(Draft)
OpenSeesPL
3D Lateral Pile-Ground Interaction

Version 0.8

User’s Manual

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Chapter 1  Introduction

OpenSeesPL is a graphical user interface (GUI) for three dimensional (3D) ground and ground-structure response. The OpenSees Finite Element (FE) Computational Analysis framework (http://opensees.berkeley.edu) is employed to conduct all analyses. The OpenSeesPL graphical interface (pre- and post-processor) is focused on facilitating a wide class of 3D studies (with additional capabilities yet under development). In the current version, OpenSeesPL may be employed to study a number of geometries and configurations of interest including:

- Linear and nonlinear (incremental plasticity based) 3D ground seismic response with capabilities for 3D excitation, and multi-layered soil strata. Multi-yield surface cohesionless (Drucker-Prager cone model), and (Mises or J2) soil models are available. The coupled solid-fluid analysis option allows for conducting liquefaction studies.

- Inclusion of a pile or shaft in the above 3D ground mesh (circular or square pile in a soil island). The pile can extend above ground and can support a bridge deck, or a point mass at the pile top. The bridge deck can be specified to only translate laterally, or to undergo both lateral translation and rotation. In addition to the seismic excitation option, the pile system may be subjected to monotonic or cyclic lateral push-over loading (in prescribed displacement or prescribed force modes). Soil within the zone occupied by the pile (as specified by pile diameter for instance) can be specified independently, allowing for a variety of useful modeling scenarios.

- Various Ground Modification scenarios may be studied by appropriate specification of the material within the pile zone. For instance, liquefaction countermeasures in the form of gravel drains, stone columns, and solidification/cementation may all be analyzed. Of particular importance and significance in these scenarios is the ability to include the effect of mild infinite-slope inclination (i.e., allowing estimates of accumulated ground deformation, effect of liquefaction countermeasures, pile-pinning effects, and liquefaction-induced lateral loading).

- Slopes and pile systems embedded in sloping ground are also currently being simulated.

In this chapter, the following sections are described:

- Overview of OpenSeesPL

- System requirements and installation of OpenSeesPL
1.1 Overview

OpenSeesPL is a FE user-interface for 3D lateral pile-ground interaction response. This interface allows conducting pushover pile analyses as well as seismic (earthquake) simulations. The FE analysis engine for this interface is the Pacific Earthquake Engineering Research (PEER) Center OpenSees Framework, developed under the leadership of Professor Gregory Fenves of UC Berkeley. For more information, please visit http://opensees.berkeley.edu/.

OpenSeesPL allows simulations for any size of pile and pile diameter. The pile cross section can be circular or square. Linear and nonlinear material properties options are available for pile definition.

OpenSeesPL allows for definition of multiple soil strata. Nonlinearity of soil materials is simulated by incremental plasticity models to allow for modeling permanent deformation and for generation of hysteretic damping. In addition, OpenSeesPL allows including user-defined soil materials.

OpenSeesPL allows for convenient pre-processing and graphical visualization of the analysis results including the deformed mesh, ground response time histories and pile responses. OpenSeesPL makes it possible for geotechnical and structural engineers/researchers to quickly build a model, run FE analysis and evaluate the performance of the pile-ground system.

OpenSeesPL was developed by Dr. Jinchi Lu (jinlu@ucsd.edu), Dr. Ahmed Elgamal (elgamal@ucsd.edu), and Dr. Zhaohui Yang (yangaaa@gmail.com). The OpenSees geotechnical simulation capabilities were developed by Dr. Zhaohui Yang and Dr. Ahmed Elgamal. For more information, please visit http://cyclic.ucsd.edu/opensees/. OpenSeesPL operates in SI and English units.

NOTE: Seismically-induced deformations are complex mechanisms. Much expertise and sound engineering judgment are necessary in interpreting the OpenSeesPL computational results.

1.2 System Requirements

OpenSeesPL runs on PC compatible systems using Windows 98, Windows NT V4.0, Windows 2000, or Windows XP. The system should have a minimum hardware configuration appropriate to the particular operating system.

Internet Explorer 3.0 or above (or compatible Browser) with Java Applet enabled is needed to view the graphic results. For best results, your system’s video should be set to 1024 by 768 or higher.
1.3 Installation

After downloading the OpenSeesPL installation file (OpenSeesPL_Setup.exe), double-click on the icon and the installation procedure will start. Once installed, the default case in OpenSeesPL is a good way to go through the steps involved in conducting an OpenSeesPL analysis. The interface will allow the user to prepare and save an input file, to run the analysis, and to display the response.

Note: Tcl/tk 8.4 must be installed in order to run OpenSeesPL. Please restart the computer after the installation of Tcl/tk 8.4 for the change to take effect.

To download Tcl/tk 8.4, please visit http://www.activestate.com/Products/ActiveTcl/.

1.4 Acknowledgments

OpenSeesPL is based on research underway since the early 1990s, and a partial list of related publications is included in the Appendix section. The OpenSeesPL graphical interface is written in Microsoft Visual C++ Professional Version 6.0 with Microsoft Foundation Class (MFC) Version 6.0. The Java Applet package used to display graphical results in OpenSeesPL is obtained from the website http://ptolemy.eecs.berkeley.edu/. GIF images are generated with GNUPLOT for MS-Windows 32 bit Version 3.7, available at http://www.gnuplot.org/.
Chapter 2  Getting Started

This chapter introduces you the OpenSeesPL simulation environment. This chapter includes:

- How to start OpenSeesPL
- Introduction to basic Windows features available in OpenSeesPL, such as menus and dialogs
- How to get help

2.1 Start-Up

On Windows start OpenSeesPL from the Start button, or from an icon on your desktop. To Start OpenSeesPL from the Start button:

1. Click Start, and then select Programs.
2. Select the OpenSeesPL folder
3. Click on OpenSeesPL

Following the opening banner, OpenSeesPL appears. Click on anywhere, the opening banner will disappear. The OpenSeesPL window is shown in Figure 2.1.

Figure 2.1: OpenSeesPL main window.
2.2 Interface

There are 3 main regions in the OpenSeesPL window – menu bar, the model input window, and the finite element mesh window.

2.2.1 Menu Bar

The menu bar, shown in Figure 2.2, offers rapid access to most of OpenSeesPL’s main features.

![OpenSeesPL menu bar](image)

**Figure 2.2: OpenSeesPL’s menu bar.**

OpenSeesPL’s main features are organized into the following menus:

- **File**: Controls reading, writing and printing of model definition parameters, and exiting OpenSeesPL.
- **Meshing**: Controls mesh generation.
- **Analyze**: Controls running analyses.
- **Display**: Controls displaying of the analysis results.
- **View**: Controls the point of view of the finite element mesh, including scale, view range, 3D rotation, and 2D plane view.
- **Help**: Get quick help on features.

2.2.2 Unit Systems

To switch between SI units and English units, please go to Menu ‘Options’ and click ‘Options’.
2.2.3 Model Input Window

The model input window controls definitions of the model and analysis options, which are organized into three regions (Figure 2.1):

- **Model Definition**: Controls definitions of pile and soil strata including material properties. Meshing parameters are also defined.
- **Loading**: Controls analysis options: pushover analysis or base shaking simulation.
- **Model Inclination**: Controls the inclination angles for the ground surface and the whole model.

2.2.4 Finite Element Mesh Window

The finite element mesh window (Figure 2.1) displays the mesh generated. Once the mesh window is focused, the mesh can be rotated by dragging the mouse, moved in 4 directions by pressing keys of LEFT ARROW, RIGHT ARROW, UP ARROW or DOWN ARROW respectively. The view can be zoomed in (by pressing key ‘F9’), out (by pressing key ‘F10’) or frame (by pressing key ‘F11’).

To display a 2D view, press key ‘F2’ (for Plane XY, where X is the longitudinal direction, Y the transverse direction), ‘F3’ (for Plane YZ, where Z is the vertical direction) or ‘F4’ (for Plane XZ). An isometric view of the mesh can be achieved by pressing key ‘F5’.

2.3 Help
OpenSeesPL features a fully integrated Help system. Detailed help is accessible by:

- Pressing the F1 key anywhere in the OpenSeesPL window.
- Selecting Help Topics from the Help menu. This calls up the Contents page of the OpenSeesPL help file.
- Clicking Help on any dialog.

Figure 2.4 shows OpenSeesPL’s Help, which supports text search, has many hypertext links, and provides detailed information on all menus and dialogs. You may send your questions and suggestions to the email addresses provided in the About OpenSeesPL box (in Help menu).

![OpenSeesPL Help](image)

Figure 2.4: OpenSeesPL Help.
Chapter 3  Definition of Model Profile

This chapter describes how to define a model profile in OpenSeesPL. This chapter includes:

- Geometry definition of pile and soil strata
- Material properties definition for pile, soil and pile-soil interfacing layer

3.1 Geometry Definition

3.1.1 Pile

To define pile geometry, click on the Pile Parameters button in the Model Input window. The pile geometry is defined by the following parameters (Figure 3.1):

**Pile Type**  The pile cross section can be circular or square.

**Pile Diameter**  The diameter (if a circular pile is chosen), or the side length (if a square pile is chosen) of the pile cross section. The value entered must be greater than zero.

**Pile Height Below Surface**  The height of the pile below the ground surface. The value entered must be greater than zero.

**Pile Height Above Surface**  The height of the pile above the ground surface. The value entered must be greater than zero.

To include a pile cap, check the checkbox “w/ Pile Cap” (Figure 3.1). The geometry of the pile cap is controlled by the following three parameters:

**Length**  The length of the pile cap (along the longitudinal direction).

**Width**  The width of the pile cap (along the transverse direction).

**Thickness**  The thickness of the pile cap (along the vertical direction).

The FE nodes of the pile cap will be constrained horizontally and vertically. In other words, the pile head will behave as a fixed head if a pile cap is included.
3.1.2 Soil Strata

To define soil strata, click on the Soil Parameters button in the Model Input window.

A total of 10 soil strata can be defined in OpenSeesPL (Figure 3.2). The profile of the soil strata can be defined by using the follow parameters:

**Thickness** The thickness for a soil layer. Definitions following a zero height will be ignored. In other words, the total number of soil layers in use will be equal to the number of the last soil layer that contain no zero values, e.g., if you need 5 strata, enter nonzero heights for Stratum #1 through Stratum #5.

**Water Table Depth** The Water Table Depth refers to the depth below ground surface. (e.g., 0.0 corresponds to a fully saturated soil profile, 1.0 is 1m below ground surface). Dry sites should specify water table depth to be equal to the entire model depth.

3.1.3 Mesh Definition

To define the finite element mesh, click on the Mesh Parameters button in the Model Input window (Figure 3.3).

3.1.3.1 General Definition
Mesh Scale  The mesh scale can be quarter mesh, half mesh or full mesh (to reduce computational effort depending on the situation at hand).

Figure 3.2: Definition of soil strata geometry and material properties.

Number of Slices  The number of mesh slices in the circumferential direction.

Number of Beam Elements for Pile Section Above Ground Surface  The number of beam elements used for the pile section above the ground surface.

3.1.3.2  Horizontal Meshing

The meshing in the horizontal direction is controlled by the following parameters (Figure 3.3b):
**Thickness of Interfacing Layer**  The thickness of the pile-soil interfacing layer (as a factor of the pile diameter D). The interfacing layer can be removed if a zero thickness is specified.

**Number of Mesh Layers after Interface (N_m)**  The number of the mesh layers after interface in the horizontal direction.

**Number of Adjacent Uniform Width Layers (N_u)**  The number of the uniform width layers after interface. $1 \leq N_u \leq N_m$.

**Width of Uniform Layer Element**  The width of each uniform layer element (as a factor of the pile diameter).

**Increasing Rate of Element Width afterwards**  The Increasing rate of the element size afterwards (e.g, if 1.2 is specified, the element size of the next layer will be 1.2 times of that of this layer).

### 3.1.3.3  Vertical Meshing

The meshing in the vertical direction is controlled by the following parameters (Figure 3.3c):

**Number of Mesh Layers**  The number of elements in the vertical direction in a layer.

**Uniform Meshing**  Uniform element size will be used if checked.

**Ratio of Top Element Height over Bottom**  The ratio of top element height over the bottom. This is activated if the **Uniform Meshing** checkbox is unchecked.

### 3.1.3.4  Mesh Scaling

The soil domain will be scaled if ‘**Re-scale Soil Domain in Horizontal Directions**’ checkbox is checked (Figure 3.3d):

**Model Length**  The length of the soil domain (along the longitudinal direction) to be scaled.

**Model Width**  The width of the soil domain (along the transverse direction) to be scaled.
Figure 3.3: Definition of meshing parameters.
Figure 3.3: (continued).
3.2 Material Properties Definition

3.2.1 Pile Properties

To define pile geometry, click on the **Pile Parameters** button in the **Model Input** window. In OpenSeesPL, the element types available for the pile are **elasticBeamColumn**, which represents elastic beam-column element, and **nonlinearBeamColumn**, which represents a nonlinear beam-column element based on the non-iterative (or iterative) force formulation. Detail information can be found in the OpenSees User Manual (Mazzoni et al. 2006).

3.2.1.1 Linear Beam Element

The material properties of the pile for the linear beam element (**elasticBeamColumn**) are defined by the following parameters (Figure 3.4):

**Young’s Modulus (E)**  Young’s Modulus of the pile.

**Mass Density**  The Mass Density of the pile.

**Moment of Inertia (I)**  The Moment of Inertia of the pile. This can be specified directly or calculated based on the pile diameter.

![Linear Beam Properties](image)

Figure 3.4: Definition of linear pile properties.

3.2.1.2 Nonlinear Beam Element

OpenSees uses the **Section** command to define the nonlinear beam-column element (a section defines the stress resultant force-deformation response at a cross section of a beam-column element). Two types of sections are available in OpenSeesPL for the nonlinear beam element (**nonlinearBeamColumn**): **Aggregator** Section or **Fiber** Section. Detail information can be found in the OpenSees User Manual (Mazzoni et al. 2006).

3.2.1.2.1 Aggregator Section
The **Aggregator** Section is defined by the following parameters in OpenSeesPL (Figure 3.5):

**Flexural Rigidity My & Mz** The Flexural Rigidity of the pile which is equal to the product of Young’s Modulus (E) and the Moment of Inertia (I). My corresponds the moment-curvature about section local y-axis and Mz corresponds the moment-curvature about section local z-axis.

**Yield Moment** The Yield Moment of the pile.

**Isotropic Hardening Parameter** The isotropic hardening Modulus.

**Kinematic Hardening Parameter** The kinematic hardening Modulus.

**Shear Rigidity Vy & Vz** The Shear Rigidity of the pile which is equal to the product of the Shear Modulus (G) and the area of the pile cross section (A). Vy corresponds the shear force-deformation along section local y-axis and Vz corresponds the shear force-deformation along section local z-axis.

**Torsional Rigidity T** The Torsional Rigidity of the pile which is equal to the product of the Shear Modulus and J..

**Axial Rigidity P** The Axial Rigidity of the pile which is equal to the product of Young’s Modulus (E) and the area of the pile cross section (A).

<table>
<thead>
<tr>
<th>Nonlinear Beam Properties</th>
<th>Aggregator Section</th>
<th>My &amp; Mz</th>
<th>Flexural Rigidity El</th>
<th>Yield Moment</th>
<th>Hardening Parameter</th>
<th>0</th>
<th>[kip-ft]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>303764 10094</td>
<td>885.07453144</td>
<td></td>
<td></td>
<td>[kip-#2]</td>
</tr>
<tr>
<td></td>
<td>Vy &amp; Vz</td>
<td></td>
<td>154266 093 914</td>
<td></td>
<td></td>
<td></td>
<td>[kip]</td>
</tr>
<tr>
<td></td>
<td>T:</td>
<td></td>
<td>102118.57867</td>
<td></td>
<td></td>
<td></td>
<td>[kip-#2]</td>
</tr>
<tr>
<td></td>
<td>P:</td>
<td></td>
<td></td>
<td></td>
<td>39077630.27</td>
<td></td>
<td>[kips]</td>
</tr>
</tbody>
</table>

Figure 3.5: Definition of nonlinear pile properties (Aggregator Section).

### 3.2.1.2.2 Fiber Section

The fiber section is only available in a full mesh situation in OpenSeesPL. The dialog of defining Fiber Section is shown in Figure 3.6 (the Fiber Section is only available to circular pile in this version of OpenSeesPL). Two materials are available: **Concrete01** and **Steel01** in this version of OpenSeesPL. **Concrete01** (Figure 3.7) is defined by the following parameters (for Core and Cover, see Figure 3.11):

**Concrete Compressive Strength** The concrete compressive strength at 28 days ($f_{pc}$ in Figure 3.7).
Concrete Strain at Maximum Strength  The concrete strain at maximum strength ($\epsilon_{psc0}$ in Figure 3.7).

Concrete Crushing Strength  The concrete crushing strength ($f_{pcu}$ in Figure 3.7).

Concrete Strain at Crushing Strength  The concrete strain at crushing strength ($\epsilon_{psU}$ in Figure 3.7).

Note that the compressive concrete parameters should be input as negative values. Typical hysteretic stress-strain relation of the Concrete01 material is shown in Figure 3.8).

Steel01 is defined by the following parameters (Figure 3.9 and Figure 3.10):

Yield Strength  The yield strength of steel.

Initial Elastic Tangent  The initial elastic tangent of steel.

Strain-hardening Ratio  The strain-hardening ratio (ratio between post-yield tangent and initial elastic tangent)

Patch (Figure 3.11) is defined by the following parameters (for both Core and Cover):

Number of Subdivisions (fibers) in the Curcumferential Direction  The number of subdivisions (fibers) in the circumferential direction of the pile circular cross section ($numSubdivCirc$ in Figure 3.12).

Number of Subdivisions (fibers) in the Radial Direction  The number of subdivisions (fibers) in the radial direction of the pile circular cross section ($numSubdivRad$ in Figure 3.12).

Internal Radius  The internal radius of the patch ($intRad$ in Figure 3.12).

External Radius  The external radius of the patch ($extRad$ in Figure 3.12).

The values of $y_{Center}$ and $z_{Center}$ (y & z-coordinates of the center of the circle) as shown in Figure 3.12 are zeros. And the $startAng$ (starting angle) and $endAng$ (ending angle) are set to 0 and 360 degrees respectively in OpenSeesPL since only a full mesh is available for fiber section nonlinear beam element).

Layer is defined by the following parameters (Figure 3.13):

Number of Reinforcing Bars along Layer  The number of reinforcing bars along layer ($numBars$ in Figure 3.13).
Area of Individual Reinforcing Bar  The area of individual reinforcing bar.

Radius of Reinforcing Layer  The radius of reinforcing layer ($radius in Figure 3.13) .

The values of $yCenter and $zCenter (y & z-coordinates of the center of the circle) as shown in Figure 3.13 are zeros. And the $startAng (starting angle) and $endAng (ending angle) are set to 0 and 360 degrees respectively in OpenSeesPL since only a full mesh is available for fiber section nonlinear beam element).

Figure 3.6: Definition of nonlinear pile properties (Fiber Section).
Figure 3.7: Material Parameters of the Concrete01 material (Mazzoni et al. 2006).

Figure 3.8: Typical hysteretic stress-strain relation of the Concrete01 material (Mazzoni et al. 2006).
Figure 3.9: Material Parameters of the Steel01 material (Mazzoni et al. 2006).

Figure 3.10: Typical hysteretic behavior of model with Isotropic hardening of the Steel01 material (Mazzoni et al. 2006).
Figure 3.11: Schematic of fiber section definition for a circular cross section (Mazzoni et al. 2006).

Figure 3.12: Schematic of patch definition for a circular cross section (Mazzoni et al. 2006)
3.2.1.3 Pile Head

The pile head is controlled by the following parameters (Figure 3.1):

Fixed or Free Head Free Head is chosen. A fixed head can be obtained by specifying a pile cap (see Section 3.2.1.4).

Pile Head Mass The mass applied at the pile head.

Axial Load The axial load applied at the pile head (positive as compression).

3.2.1.4 Pile Cap

As shown in Figure 3.1, the material properties (linear) of the pile cap is defined by the following parameters:

Young’s Modulus Young’s Modulus of the material employed for the pile cap.

Poisson’s Ratio Poisson’s Ratio of the material employed for the pile cap. Note that 3D 8-node brick elements are used for the pile cap.

Mass Density Mass Density of the material employed for the pile cap.

The connection between the pile head and the pile cap will be no rotation both longitudinally and transversely. The nodes of the pile cap are constrained in the longitudinal and vertical directions.
3.2.2 Soil Properties

3.2.2.1 Theory

In OpenSees, the soil model (Figure 3.14) for cohesionless soils is developed within the framework of multi-yield-surface plasticity (e.g., Prevost 1985). In this model, emphasis is placed on controlling the magnitude of cycle-by-cycle permanent shear strain accumulation (Figure 3.15) in clean medium to dense sands (Parra 1996; Yang 2000; Yang et al. 2003). Furthermore, appropriate loading-unloading flow rules were devised to reproduce the observed strong dilation tendency, and resulting increase in cyclic shear stiffness and strength (the “Cyclic Mobility” mechanism). The material type for the cohesionless soils in OpenSees is called \textbf{PressureDependMultiYield}.

Clay material is modeled as a nonlinear hysteretic material (Parra 1996; Yang 2000; Yang et al. 2003) with a Von Mises multi-surface (Iwan 1967; Mroz 1967) kinematic plasticity model (Figure 3.16). In this regard, focus is on reproduction of the soil hysteretic elasto-plastic shear response (including permanent deformation). In this material, plasticity is exhibited only in the deviatoric stress-strain response. The volumetric stress-strain response is linear-elastic and is independent of the deviatoric response. This constitutive model simulates monotonic or cyclic response of materials whose shear behavior is insensitive to the confinement change. Plasticity is formulated based on the multi-surface (nested surfaces) concept, with an associative flow rule (according to the well-known Provost approach). In the clay model, the nonlinear shear stress-strain back-bone curve is represented by the hyperbolic relation (Kondner 1963), defined by the two material constants, low-strain shear modulus and ultimate shear strength. The material type for the cohesive soils in OpenSees is called \textbf{PressureIndependMultiYield}.

![Figure 3.14: Multi-yield surfaces in principal stress space and deviatoric plane (Prevost 1985; Parra 1996; Yang 2000)](image-url)
Figure 3.15: Shear-effective confinement and shear stress-strain response (Yang and Elgamal 2002; Yang et al. 2003).
3.2.2.2 Predefined Materials

As shown in Figure 3.2, the soil materials can be selected from an available menu of cohesionless and cohesive soil materials (Figure 3.17). There are 18 predefined materials in OpenSeesPL. Basic model parameter values for these materials are listed in Table 3.1.

Figure 3.16: Von Mises multi-surface kinematic plasticity model (Yang 2000; Yang et al. 2003).
If ‘Cohesionless very loose’ is chosen, the user is allowed to define the residual shear strength (0.2 kPa is specified by default). The cohesionless very loose soil is same as the cohesionless loose soil except the user is allowed to specify the residual shear strength for the very loose one.

In addition, user-defined cohesionless and cohesive soil materials (U-Sand1, U-Sand2, U-Clay1 and U-Clay2) are also available to choose.

As shown in Figure 3.2, parabolic variation of soil modulus with depth is used if ‘P’ is selected. Linear variation of soil modulus with depth is used if ‘L’ is selected. And the constant soil modulus with depth is used if ‘C’ is selected;

```
1: Cohesionless very loose, silt permeability
2: Cohesionless very loose, sand permeability
3: Cohesionless very loose, gravel permeability
4: Cohesionless loose, silt permeability
5: Cohesionless loose, sand permeability
6: Cohesionless loose, gravel permeability
7: Cohesionless medium, silt permeability
8: Cohesionless medium, sand permeability
9: Cohesionless medium, gravel permeability
10: Cohesionless medium-dense, silt permeability
11: Cohesionless medium-dense, sand permeability
12: Cohesionless medium-dense, gravel permeability
13: Cohesionless dense, silt permeability
14: Cohesionless dense, sand permeability
15: Cohesionless dense, gravel permeability
16: Cohesive soft
17: Cohesive medium
18: Cohesive stiff
19: U-Sand1...
20: U-Sand2...
21: U-Clay1...
22: U-Clay2...
```

Figure 3.17: Soil materials in OpenSeesPL.
### Table 3.1: Predefined soil materials in OpenSeesPL

<table>
<thead>
<tr>
<th>Cohesionless Soil</th>
<th>Reference shear modulus $G_r$ (kPa, at $p_r'=80\text{kPa}$)</th>
<th>Reference bulk modulus $B_r$ (kPa, at $p_r'=80\text{kPa}$)</th>
<th>Friction angle $\phi$ (degrees)</th>
<th>Permeability coeff. $\lambda$ (m/s)</th>
<th>Mass density $\rho$ (ton/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose, silt permeability</td>
<td>5.5E+04</td>
<td>1.5E+05</td>
<td>29</td>
<td>1.0E-07</td>
<td>1.7</td>
</tr>
<tr>
<td>Very loose, sand permeability</td>
<td>5.5E+04</td>
<td>1.5E+05</td>
<td>29</td>
<td>1.0E-02</td>
<td>1.7</td>
</tr>
<tr>
<td>Very loose, gravel permeability</td>
<td>5.5E+04</td>
<td>1.5E+05</td>
<td>29</td>
<td>1.0E-07</td>
<td>1.7</td>
</tr>
<tr>
<td>Loose, silt permeability</td>
<td>5.5E+04</td>
<td>1.5E+05</td>
<td>29</td>
<td>1.0E-02</td>
<td>1.7</td>
</tr>
<tr>
<td>Loose, sand permeability</td>
<td>5.5E+04</td>
<td>1.5E+05</td>
<td>29</td>
<td>1.0E-07</td>
<td>1.7</td>
</tr>
<tr>
<td>Loose, gravel permeability</td>
<td>5.5E+04</td>
<td>1.5E+05</td>
<td>29</td>
<td>1.0E-02</td>
<td>1.7</td>
</tr>
<tr>
<td>Medium, silt permeability</td>
<td>7.5E+04</td>
<td>2.0E+05</td>
<td>33</td>
<td>1.0E-07</td>
<td>1.9</td>
</tr>
<tr>
<td>Medium, sand permeability</td>
<td>7.5E+04</td>
<td>2.0E+05</td>
<td>33</td>
<td>6.6E-05</td>
<td>1.9</td>
</tr>
<tr>
<td>Medium, gravel permeability</td>
<td>7.5E+04</td>
<td>2.0E+05</td>
<td>33</td>
<td>1.0E-02</td>
<td>1.9</td>
</tr>
<tr>
<td>Medium-dense, silt permeability</td>
<td>1.0E+05</td>
<td>3.0E+05</td>
<td>37</td>
<td>1.0E-07</td>
<td>2.0</td>
</tr>
<tr>
<td>Medium-dense, sand permeability</td>
<td>1.0E+05</td>
<td>3.0E+05</td>
<td>37</td>
<td>6.6E-05</td>
<td>2.0</td>
</tr>
<tr>
<td>Medium-dense, gravel permeability</td>
<td>1.0E+05</td>
<td>3.0E+05</td>
<td>37</td>
<td>1.0E-02</td>
<td>2.0</td>
</tr>
<tr>
<td>Dense, silt permeability</td>
<td>1.3E+05</td>
<td>3.9E+05</td>
<td>40</td>
<td>1.0E-02</td>
<td>2.1</td>
</tr>
<tr>
<td>Dense, sand permeability</td>
<td>1.3E+05</td>
<td>3.9E+05</td>
<td>40</td>
<td>6.6E-05</td>
<td>2.1</td>
</tr>
<tr>
<td>Dense, gravel permeability</td>
<td>1.3E+05</td>
<td>3.9E+05</td>
<td>40</td>
<td>1.0E-02</td>
<td>2.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cohesive Soil</th>
<th>Shear modulus $G$ (kPa)</th>
<th>Bulk modulus $B$ (kPa)</th>
<th>Cohesion $c$ (kPa)</th>
<th>Permeability coeff. $\lambda$ (m/s)</th>
<th>Mass density $\rho$ (ton/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft</td>
<td>1.3E+04</td>
<td>6.5E+04</td>
<td>18.0</td>
<td>1.0E-09</td>
<td>1.3</td>
</tr>
<tr>
<td>Medium</td>
<td>6.0E+04</td>
<td>3.0E+05</td>
<td>37.0</td>
<td>1.0E-09</td>
<td>1.5</td>
</tr>
<tr>
<td>Stiff</td>
<td>1.5E+05</td>
<td>7.5E+05</td>
<td>75.0</td>
<td>1.0E-09</td>
<td>1.8</td>
</tr>
</tbody>
</table>

1. Where $p_r'$ is the reference mean effective confining pressure at which soil appropriate soil properties are defined.
2. Friction angles for cohesionless soils are based on Table 7.4 (p.425) of Das, B.M. (1983).
3. Permeability values are based on Fig. 7.6 (p.210) of Holtz and Kovacs (1981).
4. Mass density is based on Table 1.4 (p.10) of Das (1995).
5. Cohesion for cohesive soils are based on Table 7.5 (p.442) of Das (1983).
Backbone Curve
At a constant confinement \( p' \), the shear stress \( \tau \) (octahedral) - shear strain \( \gamma \) (octahedral) nonlinearity is defined by a hyperbolic curve (backbone curve, see Figure 3.18):

\[
\tau = \frac{G_r \gamma}{1 + \frac{\gamma}{\gamma_r}}
\]  

(3.1)

where \( G_r \) is the low-strain shear modulus (see 3.2.2.3.1), and \( \gamma_r \) satisfies the following equation at \( p' \):

\[
\tau_f = \frac{2\sqrt{2} \sin \phi}{3 - \sin \phi} \frac{p'}{p'} = \frac{G_r \gamma_{\text{max}}}{1 + \gamma_{\text{max}} / \gamma_r}
\]  

(for sands) \hspace{2cm} (3.2a)

and,

\[
\tau_f = \frac{2\sqrt{2} \sin \phi}{3 - \sin \phi} \frac{p'}{p'} + \frac{2\sqrt{2}}{3} c = \frac{G_r \gamma_{\text{max}}}{1 + \gamma_{\text{max}} / \gamma_r}
\]  

(for clays) \hspace{2cm} (3.2b)

where \( \tau_f \) is the peak (octahedral) shear strength, \( \phi \) is the friction angle, \( c \) is the cohesion, and \( \gamma_{\text{max}} \) is the maximum shear strain (10\% is employed in OpenSeesPL).

The octahedral shear stress \( \tau \) is defined as:

\[
\tau = \frac{1}{3} \left[ (\sigma_{xx} - \sigma_{yy})^2 + (\sigma_{yy} - \sigma_{zz})^2 + (\sigma_{zz} - \sigma_{xx})^2 + 6\sigma_{xy}^2 + 6\sigma_{yz}^2 + 6\sigma_{xz}^2 \right]^{1/2}
\]

and the octahedral shear strain \( \gamma \) is defined as:

\[
\gamma = \frac{2}{3} \left[ (e_{xx} - e_{yy})^2 + (e_{yy} - e_{zz})^2 + (e_{zz} - e_{xx})^2 + 6e_{xy}^2 + 6e_{yz}^2 + 6e_{xz}^2 \right]^{1/2}
\]

The number of yield surfaces used for the predefined sands and clays is 20.
Shear stress

Shear strength

Number of yield surfaces = 5

Peak shear strain

Shear modulus = Mass density \times \text{Shear wave velocity}^2

Shear strain

Shear modulus = Mass density \times \text{Shear wave velocity}^2

Number of yield surfaces = 0

Shear modulus = Mass density \times \text{Shear wave velocity}^2

Number of yield surfaces = 1

Figure 3.18: Soil backbone curve and yield surfaces.

From Eq. (3.2), we can obtain:

\[
\gamma_r = \frac{\tau_f \gamma_{\text{max}}}{G_r \gamma_{\text{max}} - \tau_f}
\]  \hspace{1cm} (3.3a)

Or

\[
\gamma_{\text{max}} = \frac{\tau_f \gamma_r}{G_r \gamma_r - \tau_f}
\]  \hspace{1cm} (3.3b)

Substituting Eq. (3.3a) into Eq. (3.1), we can obtain:

\[
\tau = \frac{G_r \gamma}{1 + (\frac{G_r}{\tau_f} - \frac{1}{\gamma_{\text{max}}})\gamma}
\]  \hspace{1cm} (3.4)

Take Medium Sand (Table 3.1) as an example, \(G_r = 75,000 \text{ kPa}, \ p'_f = 80 \text{ kPa}, \phi = 33^\circ\).

Substituting the above values into Eq. (3.2a), we can obtain:
\[ \tau_f = \frac{2\sqrt{2} \sin(33^\circ)}{3 - \sin(33^\circ)}(80) = 50.2 \text{ kPa} \]  \hspace{1cm} (3.5)

Substituting the above into Eq. (3.4), we can obtain:

\[ \tau = \frac{(75000)\gamma}{1 + (1494 - \frac{1}{\gamma_{\text{max}}})\gamma} \]  \hspace{1cm} (3.6)

Figure 3.19 shows the backbone curves at \( \gamma_{\text{max}} = 2\%, 5\% \) and 10\%, based on Eq. (3.6).

![Figure 3.19: Backbone curves for Medium Sand.](image)

### 3.2.2.3 User-Defined Materials

User-defined materials include user-defined sand (U-Sand1 and U-Sand2) with confinement-dependent material properties, and user-defined clay (U-Clay1 and U-Clay2) with properties independent of confinement variation. To define the parameters of a user-defined material, click the list of soil materials and select ‘U-Sand1’, ‘U-Sand2’, ‘U-Clay1’, or ‘U-Clay2’ accordingly (Figure 3.17).
3.2.2.3.1 *User Defined Sand1 (U-Sand1)*

Sandy soil with confinement-dependent shear response can be defined by specifying the following parameters (see Figure 3.15 and Figure 3.20):

![U-Sand1 for Soil Layer 1](image)

**Figure 3.20:** U-Sand1 in OpenSeesPL.

**Saturated Mass Density**  The saturated mass density of the cohesionless soil.

**Reference Pressure**  The reference mean effective confining pressure \( p'_r \) at which soil appropriate soil properties below are defined.

\[ G_{\text{max}} \]  The reference low-strain shear modulus \( G_r \), specified at a reference mean effective confining pressure \( p'_r \).

\[ B_{\text{max}} \]  The reference bulk modulus \( B_r \), specified at a reference mean effective confining pressure \( p'_r \).
**Pressure Dependence Coefficient \((d)\)** A positive constant defining variations of \(G\) and \(B\) as a function of instantaneous effective confinement \(p'\):

\[
G = G_r \left( \frac{p'}{p'_r} \right)^d \quad B = B_r \left( \frac{p'}{p'_r} \right)^d
\]

(3.7)

**Peak Shear Strain** An octahedral shear strain at which the maximum shear strength is reached, specified at a reference mean effective confining pressure \(p'_r\). The suggested values are between 0.001\% and 20\%.

**Friction Angle** The friction angle \((\phi)\) at peak shear strength in degrees. The suggested values are between 5 and 65 degrees.

**Fluid Mass Density** The mass density of the fluid, which is usually 1.0 ton/m\(^3\).

**Combined Bulk Modulus** The combined undrained bulk modulus \(B_c\) relating changes in pore pressure and volumetric strain, may be approximated by:

\[
B_c \approx B_f / n
\]

(3.8)

where \(B_f\) is the bulk modulus of fluid phase \((2.2\times10^6 \text{ kPa} \text{ for water typically})\), and \(n\) the initial porosity.

**Horizontal Permeability** The permeability along the horizontal direction.

**Vertical Permeability** The permeability along the vertical direction.

**User Defined Nonlinear Shear Stress-Strain Backbone Curve:**

The nonlinear shear stress-strain backbone curve can be defined by specifying a \(G/G_{\text{max}}\) curve (Figure 3.20). To specify the \(G/G_{\text{max}}\) curve, first enter “number of points defining \(G/G_{\text{max}}\) curve” and then enter pairs of shear strain and \(G/G_{\text{max}}\) values. The maximum number of points that can be entered is 13 (the backbone curve becomes horizontal after point 13). In addition:
- If the number of points is zero, then the built-in hyperbolic curve will be used instead.
- If the number of points is 1, the material is elastic-perfectly-plastic.

The user-defined backbone curve is activated if the number of points is greater than zero. In this case, the user specified friction angle \(\phi\) is ignored. Instead, \(\phi\) is defined as follows:

\[
\sin \phi = \frac{3\sqrt{3} \sigma_m / p'_r}{6 + \sqrt{3} \sigma_m / p'_r}
\]

(3.9)
where $\sigma_m$ is the product of the last modulus and strain pair in the modulus reduction curve. Therefore, it is important to adjust the backbone curve so as to render an appropriate $\phi$. If the resulting $\phi$ is smaller than the phase transformation angle $\phi_{PT}$, $\phi_{PT}$ is set equal to $\phi$.

Also remember that improper modulus reduction curves can result in strain softening response (negative tangent shear modulus), which is not allowed in the current model formulation. Finally, note that the backbone curve varies with confinement, although the variations are small within commonly interested confinement ranges. Backbone curves at different confinements can be obtained using the OpenSees element recorder facility (Mazzoni et al. 2006).

The dilatancy/liquefaction parameters include:

**Phase Transformation (PT) Angle** The transformation angle (degrees) of the cohesionless soil.

**Contraction Parameter $c_1$** A non-negative constant defining the rate of shear-induced volume decrease (contraction) or pore pressure buildup. A larger value corresponds to faster contraction rate (Table 3.2).

The contraction rule is defined by:

$$ P^n = \frac{1 - (\eta / \eta_{PT})^2}{1 + (\eta / \eta_{PT})^2} c_1 $$  \hspace{1cm} (3.10)

where $\eta$ is the stress ratio and $\eta_{PT}$ is the stress ratio along the PT surface (Yang et al. 2003).

**Dilation Parameters $d_1$ & $d_2$** Non-negative constants defining the rate of shear-induced volume increase (dilation). Larger values correspond to stronger dilation rate (Table 3.2).

The dilation rule is defined by:

$$ P^n = \frac{1 - (\eta / \eta_{PT})^2}{1 + (\eta / \eta_{PT})^2} d_1 \exp(d_2 \gamma_d) $$  \hspace{1cm} (3.11)

where $\gamma_d$ is the octahedral shear strain accumulated during a dilation phase (Yang et al. 2003).
**Liquefaction Parameters** \( l_1, l_2 \) and \( l_3 \)  
Parameters (Table 3.2) controlling the mechanism of liquefaction-induced perfectly plastic shear strain accumulation, i.e., cyclic mobility. **Set** \( l_1 = 0 \) **to deactivate this mechanism altogether.**

(Post-liquefaction) yield domain radius:

\[
\gamma_y = l_2 \cos^3 \left( \frac{\pi p'}{2 l_1} \right) \quad (3.12)
\]

\( l_1 \) defines the effective confining pressure (e.g., 10 kPa) below which the mechanism is in effect (\( l_1 \) is actually \( p'_c \) in Figure 3.21). Smaller values should be assigned to denser sands. \( l_2 \) defines the maximum amount of perfectly plastic shear strain developed at zero effective confinement during each loading phase (\( l_2 \) is actually \( \gamma_{s,\max} \) in Figure 3.21). Smaller values should be assigned to denser sands.

Maximum extent of biased-loading yield domain (\( \gamma_y \) is actually \( \gamma_r \) in Figure 3.21)

\[
\gamma_{ry} = l_3 \gamma_y \quad (3.13)
\]

\( l_3 \) defines the maximum amount of biased perfectly plastic shear strain \( \gamma_b \) accumulated at each loading phase under biased shear loading conditions, as \( \gamma_b = l_2 \times l_3 \) (\( \gamma_{ry} \) is actually \( \gamma_r \), and \( l_3 \) is \( R \) in Figure 3.22). Typically, \( l_3 \) takes a value between 0.0 and 3.0. Smaller values should be assigned to denser sands.

**Table 3.2: Suggested values for contraction and dilation parameters**

<table>
<thead>
<tr>
<th></th>
<th>Loose Sand (15%-35%)</th>
<th>Medium Sand (35%-65%)</th>
<th>Medium-dense Sand (65%-85%)</th>
<th>Dense Sand (85%-100%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( c_1 )</td>
<td>0.21</td>
<td>0.07</td>
<td>0.05</td>
<td>0.03</td>
</tr>
<tr>
<td>( d_1 )</td>
<td>0.0</td>
<td>0.4</td>
<td>0.6</td>
<td>0.8</td>
</tr>
<tr>
<td>( d_2 )</td>
<td>0.0</td>
<td>2.0</td>
<td>3.0</td>
<td>5.0</td>
</tr>
<tr>
<td>( l_1 ) (kPa)</td>
<td>10.0</td>
<td>10.0</td>
<td>5.0</td>
<td>0.0</td>
</tr>
<tr>
<td>( l_2 )</td>
<td>0.02</td>
<td>0.01</td>
<td>0.003</td>
<td>0.0</td>
</tr>
<tr>
<td>( l_3 )</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>
Figure 3.21: Initial yield domain at low levels of effective confinement (Yang et al. 2003).
3.2.2.3.2 User Defined Sand2 (U-Sand2)

The second type of user-defined sandy soil (U-Sand2) can be defined by specifying the following parameters (Figure 3.23):
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass Density</td>
<td>1310986202 [pcf]</td>
</tr>
<tr>
<td>Reference Mean Confinement</td>
<td>0.01160304 [ksi]</td>
</tr>
<tr>
<td>Reference Shear Wave Velocity</td>
<td>984.252 [ft/s]</td>
</tr>
<tr>
<td>Confinement Dependence Coeff. (0.1-1.0)</td>
<td>0.5</td>
</tr>
<tr>
<td>Initial Lateral/Vertical Confinement Ratio (0.1-0.9)</td>
<td>0.5</td>
</tr>
<tr>
<td>Cohesion</td>
<td>0.01</td>
</tr>
<tr>
<td>Friction Angle (5-65 degrees)</td>
<td>40</td>
</tr>
<tr>
<td>Peak Shear Strain (0.001-20%)</td>
<td>3.0</td>
</tr>
<tr>
<td>Number of Yield Surfaces (0-30)</td>
<td>20</td>
</tr>
</tbody>
</table>

Figure 3.23: U-Sand2 in OpenSeesPL.

Note: All parameters shown in Figure 3.23 are defined at the reference mean confinement $p_r$.

**Mass Density** The mass density of the cohesionless soil ($\rho$). The suggested range of values are between 1 and 3 ton/m$^3$.

**Reference Shear Wave Velocity** The reference shear wave velocity ($V_{sr}$). The suggested range is between 10 and 6000 m/s. The reference shear modulus $G = \rho V_{sr}^2$.

**Reference Mean Confinement** The reference mean confinement. This is the confinement level at which shear wave velocity and peak shear strain are defined. The suggested range is between 10 kPa or larger.

**Confinement Dependence Coeff.** The confinement dependence coefficient. The suggested range is between 0.1 and 10.

**Initial Lateral/Vertical Confinement Ratio** The initial lateral/vertical stress ratio (also known as coefficient of lateral earth pressure at rest $K_0$). $K_0$ is related to Poisson’s ratio by the following relation $K_0 = \frac{\nu}{1 - \nu}$. The suggested range for $K_0$ is between 0.1 and 0.9.

**Cohesion** The suggested range is between 0 and 5000000 kPa. See Section 3.2.2.3.1 for more information.
**Friction Angle**  The suggested range is between 5 and 65 degrees. See Section 3.2.2.3.1 for more information.

**Peak Shear Strain**  The suggested range is between 0.001% and 20%. See Section 3.2.2.3.1 for more information.

**Number of Yield Surfaces NYS**  The suggested range is between 0 and 30. In particular, NYS = 0 dictates an elastic shear response (Cohesion, Friction Angle and Peak Shear Strain are ignored, see Figure 3.18), NYS = 1 indicates an elastic-perfectly plastic shear response (Peak Shear Strain is ignored, see Figure 3.18).

### 3.2.2.3.3 User Defined Clay1 (U-Clay1)

Non-liquefiable clay with shear response properties independent of confinement variation can be defined as shown in Figure 3.16 and Figure 3.24.

**Cohesion**  The apparent cohesion at zero effective confinement.

The nonlinear shear stress-strain backbone curve can be defined by specifying a $G/G_{max}$ curve (Figure 3.24). The user-defined backbone curve is activated if the number of points is greater than zero. In this case, if the user specifies $\phi=0$, cohesion $c$ will be ignored. Instead, $c$ is defined by $c=\sqrt{3}\sigma_m/2$, where $\sigma_m$ is the product of the last modulus and strain pair in the modulus reduction curve. Therefore, it is important to adjust the backbone curve so as to render an appropriate $c$.

If the user specifies $\phi>0$, this $\phi$ will be ignored. Instead, $\phi$ is defined as follows:

$$\sin \phi = \frac{3(\sqrt{3} \sigma_m - 2c)}{6 + (\sqrt{3} \sigma_m - 2c)} / p_r'$$

(3.14)

If the resulting $\phi<0$, we set $\phi=0$ and $c=\sqrt{3}\sigma_m/2$.

Also remember that improper modulus reduction curves can result in strain softening response (negative tangent shear modulus), which is not allowed in the current model formulation. Finally, note that the backbone curve varies with confinement, although the variation is small within commonly interested confinement ranges. Backbone curves at different confinements can be obtained using the OpenSees element recorder facility (Mazzoni et al. 2006).

For information about other parameters, see Section 3.2.2.3.1.
3.2.2.3.4 User Defined Clay2 (U-Clay2)

The second type of user-defined clay (U-Clay2) can be defined as shown in Figure 3.25. See Section 3.2.2.3.2 for information about parameters defining U-Clay2.

Figure 3.24: U-Clay1 in OpenSeesPL.
3.2.3 **Pile-Soil Interfacing Layer Properties**

The material for the pile-soil interfacing layer (Figure 3.2) can be selected from an available menu of cohesionless and cohesive soil materials including the elastic isotropic material. In addition, user-defined cohesionless and cohesive soil materials (U-Sand1, U-Sand2, U-Clay1 and U-Clay2) are also available to choose.

If an elastic isotropic material is selected, the user is requested to specify, Young’s Modulus, Poisson’s Ratio, Mass Density Permeability of the material used for the pile-soil interfacing layer.

3.2.4 **Material Properties for Pile Zone**

The pile zone refers to the pile domain under the ground surface. The material for the pile zone (Figure 3.2) can be selected from an available menu of cohesionless and cohesive soil materials including the elastic isotropic material. In addition, user-defined cohesionless and cohesive soil materials (U-Sand1, U-Sand2, U-Clay1 and U-Clay2) are also available to choose.

If an elastic isotropic material is selected, the user is requested to specify, Young’s Modulus, Poisson’s Ratio, Mass Density Permeability of the material used for the pile area.
3.2.5 Additional Viscous Damping

In OpenSeesPL, additional viscous Rayleigh-type damping is available of the form:

\[ C = A_m M + A_k K \]

where \( M \) is the mass matrix, \( C \) is the viscous damping matrix, \( K \) is the initial stiffness matrix. \( A_m \) and \( A_k \) are two user-specified constants.

The damping ratio curve \( \xi (f) \) is calculated based on the following equation:

\[ \xi = \frac{A_m}{4\pi f} + A_k \pi f \]

(3.15)

where \( f \) is frequency.
Figure 3.26: Rayleigh damping selection.

(1) Specification of $A_m$ and $A_k$ By Defining Damping Ratios

The user can define damping coefficients (Figure 3.26) by specifying two frequencies, $f_1$ and $f_2$ (must be between 0.1 and 50 Hz), and two damping ratios, $\xi_1$ and $\xi_2$ (suggested values are between 0.2% and 20%).

The Rayleigh damping parameters $A_m$ and $A_k$ are obtained by solving the following equations simultaneously:

$$\xi_1 = \frac{A_m}{4\pi f_1} + A_k \pi f_1$$  \hspace{1cm} (3.16a)
(2) Direct Specification of $A_m$ and $A_k$:

The user can also directly define Rayleigh damping coefficients $A_m$ and $A_k$ (Figure 3.26).
Chapter 4 Analysis Options

This chapter describes analysis options available in OpenSeesPL. This chapter includes:

- Pushover and base shaking analysis options
- Step-by-step time integration
- How to add an user-defined input motion
- Model inclination

4.1 Problem Type

4.1.1 Pushover Analysis

Two types of pushover analyses are available (Figure 4.1): Monotonic Pushover and Cyclic Pushover (Sine Wave).

Frequency  The frequency of the sine wave if Cyclic Pushover is chosen.

Total Time  The total time for the pushover. 100 loading steps will be used for one second.

4.1.1.1 Force-Based Method

The force-based method is used if the Force-Based Method radio button is chosen.

Longitudinal (X) Force  The force applied in the longitudinal direction.

Transverse (Y) Force  The force applied in the transverse direction.

Vertical (Z) Force  The force applied in the vertical direction.

Moment of X  The applied bending moment about the longitudinal direction ($M_x$).

Moment of Y  The applied bending moment about the longitudinal direction ($M_y$).

Moment of Z  The applied bending moment about the longitudinal direction ($M_z$).
4.1.1.2 Displacement-Based Method

The displacement-based method is used if the Displacement-Based Method radio button is chosen.

Longitudinal Displacement The displacement applied in the longitudinal direction.

Transverse Displacement The displacement applied in the transverse direction.

Vertical Displacement The displacement applied in the vertical direction.

Rotation around X The applied rotation around the longitudinal axis (X).

Rotation around Y The applied rotation around the transverse axis (Y).

Rotation around Z The applied rotation around the vertical axis (Z).

Figure 4.1: Pushover load pattern.
4.1.2 Base Shaking

4.1.2.1 Step-by-Step Time Integration

OpenSeesPL employs the Newmark time integration procedure with two user defined coefficients $\beta$ and $\gamma$ (Newmark 1959, Chopra 2004). Standard approaches may be adopted by appropriate specification of these constants (Figure 4.2). Default values in OpenSeesPL are $\gamma = 0.55$, and $\beta = ((\gamma + \frac{1}{2})^2) / 4$.

Computations at any time step are executed to a convergence tolerance of $10^{-4}$ (Euclidean Norm of acceleration vector), normalized by the first iteration Error Norm (predictor multi-corrector approach).

Note: An additional fluid-phase (Chan 1988) time integration parameter $\theta$ is set to 0.6 in the data file.

<table>
<thead>
<tr>
<th>$\beta$</th>
<th>$\gamma$</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$1/6$</td>
<td>$1/2$</td>
<td>Linear acceleration (conditionally stable scheme)</td>
</tr>
<tr>
<td>$1/4$</td>
<td>$1/2$</td>
<td>Average acceleration or trapezoidal rule (unconditionally stable scheme in linear analyses);</td>
</tr>
<tr>
<td>$1/12$</td>
<td>$1/2$</td>
<td>Fox-Goodwin (fourth order accurate);</td>
</tr>
</tbody>
</table>

Figure 4.2: Newmark Time Integration

4.1.2.2 Input Motion
One, two and three directions of excitations are available: longitudinal, transverse and vertical directions (Figure 2.1 and Figure 4.3).

The bedrock is assumed to be rigid, the input motion is total motion; Base seismic excitation can be defined by either of the following two methods:

i) Via a built-in input motion library. This library includes near-fault soil surface motions as well as long-duration rock outcrop motions recorded during past strong earthquakes worldwide.

ii) ‘U-Shake’, a user-defined input motion (Figure 4.4). The input motion file to be defined should consist of two columns, Time (seconds) and Acceleration (g), delimited by SPACE(S).

Below is an example of a user-defined input motion file:

```
0.00  0.000
0.02  0.005
...
19.98 0.004
20.00 0.000
```

Note that the user-defined input motion file must be placed in the subfolder “motions/”. (This subfolder also contains all provided built-in input motion files).

The amplitude of the input motion can be scaled by a factor ranging from 0.01 to 1.0. In addition, if ‘0.2g sinusoidal motion’ is chosen, the user must specify excitation frequency and number of cycles (Figure 2.1).

**Scale Factor** The amplitude of the input motion is multiplied by the Scale Factor. The Scale Factor may be positive or negative.

**Frequency** The Frequency (in Hz) has to be specified if harmonic “sinusoidal motion” is chosen.

**Number of Cycles** The Number of Cycles has to be specified if “sinusoidal motion” is chosen.
Figure 4.3: Definition of 3D base excitation.

Figure 4.4: User-defined input motion (U-Shake).
4.2 Analysis Type

The analysis type can be linear or nonlinear analysis (Figure 4.5). In a linear analysis, linear soil properties will be used while nonlinear soil properties will be used for a nonlinear case. 20-node brick element is preferred in liquefaction analyses.

![Analysis Options](image)

Figure 4.5: Analysis options.

4.3 Model Inclination

Inclined models can be defined by the following parameters (Figure 2.1):

**Ground Surface Inclination Angle along Longitudinal Direction** The inclination angle of the ground surface along the longitudinal direction (in degrees) (zero degree represents level ground surface).

**Whole Model Inclination Angle along Longitudinal Direction** The inclination angle in degrees of the whole model (zero degree represents level ground). For mildly-inclined infinite-slopes, suggested values are from 0 to 10 degrees.
Chapter 5  Running the Analysis

This chapter describes the output interfaces in OpenSeesPL. This chapter includes:

- How to manipulate the graphical output
- Displaying of the response time histories
- Viewing of the deformed mesh
- Displaying of the pile responses (time histories and profiles)

5.1 Mesh Generation

Once all model definition parameters are specified, click the Menu ‘Meshing’ and then choose ‘Generate Mesh’ to generate the mesh.

If mesh generation is failed due to any error encountered, please go to the model definition section and fix the problems accordingly.

To view the filled mesh, click the Menu ‘Meshing’ and then choose ‘Filled Mesh View’.

To view the unfilled mesh, click the Menu ‘Meshing’ and then choose ‘Unfilled Mesh View’.

5.2 Running the Analysis

To run the analysis, click “Save Model & Run Analysis” in Menu “Analyze”.

Upon the user requests to run the analysis, OpenSeesPL will check all the entries defined by the user to make sure the model is valid. Thereafter, a small window (Figure 5.1) will show the progress of the analysis.

By default, graphical output windows will be opened upon completion of the analysis.
5.3 Output

5.3.1 Tips on Manipulating Graphs

Response time histories and profiles are displayed by X-Y plot using Java Applet. Therefore, make sure to enable Java Applet in your web browser (Internet Explorer). You may also view the digital data by clicking on the link under the X-Y plot. If occasionally the graph becomes crooked, you can click on the Fill button to refresh it.

To zoom in on any region of the plot, select a box with the mouse pointer (Figure 5.2). Start at the upper left corner of the region you wish to view in more detail and drag downwards and to the right. To bring the graph to the original scale, click on the "fill" button at the upper right corner.

(a) Select a box using the mouse pointer…

(b) Then release the mouse.

Figure 5.2: Zoom in.
(a) Select a box using the mouse pointer…

(b) Then release the mouse.

Figure 5.3: Zoom out.

To zoom out, drag the mouse pointer upwards (Figure 5.3). When zooming out, a reference box is drawn that will represent the current view, and dragging will cause a box to be displayed that represents the new view. Again, click on the "fill" button at the upper right corner to bring the graph to the original scale.

5.3.2 Response Time Histories

To view the response time histories, click “Response Histories” in Menu “Display”.

The figures show the response time histories of the soil domain from the ground surface till the bottom, at a number of locations which are along the longitudinal direction crossing the pile center.

Seven types of response time histories are available:
- Longitudinal Acceleration Time History
- Longitudinal Displacement (Relative to the Base) Time History
- Transverse Acceleration Time History
- Transverse Displacement (Relative to the Base) Time History
- Vertical Acceleration Time History
- Vertical Displacement Time History
- Excess Pore Pressure Time History
- Shear Stress versus. Shear Strain
- Shear Stress versus. Effective Confinement

To zoom-in or zoom-out, use mouse to select a window. Click "fill" to get back to the original figure.
5.3.3 Deformed Mesh

To view the deformed mesh, click "Deformed Mesh" in Menu "Display".
Figure 5.5: Graph types available in the deformed mesh window.

By default, the deformed mesh is for the dynamic analysis (if ‘Due to Seismic Excitation’ is chosen) or the pushover analysis (if ‘Due to Pushover’ is chosen). However, the deformed mesh due to gravity is also available (‘Due to Gravity’ is chosen).

Types of results in the deformed mesh include (Figure 5.5):

- Vectors
- Displacement Contour Fill
- Longitudinal Displacement Contour Fill (X-disp contour)
- Transverse Displacement Contour Fill (Y-disp contour)
- Vertical Displacement Contour Fill (Z-disp contour)
- Pore Pressure Contour Fill
- Excess Pore Pressure (EPP) Contour Fill
- EPP Ratio Contour Fill
- Vertical Stress Contour Fill
- Shear Stress Contour Fill
- Stress Ratio Contour Fill
- Effective Confinement Contour Fill
The deformed mesh can be viewed in 3D or 2D (can be selected from a list of 2D cut planes, see Figure 5.6).

![Deformed Mesh](image)

Figure 5.6: 2D plane (Y = 0) view of the longitudinal displacement contour in the deformed mesh window.

To view the animation of any given type, click the “Play Animation” button. The text of the button will change to “Stop Animation” when the animation is being played. To stop the animation, click the “Stop Animation” button.

The Scale Factor can be changed to improve the viewing effects. The time between playing two frames can be defined by specifying the Animation Playing Delay (in millisecond).

Note that the animation will not be played if the current time step is in the last step and “Endless Playing” is unchecked.

At any time, the deformed mesh can be rotated by dragging the mouse, moved in 4 directions by pressing keys of LEFT ARROW, RIGHT ARROW, UP ARROW or DOWN ARROW respectively. The view can be zoomed in (by pressing key ‘F9’), out (by pressing key ‘F10’) or frame (by pressing key ‘F11’).
5.3.4 Pile Response Profiles

To view the pile response, click “Pile Response Profiles” in Menu “Display”. The figures show the response time histories and response profiles of the pile. Seven types of response are available (Figure 5.7):

- Displacement
- Acceleration
- Rotation
- Moment
- Shear
- Pressure

To zoom-in or zoom-out, use mouse to select a window. Click "fill" to get back to the original figure.

Figure 5.7: Bending moment profile displayed in the pile response window.
5.3.5 Pile Response Relationships

To view the pile response, click “Pile Response Relationships” in Menu “Display”. The figures show the response relationships of the pile. Seven types of response are available (Figure 5.8):

- Load-displacement
- Moment-curvature

To zoom-in or zoom-out, use mouse to select a window. Click "fill" to get back to the original figure.

Figure 5.8: Load-displacement curve displayed in the pile response window.
Chapter 6  Examples

This chapter presents a few examples using OpenSeesPL. This examples includes:

- Example 1: Linear Pile in Rigid Ground
- Example 2: Linear Pile in Nonlinear Loose Sand (Dry Case)
- Example 3: Linear Pile in Nonlinear Medium Sand (Dry Case)
- Example 4: Nonlinear Pile in Linear Soil
- Example 5: Saturated Medium Sand
- Example 6: Stone Columns
- Example 7: Pile in Sloping Ground

6.1 Example 1: Linear Pile in Rigid Ground

File name: \examples\cantilevelbeam.cpl

Pile Type: Circular

Pile Diameter: 1 m

Pile Height above Surface: 6 m

Young’s Modulus: 20 GPa

Moment of Inertia: 0.049087 m4

Load at pile head: 1 kN

Result:

The deflection at pile head is 7.33 E-05 m, which matches the theoretical value by using \( PL^3/3EI \), where \( P \) is the applied load, \( L \) is the height of the pile above the top of the soil, \( E \) is Young’s Modulus, and \( I \) is the Moment of Inertia.
6.2 Example 2: Linear Pile in Loose Sand

File name: \examples\LinPileNonlinLooseSand.cpl

Pile Geometry & Properties: Same as Example 1
Soil: Cohesionless loose, sand permeability, 10 m deep.
Analysis options: Nonlinear Analysis (Dry Case)
Load at pile head: 1 kN

The finite element mesh is shown in Figure 6.1.

![Finite element mesh (Example 2).](image)

Figure 6.1: Finite element mesh (Example 2).

Result:
The deflection at pile head is 1.8951 E-04 m. The displacement profile of the pile is shown in Figure 6.2.

![Displacement Profile](image)

Figure 6.2: Displacement profile (Example 2).

### 6.3 Example 3: Linear Pile in Medium Sand

**File name:** `\examples\LinPileNonlinMediumSand.cpl`

All geometry configurations and material properties are the same as Example 2 except the soil is ‘Cohesionless medium, sand permeability’.

**Result:**
The deflection at pile head is 1.7707E-04 m. The beading moment profile of the pile is shown in Figure 6.3.

![Bending Moment Profile](image.png)

Figure 6.3: Bending moment profile (Example 3).

### 6.4 Example 4: Nonlinear Pile in Linear Soil

**File name:** \examples\NonlinPileLinearSoil.cpl

**Pile:**

All geometry configurations for the pile are the same as Example 1.

Aggregator Section was used for the nonlinear pile:

- **Flexural Rigidity** $EI$: 158600 kN-m$^2$
- **Yield Moment**: 1200 kN-m
Shear Rigidity GA: 3378000 kN

**Soil:**

Material: U-Clay2

Mass Density: 1.8 ton/m³

Shear Wave Velocity: 6000 m/s

Initial lateral/vertical confinement ratio: 0.5

**Pushover:**

Monotonic, displacement-based method

**Result:**

The maximum shear force in the pile is 203.85 kN to obtain the maximum moment of 1200 kN-m and deflection of 0.5 m. The section yields after a displacement of 0.0925 m at a time of 0.37 seconds (Figure 6.4).

![Figure 6.4: Bending moment time histories (Example 4).](image)

6.5 **Example 5: Saturated Medium Sand**

**File name:** `\examples\MediumSand.cpl`
Soil:

Material: Medium Sand

Base Shaking:

User-defined motion (file: \motions\Ushake1.acc)

Figure 6.5: Excess pore pressure time histories (Example 5).

6.6 Example 6: Stone Columns

File name: \examples\StoneColumns.cpl

Soil:

Material: Dense Gravel drain in Medium Sand

Base Shaking:

User-defined motion (file: \motions\Ushake1.acc)
6.7 Example 7: Pile in Sloping Ground

File name: \examples\PileInSlopingGround.cpl

Pile Type: Circular

Pile Diameter: 1 m

Pile Height above Surface: 0 m

Pile Height below Surface: 10 m

Young’s Modulus: 20 GPa

Moment of Inertia: 0.049087 m4

Soil:

Material: Medium Sand
Base Shaking:

User-defined motion (file: \motions\Ushake1.acc)

Figure 6.7: Bending moment profile of pile (Example 7).
Appendix A  Benchmark Linear Finite Element Analysis of Laterally Loaded Single Pile Using OpenSees & Comparison with Analytical Solution

Introduction

In this study:

I) The response of a laterally loaded pile obtained using the OpenSeesPL interface is compared with the analytical elastic solution proposed by Abedzadeh and Pak (2004). Detailed information about the analytical elastic solution is provided in Appendix A-I (please see this end of Appendix A).

II) Based on the linear analysis presented below, nonlinear soil response is addressed in Appendix A-II (please see this end of Appendix A).

Laterally Loaded Pile

Pile Data

The pile employed in the OpenSees simulation is circular with a diameter of 16" (radius \(a = 8"\)) while the one for the analytical elastic solution is a cylindrical pipe pile of the same radius and a wall thickness \(h = 0.1a\). Both cases have the same pile length \(l = 33.3\) ft \((l/a = 50)\). The cross-sectional moment of inertia of the pipe pile \(I = \pi a^3h = 1286.8\) in\(^4\), which will be used for the circular pile in the OpenSees simulation.

In summary, the geometric and elastic material properties of the pile are listed below:

Radius \(a = 8"\)
Pile length \(l = 33.3\) ft
Young’s Modulus of Pile \(E_p = 29000\) ksi
Moment of Inertia of Pile \(I = 1286.8\) in\(^4\)

Soil Domain

The pile is assumed to be fully embedded in a homogeneous, isotropic, linearly elastic half-space. The elastic properties of the soil are assumed constant along the depth (in order to compare with the analytical elastic solution) and are listed below:

Shear Modulus of Soil \(G_s = 7.98\) ksi
Bulk Modulus of Soil \(B = 13.288\) ksi (i.e., Poisson’s ratio \(\nu_s = 0.25\))
Submerged Unit Weight \(\gamma'\) = 62.8 pcf

The ratio of Young’s Modulus of Pile \((E_p)\) to the Shear Modulus of Soil \((G_s)\):
\( \frac{E_p}{G_s} = 3634 \) (which will be used later to obtain the analytical elastic solution by interpolation).

**Lateral Load**

The pile head (free head condition), which is located at the ground surface, is subjected to a horizontal load \( (H) \) of 31.5 kips.

**Finite Element Simulation**

In view of symmetry, a half-mesh is studied as shown in Figure A.1. For comparison, both 8-node and 20-node elements are used (2,900 8-node brick elements, 20 beam-column elements and 189 rigid beam-column elements in total) in the OpenSeesPL simulation. Length of the mesh in the longitudinal direction is 520 ft, with 260 ft transversally (in this half-mesh configuration, resulting in a 520 ft x 520 ssoil domain in plan view). Layer thickness is 66 ft (the bottom of the soil domain is 32.7 ft below the pile tip, so as to mimic the analytical half-space solution).

The floating pile is modeled by beam-column elements, and rigid beam-column elements are used to model the pile size (diameter).

The following boundary conditions are enforced:

I) The bottom of the domain is fixed in the longitudinal \( (x) \), transverse \( (y) \), and vertical \( (z) \) directions.

II) Left, right and back planes of the mesh are fixed in \( x \) and \( y \) directions (the lateral directions) and free in \( z \) direction.

III) Plane of symmetry is fixed in \( y \) direction and free in \( z \) and \( x \) direction (to model the full-mesh 3D solution).

The lateral load is applied at the pile head (ground level) in \( x \) (longitudinal) direction.

The above simulations were performed using OpenSeesPL (Lu et al., 2006).

**Simulation Results and Comparison with Elastic Solution**

Deflection and bending moment response profiles obtained from OpenSees are shown in Figure A.2 and Figure A.3, along with the analytical elastic solution by Abedzadeh and Pak (2004) for comparison (note that the elastic solution was obtained by performing a linear interpolation of the normalized deflections and moments shown in Figure A.4 and Figure A.5 for \( \frac{E_p}{G_s} = 3634 \)).

The pile head deflection and the maximum bending moment from OpenSees and the elastic solution are also listed in Table A.1. In general, the numerical results match well with the analytical elastic solution. The pile head deflection from the 20-node element
mesh (0.043") is almost identical to the elastic solution (0.042").

For nonlinear run, please see Appendix A-II.

Figure A.1: Finite element mesh employed in this study.
Table A.1: Comparison of OpenSees results and the analytical elastic solution.

<table>
<thead>
<tr>
<th></th>
<th>OpenSees Results</th>
<th>Elastic solution by Abedzadeh and Pak (2004)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>8-node element</td>
<td>20-node element</td>
</tr>
<tr>
<td>Pile head deflection (in)</td>
<td>0.039</td>
<td>0.043</td>
</tr>
<tr>
<td>Maximum moment $M_{\text{max}}$ (kip-ft)</td>
<td>30</td>
<td>31</td>
</tr>
<tr>
<td>Depth where $M_{\text{max}}$ occurs (ft)</td>
<td>2.87</td>
<td>2.87</td>
</tr>
</tbody>
</table>

Figure A.2: Comparison of pile deflection profiles ($v_s=.25, \ l/a=50$).
Figure A.3: Comparison of pile bending moment profiles ($\nu_s=.25$, $l/a=50$).
Appendix A-I: Elastic Solution of the Response of a Laterally Loaded Pile in a Semi-Infinite Soil Medium with Constant Modulus along Depth


Consider a flexible cylindrical pipe pile of radius $a$, length $l$, a wall thickness $h \ll a$ (note that the moment of inertia $I = \pi a^3 h$). The pile is assumed to be fully embedded in a homogenous, isotropic, linearly elastic half-space with a shear modulus $G_s$ and a Poisson’s ratio $\nu_s = 0.25$.

Using Eqs. (78)-(83) and Figure 9 of the above reference, the pile response ($h/a = 0.1$, $l/a = 50$) under an applied pure pile-head horizontal load is shown in Figure A.4 and Figure A.5, where,

$E_p$ – Young’s Modulus of Pile
$G_s$ – Shear Modulus of Soil
$w$ – Pile deflection (in)
$H$ – Horizontal load (kip)
$z$ – Pile depth (ft)
Figure A.4: Sample pile deflection ($h/a = 1, \ell/a = 50$) under an applied pure pile-head horizontal load (Abedzadeh and Pak, 2004).
Figure A.5: Sample pile bending moment ($h/a=0.1$, $l/a=50$) under an applied pure pile-head horizontal load (Abedzadeh and Pak, 2004).
Appendix A-II: Nonlinear Response of the Single Pile Model

In the nonlinear run, the same material properties of the linear run are employed except the soil now assumed to be a clay material with a maximum shear strength or cohesion = 5.1 psi, in the range of a Medium Clay. This maximum shear strength is achieved at a specified strain $\gamma_{\text{max}} = 10\%$.

The lateral load ($H$) is applied at an increment of 0.7875 kips and the final load is 94.5 kips (= $3 \times 31.5$ kips). The 8-node brick element mesh is employed in this nonlinear analysis (Figure A.1).

Simulation Results

Figure A.6 shows the load-deflection curve for the nonlinear run, along with the linear result (for the 8-node brick element mesh; the final lateral load is also extended to 94.5 kips) as described in the previous sections for comparison. It is seen from Figure A.6 that nearly linear behavior is exhibited in the nonlinear run for only low levels of applied lateral load (less than 10 kips).

![Figure A.6: Comparison of the load-deflection curves for the linear and nonlinear runs.](image-url)
The pile deflection profiles for both linear and nonlinear cases are displayed in Figure 7. For comparison, the linear and nonlinear responses at the lateral load of 31.5 kips, 63 kips (= 2 x 31.5), and 94.5 kips (= 3 x 31.5) are shown (Figure A.7). The bending moment profiles for the 3 load levels are shown in Figure A.8a-c.

Figure A.7: Comparison of the pile deflection profiles for the linear and nonlinear runs.
The pile head deflections and the maximum bending moments for both linear and nonlinear analyses are listed in Table A.2. The stress ratio contour fill of the nonlinear run is displayed in Figure A.9.

Figure A.7: (continued).

Figure A.8: Comparison of the pile bending moment profiles for the linear and nonlinear runs.
b) \( H = 63 \text{ kips} \)

c) \( H = 94.5 \text{ kips} \)

Figure A.8: (continued).
Table A.2: OpenSees simulation results for the linear and nonlinear runs.

<table>
<thead>
<tr>
<th></th>
<th>$H = 31.5$ kips</th>
<th>$H = 63$ kips</th>
<th>$H = 94.5$ kips</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Linear</td>
<td>Nonlinear</td>
<td>Linear</td>
</tr>
<tr>
<td>Pile head deflection (in)</td>
<td>0.039</td>
<td>0.07</td>
<td>0.078</td>
</tr>
<tr>
<td>Maximum moment $M_{\text{max}}$ (kip-ft)</td>
<td>30</td>
<td>48.2</td>
<td>60</td>
</tr>
<tr>
<td>Depth where $M_{\text{max}}$ occurs (ft)</td>
<td>2.9</td>
<td>3.8</td>
<td>2.9</td>
</tr>
</tbody>
</table>

Figure A.9: Stress ratio contour fill of the nonlinear run at different load levels (red color shows yielded soil elements).
Appendix B  Finite Element Analysis of Arkansas Test Series Pile #2 Using Opensees (with LPILE Comparison)

Introduction

In this study, we conduct a finite element simulation of Pile No. 2 of the Arkansas test series (Alizadeh and Davisson 1970) using the OpenSeesPL interface. This pipe pile is subjected to lateral loads. Comparison with LPILE is also included in Appendix B-I (please see the end of Appendix B).

Laterally Loaded Pile

Pile Data

The pile employed in the OpenSees simulation is circular with a diameter of 16" (radius \( a = 8" \)) while the one for the experimental test is a cylindrical pipe pile of the same radius and a wall thickness \( h = 0.312" \). The cross-sectional moment of inertia of the pipe pile \( I = 838.2 \text{ in}^4 \) (Bowles 1988, pages 777-778), which will be used for the circular pile in the OpenSees simulation.

The geometric and elastic material properties of the pile are listed below (Bowles 1988):

- Diameter = 16" or Radius \( a = 8" \)
- Pile length \( l = 52.9 \text{ ft} \)
- Young’s Modulus of Pile \( E_p = 29000 \text{ ksi} \)
- Moment of Inertia of Pile \( I = 838.2 \text{ in}^4 \)

Soil Domain

In this section, the pile is embedded in a uniform soil layer (pile top is 0.1’ above the ground line). Linear and nonlinear soil responses are investigated. The Medium density (relative) granular soil type (Lu et al. 2006) is selected in this initial attempt. The material properties of the soil are listed below:

- At the reference confinement of 80 kPa (or 11.6 psi), the Shear Modulus of Soil \( G_s = 10.88 \text{ ksi} \) and the Bulk Modulus of Soil \( B = 29 \text{ ksi} \) (i.e., Poisson’s ratio \( \nu_s = 0.33 \)), see Lu et al. 2006.
- Submerged Unit Weight \( \gamma' = 62.8 \text{ pcf} \) (Bowles 1988)
- For nonlinear analysis, the Friction Angle \( \phi = 32° \) (Bowles 1988) and the peak shear stress occurs at a shear strain \( \gamma_{max} = 10% \) (at the 11.6 psi confinement)
Lateral Load

The pile head (with a free head condition), which is 0.1' above the ground surface, is subjected to horizontal loads \((H)\) of 21 kips, 31.5 kips and 43 kips (Bowles 1988).

Finite Element Simulation

In view of symmetry, a half-mesh (2,900 8-node brick elements, 23 beam-column elements and 207 rigid beam-column elements in total) is studied as shown in Figure B.1. Length of the mesh in the longitudinal direction is 520 ft, with 260 ft transversally (in this half-mesh configuration, resulting in a 520 ft x 520 soil domain in plan view). Layer thickness is 80 ft (the bottom of the soil domain is 27.2 ft below the pile tip, so as to mimic the analytical half-space solution).

The floating pile is modeled by beam-column elements, and rigid beam-column elements are used to model the pile size (diameter).

The following boundary conditions are enforced:

I) The bottom of the domain is fixed in the longitudinal \((x)\), transverse \((y)\), and vertical \((z)\) directions.

II) Left, right and back planes of the mesh are fixed in \(x\) and \(y\) directions (the lateral directions) and free in \(z\) direction.

III) Plane of symmetry is fixed in \(y\) direction and free in \(z\) and \(x\) direction (to model the full-mesh 3D solution).

The lateral load is applied at the pile head (ground level) in \(x\) (longitudinal) direction.

The above simulations were performed using OpenSeesPL (Lu et al. 2006).

Simulation Results

The pile deflections at the ground line and the maximum bending moments for the linear and nonlinear analyses are listed in Table B.1, along with the experimental measurements for comparison (Alizadeh and Davisson 1970; Bowles 1988).

Figure B.2 shows the load-deflection curve for the linear and nonlinear runs. Comparison of the pile deflection profiles for the linear and nonlinear analyses are displayed in Figure B.3a-c. The bending moment profiles for the 3 load levels are shown in Figure B.4a-c, along with the observed for comparison (Alizadeh and Davisson 1970). The stress ratio contour fill of the nonlinear run is displayed in Figure B.5.

Comparison with LPILE is included in Appendix B-I.
Figure B.1: Finite element mesh employed in this study.
Table B.1: OpenSees Simulation Results and Experimental Measurements.

<table>
<thead>
<tr>
<th>Analysis type</th>
<th>Pile deflection at ground line (in)</th>
<th>Max. bending moment $M_{\text{max}}$ (kip-ft)</th>
<th>$M_{\text{max}}$ depth (ft)</th>
<th>Profile displays</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>H = 21 kips</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Experimental</td>
<td>0.17</td>
<td>62</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>Case 1 Linear soil</td>
<td>0.085</td>
<td>35.1</td>
<td>3.1</td>
<td>Figures</td>
</tr>
<tr>
<td>Case 2 Nonlinear soil</td>
<td>0.31</td>
<td>70.5</td>
<td>6.8</td>
<td></td>
</tr>
<tr>
<td><strong>H = 31.5 kips</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Experimental</td>
<td>0.26</td>
<td>85</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Case 3 Linear soil</td>
<td>0.13</td>
<td>52.6</td>
<td>3.1</td>
<td>Figures</td>
</tr>
<tr>
<td>Case 4 Nonlinear soil</td>
<td>0.56</td>
<td>115.5</td>
<td>6.8</td>
<td></td>
</tr>
<tr>
<td><strong>H = 43 kips</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Experimental</td>
<td>0.4</td>
<td>120</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Case 5 Linear soil</td>
<td>0.17</td>
<td>70.1</td>
<td>3.1</td>
<td>Figures</td>
</tr>
<tr>
<td>Case 6 Nonlinear soil</td>
<td>0.89</td>
<td>164.7</td>
<td>6.8</td>
<td></td>
</tr>
</tbody>
</table>

Figure B.2: Comparison of the load-deflection curves for the linear and nonlinear runs.
Figure B.3: Comparison of the pile deflection profiles for the linear and nonlinear runs.
f) $H = 43$ kips

Figure B.3: (continued).

a) $H = 21$ kips

Figure B.4: Comparison of the pile bending moment profiles for the linear and nonlinear runs.
b) $H = 31.5$ kips

c) $H = 43$ kips

Figure B.4: (continued).
Figure B.5: Stress ratio contour fill of the nonlinear run at different load levels (red color shows yielded soil elements).
Appendix B-I: Comparison with LPILE

In the LPILE run, a p-y modulus of 90 psi is employed (p-y multiplier = 1.0). All other properties are the same as described earlier.

Figure B.6: Comparison of the pile deflection profiles for the linear and nonlinear runs.
c) $H = 43$ kips

Figure B.6: (continued).

a) $H = 21$ kips

Figure B.7: Comparison of the pile bending moment profiles for the linear and nonlinear runs.
b) $H = 31.5$ kips

c) $H = 43$ kips

Figure B.7: (continued).
Appendix C  Finite Element Analysis of Standard CalTrans 16" CIDH Pile Using OpenSees for General Comparison with LPILE (with Default P-Y Multiplier = 1.0)

Introduction

In this study, we conduct a finite element simulation of the standard Caltran 16" CIDH pile using the 3D OpenSeesPL interface. The simulated pile responses are compared with LPILE results.

Laterally Loaded Pile

Pile Data

The geometric and elastic material properties of the pile are listed below:

Diameter $D = 16"$
Pile length $l = 35$ ft
Moment of Inertia of Pile $I = 850$ in$^4$
Young’s Modulus of Pile $E_c = 4030$ ksi

In this initial study, the pile was modeled to remain linear (also in view of the applied load levels).

Soil Domain

Linear and nonlinear soil responses are investigated. The Medium relative-density granular soil type (Lu et al. 2006) is selected in the analyses. The material properties of the soil are listed below:

At the reference confinement of 80 kPa (or 11.6 psi), the Shear Modulus of Soil $G_s = 10.88$ ksi and the Bulk Modulus of Soil $B = 29$ ksi (i.e., Poisson’s ratio $\nu_s = 0.33$), see Lu et al. 2006.

Effective Unit Weight $\gamma' = 110$ pcf (given by CalTrans)

For nonlinear analysis, the Friction Angle $\varphi = 33^\circ$ (given by CalTrans) and the peak shear stress occurs at a shear strain $\gamma_{\text{max}} = 10\%$ (at the 11.6 psi confinement). The parameter $\gamma_{\text{max}}$ along with the shear modulus define the nonlinear soil stress-strain curve. Other values of $\gamma_{\text{max}}$ should be explored in the future.
Lateral Load

Two load cases (Table 1) are studied. The loads are applied at the pile head.

<table>
<thead>
<tr>
<th>Table C.1: Load cases for the study.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear (kips)</td>
</tr>
<tr>
<td>Load case 1*</td>
</tr>
<tr>
<td>Load case 2**</td>
</tr>
</tbody>
</table>

* Fixed pile head connection
** Apply moment in opposite direction of shear.

Finite Element Simulation

In view of symmetry, a half-mesh (2,900 8-node brick elements, 19 beam-column elements and 180 rigid beam-column elements in total) is studied as shown in Figure C.1. Length of the mesh in the longitudinal direction is 520 ft, with 260 ft transversally (in this half-mesh configuration, resulting in a 520 ft x 520 soil domain in plan view). Layer thickness is 60 ft (the bottom of the soil domain is 25 ft below the pile tip, so as to mimic the analytical half-space solution).

The floating pile is modeled by beam-column elements (Mazzoni et al. 2006), and rigid beam-column elements are used to model the pile size (diameter).

The following boundary conditions are enforced:

I) The bottom of the domain is fixed in the longitudinal (x), transverse (y), and vertical (z) directions.

II) Left, right and back planes of the mesh are fixed in x and y directions (the lateral directions) and free in z direction.

III) Plane of symmetry is fixed in y direction and free in z and x direction (to model the full-mesh 3D solution).

The lateral load is applied at the pile head (ground level) in x (longitudinal) direction.

The above simulations were performed using OpenSeesPL (Lu et al. 2006).

Simulation Results

The pile head deflections and the maximum bending moments for the linear and nonlinear analyses are listed in Table 2, along with LPILE results for comparison (see Appendix for partial output of LPILE results).

Figures C.2-C.5 show comparisons of the pile deflection, rotation, bending moment and shear force profiles, respectively, for load case 1. Figures C.6-C.9 show comparisons of
the pile deflection, rotation, bending moment and shear force profiles, respectively, for load case 2. The stress ratio contour fill of the nonlinear runs for load cases 1 & 2 are displayed in Figures C.10 & C.11.

Figure C.1: Finite element mesh employed in this study.
Table C.2: CalTrans CIDH Pile OpenSees Simulation and LPILE Results.

<table>
<thead>
<tr>
<th>Analysis type</th>
<th>Pile head deflection (in)</th>
<th>Max. bending moment $M_{\text{max}}$ (kip-ft)</th>
<th>$M_{\text{max}}$ depth (ft)</th>
<th>Profile displays</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Case 1 Fixed Head H = 16 kips</td>
<td>LPILE 0.24</td>
<td>-48.2</td>
<td>0</td>
<td>Figures 2 &amp; 4</td>
</tr>
<tr>
<td></td>
<td>Linear soil 0.038</td>
<td>-20.8</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Nonlinear soil 0.092</td>
<td>-32.3</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Load Case 2 Free Head M = -100 kip-ft applied opposite to shear</td>
<td>LPILE -0.094</td>
<td>-100</td>
<td>0</td>
<td>Figures 6 &amp; 8</td>
</tr>
<tr>
<td></td>
<td>Linear soil -0.06</td>
<td>-96.7</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Nonlinear soil -0.094</td>
<td>-96.9</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

Figure C.2: Comparison of pile deflection profiles for load case 1.
Figure C.3: Comparison of pile rotation profiles for load case 1.

Figure C.4: Comparison of bending moment profiles for load case 1.
Figure C.5: Comparison of shear force profiles for load case 1.

Figure C.6: Comparison of pile deflection profiles for load case 2.
Figure C.7: Comparison of pile rotation profiles for load case 2.

Figure C.8: Comparison of bending moment profiles for load case 2.
Figure C.9: Comparison of shear force profiles for load case 2.

Figure C.10: Stress ratio contour fill for load case 1 (red color shows yielded soil elements).

Figure C.11: Stress ratio contour fill for load case 2 (red color shows yielded soil elements).
Appendix D  Finite Element Analysis of Caltrans 42" CIDH Pile Using OpenSees for General Comparison with LPILE (with Default P-Y Multiplier = 1.0)

Introduction

In this study, we conduct a finite element simulation of a CalTrans 42" CIDH pile using the 3D OpenSeesPL interface. The simulated pile responses are compared with LPILE results.

Laterally Loaded Pile

Pile Data

The geometric and elastic material properties of the pipe pile are listed below:

- Diameter $D = 42"$ or radius $a = 21"$
- Wall thickness $h = 0.75"$
- Pile length $l = 35$ ft
- Moment of Inertia of Pile $I = \pi a^2 h = 21,821$ in$^4$
- Young’s Modulus of Pile $E_s = 29,000$ ksi

In this initial study, the pile was modeled to remain linear (also in view of the applied load levels).

Soil Domain

Linear and nonlinear soil responses are investigated. The Medium relative-density granular soil type (Lu et al. 2006) is selected in the analyses. The material properties of the soil are listed below:

- At the reference confinement of 80 kPa (or 11.6 psi), the Shear Modulus of Soil $G_s = 10.88$ ksi and the Bulk Modulus of Soil $B = 29$ ksi (i.e., Poisson’s ratio $\nu_s = 0.33$), see Lu et al. 2006.
- Unit Weight $\gamma = 110$ pcf

For nonlinear analysis, the Friction Angle $\phi = 33^\circ$ and the peak shear stress occurs at a shear strain $\gamma_{\text{max}} = 10\%$ (at the 11.6 psi confinement). The parameter $\gamma_{\text{max}}$ along with the shear modulus define the nonlinear soil stress-strain curve. Other values of $\gamma_{\text{max}}$ should be explored in the future.
Lateral Load

A total of six load cases (Table 1) are studied. The loads are applied at the pile head.

Table D.1: Load cases for the study.

<table>
<thead>
<tr>
<th>Load case</th>
<th>Pile head condition</th>
<th>Shear (kips)</th>
<th>Moment (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load case 1</td>
<td>Fixed head</td>
<td>64</td>
<td>0</td>
</tr>
<tr>
<td>Load case 2</td>
<td>Fixed head</td>
<td>128</td>
<td>0</td>
</tr>
<tr>
<td>Load case 3</td>
<td>Fixed head</td>
<td>256</td>
<td>0</td>
</tr>
<tr>
<td>Load case 4</td>
<td>Free head</td>
<td>64</td>
<td>0</td>
</tr>
<tr>
<td>Load case 5</td>
<td>Free head</td>
<td>128</td>
<td>0</td>
</tr>
<tr>
<td>Load case 6</td>
<td>Free head</td>
<td>256</td>
<td>0</td>
</tr>
</tbody>
</table>

Finite Element Simulation

In view of symmetry, a half-mesh (2,900 8-node brick elements, 19 beam-column elements and 180 rigid beam-column elements in total) is studied as shown in Figure D.1. Length of the mesh in the longitudinal direction is 1360 ft, with 680 ft transversally (in this half-mesh configuration, resulting in a 1360 ft x 1360 soil domain in plan view). Layer thickness is 60 ft (the bottom of the soil domain is 25 ft below the pile tip, so as to mimic the analytical half-space solution).

The floating pile is modeled by beam-column elements (Mazzoni et al. 2006), and rigid beam-column elements are used to model the pile size (diameter).

The following boundary conditions are enforced:
   I) The bottom of the domain is fixed in the longitudinal (x), transverse (y), and vertical (z) directions.
   II) Left, right and back planes of the mesh are fixed in x and y directions (the lateral directions) and free in z direction.
   III) Plane of symmetry is fixed in y direction and free in z and x direction (to model the full-mesh 3D solution).

The lateral load is applied at the pile head (ground level) in x (longitudinal) direction.

The above simulations were performed using OpenSeesPL (Lu et al. 2006).

Simulation Results

Figures D.2-D.5 show comparisons of the pile deflection, rotation, bending moment and shear force profiles, respectively, for the fixed-head condition (load cases 1, 2 & 3), along with LPILE results for comparison. Figures D.6-D.9 show comparisons of the pile deflection, rotation, bending moment and shear force profiles, respectively, for the free-head condition (load cases 4, 5 & 6), also along with LPILE results for comparison. The
stress ratio contour fill of the nonlinear runs for the fixed and free head conditions are displayed in Figures D.10 & D.11.

Comparisons of the linear and nonlinear responses using OpenSees are shown in Appendix (Figures D.12-D.19).

(a) Isometric view

(b) Pile head close-up

Figure D.1: Finite element mesh employed in this study.
Figure D.2: Comparison of pile deflection profiles for the fixed-head condition.

Figure D.3: Comparison of pile rotation profiles for the fixed-head condition.
Figure D.4: Comparison of bending moment profiles for the fixed-head condition.

Figure D.5: Comparison of shear force profiles for the fixed-head condition.
Figure D.6: Comparison of pile deflection profiles for the free-head condition.

Figure D.7: Comparison of pile rotation profiles for the free-head condition.
Figure D.8: Comparison of bending moment profiles for the free-head condition.

Figure D.9: Comparison of shear force profiles for the free-head condition.
a) lateral load = 64 kips (left: plan view; right: side view)

b) lateral load = 128 kips (left: plan view; right: side view)

c) lateral load = 256 kips (left: plan view; right: side view)

Figure D.10: Stress ratio contour fill of the nonlinear run for the fixed-head condition (red color shows yielded soil elements).
a) lateral load = 64 kips (left: plan view; right: side view)

b) lateral load = 128 kips (left: plan view; right: side view)

c) lateral load = 256 kips (left: plan view; right: side view)

Figure D.11: Stress ratio contour fill of the nonlinear run for the free-head condition (red color shows yielded soil elements).
Appendix D-I: OpenSees Simulation Results

Figure D.12: Comparison of pile deflection profiles for the fixed-head condition.
Figure D.13: Comparison of pile rotation profiles for the fixed-head condition.

Figure D.14: Comparison of bending moment profiles for the fixed-head condition.

Figure D.15: Comparison of shear force profiles for the fixed-head condition.
Figure D.16: Comparison of pile deflection profiles for the free-head condition.

Figure D.17: Comparison of pile rotation profiles for the free-head condition.
Figure D.18: Comparison of bending moment profiles for the free-head condition.

Figure D.19: Comparison of shear force profiles for the free-head condition.
Appendix E  Finite Element Analysis of Standard CalTrans 16" CIDH Pile Subjected to Axial Load

Introduction

In this study, we conduct a finite element simulation of the standard Caltran 16" CIDH pile using the 3D OpenSeesPL interface. The simulated pile is subjected to axial load.

Axially Loaded Pile

Pile Data

The geometric and elastic material properties of the pile are listed below:

- Diameter $D = 16''$
- Pile length $l = 35$ ft
- Moment of Inertia of Pile $I = 850$ in$^4$
- Young’s Modulus of Pile $E_c = 4030$ ksi

In this initial study, the pile was modeled to remain linear (also in view of the applied load levels).

Soil Domain

Nonlinear soil response is investigated. The Medium relative-density granular soil type (Lu et al. 2006) is selected in the analyses. The material properties of the soil are listed below:

- At the reference confinement of 80 kPa (or 11.6 psi), the Shear Modulus of Soil $G_s = 10.88$ ksi and the Bulk Modulus of Soil $B = 29$ ksi (i.e., Poisson’s ratio $\nu_s = 0.33$), see Lu et al. 2006.
- Effective Unit Weight $\gamma' = 110$ pcf (given by CalTrans)

For nonlinear analysis, the Friction Angle $\varphi = 33^\circ$ (given by CalTrans) and the peak shear stress occurs at a shear strain $\gamma_{\text{max}} = 10\%$ (at the 11.6 psi confinement). The parameter $\gamma_{\text{max}}$ along with the shear modulus define the nonlinear soil stress-strain curve. Other values of $\gamma_{\text{max}}$ should be explored in the future.

Axial Load

An axial load of 243 kips is applied at the pile head (free head connection).
Finite Element Simulation

In view of symmetry, a half-mesh (2,900 8-node brick elements, 19 beam-column elements and 180 rigid beam-column elements in total) is studied as shown in Figure E.1. Length of the mesh in the longitudinal direction is 520 ft, with 260 ft transversely (in this half-mesh configuration, resulting in a 520 ft x 520 soil domain in plan view). Layer thickness is 60 ft (the bottom of the soil domain is 25 ft below the pile tip, so as to mimic the analytical half-space solution).

The floating pile is modeled by beam-column elements (Mazzoni et al. 2006), and rigid beam-column elements are used to model the pile size (diameter).

The following boundary conditions are enforced:

I) The bottom of the domain is fixed in the longitudinal (x), transverse (y), and vertical (z) directions.
II) Left, right and back planes of the mesh are fixed in x and y directions (the lateral directions) and free in z direction.
III) Plane of symmetry is fixed in y direction and free in z and x direction (to model the full-mesh 3D solution).

The axial load is applied at the pile head (ground level) in z (vertical) direction.

The above simulations were performed using OpenSeesPL (Lu et al. 2006).

Simulation Results

The pile vertical displacement and axial force profiles at the axial load of 243 kips are shown in Figure E.2. The final deformed mesh is shown in Figure E.3. Figure E.4 displays the stress ratio contour fill.
Figure E.1: Finite element mesh employed in this study.

(a) Isometric view

(b) Pile head close-up
Figure E.2: Pile profile response at the axial load of 243 kips.

Figure E.3: Close-up of final deformed mesh (factor of 120).
Figure E.4: Stress ratio contour fill for the nonlinear analysis (red color shows yielded soil elements).
References


OpenSeesPL-Related References


