# SITE RESPONSE ANALYSIS INCLUDING LIQUEFACTION

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# SITE RESPONSE ANALYSIS INCLUDING LIQUEFACTION

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## Presentation Outline

- Presentation Objectives
- Seismic Response Methodology
- Site Response Analysis for this study
- Application to NMSZ Bridge Sites
- Simulated vs. Observed Near-Field Ground Motions
- Summary & Conclusions



## Objectives

- Define the required dynamic soil properties for site response
- Obtain ground motions at ground surface in time domain modeling
- Study effects of deep Soils – high confinement
- Examine the liquefaction potential at the sites

## Properties of Earthquakes

- Anomalously high frequency and long duration
- Large influenced area
- Long recurrence interval, but the probability of recurrence is high in next 50 years

Source :The Center for Earthquake Research and Information (CERI) at The University of Memphis

| Magnitude | Recurrence Interval | Probability of Recurrence in the years 2000-2015 | Probability of Recurrence in the years 2000-2050 |
|---|---|---|---|
| >= 6.0 | 70+/-15 years | 40 - 70% | 88 - 98% |
| >= 7.5 | 250+/-60 years | 6.0 - 9.5% | 21 - 33% |
| >= 8.0 | 550+/-125 years | 0.4 - 1.1% | 1.6 - 4.3% |

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## Bridge Foundation Damage

- A large amount of bridge foundation (pile foundations) damage and failure were observed in the 1964 Alaska, 1989 Loma Prieta, 1995 Kobe, 1999 Chi-Chi, 1999 Izmit earthquakes (Magnitude ranging from 6.4 to 8.3 ).
- These failures have been found primarily due to two factors:
  - Loss of lateral soil support may occur due to liquefaction of cohesionless soils or strain softening of cohesive soils near the pile head, and
  - Large loads and displacements due to laterally spreading soil deposit after liquefaction.

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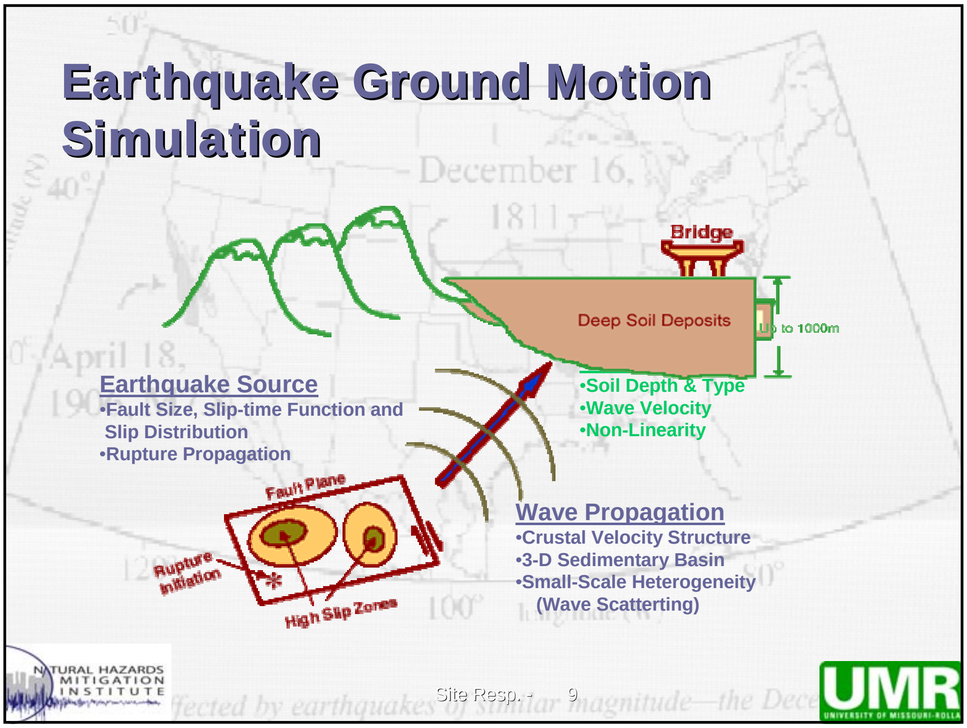
## Shi-Wei Bridge Collapse

Shi-wei Bridge Collapse during the Chi-Chi Earthquake



## Bridges in the NMSZ

- Similar sub structure and foundation conditions as the Shi-wei Bridge.
- Bridge decks supported on steel rocker bearings with multiple expansion joints.
- It is necessary to study SPSI to understand the seismic behavior of highway bridges.
- The purpose of this research is to study the dynamic soil properties in the NMSZ and the current analytical methods for SPSI and develop a sound approach for the fully-coupled SPSI analysis in the NMSZ.


Earthquake Ground Motion Simulation
Bridge
Deep Soil Deposits
Up to 1000m
Earthquake Source
•Fault Size, Slip-time Function and Slip Distribution
•Rupture Propagation
•Soil Depth & Type
•Wave Velocity
•Non-Linearity
Fault Plane
Rupture Initiation
High Slip Zones
Wave Propagation
•Crustal Velocity Structure
•3-D Sedimentary Basin
•Small-Scale Heterogeneity
(Wave Scattering)
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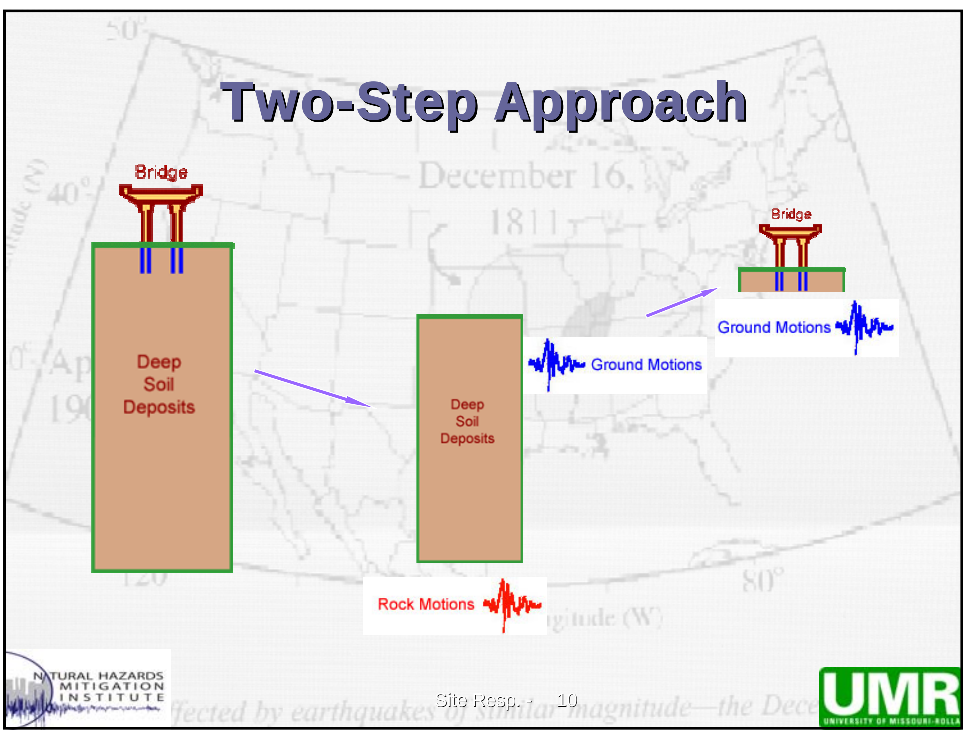
Two-Step Approach
Bridge
Deep Soil Deposits
Deep Soil Deposits
Ground Motions
Bridge
Ground Motions
Rock Motions
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## Seismic Site Response

- Seismic site response is usually referred to as the propagation of seismic waves from an input base rock to the ground surface through the local site soils.
- Since the 1970’s methodologies have been developed to analyze this process using equivalent-linear or nonlinear methods.



## Seismic Site Response

***Equivalent linear methods in the Frequency Domain:***

- SHAKE (Schnabel et al., 1972) → 1D
- FLUSH (Lysmer et al. 1975) → 2-D
- RASCALS, Silva (1992) → deep soils
- Assimaki (2001) introduced frequency-dependent soil parameters.

# Seismic Site Response

***1D Nonlinear Methods in the Time Domain:***

| Program | Soil model | Method | Stress | Reference |
|---|---|---|---|---|
| CHARSOIL | Ramberg-Osgood | Characteristics | Total | Streeter et al. (1973) |
| DESRA-2 | Hyperbolic | Finite element | Effective | Lee and Finn (1978, 1991) |
| DESRAMOD2 | Hyperbolic | Finite element | Effective | Vucetic (1998) |
| DESRA-MUSC | Hyperbolic | Finite element | Effective | Qiu(1998) |
| D-MOD(derived from DESRA-2) | M-K-Z (Matasovic, Konder, and Zelasko) | Finite element | Effective | Matasovic (1993) |
| MASH | Martin-Davidenkov | Finite element | Effective | Martin and Seed (1978) |
| DYNA1D | Nested yield surface | Finite element | Effective | Prevost (1989) |
| TESS | HDCP (Hardin-Drnevich-Cundall-Pyke) | Finite difference | Effective | Pyke (1979, 1985, 1992) |
| SUMDES | Hypoplasticity | Finite element | Effective | Li et al. (1992) |
| DEEPSOIL (derived from D-MOD) | Modified hyperbolic with extended Masing criteria | Finite element | Total | Hashash and Park (2001) |

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# Seismic Site Response

***1D Nonlinear Methods in the Time Domain:***

- There are many nonlinear, 1D ground response analysis computer programs using direct numerical integration in the time domain.

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## Seismic Site Response

### *2D Nonlinear Methods in the Time Domain:*

- 1D methods are useful for level or gently sloping sites with parallel material boundaries. However, problems such as sloping or irregular ground surfaces, the presence of heavy, stiff, or embedded structures, or walls and tunnels all require 2D or even 3D analysis.



## Seismic Site Response

### *2D Nonlinear Methods in the Time Domain:*

| Program | Soil model | Method | Stress | Reference |
|---|---|---|---|---|
| TARA-3 | Hyperbolic | Finite element | Effective | Finn et al. (1986) |
| DYNAFLOW | Multiple yield surface | Finite element | Effective | Prevost (1986) |
| DIANA | Different advanced models | Finite element | Effective | Kawai (1985) |
| FLAC | Hyperbolic (Finn and Byrne model) | Finite difference | Effective | Commercial |
| DYSAC2 | Hypoplasticity | Finite element | Effective | - |

## Recent Use of Site Response Methods

- Yu et al. (1993) studied the nonlinear behavior of soil using DESRA2 (Lee and Finn, 1978)
- Ni et al. (1997) extended this work to include deep saturated soil deposits accounting for the influence of pore pressure and stress-dependent damping and shear modulus ratio variations with shear strain (EPRI, 1993).



## Recent Use of Site Response Methods

- Ni et al. (2000) studied the nonlinearity of soil properties of shallow soil (upper 30 m).
- Assimaki et al. (2000) developed a simple four-parameter model to do site response of deep cohesionless soil (1 km deep) accounting for the stress-dependent modulus and damping ratio

## Recent Use of Site Response Methods

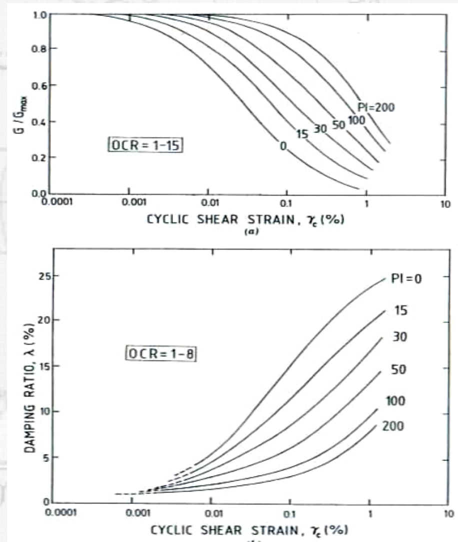
- Romero and Rix (2001) studied the site response in the Central United States using the equivalent method RASCALS.
- Hashash et al. (2001) developed a new model accounting for the effect of high confining pressure on modulus degradation and damping ratio of deep soil.
- In 2002 this method used full Rayleigh damping formulation to represent the viscous damping of soils.



## Development of New Deep Ground Response Analysis

# Nonlinear Soil Properties

- Quite nonlinear Soil properties under seismic loading condition.
- In Vucetic & Dobry ‘s curves, for a given shear strain g, PI increases, G/G$_{max}$ rises and l reduced.

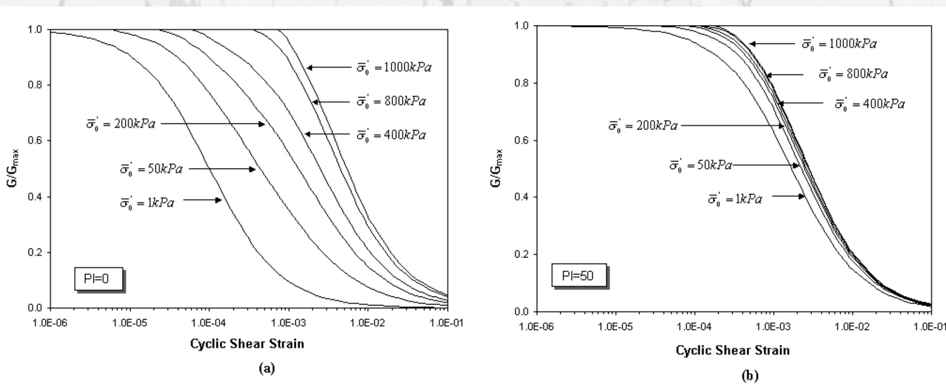



(Vucetic and Dobry, 1991)



# Effect of Confining Pressure

Ishibashi (1992) pointed out that the method of Vucetic & Dobry didn’t include one of the significant parameters, the effective mean normal stress.



# Unified Formula

## Shear Modulus

$$\frac{G}{G_{\max}} = K(\gamma, PI)(\sigma'_m)^{m(\gamma, PI)}$$

$$K(\gamma, PI) = 0.5\left\{1 + \tanh\left[\ln\left(\frac{0.000101 + n(PI)}{\gamma}\right)^{0.492}\right]\right\}$$

$$m(\gamma, PI) = 0.272\left\{1 - \tanh\left[\ln\left(\frac{0.000556}{\gamma}\right)^{0.4}\right]\right\}\exp(-0.0145\, PI^{1.3})$$

$$n(PI) = \begin{cases} 0.0 & \text{for } PI = 0 \\ 3.37 \times 10^{-6}\, PI^{1.404} & \text{for } 0 < PI \le 15 \\ 7.0 \times 10^{-7}\, PI^{1.976} & \text{for } 15 < PI \le 70 \\ 2.7 \times 10^{-5}\, PI^{1.115} & \text{for } PI > 70 \end{cases}$$

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# Unified Formula (contd.)

## Damping Ratio

$$\lambda = \frac{0.333(1 + \exp(-0.0145\, PI^{1.3}))}{2}\left\{0.586\left(\frac{G}{G_{\max}}\right)^2 - 1.547\left(\frac{G}{G_{\max}}\right) + 1\right\}$$

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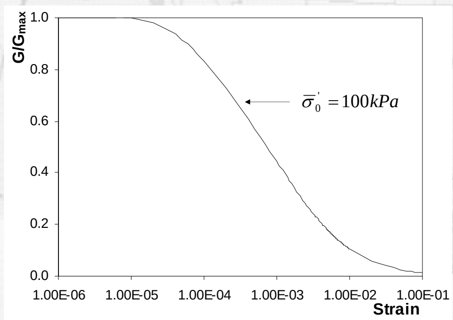
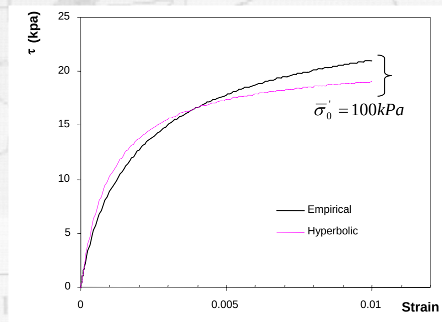
# Backbone Curve

- **The shear modulus degradation curve presented in previous slide can be described as the backbone curve in stress-strain field.**

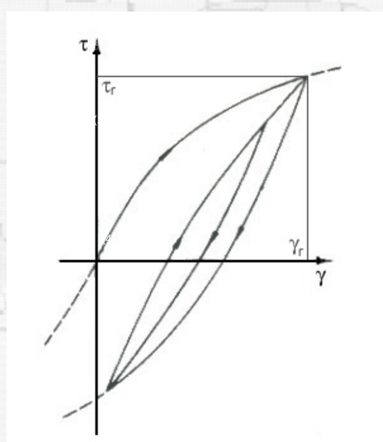





# Extended Masing Criteria

- The extended Masing criteria (1926) are used to govern the unloading-reloading behavior of soil.




Extended Masing Criteria

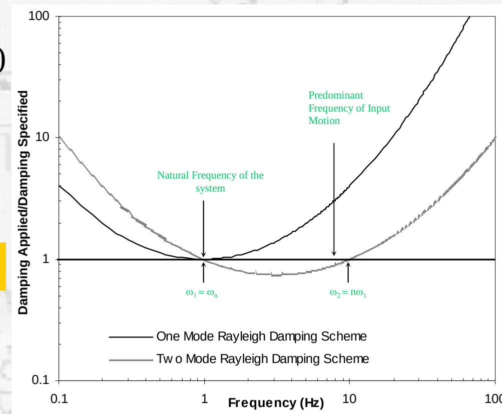
# Finite Element Approach

## Global Dynamic Equation

$$[M]\{\ddot{u}\}+[C]\{\dot{u}\}+[K]\{u\}=P(t)$$

*where*

$$P(t)=-[M]\{I\}\ddot{u}_g(t)$$




## Rayleigh Damping Formulation

$$[c]_{el} = \alpha[m]_{el} + \beta[k]_{el}$$

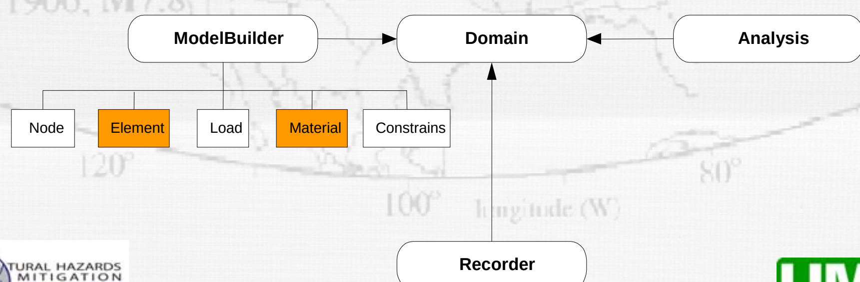
$$\alpha = 2\lambda\omega_1\omega_2/(\omega_1+\omega_2)$$

$$\beta = 2\lambda/(\omega_1+\omega_2)$$

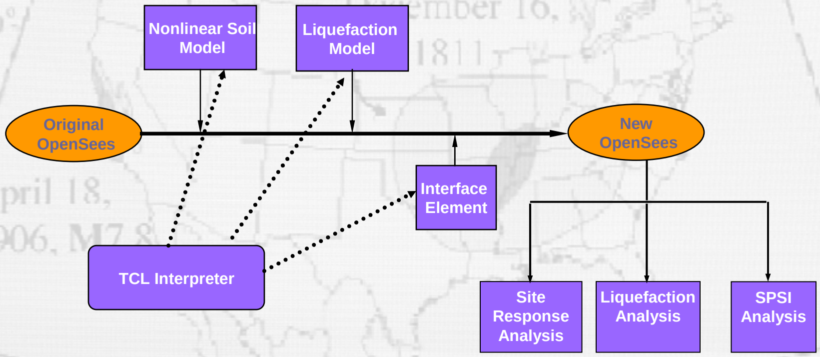


# OpenSees Framework

- OpenSees - Open System for Earthquake Engineering Simulation
- OpenSees developed by PEER is a software framework to create models and analysis methods to simulate structural and geotechnical systems under earthquake loading.
- C++ language is used as compiler and finite element method is used for analysis.
- Tool Command Language (TCL) is used as interpreter to create commands.



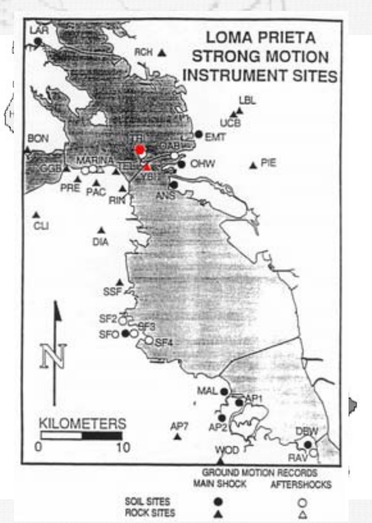
# Work Chart of Programming





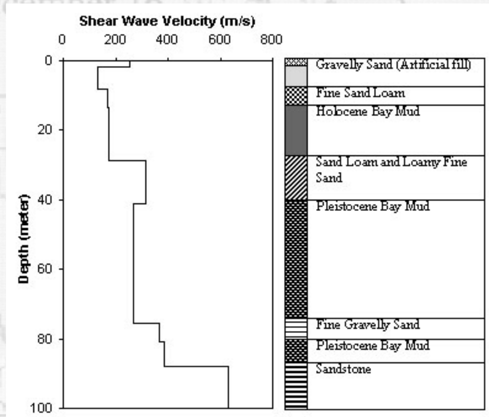

# Site for Validation

- **Treasure Island** (TRI) man-made island
- Yerba Buena Island (YBI) – large base rock output, 2 km away from Treasure Island.
- Both islands are located 70~75 km northwest of the epicenter



# Treasure Island Soil Profile

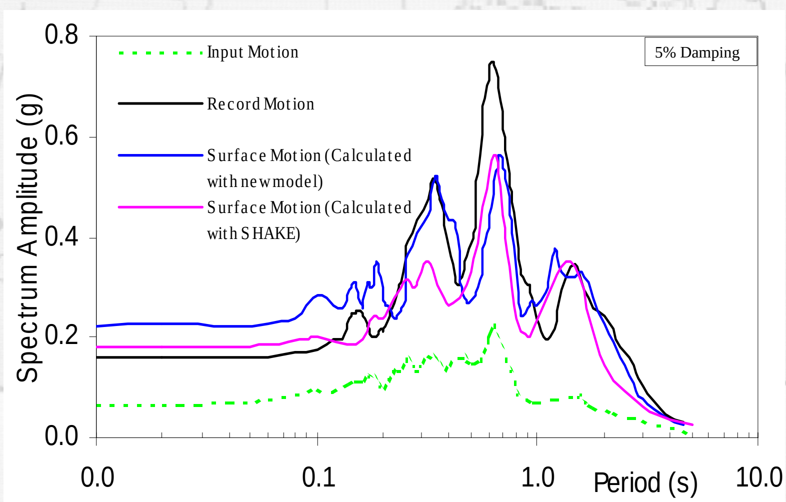
Treasure Island site consists of about 13m sandy fill, underlain by about 16 m thick of Young Bay Mud. Underlying the Young Bay Mud are alternating layers of dense sand and Old Bay Mud to a depth of about 89 m.



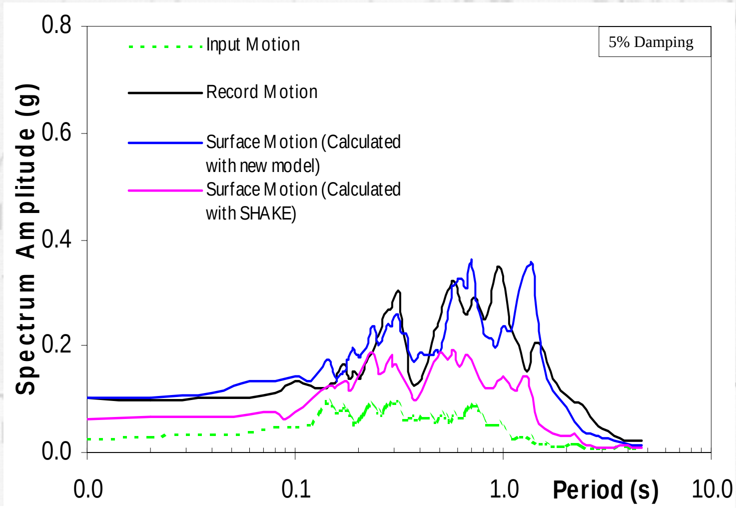

(Gibbs et al, 1992)



# Response Spectra Comparison (90°)



## Response Spectra Comparison (00°)

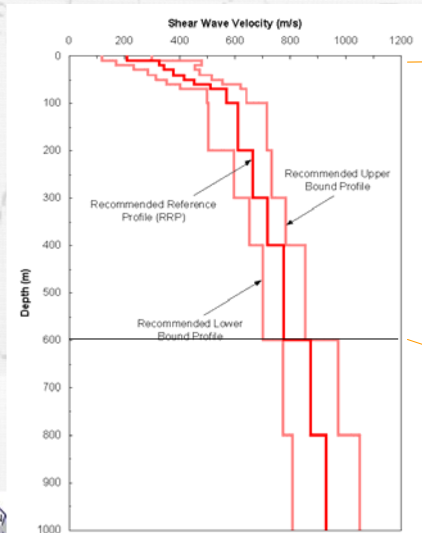


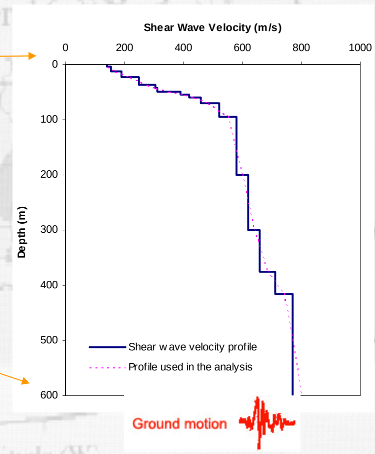



## Application in the NMSZ

- The new soil model is applied a highway bridge site near Hayti, Missouri in the NMSZ.
- The thickness of the sediment at the study site is estimated at about 600 m.
- The shallow shear wave velocity profile was based on cross-hole testing data measure at the study site. The deeper soil profile was inferred to the several deep wells in Mississippi Embayment area.

# Shear Wave Velocity Profile




M=6.5, PGA=0.148g



# Site Response Analysis

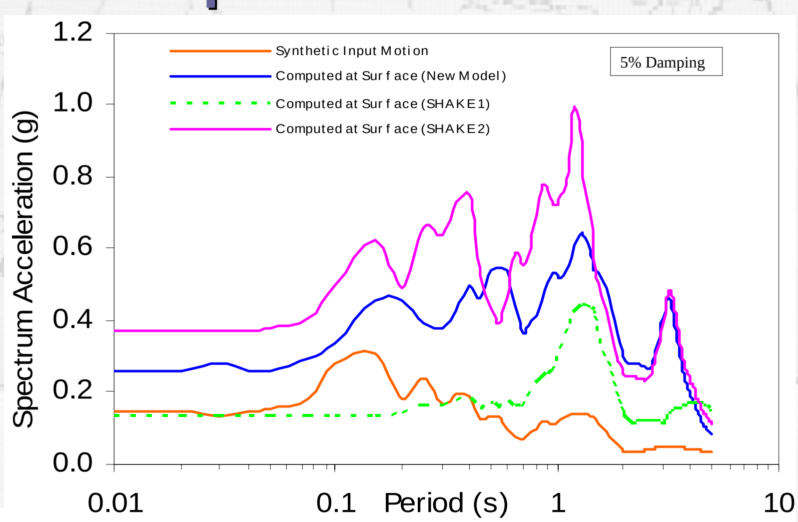
- The composite source model program was used to develop the synthetic ground motions.
- Three cases were studied for the site response analysis. One is in the new model and two are in SHAKE.
  - New model.
  - SHAKE1. Vucetic and Dobry's curves developed in the database of SHAKE are used for the whole soil profile.
  - SHAKE2. Modified modulus degradation curve and damping curves for the deep soil layers (Ishibashi and Zhang, 1993).

## Comparison of PGA

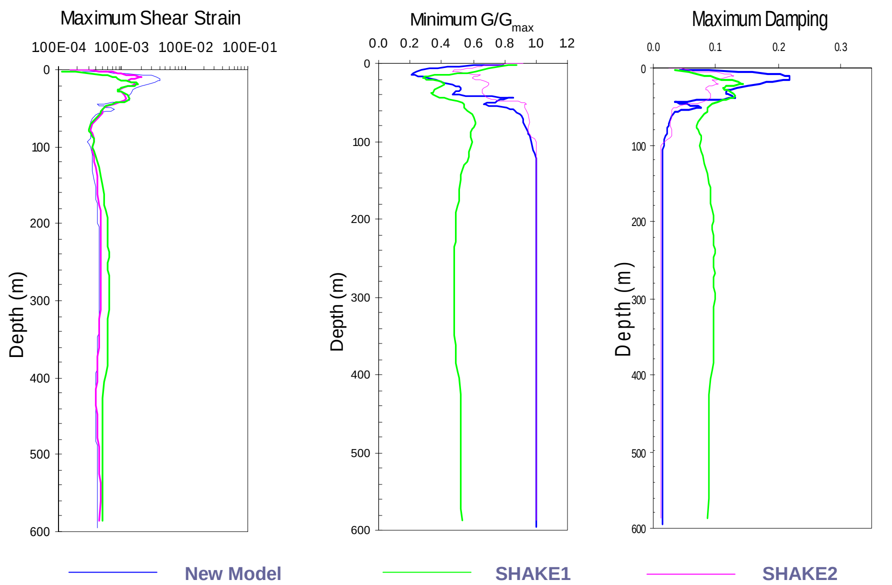
| Ground Motions | PGA (g) |
|---|---|
| Synthetic Input Motion (rock) | 0.148 |
| Computed at Surface (New Model) | 0.259 |
| Computed at Surface (SHAKE1) | 0.133 |
| Computed at Surface (SHAKE2) | 0.374 |



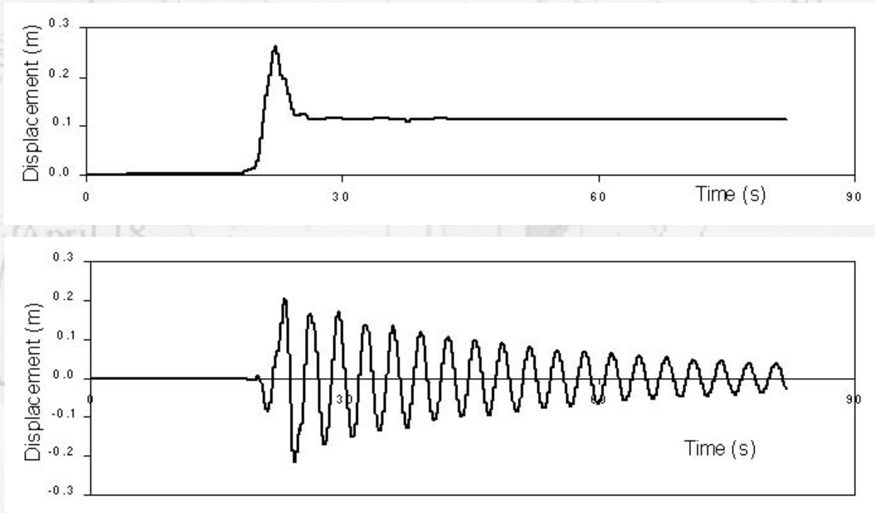
## Comparison of Response Spectra for NMSZ



# Profile of Dynamic Properties




# Near Field Study




*Displacement Time Histories at A1466 M=6.5 (Composite Source) (a) Input (b) Surface*

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## Liquefaction Considerations in the NMSZ



## Liquefaction Considerations - NMSZ-

- Shallow sediments in the NMSZ consist of silts, sands and low plastic soil that have high potential for liquefaction.
- Lots of liquefaction vestige, such as sand boiling and landslides, can be still found today for 1811-1812 earthquakes.
- Computational techniques that include liquefaction modeling are important for the performance evaluation of infrastructure built on these foundation soils.

## Pore Water Pressure Generation Model

- Martin et al. (1975)'s four-parameter pore water pressure generation model.

$$\Delta\varepsilon_v = C_1(\gamma - C_2\varepsilon_v) + \frac{C_3\varepsilon_v^2}{\gamma + C_4\varepsilon_v}$$

- Byrne (1991)'s two-parameter pore water pressure generation model.

$$\Delta\varepsilon_v = C_1\gamma \exp(-C_2(\frac{\varepsilon_v}{\gamma}))$$



## Parameters for Byrne's Model

- The value of $C_1$ and $C_2$ can be empirically determined from the relative density or the normalized penetration value.

$$C_1 = 7600(D_r)^{-2.5} \quad \text{or} \quad C_1 = 8.7(N_1)_{60}^{-1.25}$$

- The parameter $C_2$ has been found to be a constant fraction of $C_1$ as follows .

$$C_2 = 0.4 / C_1$$

## Application in Earthquake Problem

- The equation can be written in the incremental form by assuming that the volumetric strain develops linearly with shear strain during any half cycle (Byrne & Mclintyre, 1995).

$$d\varepsilon_v = 0.25 C_1 d\gamma \exp(-C_2 (\frac{\varepsilon_v}{\gamma}))$$

- After the incremental change in volumetric strain is determined, the incremental change in pore water pressure can be obtained as follows:

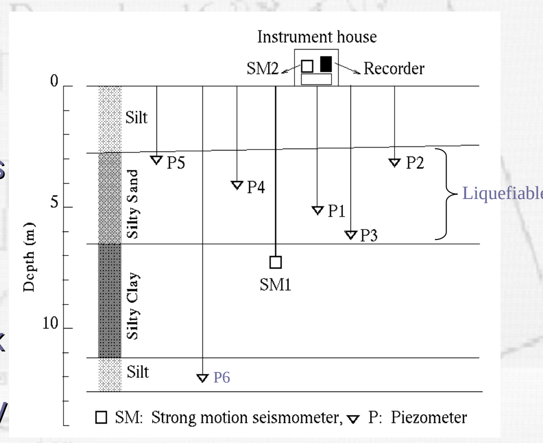
$$du = M d\varepsilon_v$$

- The model is loosely coupled into the nonlinear soil model. At the end of each time step, the pore water pressure is updated based on the increment of shear strain of this step.

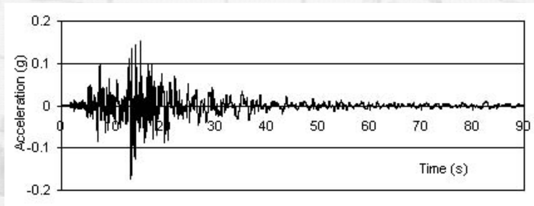


## Field Verification

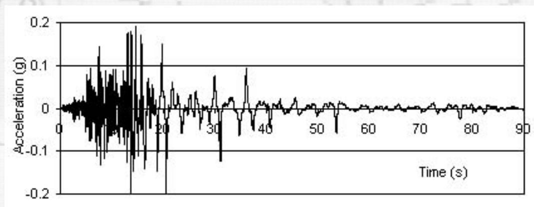
The pore water pressure generation model described above was verified using the records at the Wildlife site during the 1987 Superstition Hills Earthquake ($M_s$ =6.6). The site stratigraphy consists of a silt layer approximately 2.5m thick underlain by a 4.3 m thick layer of loose silty-sand, underlain by a stiff to very stiff clay.



## Acceleration Time Series



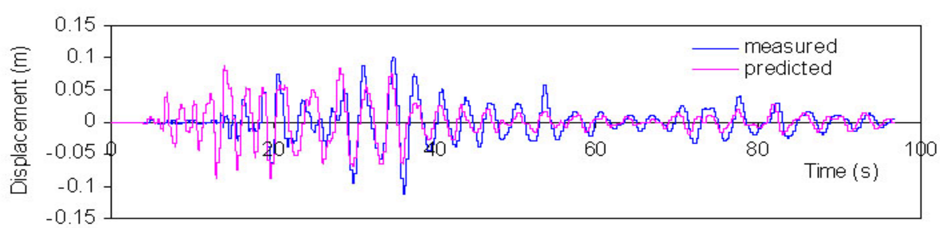

Downhole Acceleration Time Series




Uphole Acceleration Time Series

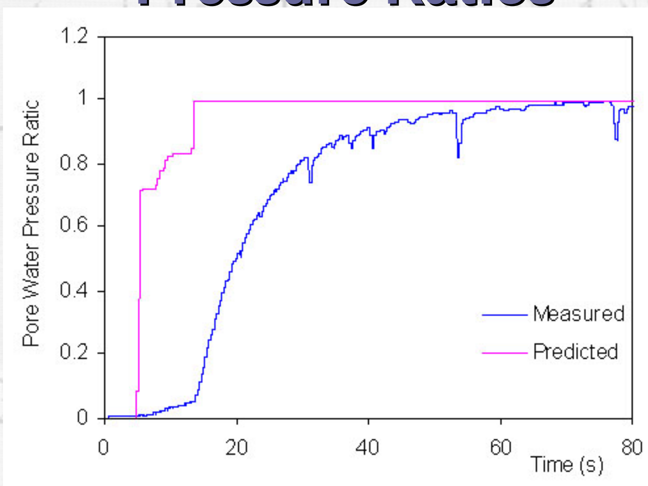


## Comparison for Relative Displacement




Comparison of Measured and Predicted Relative Displacement Time Histories

## Comparison for Pore Water Pressure Ratios

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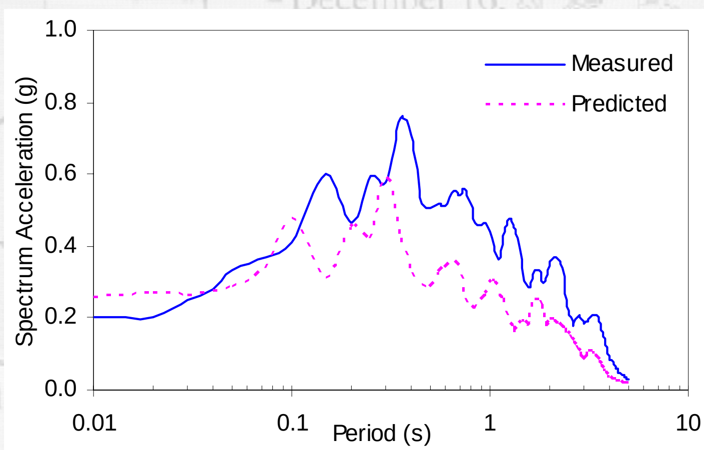
## Comparison for Response Spectra at Surface

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# Liquefaction Analysis in the NMSZ



# Liquefaction Analysis in the NMSZ

- Liquefaction analysis was performed at the same bridge site and the same soil profile was used.
- The synthetic motions with different energy levels were used.
- The pore water pressure generation model was used to examine the liquefaction performance of the near surface soil layers (around 60m).
- The parameters for the pore water pressure generation model were estimated from the SPT and CPT test data.

# Synthetic Input Motions

## Summary of the Synthetic Motions

| Magnitude | M =6.5 | | | | | M =7.0 | | | | | M =7.5 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Series No. | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 |
| $a_{max}$ (g) FP | 0.18 | 0.27 | 0.23 | 0.18 | 0.13 | 0.45 | 0.54 | 0.39 | 0.47 | 0.31 | 0.78 | 0.55 | 0.85 | 1.03 | 0.68 |
| $a_{max}$ (g) FN | 0.15 | 0.24 | 0.27 | 0.20 | 0.12 | 0.42 | 0.47 | 0.32 | 0.41 | 0.35 | 1.10 | 0.73 | 0.94 | 1.02 | 0.79 |

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# Results for M=6.5 Earthquakes

| Layer No. | Depth (m) | Soil Type | Max Pore Water Pressure Ratio | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | FP Direction | | | | | FN Direction | | | | |
| Series No. | | | 1 | 2 | 3 | 4 | 5 | 1 | 2 | 3 | 4 | 5 |
| 1 | 5.5~7.4 | Sandy Silt | 0.18 | 0.56 | 0.16 | 0.15 | 0.13 | 0.18 | 0.63 | 0.84 | 0.96 | 0.19 |
| 2 | 7.4~11.8 | Loose Sandy Silt | 0.30 | 0.68 | 0.24 | 0.25 | 0.22 | 0.37 | 0.76 | **1.00** | **1.00** | 0.31 |
| 3 | 11.8~18.2 | Medium Dense Sand | 0.13 | 0.27 | 0.11 | 0.11 | 0.10 | 0.13 | 0.30 | 0.40 | 0.46 | 0.13 |
| 4 | 18.2~22.5 | Dense Sand | 0.05 | 0.16 | 0.06 | 0.07 | 0.06 | 0.10 | 0.18 | 0.23 | 0.27 | 0.08 |
| 5 | 22.5~39.3 | Dense Sand | 0.03 | 0.06 | 0.02 | 0.03 | 0.02 | 0.04 | 0.07 | 0.09 | 0.09 | 0.03 |

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# Results for M=7.0 Earthquakes

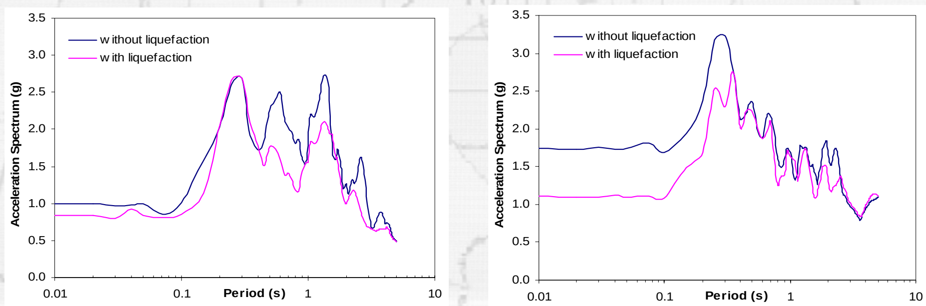
| Layer No. | Depth (m) | Soil Type | Max Pore Water Pressure Ratio | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | FP Direction | | | | | FN Direction | | | | |
| Series No. | | | 6 | 7 | 8 | 9 | 10 | 6 | 7 | 8 | 9 | 10 |
| 1 | 5.5~7.4 | Sandy Silt | 0.93 | 0.98 | 0.87 | 1.00 | 0.38 | 0.98 | 0.84 | 0.97 | 0.92 | 1.00 |
| 2 | 7.4~11.8 | Loose Sandy Silt | 1.00 | 1.00 | 1.00 | 1.00 | 0.49 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| 3 | 11.8~18.2 | Medium Dense Sand | 0.50 | 0.56 | 0.42 | 0.64 | 0.21 | 0.48 | 0.41 | 0.49 | 0.50 | 0.55 |
| 4 | 18.2~22.5 | Dense Sand | 0.33 | 0.37 | 0.26 | 0.48 | 0.14 | 0.31 | 0.26 | 0.32 | 0.31 | 0.38 |
| 5 | 22.5~39.3 | Dense Sand | 0.14 | 0.17 | 0.10 | 0.20 | 0.06 | 0.13 | 0.12 | 0.13 | 0.12 | 0.14 |



# Results for M=7.5 Earthquakes

| Layer No. | Depth (m) | Soil Type | Max Pore Water Pressure Ratio | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | FP Direction | | | | | FN Direction | | | | |
| Series No. | | | 11 | 12 | 13 | 14 | 15 | 11 | 12 | 13 | 14 | 15 |
| 1 | 5.5~7.4 | Sandy Silt | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| 2 | 7.4~11.8 | Loose Sandy Silt | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| 3 | 11.8~18.2 | Medium Dense Sand | 0.85 | 0.50 | 0.59 | 0.55 | 0.66 | 1.00 | 0.60 | 0.93 | 0.92 | 0.66 |
| 4 | 18.2~22.5 | Dense Sand | 0.64 | 0.36 | 0.44 | 0.39 | 0.48 | 0.84 | 0.64 | 0.67 | 0.65 | 0.48 |
| 5 | 22.5~39.3 | Dense Sand | 0.28 | 0.19 | 0.21 | 0.20 | 0.22 | 0.36 | 0.33 | 0.32 | 0.21 | 0.24 |

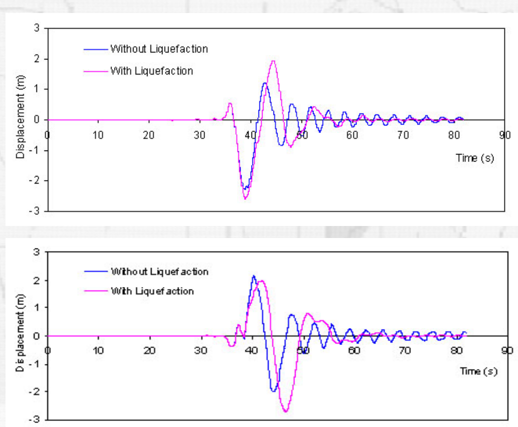
# Comparison: Response Spectra




Comparisons of the Computed Response Spectra for Motion No. 11
(a) in Parallel Direction (b) in Normal Direction



# Comparison: Displacement Histories




Comparison of the Displacement Time Histories at Ground Surface for
Motion No. 11 (a) in Parallel Direction (b) in Normal Direction

# Summary & Conclusions



# Summary of Findings

- A new nonlinear soil model was developed to take into account the influence of the confining pressure on the site response analysis of deep soil deposits.
- Results from the site response analysis indicates that ignoring the influence of confining pressure on site response analysis will significantly underestimate the ground response in deep soil sites.

## Summary of Findings

- A two-parameter pore water pressure generation model is loosely coupled into the nonlinear soil model. Preliminary results show that the liquefaction could happen for M=6.5 or larger earthquakes in this area.
- Near field effects have been studied. After the seismic waves propagate through the deep soil deposit, the fling effect is not present while the pulse is still found in the surface motions. These preliminary findings are in agreement with the lack of evidence of surface ground rupture due to previous earthquakes in the NMSZ.



## Summary of Findings

- Near field energy pulse could be transmitted to the piles and other bridge components after propagating through the inelastic behavior of pile-soil interaction. However, near-field properties in the superstructure are not as significant as when the degradation of soil springs due to the pore water pressure is considered.

## Thank You!

## Questions / Comments

The End