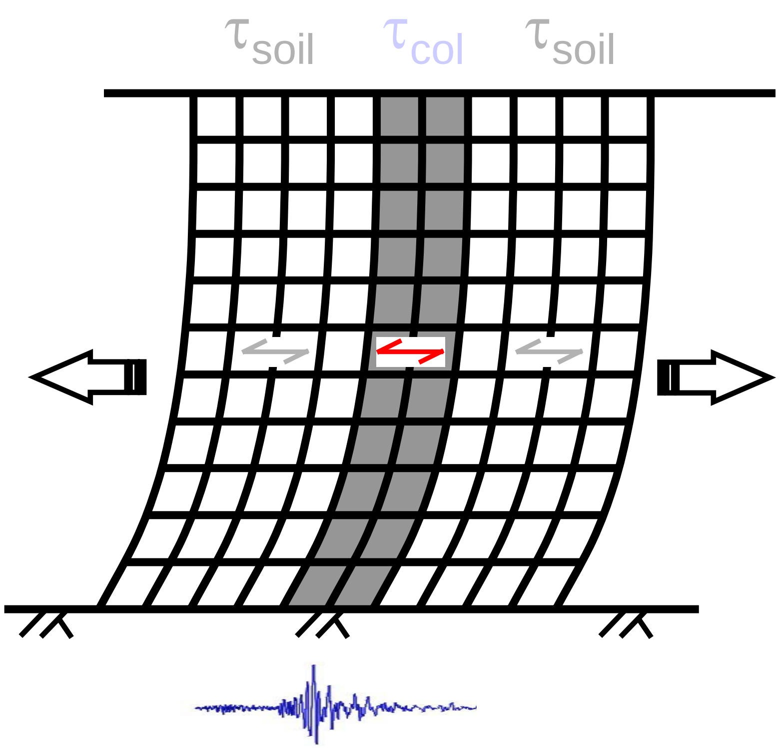

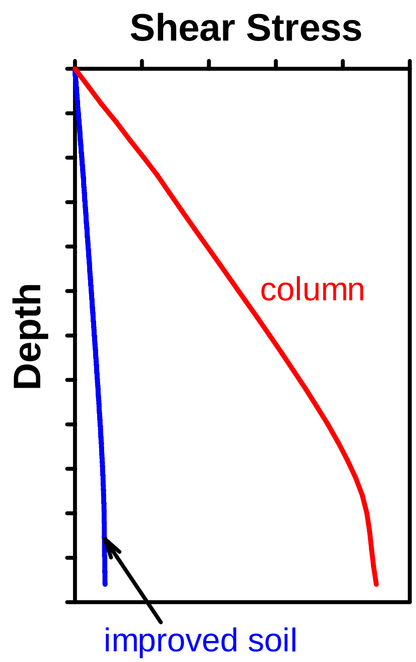

$$CSR = 0.65 \frac{\tau_{max}}{\sigma'_v} \frac{1}{MSF} r_d$$

$$CSR_{treated} = K_G \cdot CSR_{untreated}$$

- $G_{column}/G_{soil} = 10$
- Replacement ratio 20%

For shear beam behavior $K_G < 1$

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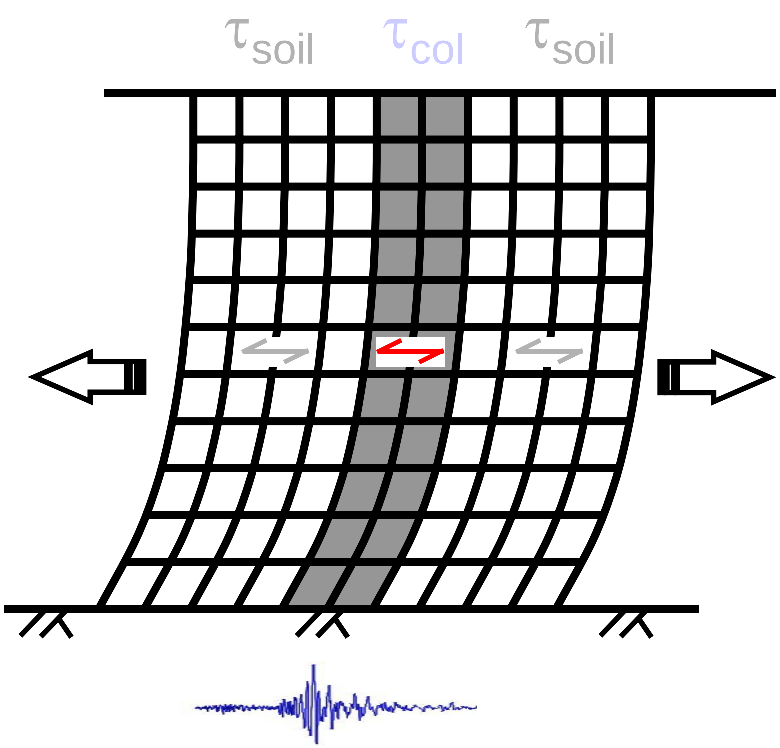
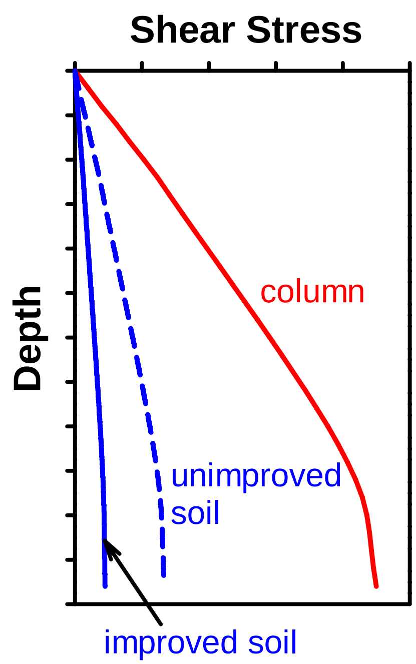
Shear Stress Reduction

Olgun (2010)

- Column <u>assumed</u> to deform as a shear beam
- $\tau_{column}/\tau_{soil} = G_{column}/G_{soil}$





$$CSR = 0.65 \frac{\tau_{max}}{\sigma'_v} \frac{1}{MSF} r_d$$

$$CSR_{treated} = K_G \cdot CSR_{untreated}$$

- $G_{column}/G_{soil} = 10$
- Replacement ratio 20%
- 65% reduction in CSR

For shear beam behavior $K_G < 1$

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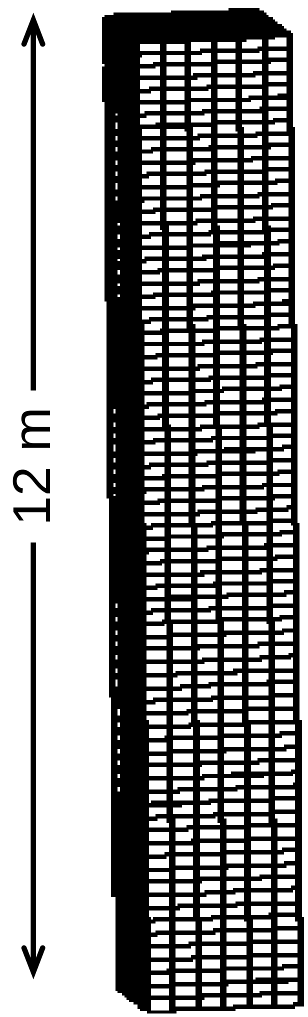
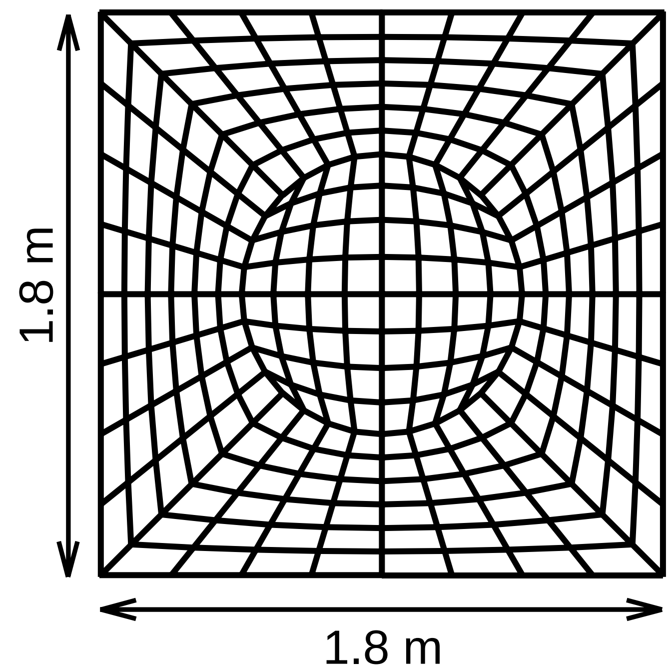
Numerical Analyses

Olgun (2010)



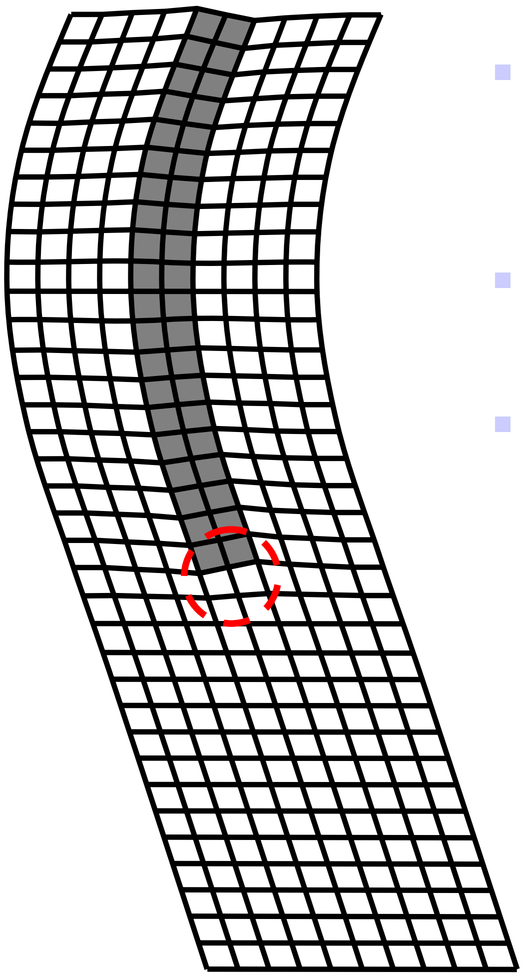
- 3D finite element analyses with Dynaflow
- ~14,000 elements
- 6 m long stiff column
- A repeating sequence of columns (blanket treatment)
- A range of column-to-soil stiffness ratios
- Various column diameters and spacings
- Model shaken at the base along both horizontal directions



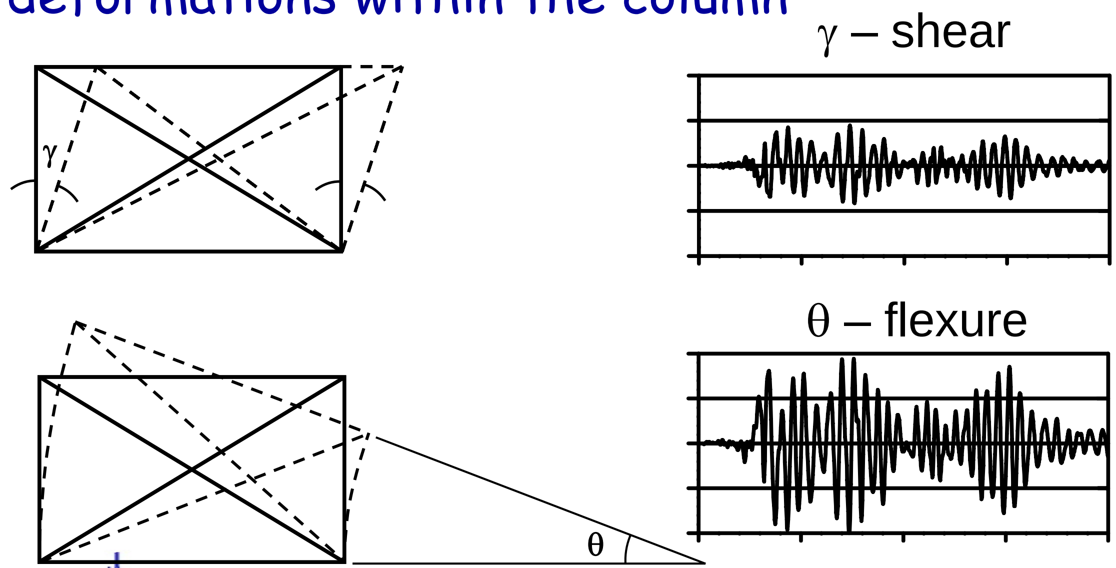
- 90 cm column diameter
- S/D = 2 replacement ratio of about 20%

Results – Schematic of the Deformed Mesh

Olgun (2010)



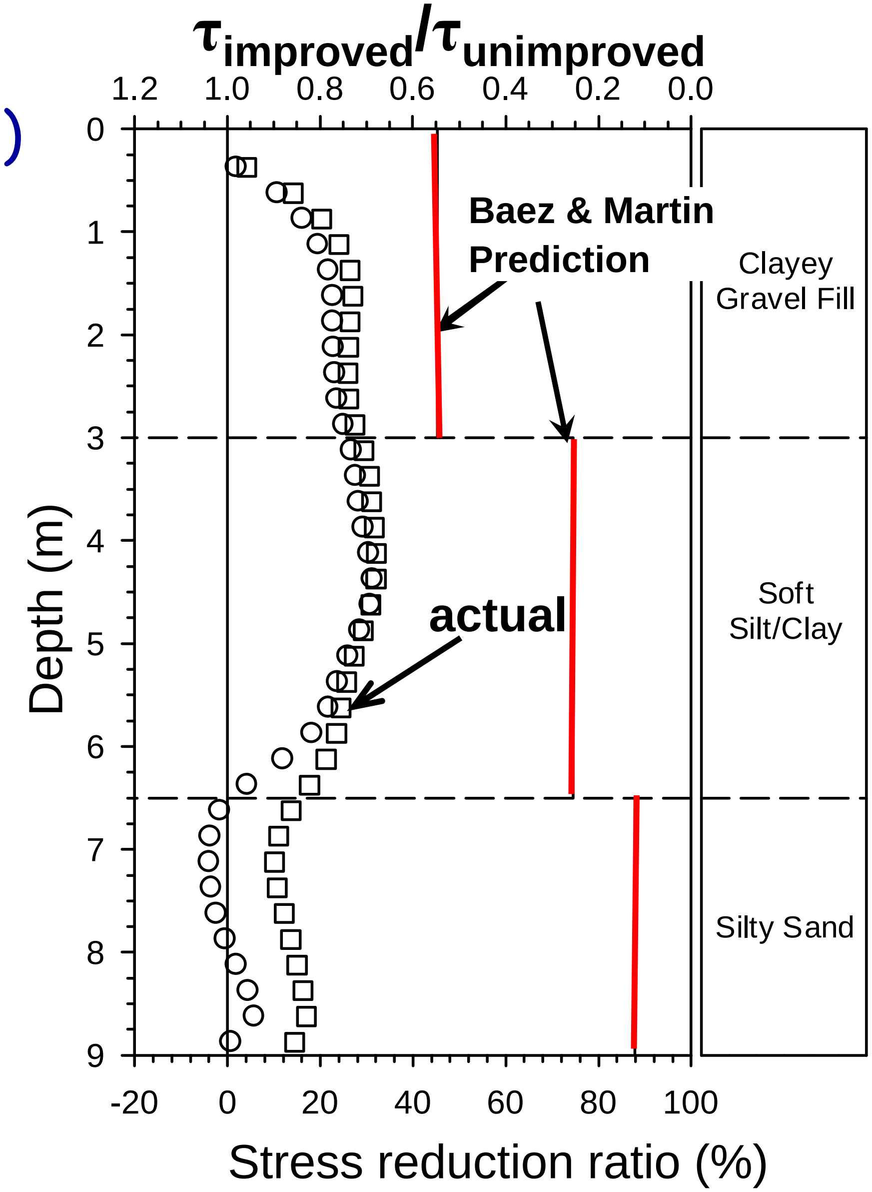
- Soil moves in pure shear, except near the column side and tips (i.e. end effects)
- Apparent flexural deformation of the stiff column within the soft ground – if column stiffer, then more flexure less shear
- Deformation mode of the column investigated in further detail
- Relative values of shear (γ) and flexural (θ) deformations within the column



Result? Actual shear stress reduction much less than Baez and Martin (1994) Predict:

Martin and Olgun (2009)

- τ_{zx} and τ_{zy} for 3-D analysis shown



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2010-11 NZ EQs – Key geotech issues:

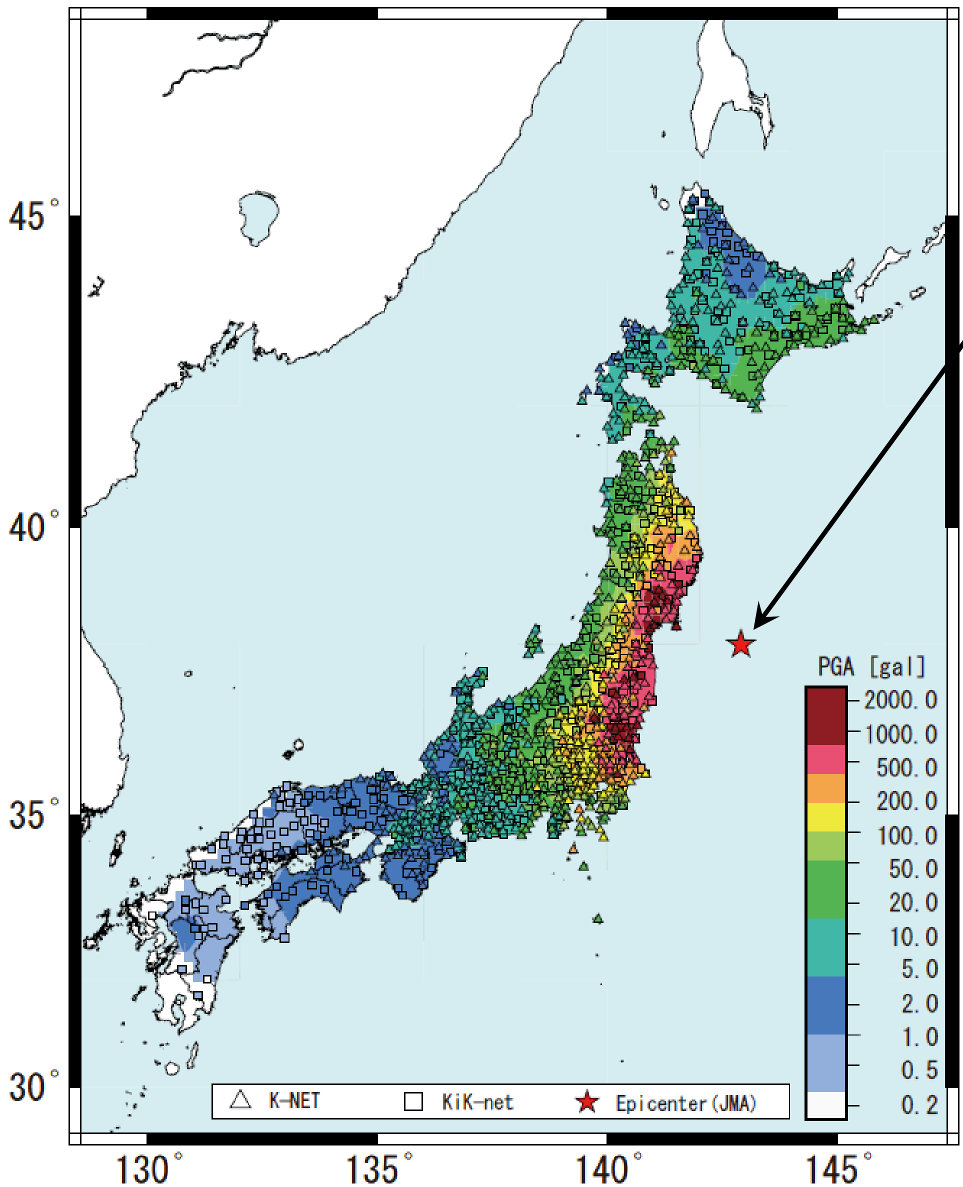
- 3 strong, damaging earthquakes; one w/ very high PGAs
- Ground motions recorded w/ dense array of instruments
- Events produced widespread and spectacular liquefaction
- Liquefaction damages were well-documented
- Numerous well-documented improved-soil sites; some performed well, some did not
- Excellent geotechnical site characterization overall

Some key lessons: 1.) better liquefaction damage prediction models (LPI); 2.) better design methods for improved ground; major NSF study underway

To Japan….

March 2011 M9 Tohoku, Japan EQ & Tsunami

Peak Ground Acceleration (surface)



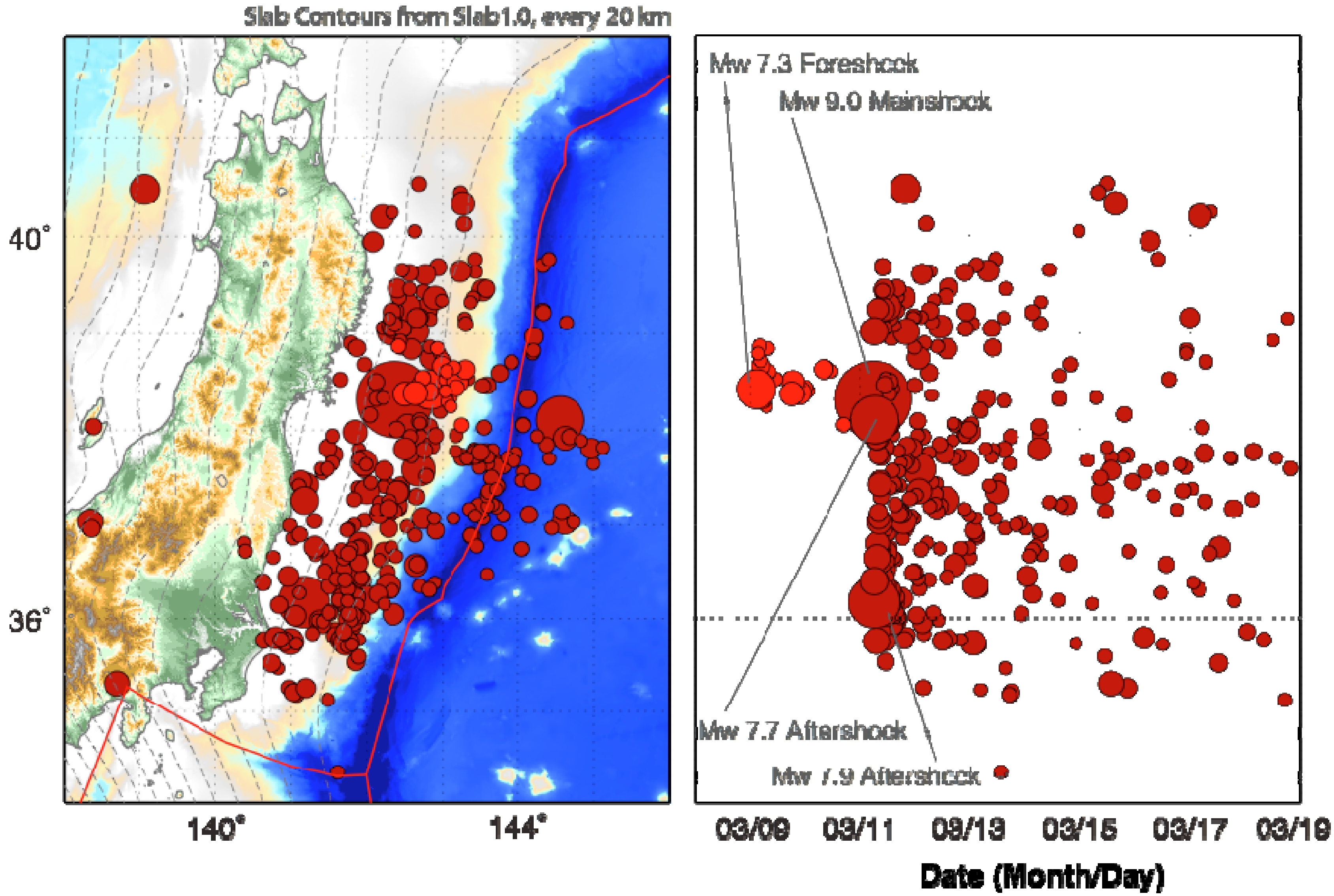
M9 epicenter ~130 km off coast of Sendai

Focus of our field study:

1. Liquefaction
2. Improved soil sites
3. Response of ports and waterfront structures

(Plot from K-NET website)

M9 Tohoku, Japan EQ, Aftershocks, and Foreshock



M9: PGAs >1g; duration~ 180 secs; M7.7 PGA 0.9g; duration ~120 sec

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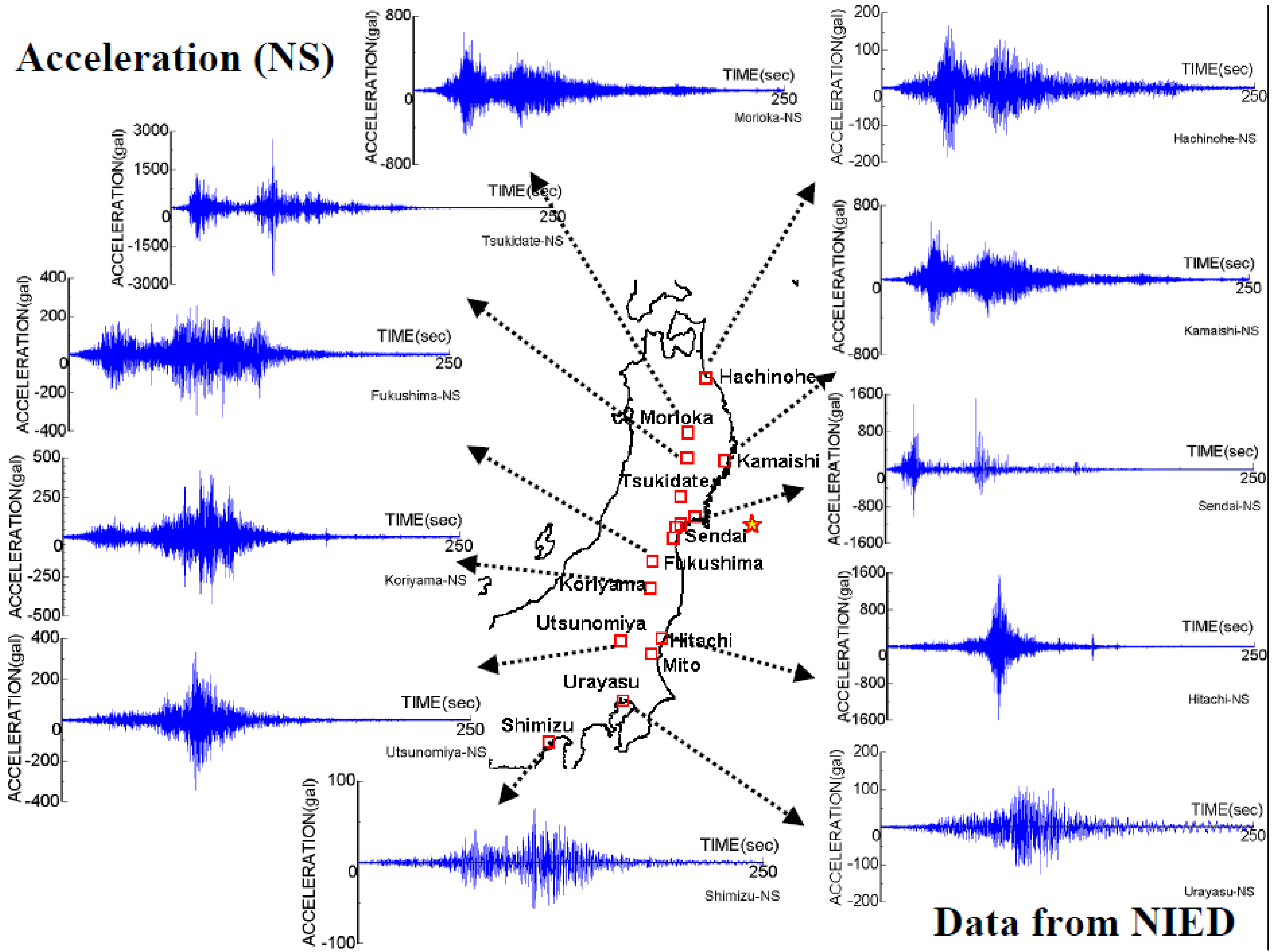
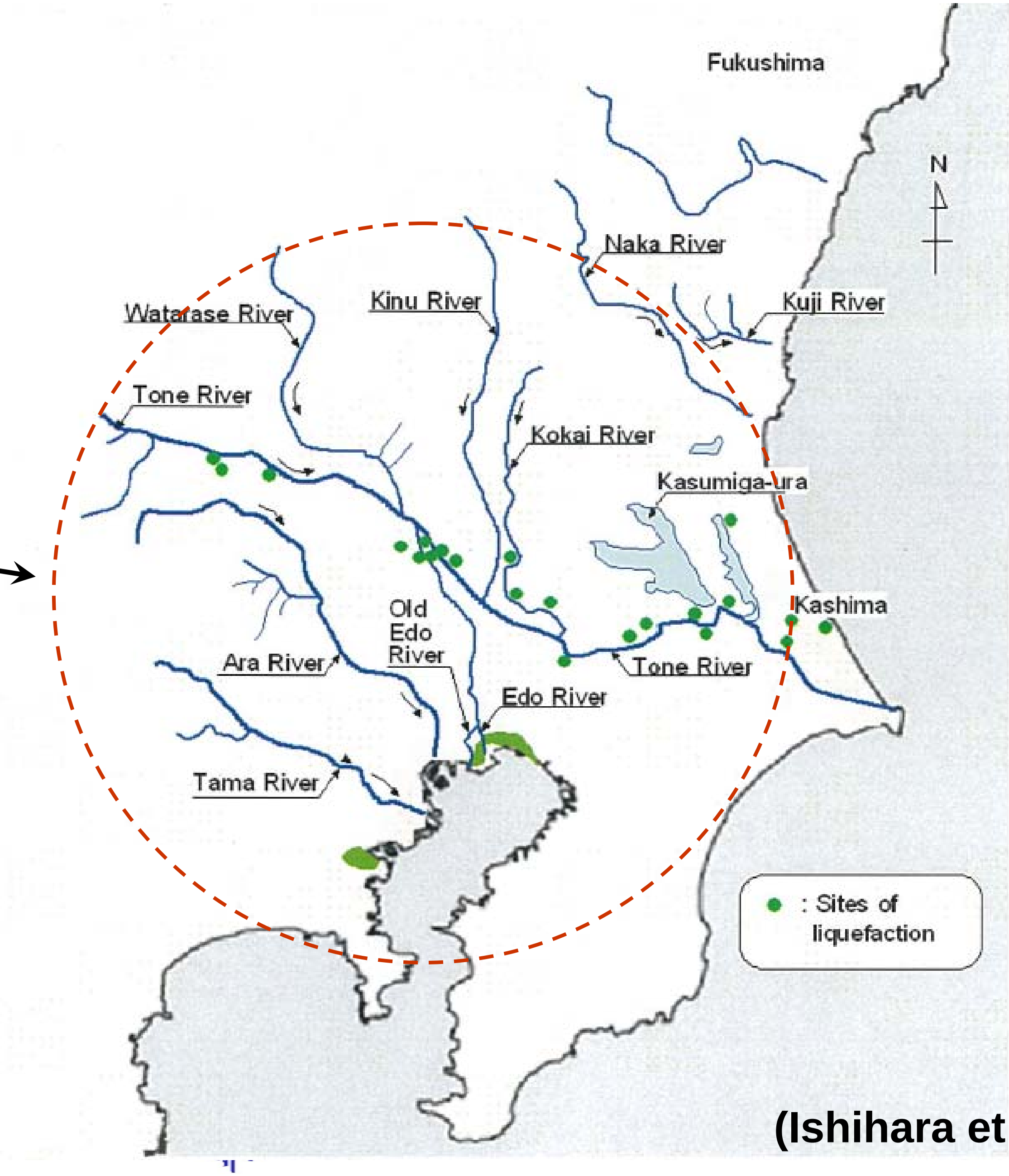
M9 Ground Motions – high amplitude, long duration

(M9: PGAs >1g, duration ~ 180 secs; M7.9 PGA 0.9g, duration ~120 secs)



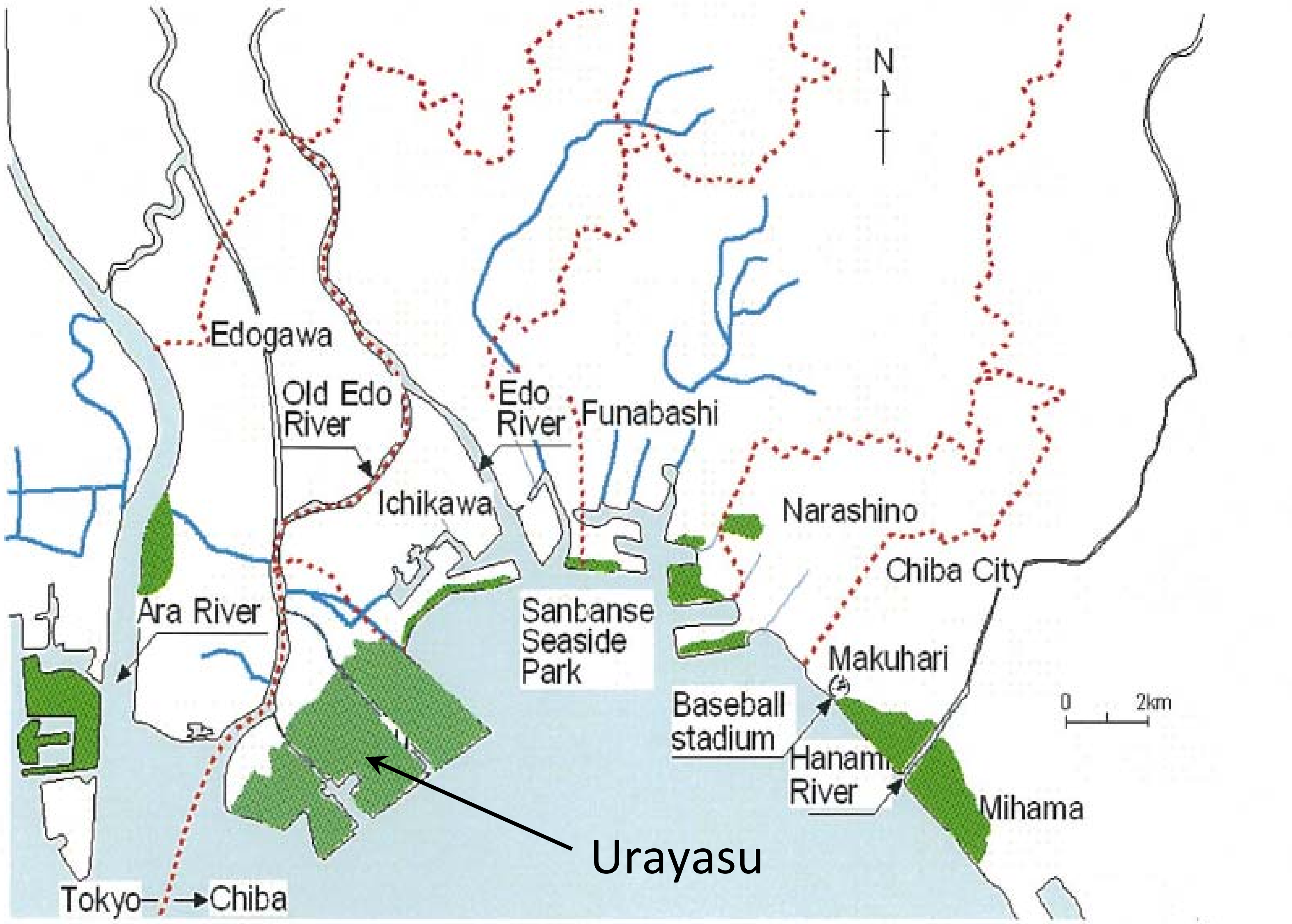
M9 Tohoku liquefaction areas – Kanto region

Kanto region



(Ishihara et al., 2012)

Liquefaction - Tokyo Bay - M9 PGA 0.2g; 150 secs.



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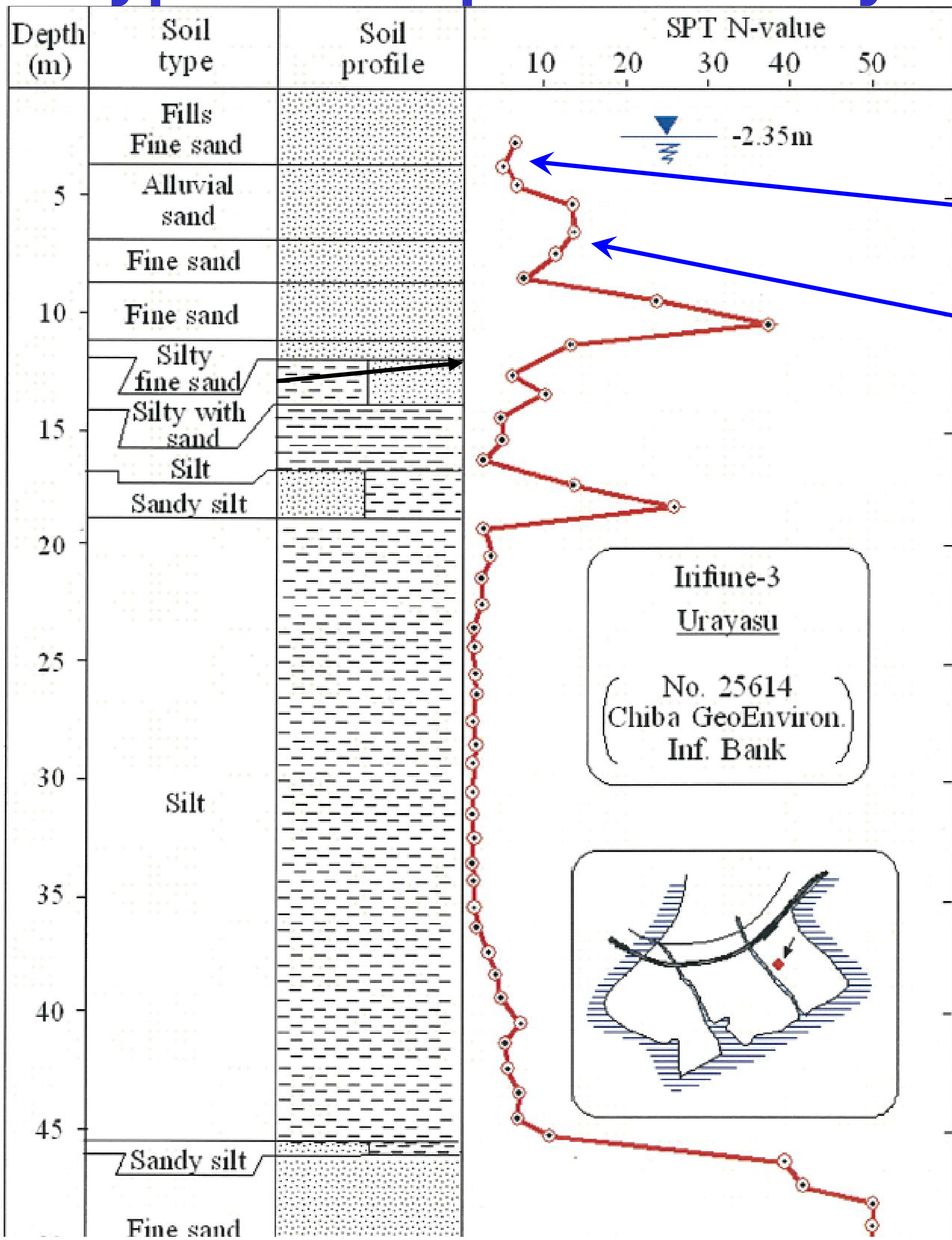
(Ishihara et al., 2012)

Typical soil profile – Urayasu - Midtown



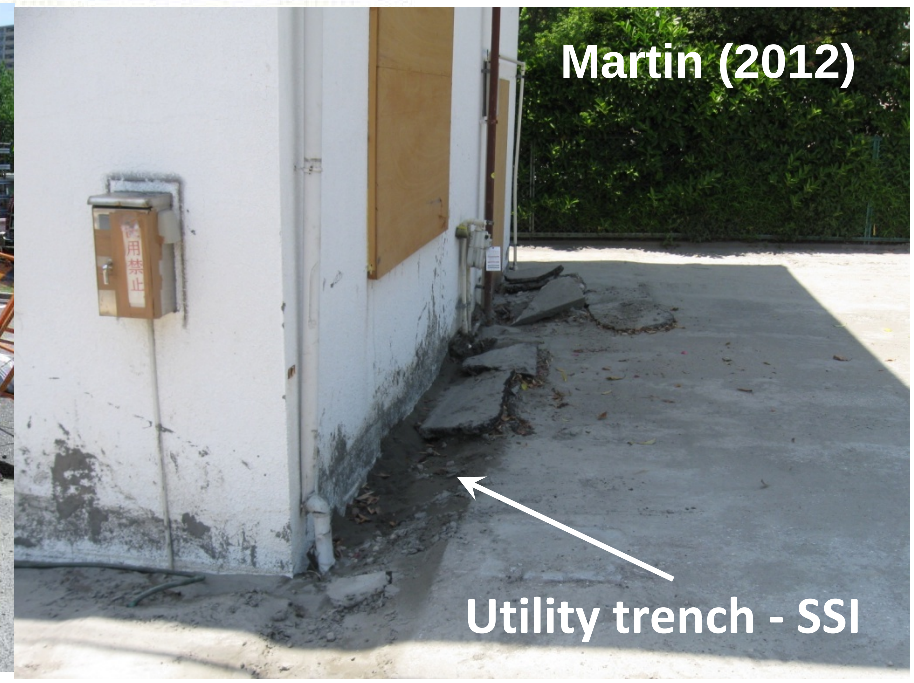
(Ishihara et al., 2012)

Fill commonly liquefied

Alluvium did not; aging due to fines?

- No distress in alluvial soils anywhere
- apparent extra cyclic resistance of alluvial sands not fully reflected by SPTs
- Fills did not liquefy in M7.9 aftershock, only M9

Some sites with significant liquefaction in midtown



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BABiES R US
Martin (2012)
Martin (2012)
Martin (2012)
Martin (2012)

Liquefaction along waterfront Urayasu

Martin (2012)

Tokyo Bay: Successful Ground Treatment

Key geotechnical lessons – Kanto region:

- High-amplitude, long duration EQ – unique opportunity to test geotech models and infrastructure response
- Ground motions well recorded; first well-recorded mega-thrust EQ (implications for Pacific NW)
- Liquefaction up to 400 km from epicenter (150 km from rupture)
- Fills were more vulnerable than natural soils w/fines

<u>Some key lessons</u>: 1.) Liquefaction triggering- aging effects on cyclic resistance of alluvial soils not fully captured by CPT, SPT; due to aging of fines?

2.) Magnitude Scaling Factors (MSF) for M9 – fill soils in Tokyo would not have liquefied if not for 150 seconds of shaking;

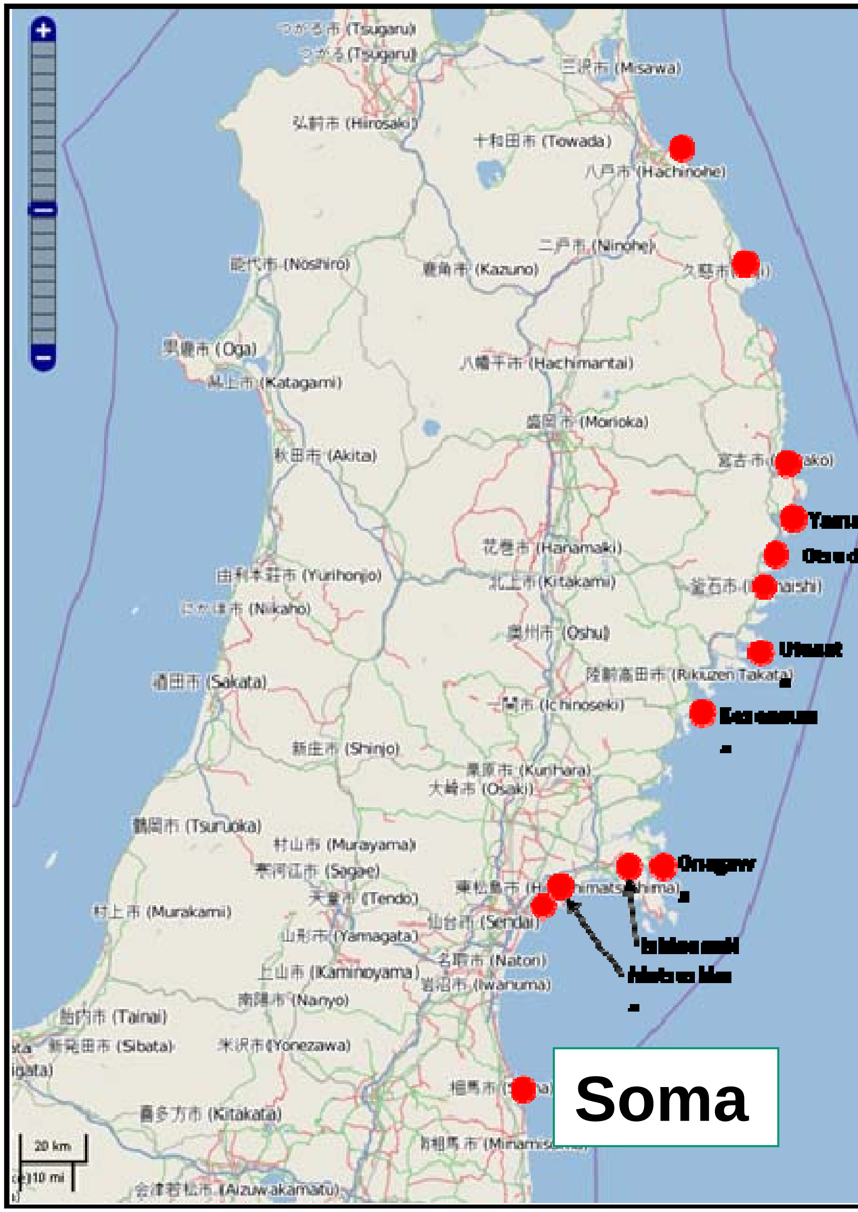
3.) Soil-structure interaction effects– utility trenches, foundations;

4.) Settlement methods - calibration

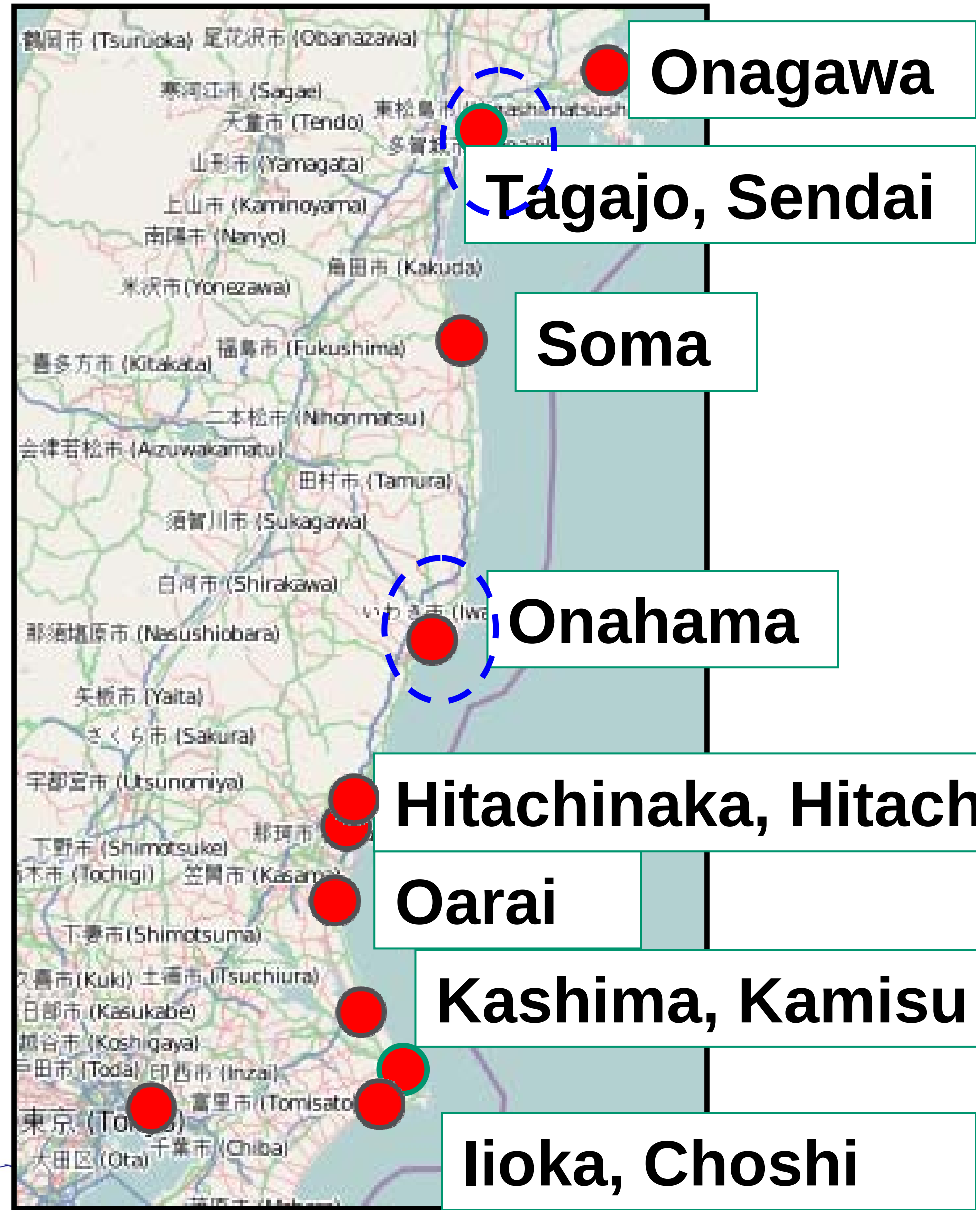
5.) Ground improvement effectiveness

Ports and waterfronts

NORTHERN PORTS



SOUTHERN PORTS (our team)



Summary observations at ports, waterfronts:

- Damages strongly related to geotechnical conditions
- Despite PGAs >0.5g recorded at numerous ports, and 5 close-by recording sites with PGAs >1.4 g, quay walls performed surprisingly well ***<u>except where liquefaction was present</u>***
- The tsunami exacerbated liquefaction-related damages
- Ports developed mainly in natural soils (cut ports), performed better than ports with fills (fill ports), unless fills were improved
- Many ports contained natural soils with SPT, CPT values that should have liquefied under M9 and high-PGA loadings;
- For example SPT $N_{1,60}$ = 25, with 0.5g PGA should have FS_{Liqn} <0.5 for M9 with current consensus MSF (0.65)
- Improved soil sites performed well relative to non-treated sites

Sendai Port (Tagajo)

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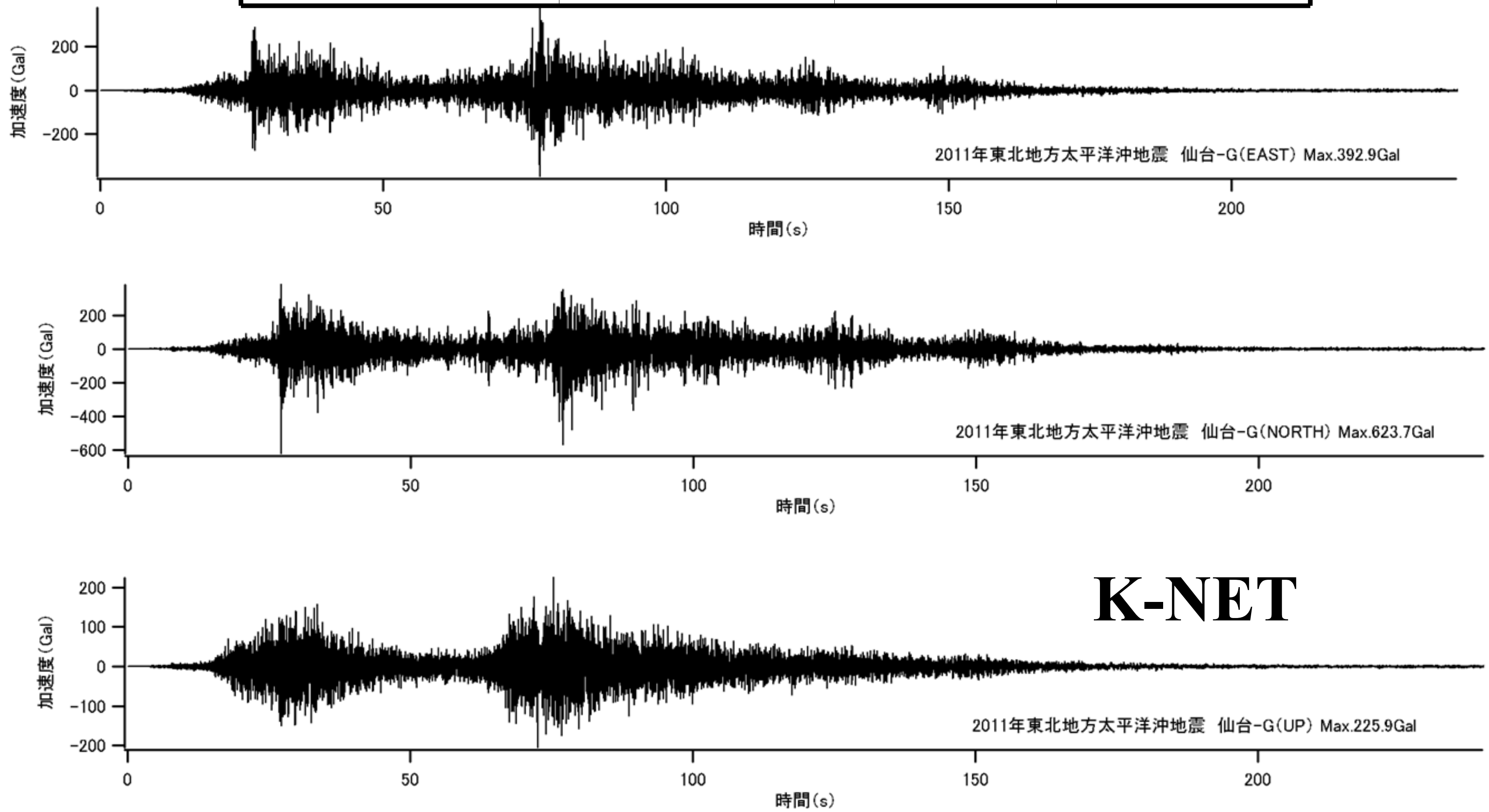
Sendai Port PGA Time Histories (PARI)
PGA: 0.40/0.64g

Sendai Port and Nippon Oil	Sendai Port	PARI	0.636g
	Shiogama	KNET	2.009g
	Sendai	KNET	1.548g



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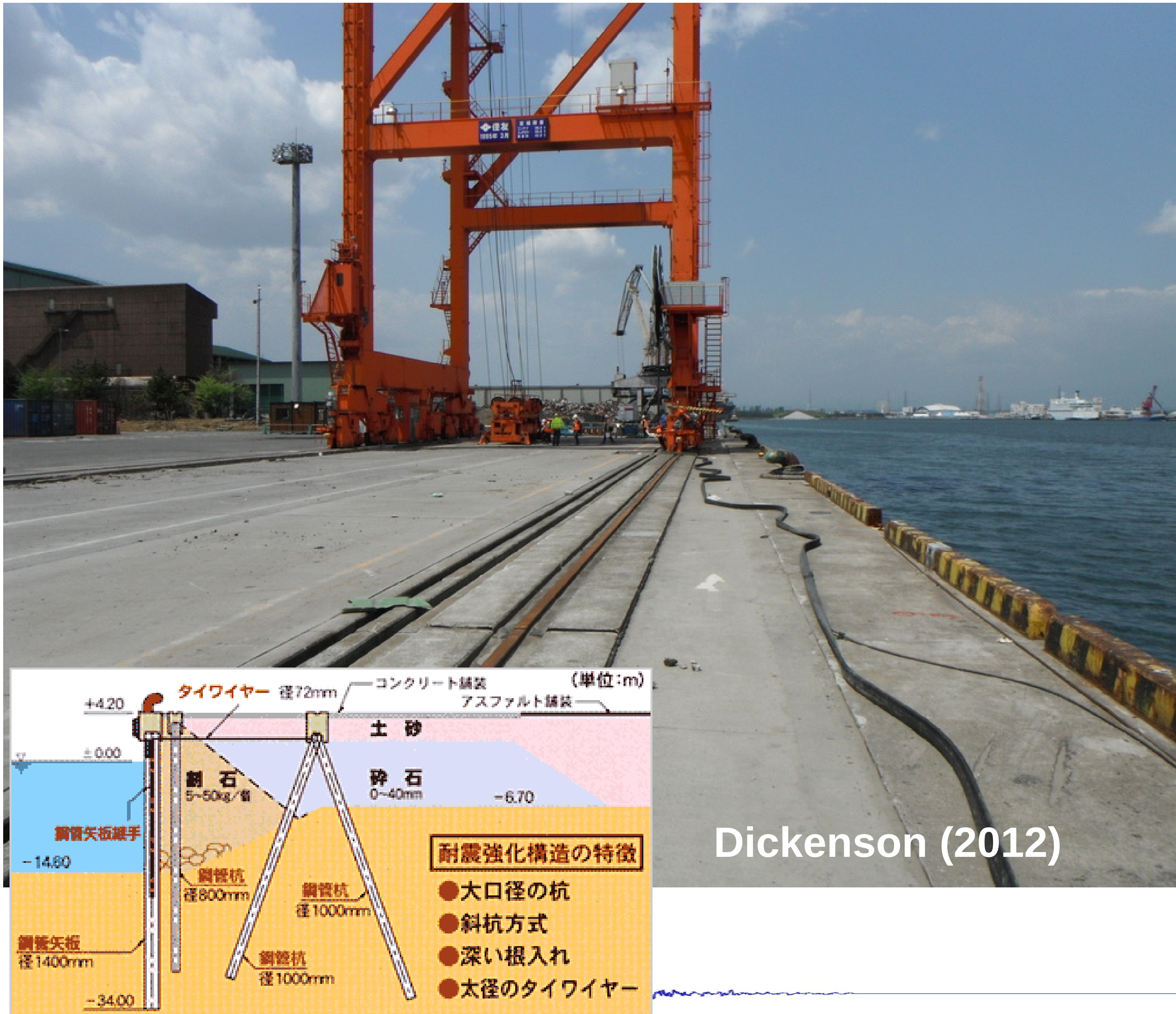
Sendai Port – Part natural soils, part fill

Liquefaction along old shore line

Dickenson (2012)

Sheetpile wall failure in fill section

Undamaged Sheetpile Wharf – natural soil section



- **with PGAs of 0.6g, 1.5g, and 2g nearby, current models would predict very large movements**
- **Why no liq'n?**

Sendai Port: Successful Ground Treatment

Sendai Port: Successful Ground Treatment

Sendai Port: Hazards Associated with Liquid Fuels

Liquefaction and ground improvement

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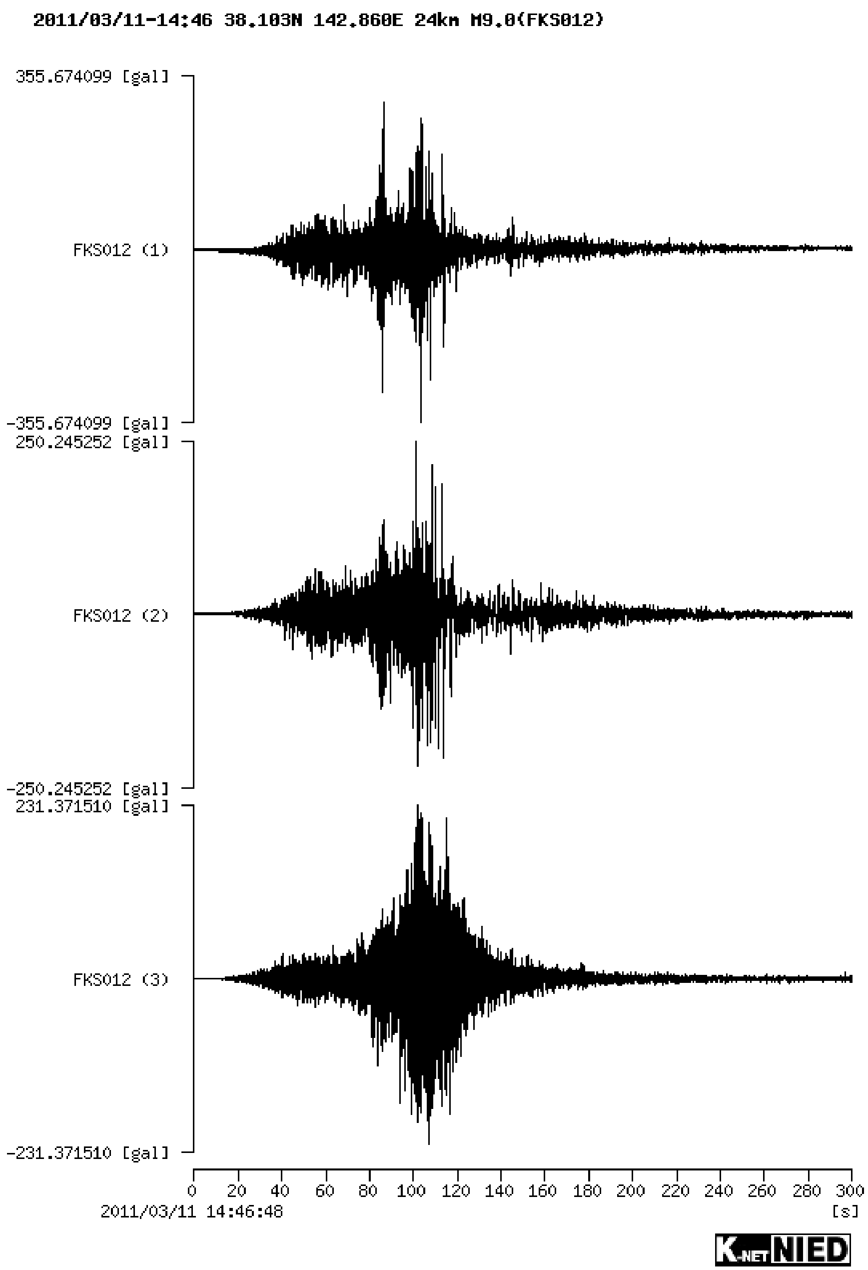


Strong Ground Motions: Onahama Port Area

FKS012: 0.36/0.26g

Deep soft soils

Port HQ: 1.46/1.11g

Shallow stiff soils

Onahama Port: Wharf No. 3 (sheetpile wall)

Onahama Port: Wharf No. 3 (cassions)

Onahama Port: Ground Failures adjacent to Pier

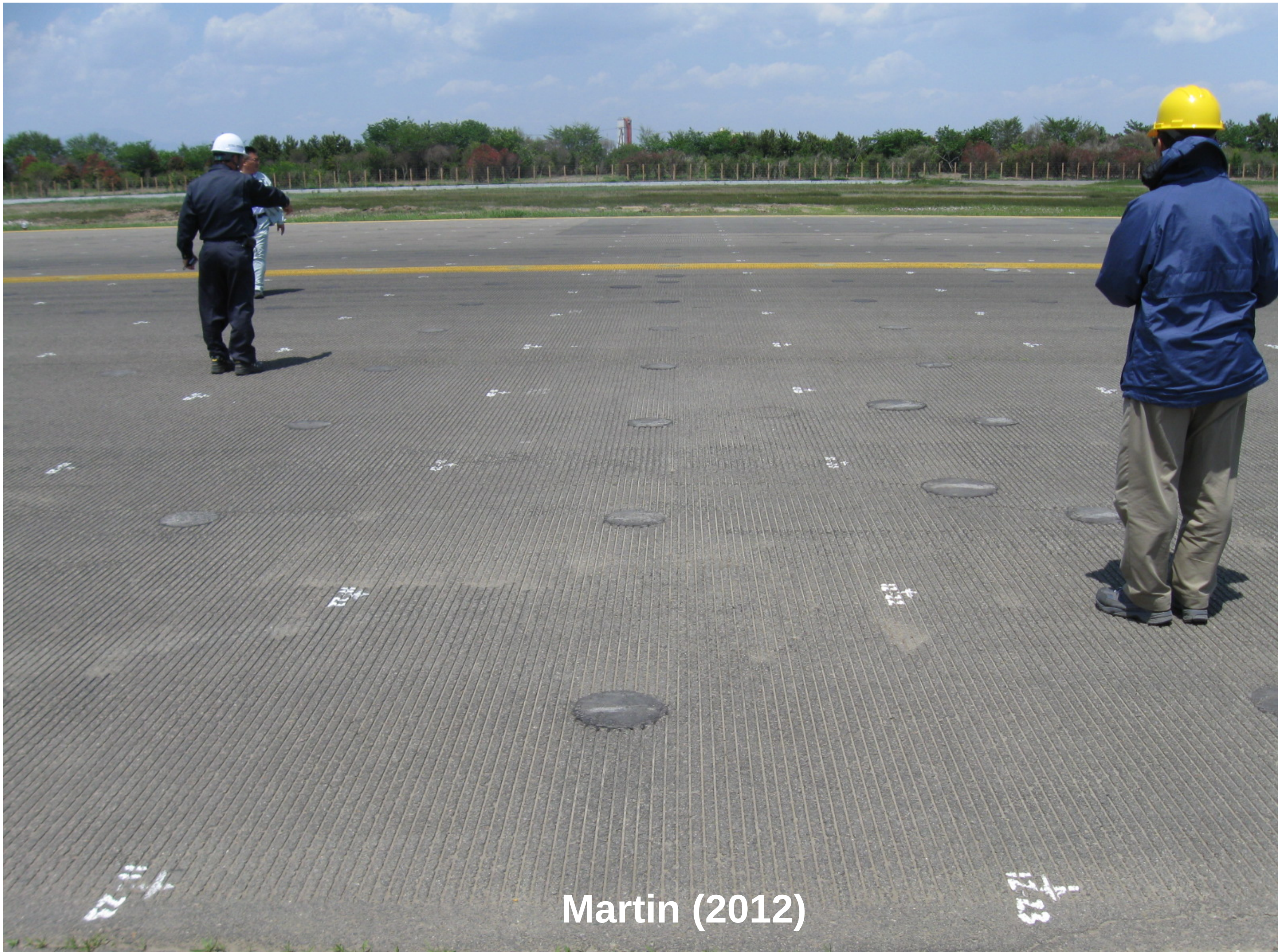
Onahama Port: Successful Ground Treatment

Sendai Airport

Sendai Airport: Successful Ground Treatment

Sendai Airport: Successful Ground Treatment

Movement along culvert near runway



Martin (2012)

Tsunami Effects - Soma Port: Wharf No. 2

Wave-Dissipating Block Displaced by Tsunami


Martin (2012)


Martin (2012)


Dickenson (2012)

Pile Capacity?

M9 Japan EQ & Tsunami – key geotech lessons

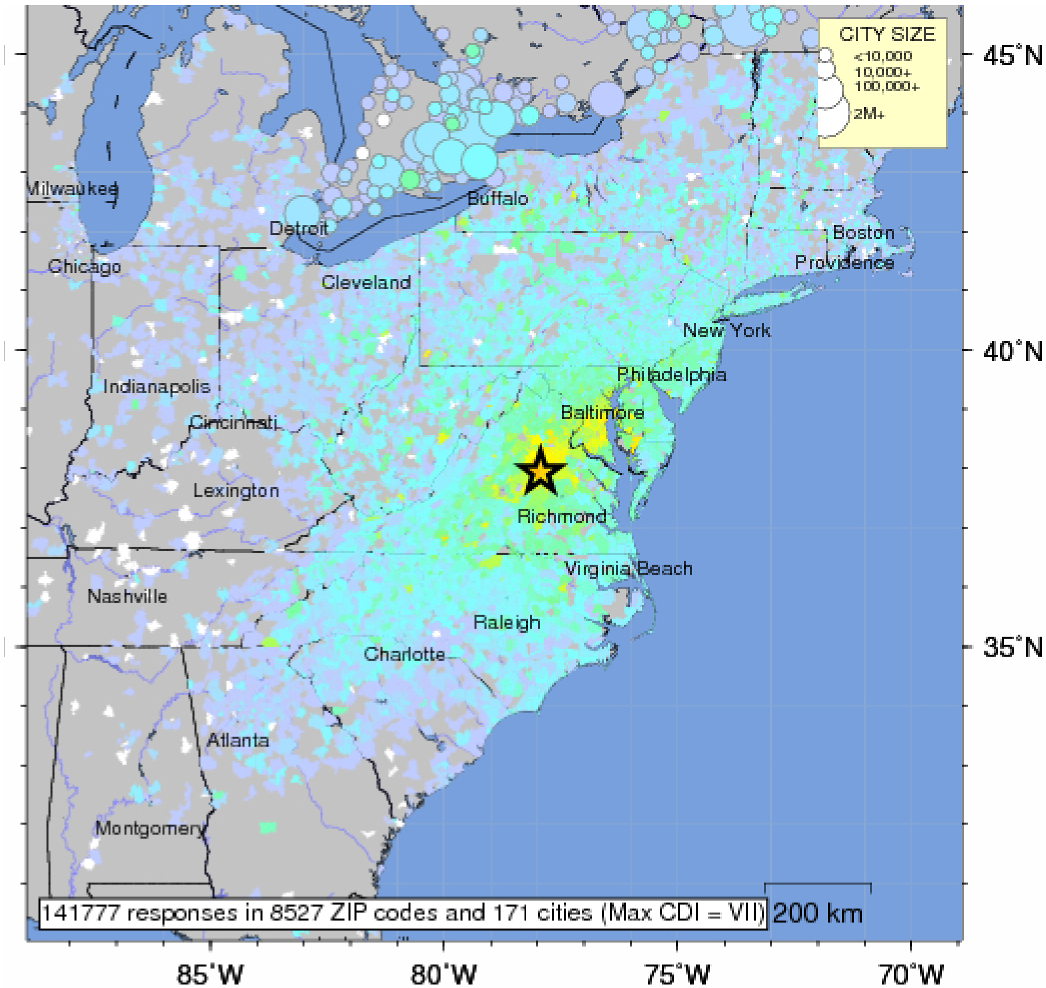
- High-amplitude, long duration EQ – unique opportunity to test geotech models and infrastructure response
- In absence of liquefaction, pile-supported wharfs, sheetpile walls, and gravity walls (caissons) performed very well under inertial loading
- Tsunami exacerbated the inertial (seismic) damages and complicated field interpretations
- Improved soil sites performed well relative to non-treated sites; ports on natural soils performed better than fill ports

Key lessons: 1.) lateral seismic coefficient K_H - the acceptable performance of gravity walls and sheetpile walls will be useful for refining design seismic coefficients based on PGA, or may lead to alternative ground motion parameters for design

2.) MSFs, study of why no liquefaction in high N value soils? 3.) Calibration of deformation models; 4.) effectiveness of ground improvement

To Virginia….

23 Aug 2011 M5.8 Mineral, Virginia, EQ



INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+
SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	V. Heavy

Processed: Sun Sep 18 04:33:19 2011

USGS (2011)

- Was “500-year” event; PGA = 0.27g 20 km from source
- Most felt earthquake in US history – low attenuation, high population density
- Felt from GA to Canada
- Shaking intensity followed regional geologic structure; shaking intensity and damage patterns selective, related to geology/soil conditions
- DC felt strong shaking; EQ felt more like an M6 there

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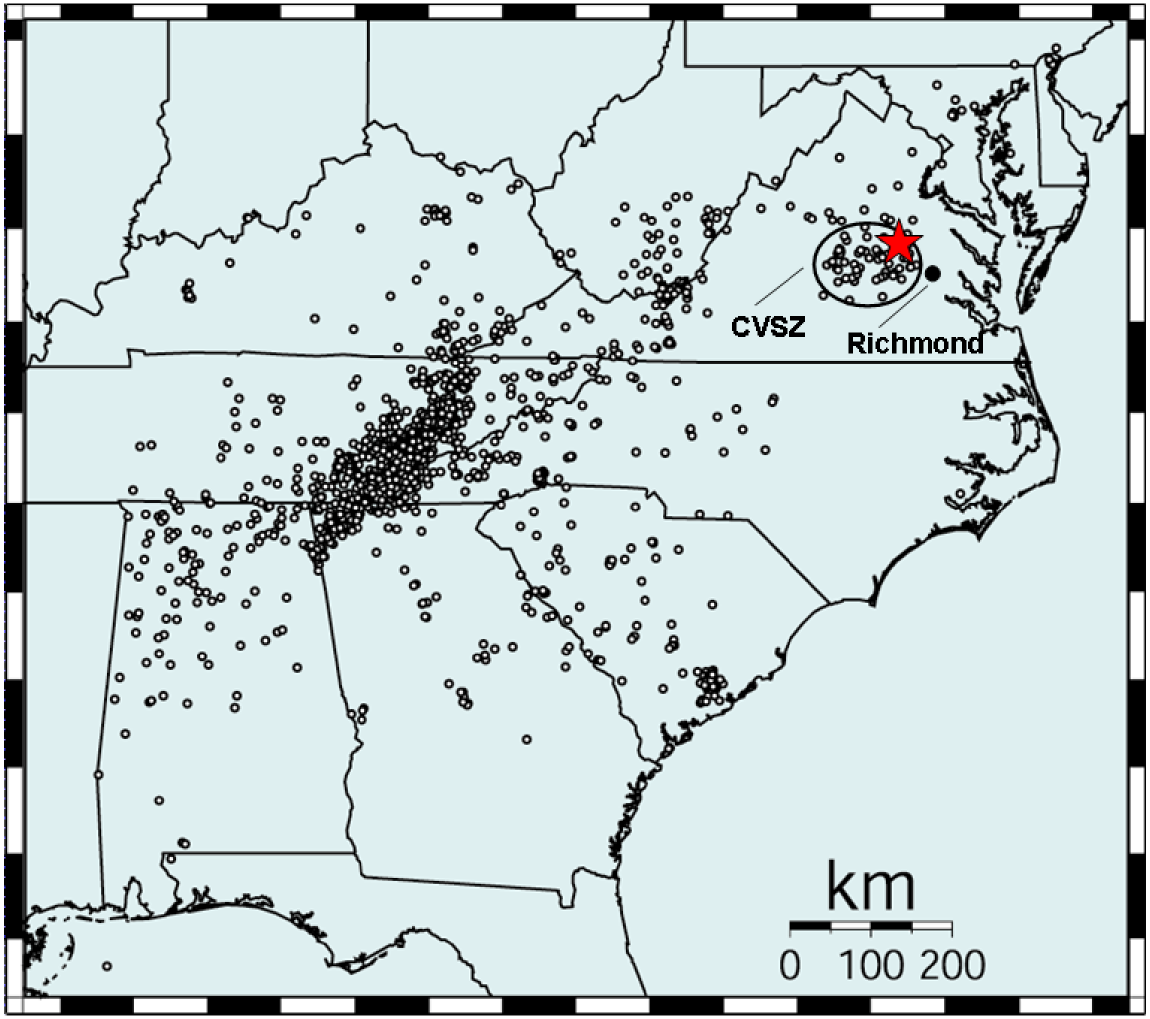
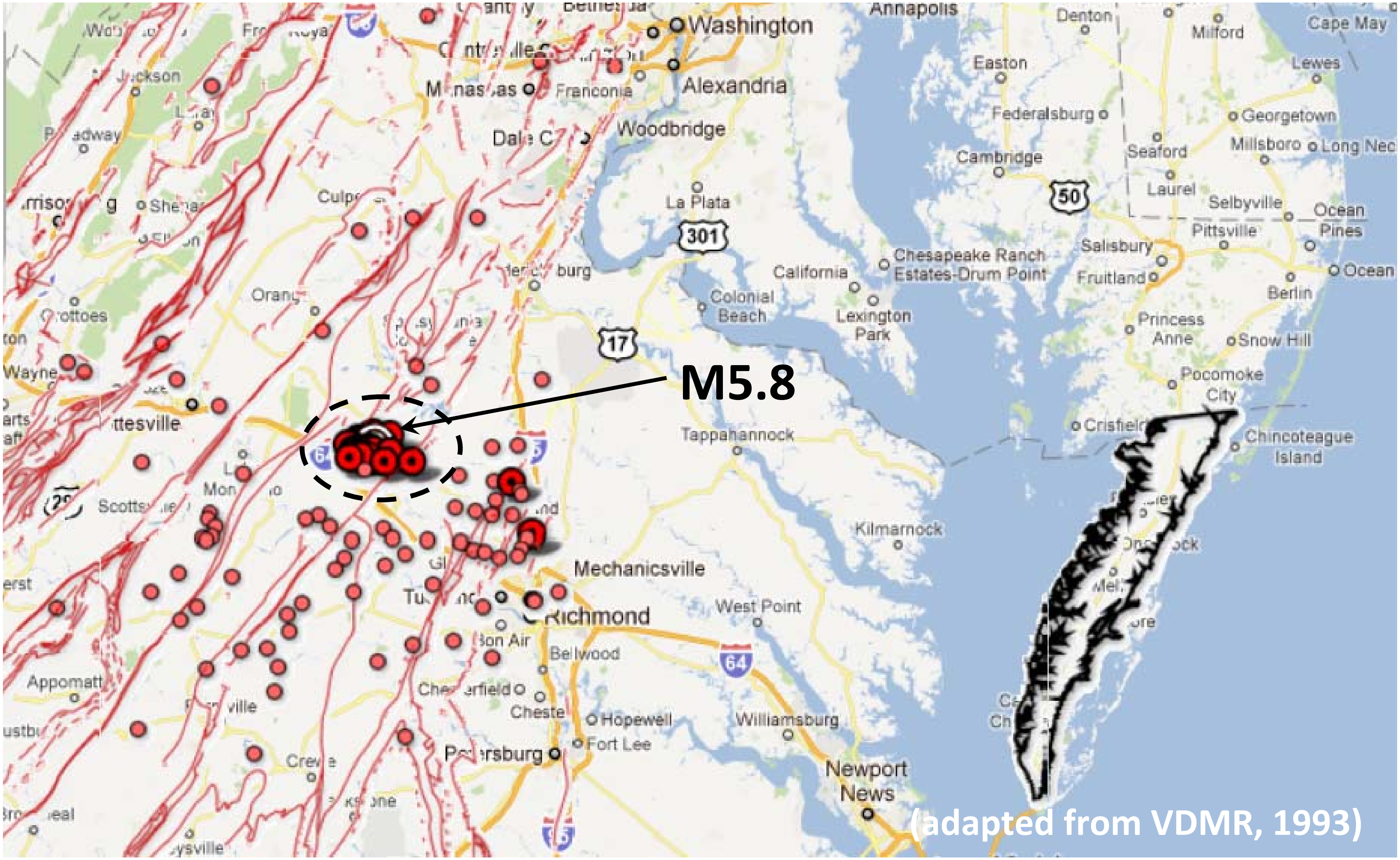
Central VA Seismic Zone (CVSZ) – VTSO



VTSO (1977-1999) recorded EQS in CEUS

- Felt earthquakes in CVZS since 1774
- About 1 felt EQ/yr.

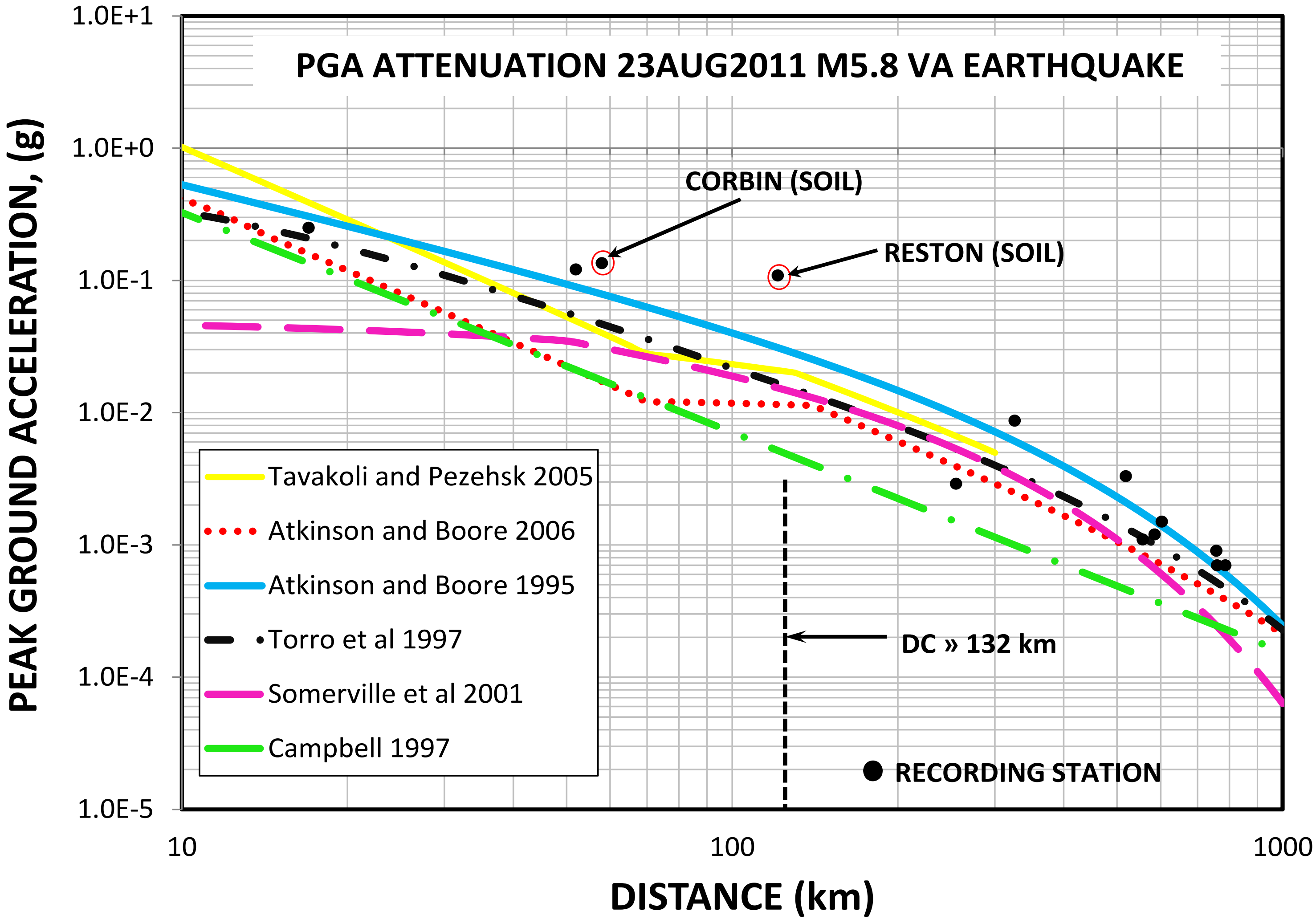
Region has NE trending faults – all "old"



Summary of key Issues

- Event was poorly recorded; no usable recordings in DC
- No geotechnical failures; but important geotech observations
- Shaking intensity and damage pattern strongly related to geological and geotechnical conditions
- Shaking intensity strongest along NE-SW corridor along strike of regional geological structure
- Intensity in DC (132 distance) had stronger shaking relative to many areas close to source; some structures got into trouble
- Zones of heaviest damage were on soft deposits with high velocity impedance contrast
- Attenuation models did not adequately capture motions; too poorly recorded to make real gains for efforts such as NGA east
- NAPS performed well; still implications for nuclear facilities
- Highlighted under-appreciated seismic vulnerability of CEUS

MEASURED ACCELERATIONS AND ATTENUATION MODELS



Eddy & Martin (2012)

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Summary of Observations

- ➢ Moderate damage in epicentral region to new and old structures
- ➢ Light to moderate damage 120 to 180 km away in DC region
- ➢ Damage concentrated mainly in unreinforced masonry & precast connections

LOUISA, VA

MINERAL, VA

WASHINGTON,DC

ALEXANDRIA, VA

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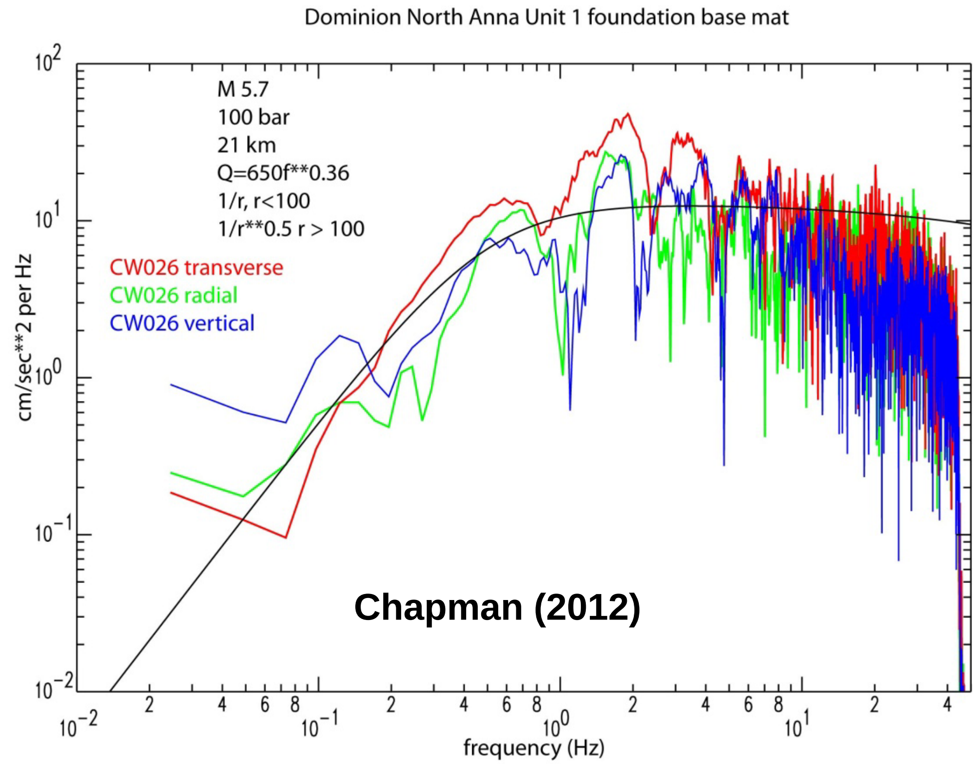
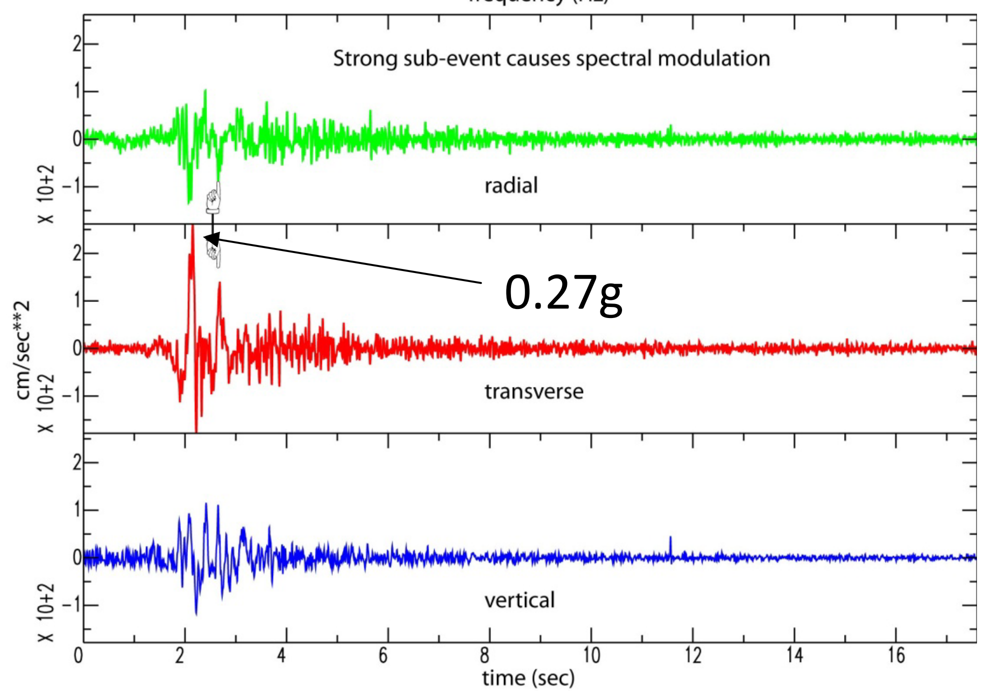
NAPS Performed well although design basis was exceeded





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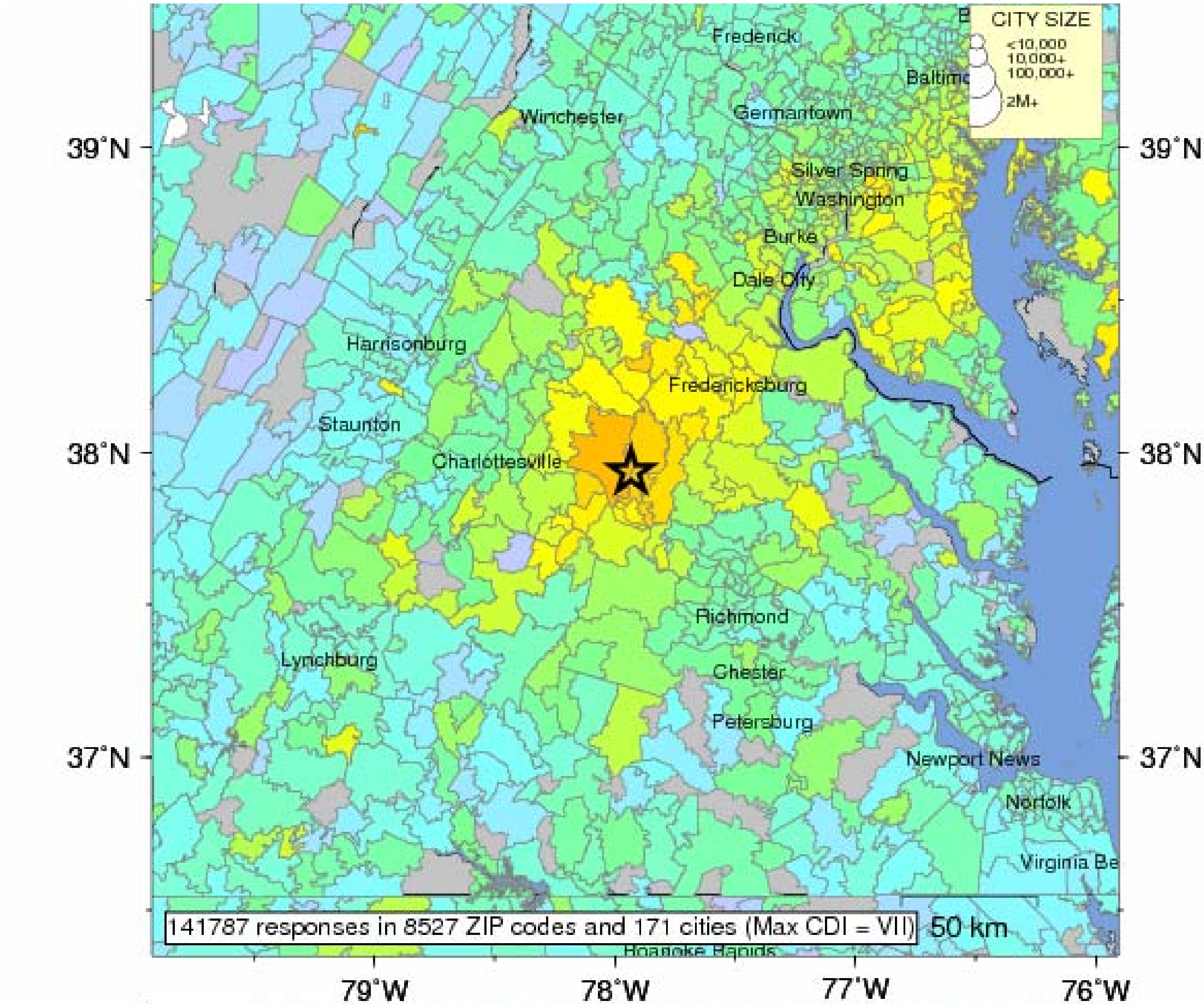
Uptick in Intensity in DC region



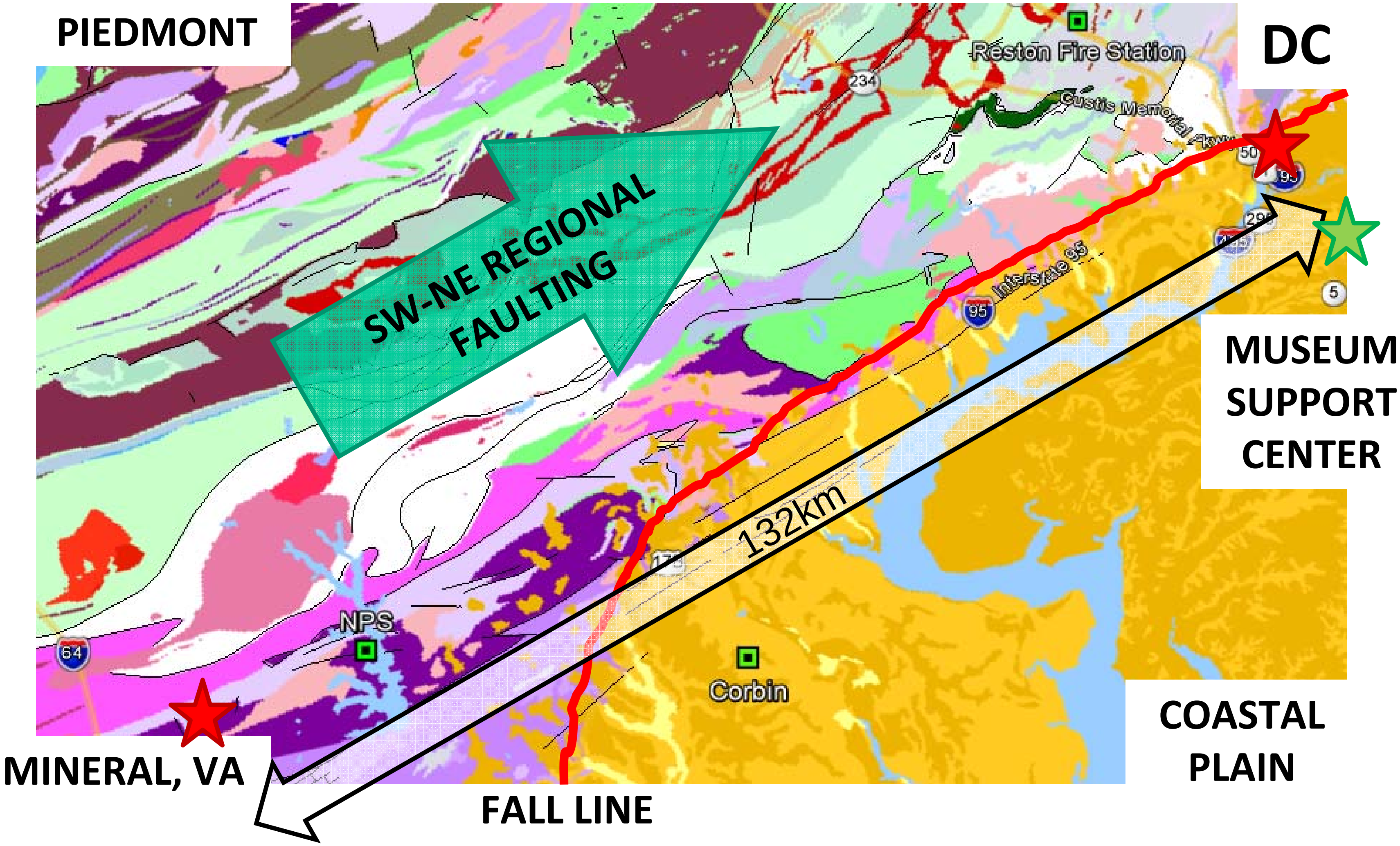
INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+
SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	V. Heavy

USGS (2011)

Vienna, VA (DC region)

(adapted from MSNBC)

INCREASED INTENSITY OF SHAKING IN DC



Eddy & Martin (2012)

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Increased Intensity of shaking in DC

- The event had strong directivity; highest shaking energies were produced in the NE direction – directly toward DC

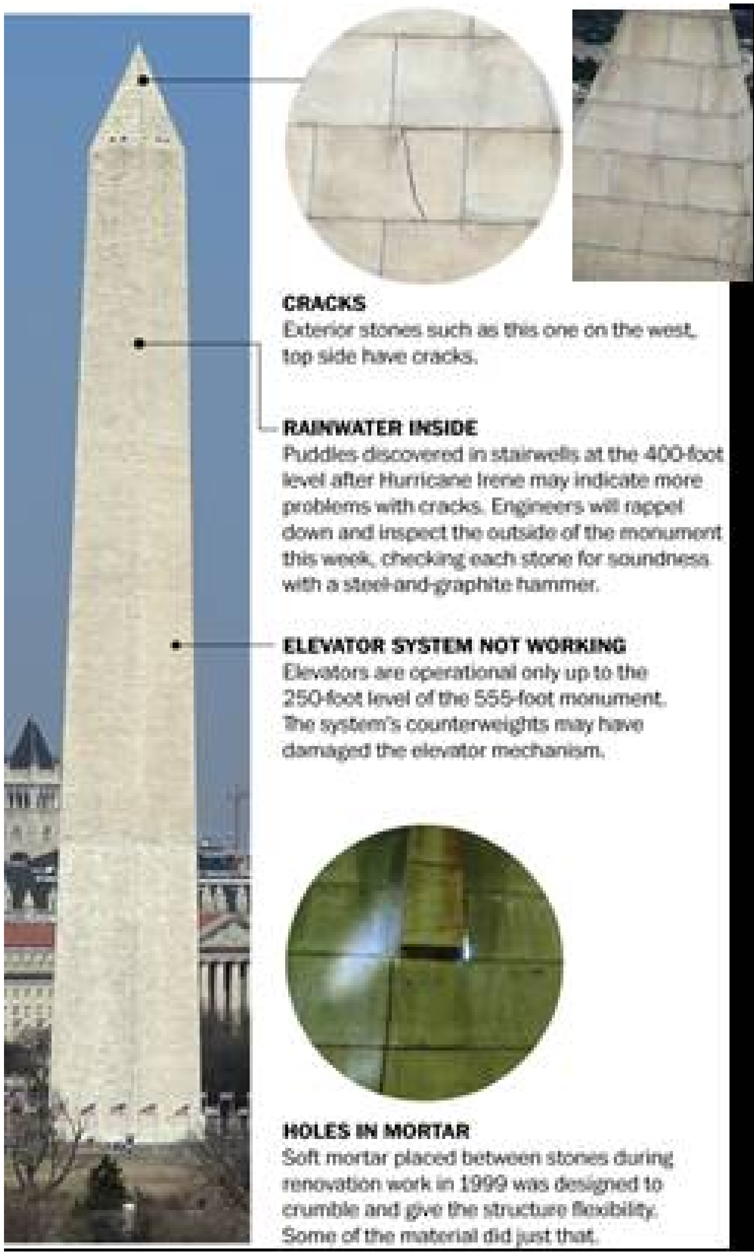
- Directivity related to mechanics of complicated fault rupture – ruptured fault trended SW-NE and was actually 3 smaller earthquakes separated by about 1 sec– **this could be CEUS characteristic**

- Latter events occurred on the NE end of the fault, exaggerating the directivity “slingshot” effect toward DC

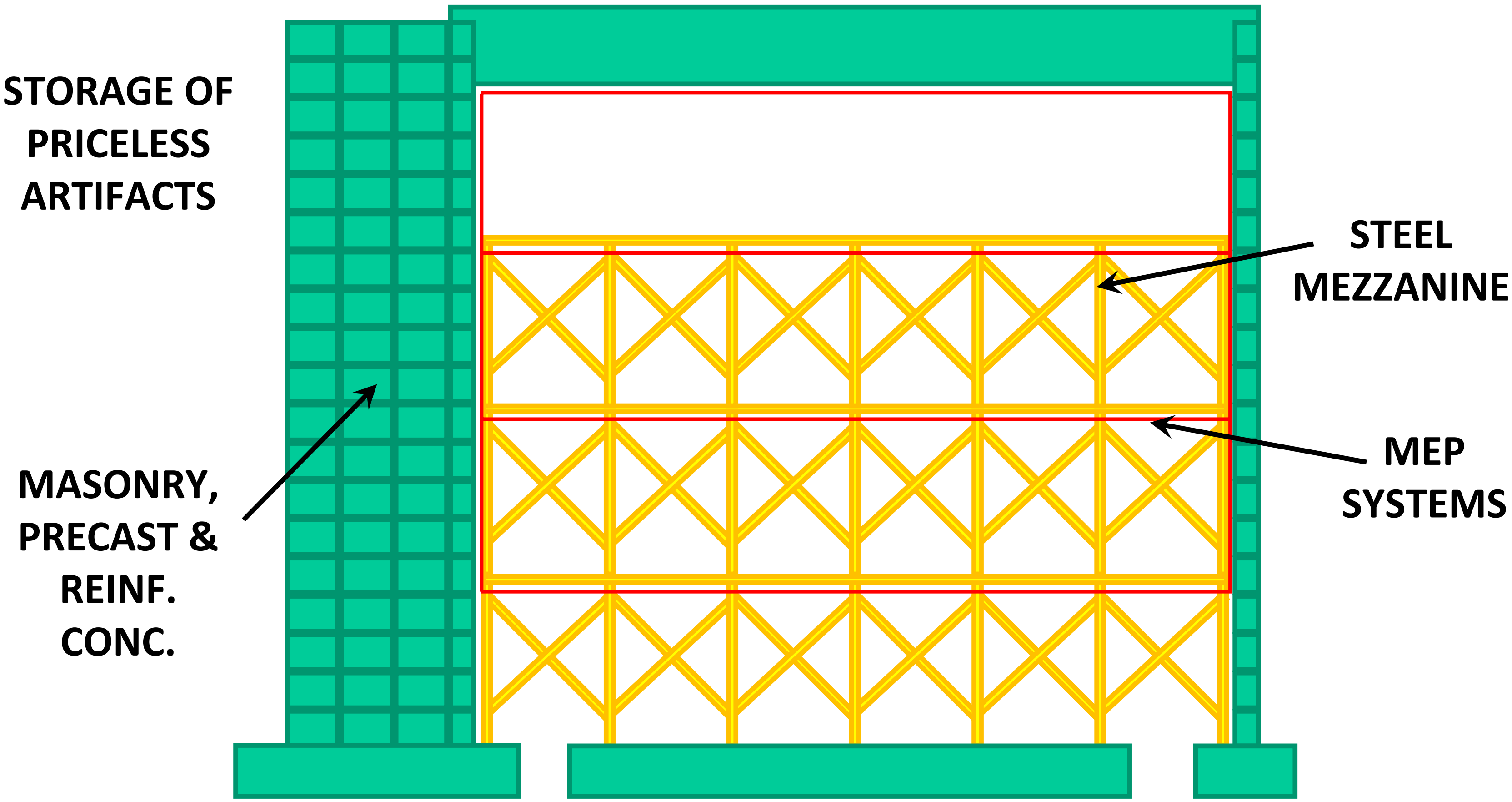
Increased Intensity of shaking in DC

- In addition to generating maximum energies in the NE direction, regional faulting is also oriented SW-NE
- Thus the direction of the strongest energy was also oriented along the strike of regional geological faulting. This provided higher apparent velocities and a more efficient propagation path toward the NE – directly toward DC
- Also, the Fall Line is oriented in SW-NE direction; hard rock is close to the surface and overlain by thin stack of sediments; this Vs impedance contrast strongly amplifies ground motions– very hard rock close to surface is CEUS characteristic
- These factors, combined, meant that the event felt more like an M6 event in DC than M5.8 – rock PGAs 0.02-.04g

Dynamic Soil–Structure Interaction Analysis and 3-D EQ Motion Simulations



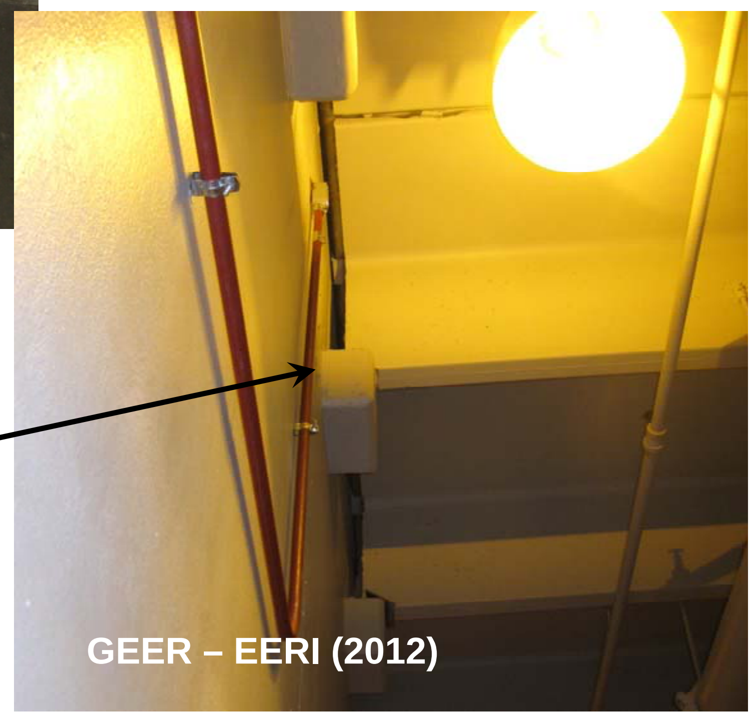
MUSEUM SUPPORT CENTER - OBSERVED DAMAGE



- STEEL MEZZANINE AND MASONRY/PRECAST SHAKING AT DIFFERENT PERIODS
- DAMAGE TO SPRINKLER SYSTEM
- STRUCTURE NEAR FAILURE

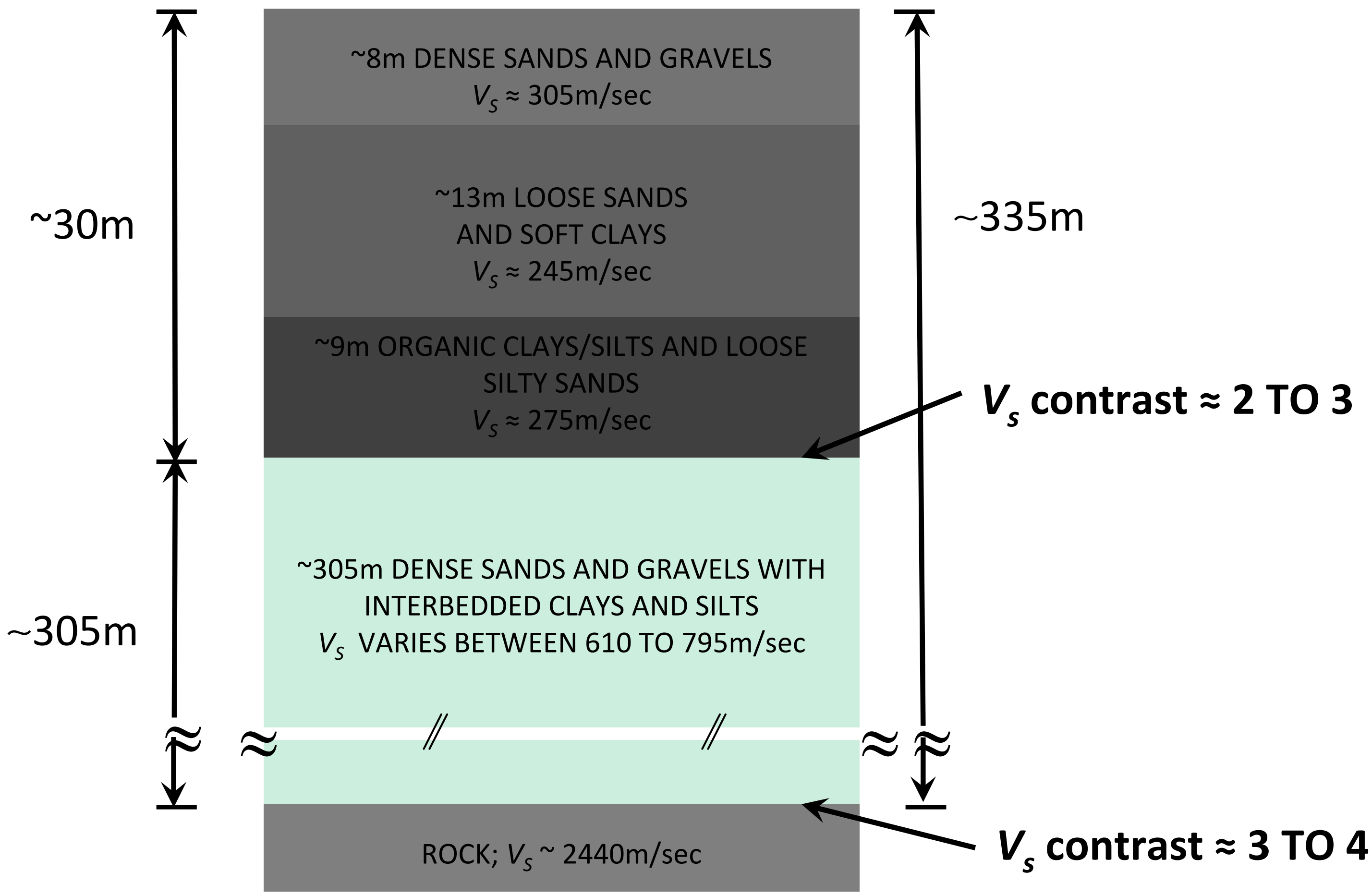
What if event had been closer, last longer?

BUCKLING & TENSION YIELDING OF X-BRACES



REDUCTION IN BEARING DUE TO PERMANENT MOVEMENT

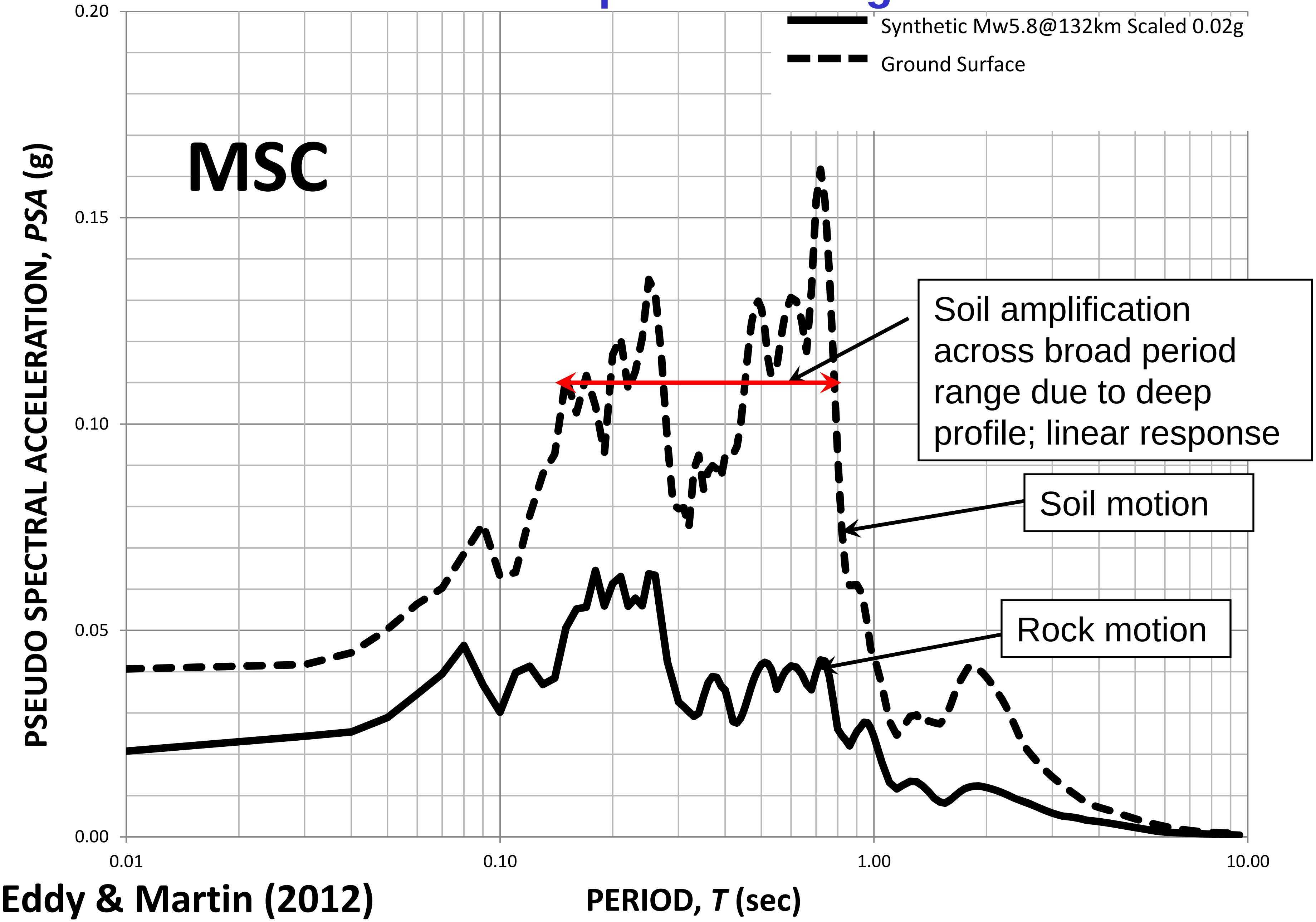
MUSEUM SUPPORT CENTER (MSC) – SOIL PROFILE



GEER VA (2012)

DRM-VT

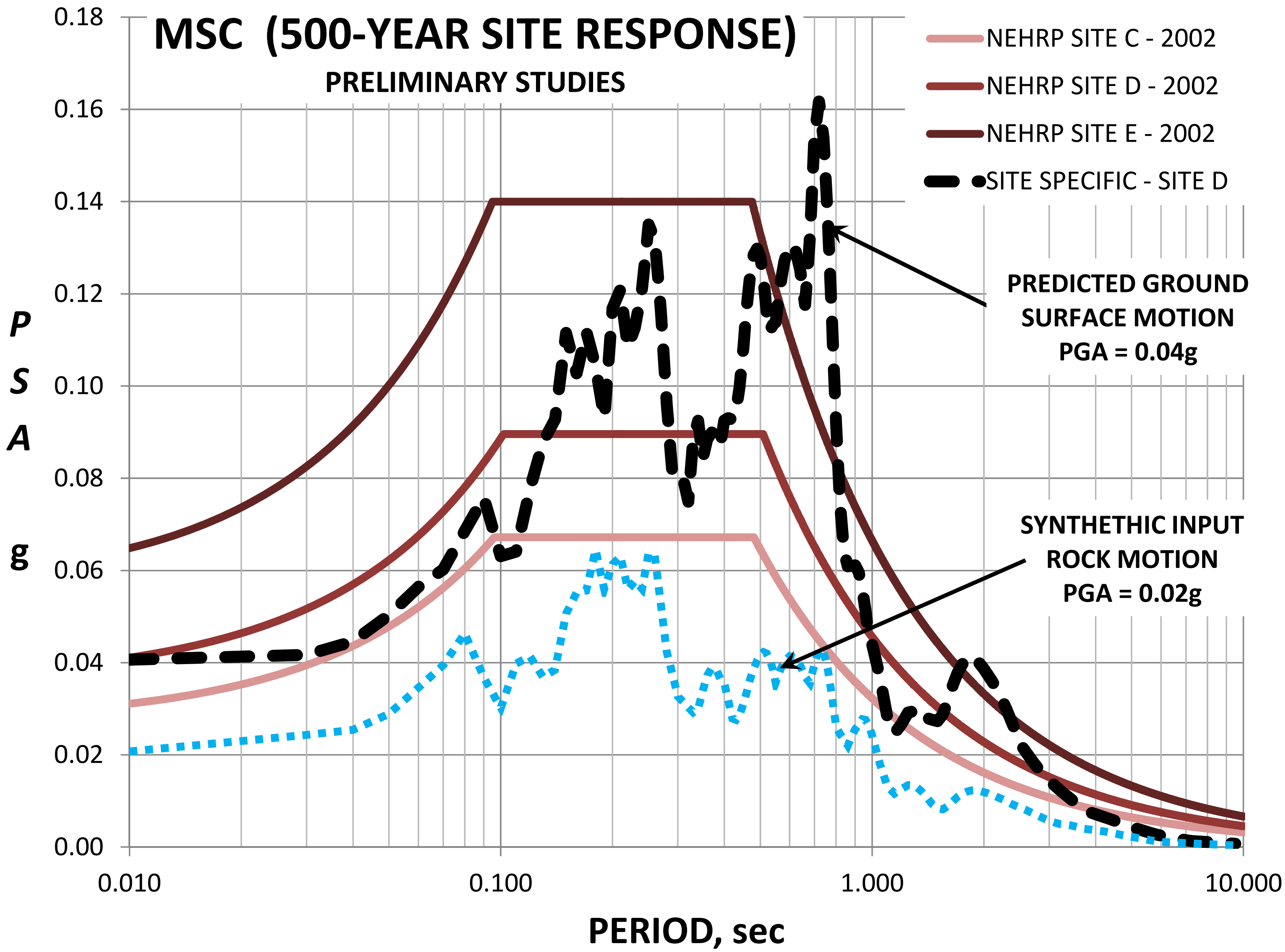
Estimated Response During M5.8



Eddy & Martin (2012)

DRM-VT

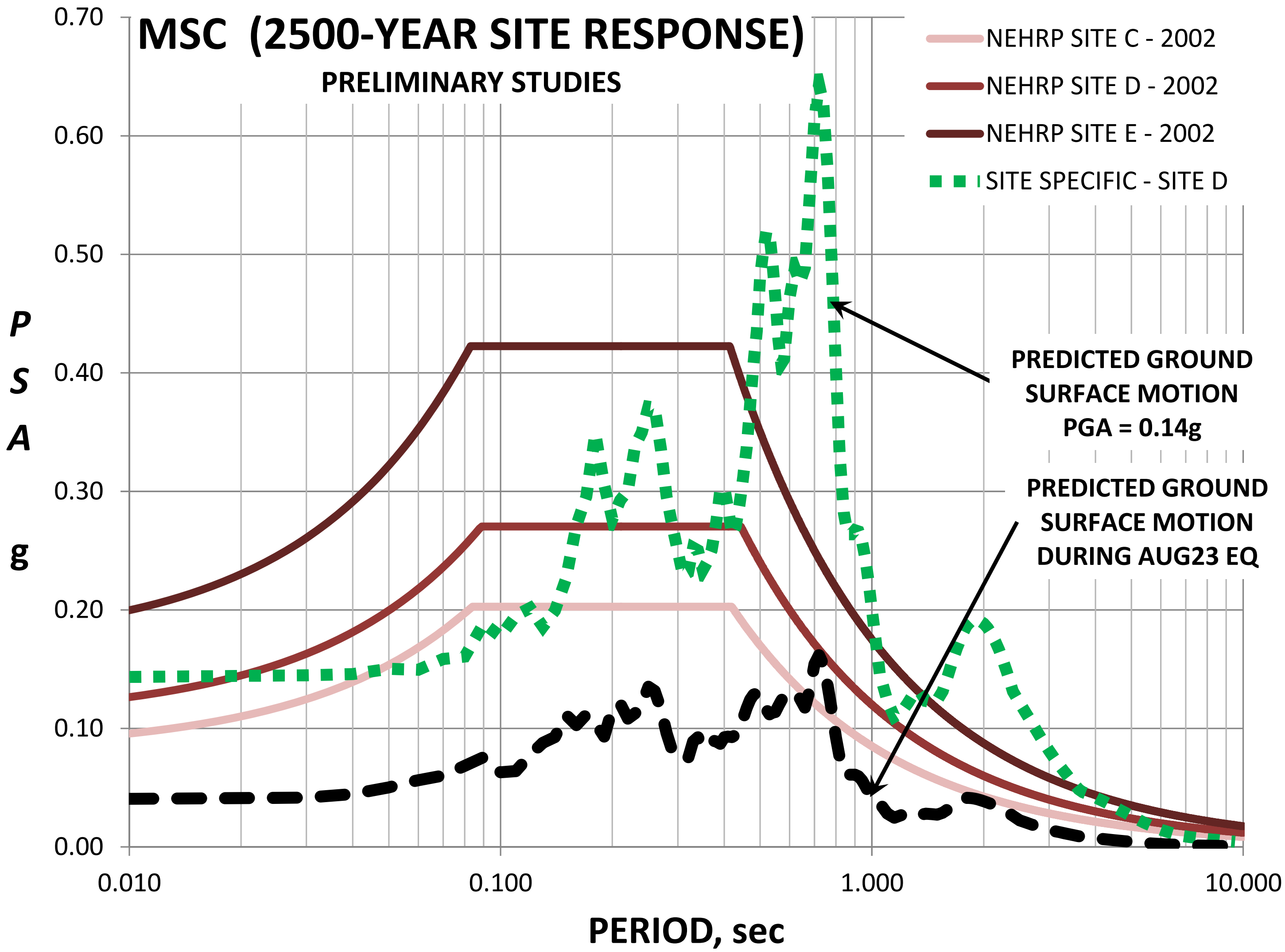
Response Not Captured by Current NEHRP/IBC Code



Eddy & Martin (2012)

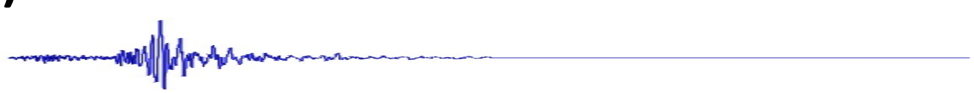
DRM-VT

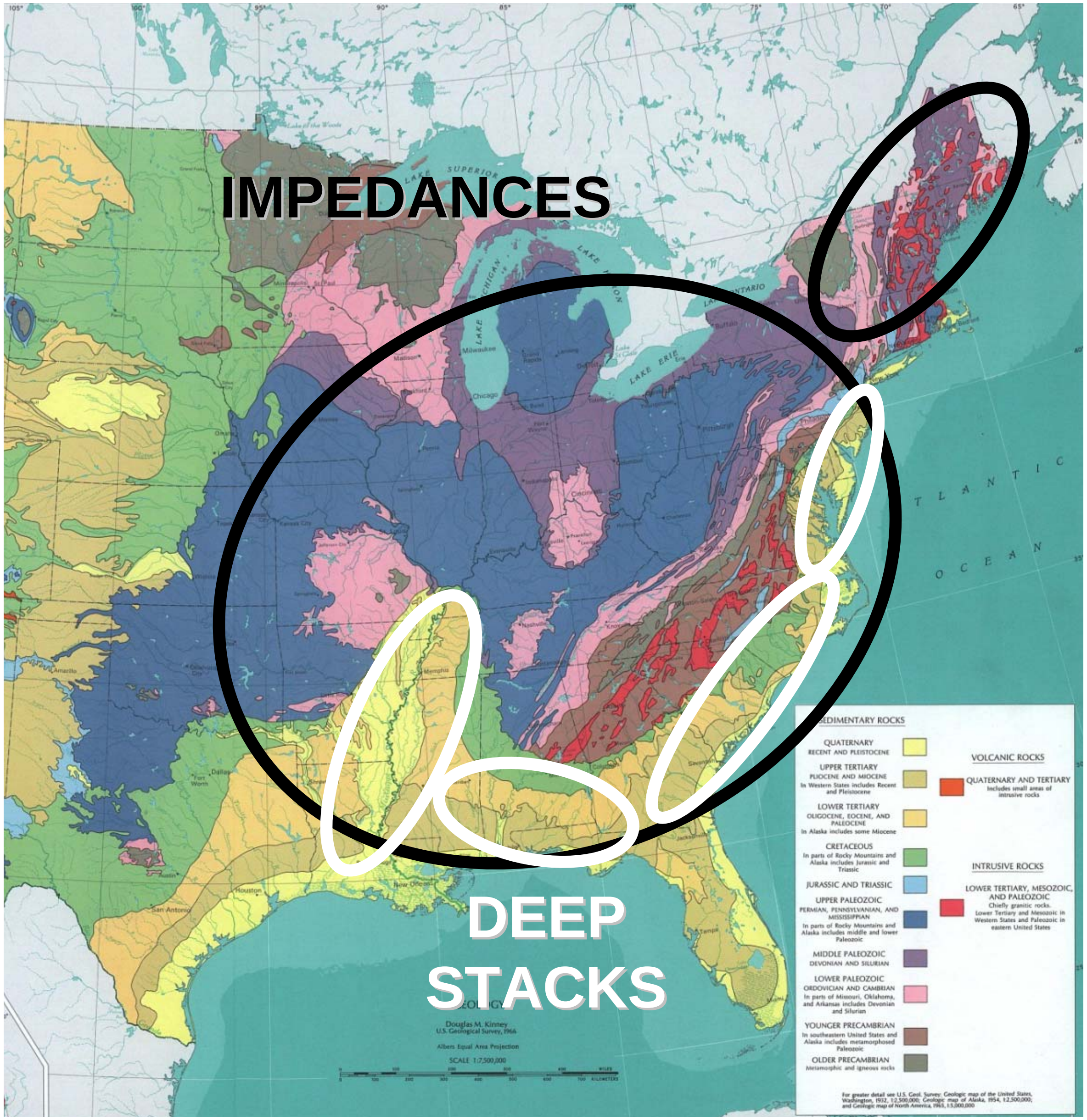
WHAT IF IT WAS THE IBC MAXIMUM CONSIDERED EQ?



Eddy & Martin (2012)

DRM-VT





- Areas of caution
- Must do site-specific analyses here

Eddy & Martin (2012)

M5.8 Virginia – key geotech lessons:

- Event highlighted vulnerability of CEUS to EQs
- Serves as teachable moment
- Social/awareness/educations lessons to be learned as well as engineering lessons
- We missed out not having adequate instrumentation
- Detailed studies ongoing

Key lessons: 1.) Region-specific soil amplification factors, Fa & Fv, needed for simplified design; 2.) finite-fault modeling studies possible; 3.) seismic hazard analysis implications, especially attenuation; 4.) complicated fault rupture may be common to CEUS events; 5.) new NRC regulations

Overall Summary

- NZ – Liquefaction damage prediction models, soil improvement for seismic design
- Japan – Liquefaction triggering and deformations, seismic design of ports and waterfront structures, soil –structure interaction, soil improvement for seismic design
- VA – CEUS seismic hazard and region-specific simplified seismic design procedures