Recent Developments in Computer Simulations and Visualization for Geotechnical Earthquake Engineering Problems

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Abstract

This paper presents some recent developments in computer simulations and visualization for geotechnical earthquake engineering. In particular, presented are developments centered on soil-foundation-structure (SFS) interactions modeling using the finite element method.

The soil-foundation-structure interaction is one of the most challenging problems in earthquake engineering. The computability of such a non-linear, dynamic problem is even more challenging. The main problems can be classified into:

von Neumann computability: also called Physical problem computability refers to how well a physical process can be computed or how well a simulation can predict the response of a mechanical system, and

Turing computability: also called Computer Science (CompSci) computability problem states that the discretized problem is computable if there exists an algorithm that can solve the problem in a finite number of steps.

This paper presents issues that concern both definitions of computability. Moreover, a connection between Physical and CompSci computability is discussed in some details. In particular, illustrative numerical simulation results for single pile and pile group behavior during static and dynamic events are presented to illustrate current endeavor in SFS interaction modeling, simulations and visualization within Authors research group. Discussed are also future plans, in particular related to collaborative grid-enabled modeling, simulation and visualization efforts within the research group and on a much wider, National and International scale.

1 INTRODUCTION

Recent earthquakes have once more time shown the importance of the Soil–Foundation–Structure interaction on seismic behavior of infrastructure. For example, collapse of the Hanshin expressway in Kobe can be to some extent attributed to design flaws including neglect of the SFS interaction effects (e.g. Gazetas and Mylonakis [8]).

It is widely believed that the SFS interaction is beneficial to the behavior of the structural system under earthquake loading.

The presence of deformable soil below and adjacent to a structure effects its seismic response in many ways. First, a flexibly-supported structure has different vibration characteristics, longer fundamental period than the same structure if rigidly supported. Secondly, part of the vibration energy of the flexibly supported structure is dissipated into the supporting and surrounding inelastic soil by radiation of waves and by hysteretic action.

The Applied Technology Council’s development of seismic regulations (known as ATC-3) propose simple formulae for computing fundamental period ($\tilde{T}$) and the effective dumping ratio ($\tilde{\beta}$) of structures founded on mat foundations on a homogeneous half-space.
All codes today use an idealized envelope response spectra which attain constant acceleration values up to certain period (of order of 0.4 second to 1.0 second at most) and then decrease monotonically with period (for example as $T^{-2/3}$).

As a consequence, SFSI leads to smaller accelerations and stresses in the structure and thereby smaller forces onto the foundation. The beneficial role of SFSI has been essentially turned into dogma for many structural engineers.

Even the NEHRP-94 seismic code states that:

"These [seismic] forces therefore can be evaluated conservatively without the adjustments recommended in Sec. 2.5 [i.e. for SFSI effects]"

Eventhough design spectra are derived on a conservative basis, and the above statement may hold for large class of structures, there are case histories that show that the perceived role of SFSI is an over-simplification and may lead to unsafe design.

For example, Figure 1(a) shows comparison of four response spectra: Bucharest (1977), Mexico City (1985), Kobe Fukai (1995), and Kobe Takatori (1995), in terms of spectral amplifications. It is important to note that all the recorded response spectra increase with period and attain their maxima at periods exceeding 1.0sec. This increase in the fundamental period (due to SFSI) will lead to increased rather than reduced response. Figure 1(b) the damage after the earthquake.

![Figure 1: (a) Comparison of a typical seismic code design spectrum to the actual response spectra of several catastrophic earthquakes with strong long period components ($\xi = 5\%$) eg. Gazetas and Mylonakis ([8]); (b) Hanshin Expressway after Kobe Earthquake of 1995.](image)

It is clear that there is a need for a new set of tools, that will help design engineers in predicting behavior of constructed facilities, in particular in conjunction with seismic effects and the SFS interaction behavior. This paper presents some recent developments in that direction related to numerical modeling.

## 2 FINITE ELEMENT MODELING

In this section we briefly describe the incremental elastic–plastic finite element formulations that forms basis for our work. In the first part of this section we describe the small deformation elastic–plastic finite element formulation. We give some insight and examples on using elastic–plastic modeling for soils with Template Elastic–Plastic Constitutive Driver, implemented into OpenSees. We also describe the fully coupled $u-p-U$ formulation which has been implemented into OpenSees and is undergoing testing. We also give very brief overview of the large deformation $u-p-U$
formulation which is under development. In addition to that, we describe recent and ongoing work related to the generation of input motions for seismic simulations and to some extent the software and hardware platforms used in our work.

2.1 Elastic–Plastic Finite Element Formulation

Our elastic plastic finite element formulation, for small deformations, is based on the additive decomposition of the strain tensor. In addition to that, we assume existence of the yield surface, plastic flow directions and a number of internal variables with their evolution laws (Jeremić et al. [13, 14]). The resulting incremental constitutive equations are usually solved using either explicit or implicit integration methods.

The implicit algorithm (Backward Euler) for integrating constitutive problem is based on using the final point in the stress–internal variable space for finding all the relevant derivatives and variables. Since this point is not known in advance, a set of Newton iterations is used to advance the solution toward the final solution for each increment. To this end, resulting iterative change in stress tensor and internal variables space is obtained after some tensor algebra and reads (Jeremić and Sture [13]):

\[
d\sigma_{mn} = -\left( \frac{n+1}{n+1} n+1 T_{mn}^{ij} - \frac{n+1}{n+1} H_{kl}^{ij} - n+1 \frac{\partial m_{kl}}{\partial \sigma_{mn}} \right) \partial_{mn}^{ij} + \frac{n+1}{n+1} \frac{\partial F}{\partial \sigma_{mn}} \left. \frac{\partial m_{kl}}{\partial q_{s}} \right|_{n+1} \frac{\partial q_{s}}{\partial \sigma_{mn}} \right) h_{s}
\]

where we have introduced the fourth order tensors \( T_{ijmn} \) and \( H_{kl}^{ij} \):

\[
n+1 T_{ijmn} = \delta_{im} \delta_{nj} + (\Delta \lambda) T_{ijmn}^{kl} \frac{\partial m_{kl}}{\partial \sigma_{mn}} \bigg|_{n+1} ; \quad n+1 H_{kl} = n+1 m_{kl} + \lambda \frac{\partial m_{kl}}{\partial q_{s}} \bigg|_{n+1} h_{s}
\]

where \( m_{mn} = \partial F/\partial \sigma_{mn}, \xi_{s} = \partial F/\partial q_{s} \) and \( dq_{s} = d\lambda h_{s}(\sigma_{ij}, q_{s}) \). Iterative procedure is continued until the objective function is satisfied given a certain tolerance.

The use of the constitutive stiffness tensor is essential in constructing the finite element matrices. Based on implicit integration schemes, described above, we derive a consistent algorithmic stiffness tensor \( E_{pqmn}^{alp} \) (eg. Jeremić and Sture [13]) which reads

\[
E_{pqmn}^{alp} = R_{pqmn} - \frac{R_{pqkl}^{n+1} H_{kl}^{ij} n+1 m_{ij} R_{ijmn}^{n+1} H_{kl}^{ij} n+1 m_{ij} R_{ijmn}^{n+1} H_{kl}^{ij} n+1 m_{ij} R_{ijmn}^{n+1} H_{kl}^{ij}}{n+1 n_{ot} R_{otrs}^{n+1} H_{rs}^{ij} n+1 m_{ij} R_{ijmn}^{n+1} H_{kl}^{ij} n+1 m_{ij} R_{ijmn}^{n+1} H_{kl}^{ij} n+1 m_{ij} R_{ijmn}^{n+1} H_{kl}^{ij}}
\]

where

\[
R_{mnkl} = (n+1 T_{ijmn})^{-1} E_{ijkl}
\]

while \( T_{ijmn} \) and \( H_{kl} \) were defined in equation (3). If used properly, together with a global iterative method based on Newton scheme, it provides for very fast convergence\(^1\).

The finite element stiffness matrix can be obtained through the weak form of equilibrium (eg. Zienkiewicz and Taylor [24]) and reads (in indicial notation)

\[
k_{a,c}^{E} = \int_{V_{m}} H_{I,b} C^{/A} E_{abcd} H_{J,d} dV_{m}
\]

\(^1\)The full quadratic convergence can only be achieved for material models described through quadratic functions, for example von Mises.
In the above equation, the constitutive stiffness tensor $C^{A_{abcd}}$ follows the algorithmic format from Eq. 4.

2.2 Template Elastic–Plastic Computations

In this part of paper we present some details of a new approach to computations in elasto–plastic geomechanics. The approach is based on the object oriented design philosophy and observations on similarity of most incremental elastic–plastic material models. This new approach to elastic–plastic computations in geomechanics allows for creation of template material models. The analysis of template material models will in turn allow for an easy implementation of other elastic–plastic material models based on the object oriented design principles. Detailed description of the Template Elastic–Plastic Computational Tools is given by [14] (and is available in preprint at http://sokocalo.engr.ucdavis.edu/~jeremic/publications/CGM0102.pdf).

In order to illustrate how different yield criteria, flow directions and hardening/softening rules can be combined to create elastic–plastic material models we present some examples bellow. Figure 2 shows results for cyclic triaxial loading of a normally consolidated sand specimen using the Drucker-Prager yield surface, the Manzari-Dafalias flow direction and the bounding surface hardening rule [17]. The load displacement curve shows near saturation after few cycles while the volumetric response is compressive. Detailed description of the template elastic–plastic constitutive driver and models is given in a recent paper [14].

2.3 Fully Coupled, Solid–Fluid Formulation and Implementation

In this section we briefly describe the formulation and implementation of the fully coupled, solid fluid solid finite elements. The formulations described below follows earlier works by [23] and is based on the effective stress principle and incremental elastic–plastic formulation presented above. The formulation is developed in small deformation setting. The general, large deformation formulation is under development and is presented in some details in next section. After some algebra we can write the system of discretized equations (main unknowns are: solid phase displacements $u_i$, pore pressures $p$ and fluid phase displacements $U_i$) in the following form:
\[
\begin{bmatrix}
(M_s)_{KijL} & 0 & 0 \\
0 & 0 & 0 \\
0 & 0 & (M_f)_{KijL}
\end{bmatrix}
\begin{bmatrix}
\ddot{u}_{Lj} \\
\ddot{p}_L \\
\ddot{U}_{Lj}
\end{bmatrix}
+ \begin{bmatrix}
(C_1)_{KijL} & 0 & -(C_2)_{KijL} \\
0 & 0 & 0 \\
-(C_2)_{LjiK} & (C_3)_{KijL}
\end{bmatrix}
\begin{bmatrix}
\ddot{u}_{Lj} \\
\ddot{p}_L \\
\ddot{U}_{Lj}
\end{bmatrix}
+ \begin{bmatrix}
(K^{EP})_{KijL} & (G_1)_{KijL} & 0 \\
(G_1)_{LjiK} & (P)_{KL} & (G_2)_{LjiK} \\
0 & (G_2)_{KijL} & 0
\end{bmatrix}
\begin{bmatrix}
\ddot{u}_{Lj} \\
\ddot{p}_L \\
\ddot{U}_{Lj}
\end{bmatrix}
= \begin{bmatrix}
(f_\xi)_{Ki} \\
(f_p)_{K} \\
(f_f)_{Ki}
\end{bmatrix}
\] (5)

2.4 Fully Coupled Analysis of Soils in Large Deformation Realm

We have also started a development of the finite element formulation for solving fully coupled, solid-fluid problems in geomechanics. The formulations is based on earlier works of [4], [23], [5] and [11].

The formulation is quite general in that it can handle elastic-plastic geomaterials subjected to arbitrarily large deformations. The formulation is also capable of treating compressible and/or incompressible pore fluid. We assume Lagrangian format for the solid phase while deformations of the fluid phase are written in Eulerian format. It is shown that the multiplicative decomposition of the deformation gradient, utilized in the large deformation elastic-plastic formulation for solid phase, can be used to precisely control the volumetric response of the fluid phase. The fluid content is decomposed additively. The basic unknown variables are the absolute displacement of solid phase \( u_i \), pore pressure \( p \), and relative displacement of fluid phase \( U_i \). The strong form is based on the: Equation of Motion (6), Fluid Mass Conduction (7), and Fluid Mass Conservation (8).

\[
P_{ij,j} + (r_o + m) b_i - \rho_o (1 - n) \ddot{u}_i - (m + \rho_0^{fl}) n \ddot{U}_i = 0
\] (6)

\[
Jn F_{ij}^{-1} \ddot{U}_i + k_{ij} \left[ p_{ij} + \rho^{fl} F_{ij} b_i - \rho^{fl} F_{ij} \ddot{U}_i - \rho^{fl} F_{ij} a_i \right] = 0
\] (7)

\[
(1 - n) \ddot{E}_{ii} \rho^{fl} + \left( \frac{n}{K^{fl}} + \frac{(1 - n)}{K_s} \right) (\ddot{p} + p_i \ddot{u}_i) + J \rho^{fl} n \left( F_{ij}^{-1} \ddot{U}_{ij,j} + F_{ij}^{-1} \ddot{U}_i \right) = 0
\] (8)

where we have used the following notation: \( P_{ij} \) First Piola–Kirchhoff stress tensor; \( r_o \) Mass density; \( m \), Fluid mass content per initial volume; \( b_i \) Body force; \( \rho_o \) Initial mass density for solid phase; \( \rho_0^{fl} \) Initial mass density for fluid phase; \( n \) Porosity; \( F_{ij} \) deformation Gradient; \( J \) Jacobian of the transformation \( J = \det F_{ij} \); \( k_{ij} \) Permeability tensor (anisotropic); \( \rho^{fl} \) Current mass density for fluid phase; \( a_i \) Tortuosity vector; \( D_{km} \) Lagrange strain rate; \( E_{IJ} \) Lagrange strain tensor; \( K_s \) Bulk modulus of the solid phase; and \( K^{fl} \) Bulk modulus of the fluid phase.

The development of this formulation is nearing the end and we plan on starting an implementation in OpenSees fairly soon.

2.5 Input Motions

The very important issue of input motion for any numerical seismic simulations has received much attention in last years. The problems related to this issue can be roughly separated to those related to the (a) absorbing boundaries and (b) what actual seismic input motions should be used for a SFS system. Recent work by Professor Bielak and his group at CMU (Bielak et al. [6, 1, 2, 3, 22]) provides possible solutions for some of the problems. In particular the development of the Domain
Reduction Method (DRM) as well as generation of seismic input motions using Quake program ([www-2.cs.cmu.edu/~quake/](http://www-2.cs.cmu.edu/~quake/)) or any other acceptable input motion generator (SHAKE, Green’s functions, ...) will be used in near future for seismic simulations with OpenSees. For example, Figure 3 shows how a finite element model might look like if DRM is used for analysis.

Figure 3: Plastic soil bowls used in conjunction with DRM will lead to: (a) significant reduction in SFS model size, (b) application of appropriate seismic input motions, and (c) solving the problem of non-reflecting boundaries.

2.6 Simulation Platform

An open source finite element platform, named Open System for Earthquake Engineering Simulations (OpenSees) is used for most of our numerical work. OpenSees features a number of finite elements, material models, solution algorithms. In addition to that, it is available in source code, or in executable format, a large number of examples and an ever growing body of documentation can be obtained from the following web-site [www.opensees.org](http://www.opensees.org).

2.7 Parallel Simulations

Large scale finite element simulations can only be feasible if fast computational hardware is available. One possible option in pursuing fast computer platform is in using the distributed memory parallel computer architecture based on the Beowulf concept (cf. Sterling et al. [20, 21], Reschke et al. [18], Ridge et al [19]). Beowulf type parallel computer feature (a) commodity microprocessors and data buses, allowing us to utilize reliable, low-cost, off-the-shelf technology, (b) fast, switched Ethernet connectivity between nodes, and (c) public-domain, portable compilers and message passing libraries. Specifically, a distributed memory parallel computer GeoWulf (Fig. 4) (Jeremić et al. [16]) has been developed and is used for some time now at UC Davis. GeoWulf consists of 16 heterogenous node computers based on the Intel Pentium II (8 nodes) and AMD Duron (8 nodes) processors, one controller computer (dual Pentium III) and one service computer (AMD Duron). It is running LINUX operating system, a public domain, open source POSIX compliant version of UNIX. Inter-computer communication use the public-domain message passing protocol MPI.
Use of distributed memory parallel computer architecture requires designing the finite element code in such way to take maximum advantage of distributed processing power available. The so-called plastic domain decomposition method is currently under development (Jeremić et al. [15]) and will allow for almost linear speed-up as the number of computer nodes is increased.

3 EXAMPLES SIMULATIONS

We have also performed a number of SFS interaction simulations to test the concepts described above. We have performed both static and dynamic simulations. Below we present some illustrative examples.

3.1 Static Pushover Tests for Layered Systems

A number of static pushover tests were simulated for a single pile. In the analysis currently being done the behavior of a single pile system in layered soils is being analyzed. Figure 5 shows plastic zones (plastiﬁed Gauss-points) for the two cases. Case #1 comprises layers of sand and clay; case #2 comprises a single layer of clay throughout the soil. Automatically generated of P–Y curves from numerical simulation results are shown in Figures 6, 7.

![Figure 5: Plastic zone at the end of loading for (a) case 1, layers of sand and clay; (b) case 2, single layer of clay.](image)

![Figure 6: (a) Moment, shear force, and pressure distribution (clay–sand profile); (b) p-y curves (clay–sand profile).](image)
3.2 Humboldt Bay Bridge Retrofit

The Humboldt Bay Bridge simulations comprise before and after retrofit analysis of one of the foundation systems during a synthetic earthquake, provided by Dr. Abbas Abghari of Caltrans. Figure 8 shows the finite element models for before and after the retrofit cases as well selected response (displacements and accelerations) for before and after the retrofit models. This simulation was not intended to give definite answers on the actual retrofit, so an in-depth study of the differences was not performed. The goal was to show that OpenSees can indeed perform such simulations. The material models used for soil are a version of rotational kinematic hardening Drucker–Prager (for dense sand). The initial friction angle was at 37°.

4 VISUALIZATIONS

Our work on visualization is guided by the need to understand the vast output of 3D simulations of soil–foundation–structure interactions. Current work is on two fronts. The first front is our work on trying to understand the output in terms of visualizing stress, strains, plastic zones and other relevant indicators of the state in which geotechnical solid is. The second front is directed toward visualizing the actual numerical procedures used in computations. The goal of this second visualization focus is in understanding the working of numerical algorithms and possibly debugging their unusual behavior.

Figures 9(a,b) shows hyperstreamsurfaces for minor principal stress (compression) for (a) a single–pile and (b) a four–pile group example (cf. Jeremić et al. [12]). In Figure 9(a), the hyperstreamsurface starts at the center line of a pile and extends through the concrete (in both directions) until it reaches surrounding soil. The hyperstreamsurface shown in Figure 9(a) reveals the stress field in more details. We notice the twisting and “discoloring” of the hyperstreamsurface toward the bottom of the pile. This can be explained by the fact that the stress field in the concrete pile is far away from the main bending effects close to the surface. The state of stress thus deviates just by a small amount from the initial state induced by the self weight. Such small change in stresses leads to a near–uniform green hue coloring. A “kink” in direction and color is noticed close to the surface. This is a part of the hyperstreamsurface that ends in the soil and thus, by moving from stiff (concrete) toward soft (soil) medium, has much smaller minor (compressive) stresses.

Figure 9(b) represents the minor (compressive) principal stress hyperstreamsurface for the four–pile group. In this figure, we have removed one pile from the model to simplify the view. An interesting effect is “shading,” apparent behind the left pile in this figure. The hyperstreamsurface
Figure 8: Humboldt Bay Bridge, before and after the retrofit finite element models; and calculated response.

Figure 9: Hyperstreamsurfaces for minor principal stress (compression) for (a) a single-pile and (b) a four-pile group.
starts, at the center of that pile and extends outside, ending in soil. This pile group is much stiffer than the one-pile example. The effects of bending are much smaller, and the four-pile group behaves like a stiff frame embedded in soil. The observed deformation pattern is thus closer to horizontal translation with a large horizontal resistance and small bending. The hyperstreamlinesurface extends into the soil outside the pile group, with apparent compressive stress. However, just behind the pile, but inside the pile group, the minor principal stress changes significantly in value and then curves by almost 90°. The twist of the minor principal stress hyperstreamlinesurface clearly demonstrates the pile-group effect: Maximal compressive stress (minor principal stress) is acting between two piles in direction perpendicular to the pile loading direction. This findings is contrary to common belief about the state of stresses in pile groups. More important, a large “shading” effect is observed, which, when combined with the twisting of the minor principal stress, leads us to conclude that the two back piles are mostly loaded in direction perpendicular to the global loading direction.

In addition to visualizing the results from numerical simulations (or experiments) it is important to be able to visualize numerical procedures and perform a visual check. For example, Figure 10 shows visualization of the constitutive integration algorithms. In particular, Fig. 10(a) shows visualization of the implicit algorithm iterations applied to the B Material Model ([10]), while 10(b) shows a visualization of the implicit algorithm iterations which failed and were then corrected using line search method ([9]).

Figure 10: Visualization of the constitutive integration algorithms: (a) Implicit algorithm applied to the B Material Model, and (b) Failed implicit iterations and the corrected version of the algorithm using the line search method.

Use of existing and development new Visualization Tools is very important in order to comprehend vast amount of data generated through simulations. Three dimensional, stereoscopic views can be achieved through an Immersive WorkBench (IW) [7], virtual modeling display for scale models and environments. In addition to the IW, computers with high resolution monitors and mouse can and will be used. Use of IW makes it easy to manipulate the complete model of a pile and visualize in 3D, stress and strain tensors, pore pressure fields, displacements and wave propagation patterns. For example, Figure 11(a) shows couple of graduate students and myself, using the IW at UCD-CIPIC, in an experimental lecture on the evolutions of stress distributions in soil around
a pile group during an earthquake.

Figure 11: 3D goggles, and the Immersive WorkBench at the UC Davis, PI with couple of students examines results from a dynamic soil–foundation–structure interaction computer simulations. b) Hyperstreamsurfaces for a front pile in a four pile group (cf. Jeremić et al. [12]) (in VT this is rendered in 3D).

Acknowledgment
This work was supported in part by the Earthquake Engineering Research Centers Program of the National Science Foundation under Award Number EEC-9701568. The author wishes to acknowledge help and support from Professors Gregory Fenves, Ahmed Elgamal, Jacobo Bielak, and Dr. Francis McKenna. The Author would also like to thank his PhD students Zhaohui Yang and Xiaoyan Wu for diligent work of the highest quality.

References


