Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1614

COMPUTATIONAL MODELING OF SOIL-FOUNDATION STRUCTURAL SYSTEMS

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ABSTRACT

Numerical modeling of soil-foundation-structure seismic response is increasingly generating valuable insights. Based on the presented computational simulations, system as well as component behavior reveal mechanisms that may qualitatively and quantitatively influence the state of practice and design. Potential seismically-induced ground displacement effects are systematically imposed along with the loads due to dynamic excitation. Mitigation approaches may be also represented, and the extent thereof may be assessed. In such scenarios, high fidelity simulations are permitted by three-dimensional modeling. Pre- and post-processing and visualization tools are also an integral component. OpenSeesPL, a graphical user interface, is a step in this direction with capabilities for simulating footings, piles and pile groups, and ground modification scenarios.

Introduction

The continued advances in computational software and hardware are now permitting the systematic use of three-dimensional (3D) simulation for a wide class of geotechnical earthquake engineering applications. In this paper, representative recent studies are summarized including: i) seismic response of a full bridge-foundation-ground system, ii) pile group behavior, and iii) ground modification. For such 3D modeling environments, a graphical user interface (OpenSeesPL) permits the efficient pre- and post-processing and visualization of the resulting seismic response.

The holistic analysis of structures including the foundation and supporting ground , all within a unified framework, allows for: i) more realistic application of the dynamic loading in the form of incident seismic waves, ii) estimation of the potential seismically-induced ground deformation and its effect on the foundation and the super-structure, and iii) more accurate representation of the involved nonlinear structural and soil-structure interaction mechanisms, and the interplay between these effects throughout the earthquake shaking event.

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Bridge-Foundation-Ground System

The Humboldt Bay Middle Channel Bridge is a PEER Testbed that motivated seismic computational simulation efforts of entire ground-structure systems. The bridge (Figs. 1-3) is a 330m long, 9-span structure, supported on the cap beams of single pier bents with both longitudinal and transversal shear keys to prevent unseating.

Initially, 2D studies were undertaken (Zhang et al. 2008), followed by a full 3D investigation (Yan 2006, Elgamal et al. 2008). Development of the 3D ground structure Finite Element (FE) mesh involved (Figs. 3-5):
i) Representation of the essential structural and foundation elements of bridge. In this regard, the foundation under each pier was modeled by a 2x2 pile group. Stiff strengthened zones were included below the bridge approach ramps.
ii) Placement of the mesh lateral and vertical boundaries as far away as possible from the bridge, its foundation, and approach ramps.
iii) Employment of the largest possible FE mesh within the limitation of in-core execution of the computations on a 32 bit Windows-based Personal Computer. The soil elements were configured to be relatively small around the bridge and its foundation, becoming gradually larger towards the outer mesh boundaries (Fig. 5).
iv) Provision for exploring the impact of permanent ground deformation, by inclusion of a relatively soft soil stratum at shallow depth.

Figure 1. Humboldt Bay bridge.

Figure 2. Bridge (close-up view).

A nonlinear multi-yield surface J_2 soil model was employed. Nonlinear fiber elements were employed to model the bridge piers and piles (details are provided in the Appendix).

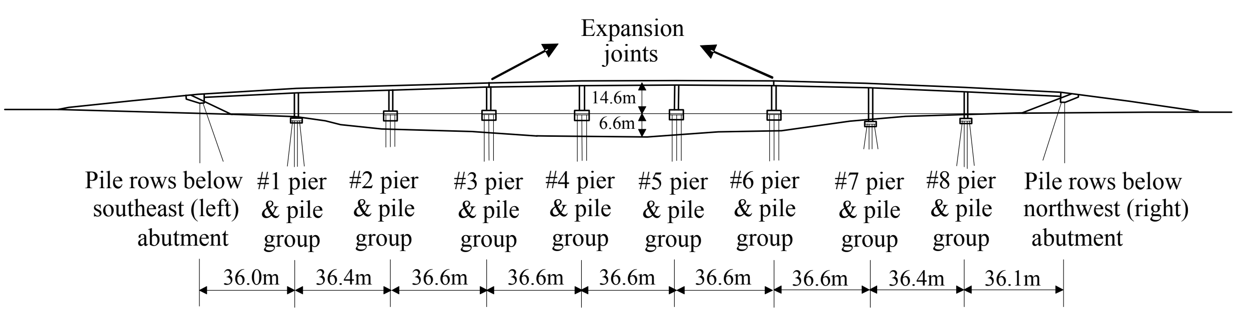


Figure 3. Schematic of bridge, pile foundations, and approach ramps.

As part of the Humboldt Bay Middle Channel Bridge PEER Testbed activities, the September 16, 1978 Tabas earthquake record was selected as a potential site-specific rock outcrop motion at a hazard level of 10% probability of exceedance in 50 years (Somerville and Collins 2002). This Tabas Earthquake record was employed in this study (Yan 2006, Elgamal et al. 2008) to derive an incident earthquake motion along the FE mesh base.

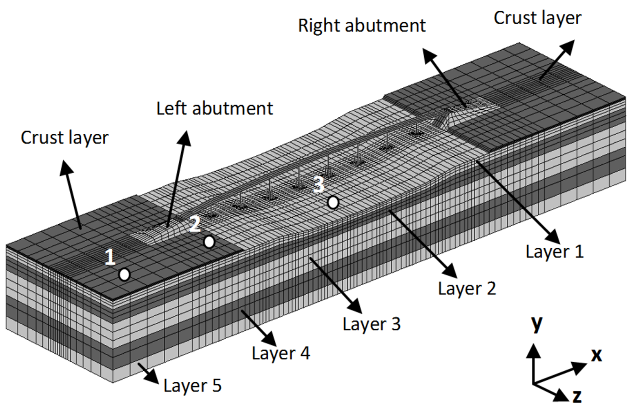


Figure 4. 3D bridge and soil layers (Table 1).

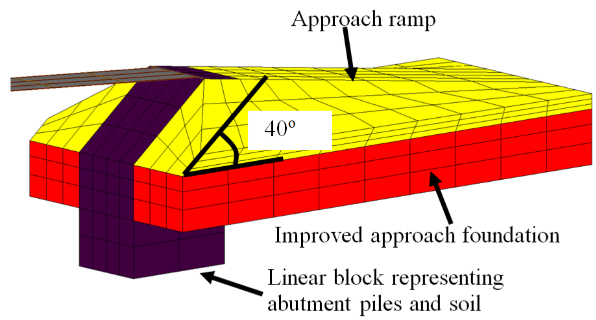


Figure 5. Abutment and approach ramp zone.

Ground motion was imparted in the form of vertically incident waves. For that purpose, a protocol for handling the base boundary condition was careful defined (Fig. 6) and executed (to permit staged loading in terms of application of own weight of the ground and structure, transition to the nonlinear material models, and imparting the incident wave ground motion). Employing this protocol, it was verified that the resulting free-field seismic motion (location 1 in Fig. 4) was essentially identical (Fig. 7) to that of a shear beam model (of the same layering profile, Yan 2006).

Table 1. Soil profile properties (Figs. 5, 6).

Soil Layer	**Mass Density kg/m^3**	**Shear Modulus G (kPa)**	**Shear strength s_u (kPa)**
Abutment	2000	30000	30
Crust layer	1500	60000	40
Layer 1	1300	19000	10
Approach Foundation	1500	25000	25
Layer 2	1500	60000	40
Layer 3	1800	196000	75
Layer 4	1900	335000	75
Layer 5	1900	475000	75

Among the observations from this study are:

1) Permanent ground deformation might have a major impact on the overall bridge deformation pattern (Fig. 8). Translation of the pile groups towards the center of the underlying waterway (Fig. 8) may induce significant moments and shear forces in the bridge piers.

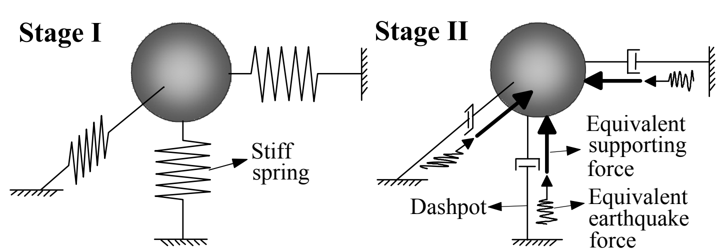


Figure 6. Boundary condition along the FE model base; I: own weight; and II: earthquake analysis.

2) Settlement and lateral translation of the bridge abutments may induce very large destructive forces into the bridge super-structure (Fig. 8). In the employed fixed bridge-abutment

connection, very high permanent shear forces and bending moments were observed as a consequence of this mechanism (Fig. 9).

3) A noticeable difference in seismic motion at the ground surface occurred (Fig. 10). This difference was mainly due to the presence of the upper crust layer above the specified soft and relatively low strength shallow stratum (layer 1 in Table 1). Essentially, a form of base isolation emanates from such a stratification profile.

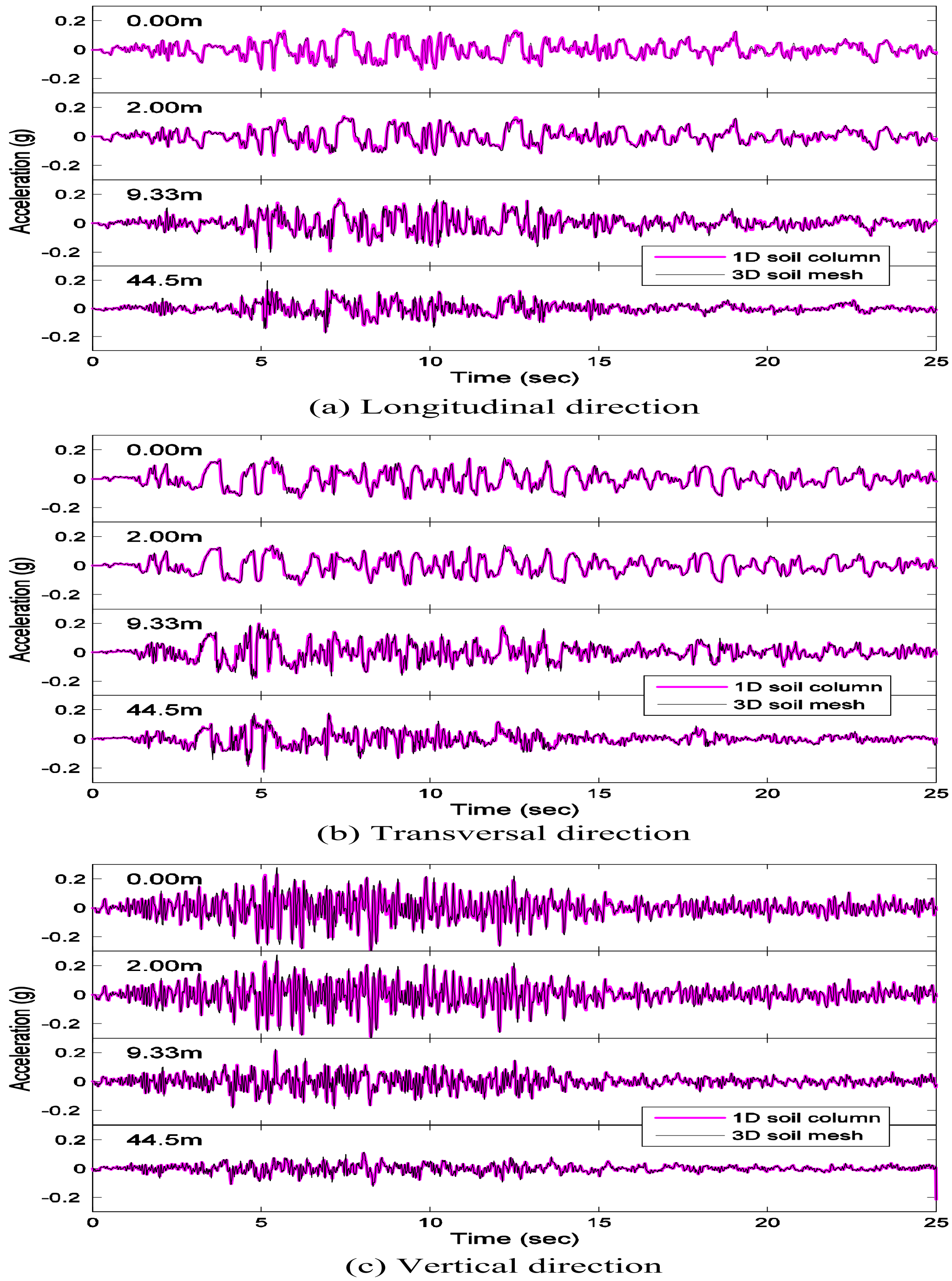
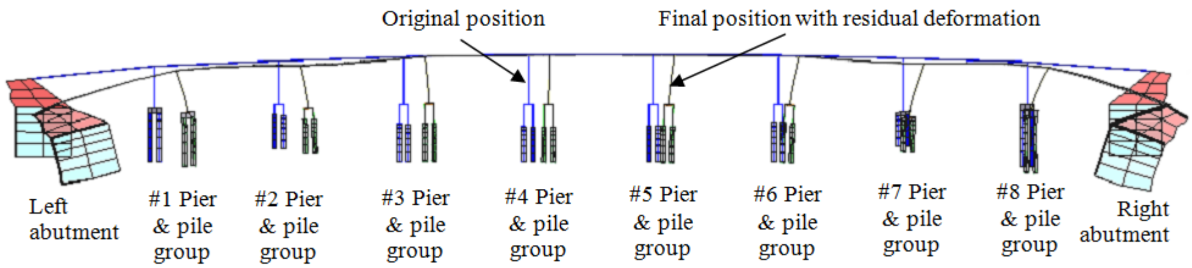
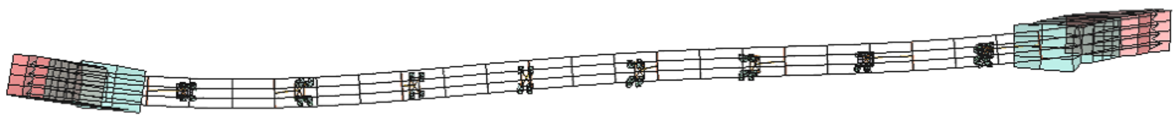


Figure 7. Computed free-field ground motion along the soil profile depth.

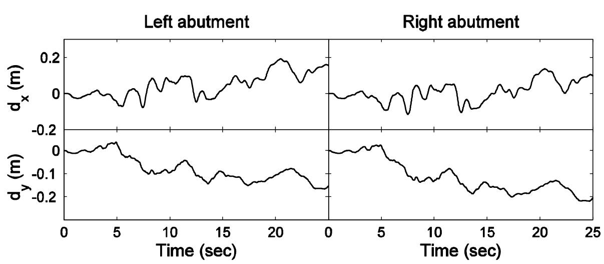


(a) Elevation view (displacement scaled by a factor of 50)

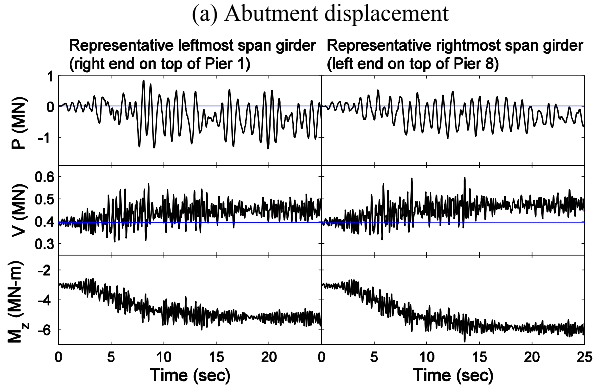


(b) Plan view (displacement scaled by a factor of 150)

Figure 8. Elevation and plan views of the bridge system after earthquake shaking.



(a) Abutment displacement



(b) Bridge response

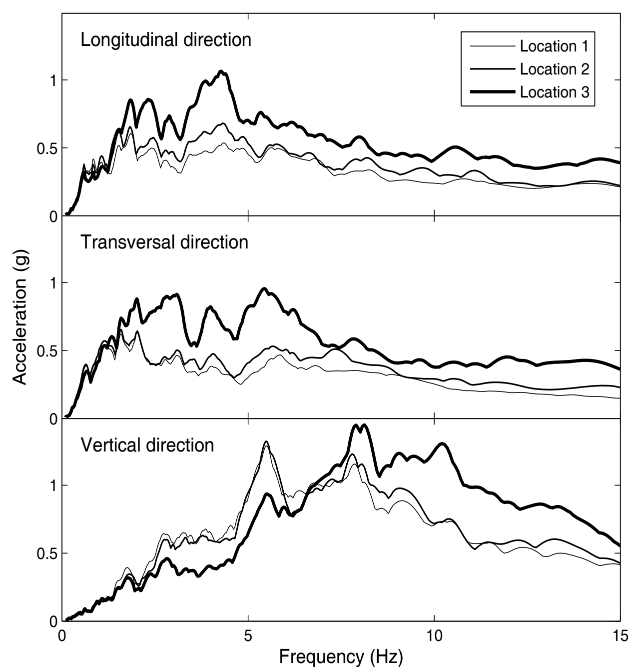


Figure 9. a) Abutment long. displ. d_x, and settlement d_y; b) axial force P, vert. shear force V, and longitudinal moment M_z in bridge girders.

Figure 10. Linear elastic acceleration response spectra (5 percent damped) for locations 1-3 in Fig. 4.

4) In 3D space, inspection of the results in order to draw insights can be a daunting task. In this regard, a parallel investment in advanced visualization techniques would be a worthwhile undertaking (Figure 11).

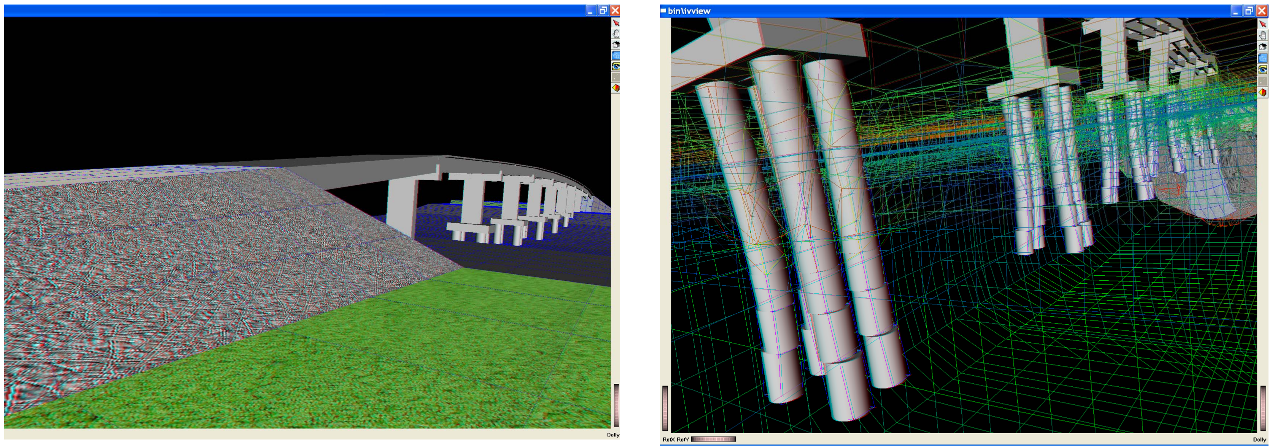


Figure 11. Visualization by Immersaview (http://www.evl.uic.edu/cavern/agave/immersaview/).

OpenSeesPL: A Graphical User Interface

In order to facilitate the efficient execution of 3D ground-foundation computational simulations, a pre- and post-processor graphical interface is under development (Lu 2006, Lu et al. 2006). Currently, this interface permits the analysis of footings, piles, pile groups, and cellular ground modification under static and seismic loading conditions. Recent studies using this interface include:

Pile-Group Push-Over Analysis

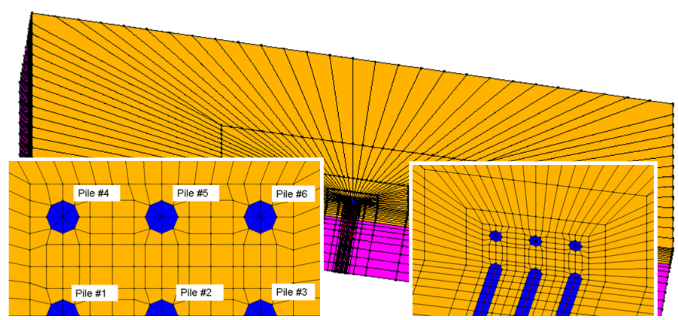


Figure 12. 3x3 Pile group (1/2 mesh due to symmetry).

Elgamal et al. (2009a) conducted a pilot study to illustrate salient pile-group interaction mechanisms (Fig. 12) under lateral loading conditions. Initially, calibration was undertaken for the scenario of a single pile in a homogenous half-space, dictating the use of an appropriate large soil mesh (in terms of element size and location of mesh boundaries). A nonlinear J_2 plasticity model was then employed for the soil domain, and the impact of pile spacing was systematically studied (free head piles in all cases).

Figure 13 depicts the displacement fields (plan view) for pile spacing configurations (where D stands for pile diameter). At close pile spacing (e.g., 3D), it is evident that the entire soil mass between the piles is translating. Thus, the piles end up sharing the available lateral resistance of the surrounding soil. As the spacing increases, each pile is surrounded by an adequate soil domain (independent of the other piles), thus allowing the single pile resistance to be gradually achieved at a spacing of about 8D (Fig. 13).

Ground Modification

For scenarios of ground modification, a representative cell (within a large remediated area) may be studied (using the periodic boundary concept (Law and Lam 2001) along the mesh lateral sides (Fig. 14). Following this logic, OpenSeesPL was employed recently to study liquefaction-induced lateral ground displacement mitigation, by the stone column and pile-pinning approaches (Elgamal et al. 2009b). Figs. 14 and 15 show the results for a 10 *m* thick mildly-inclined (4 degrees) saturated silt layer (permeability $k = 1 \times 10^{-7}$ m/sec), based on Nevada Sand properties at a medium relative density D_r of about 40% (Elgamal et al. 2003; Yang et al. 2003). The modeling approach (elements and pressure dependent liquefaction soil model) are briefly portrayed in the Appendix.

As shown in Fig. 15, 3 simulations were performed. Case **MS** represents the original benchmark **M**edium D_r **S**ilt unremediated situation (essentially a 1D shear wave propagation situation). In order to reduce the extent of liquefaction-induced lateral deformation, remediation by **S**tone **C**olumns (Case **SC**) and by the pile-pinning effect were investigated with an area replacement ratio A_{rr} = 20% (Figs. 14,15). Diameter of the stone column or pile was set at 0.6 m and the surrounding area was adjusted accordingly. The stone column properties were represented by dense sand properties (Lu et al. 2006) and a representative gravel permeability of $k = 1 \times 10^{-2}$ *m/s*. The pile had a bending stiffness $EI = 1.27 \times 10^{5}$ $kN{\cdot}m^2$.

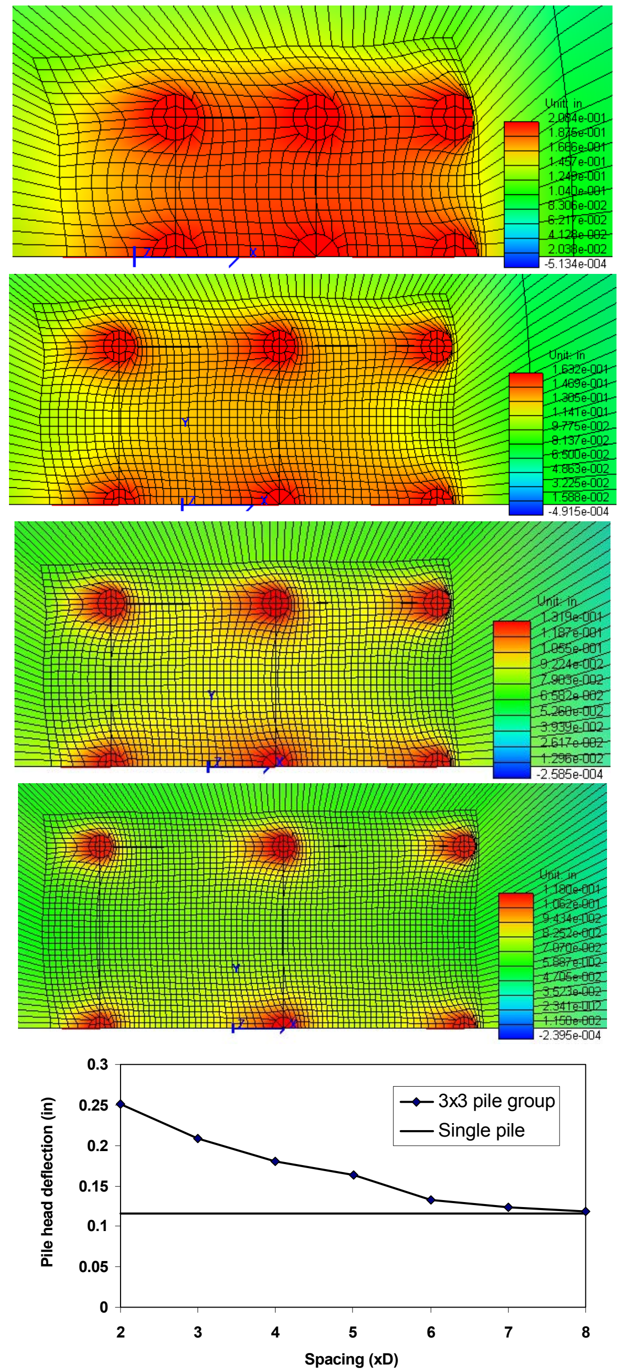
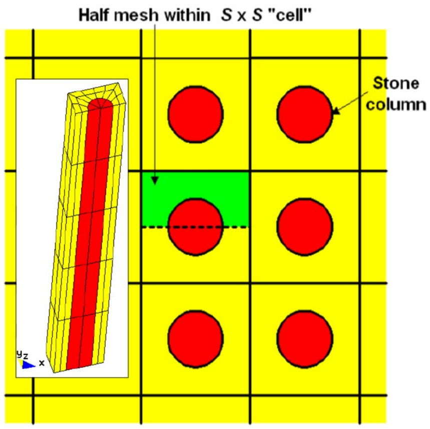


Figure 13. 3x3 pile group displacement field (3D, 5D, 6D, and 8D, top to bottom, and pile head deflection.

Fig. 15 shows the lateral displacement time histories at the model surface center for the 3 cases. The mild 4 degrees inclination imposes a static driving shear stress component (due to gravity), causing the accumulated longitudinal downslope deformation (Fig. 15). For Case SC, the final lateral displacement was reduced to 0.5 m, compared to 1.7 m in Case MS (the free-field response). There is essentially no lateral displacement in the pile-pinning case, showing this approach to be highly viable for cellular remediation.



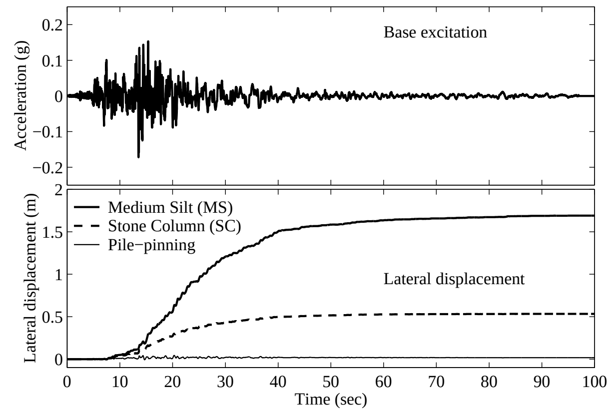


Figure 14. Cellular ground modification and FE mesh (1/2 due to symmetry).

Figure 15. Base acceleration and ground surface displacement.

Conclusions

Three-dimensional computational modeling is increasingly becoming an efficient approach for seismic modeling of soil-foundation-structure systems. Visualization of the results is a particularly important integral component. Scenario-specific graphical interfaces such as OpenSeesPL (pre- and post-processor) promote wider adoption. Such routine usage will in turn enhance the environment for advancement of the underlying engineering sciences.

Acknowledgments

This research was supported by the Pacific Earthquake Engineering Research (PEER) Center, and by the National Science Foundation (Grant No. OCI-0749227). Mr. Atul Nayak, UCSD, led the reported advanced visualization efforts.

Appendix

The employed FE analysis platform OpenSees http://opensees.berkeley.edu) includes a large library of element and material models that are particularly suited to earthquake engineering simulation (Mazzoni et al. 2006). Among the main capabilities accessible via the user interface OpenSeesPL are:

1) In OpenSeesPL, the OpenSees beam-column linear, bilinear and fiber force-based elements may be directly accessed (Spacone et al. 1996; De Sousa 2000; McKenna and Fenves 2001). For the fiber element, the uni-axial Kent-Scott-Park model (Kent and Park 1971; Scott et al. 1982; Mander et al. 1988) with degraded linear unloading/reloading stiffness (Fig.16) is used to model the concrete. The reinforcing steel is represented by a uni-axial bilinear inelastic model with kinematic hardening (equivalent to 1-D J_2 plasticity model with linear kinematic hardening) as shown in Fig.16.

2) For the soil domain, 3D brick elements are included in OpenSees with coupled solid-fluid capabilities (Yang 2000, Yang and Elgamal 2002), following the original *u-p* formulation (Chan 1988). In addition, multi-yield surface soil models (Yang et al. 2003, 2004) are available for the

pressure-independent (J_2 plasticity) and pressure-dependent Drucker-Prager scenarios (Fig. 17). The above soil elements and models allow for simulation of dry/fully saturated soil conditions.

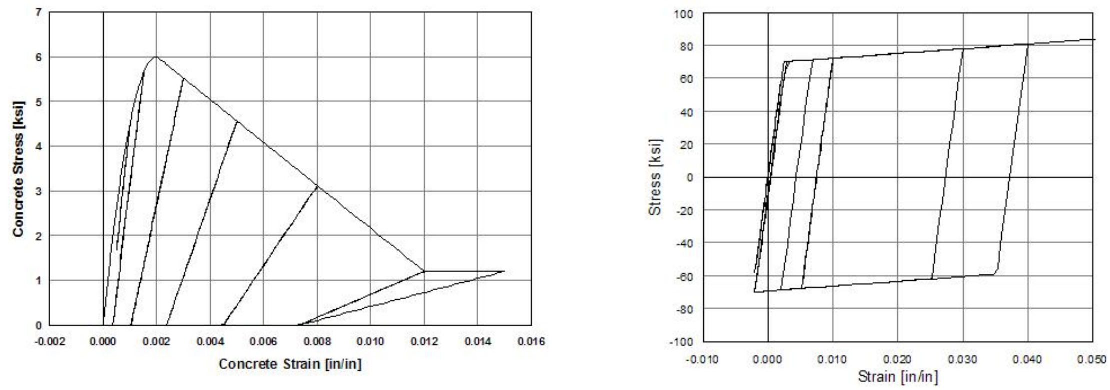


Figure 16. Concrete Kent-Scott-Park model with degraded linear unloading/reloading stiffness, and steel bilinear inelastic model with linear kinematic hardening.

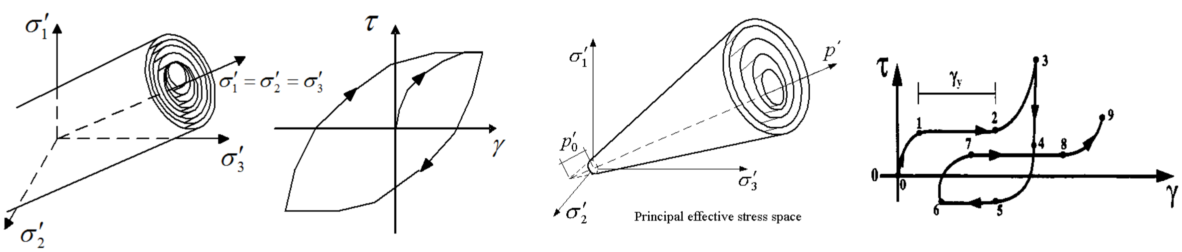


Figure 17. Multi-yield surface soil models available in OpenSees (J_2 and Drucker-Prager).

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