ABSTRACT

SUAREZ, VINICIO. Implementation of Direct Displacement Based Design for Pile and Drilled Shaft Bents. (Under the direction of Dr. Mervyn Kowalsky)

The work in this thesis attempts to implement the Direct Displacement Based Design (DDBD) method to the seismic design of long reinforced concrete pile and drilled shaft bents embedded in soft soils. DDBD has been successfully used to design bridge columns that are fixed at ground level and without soil interaction. The implementation of DDBD for column bents, however, requires the consideration of soil-structure interaction effects—namely added flexibility and damping. The main objective of this research is to develop an equivalent model to predict yield displacement and ductility and to assess the equivalent viscous damping as a function of ductility demand and soil type.

The proposed equivalent cantilever model replaces a nonlinear soil-column system. In the equivalent model, the column is considered fixed at some depth below ground at the point of maximum moment and possible formation of an underground plastic hinge. The yield displacement of the column is matched with the yield displacement of the soil-column model by introducing a coefficient and the energy dissipation characteristics are matched by the introduction of equivalent viscous damping as function of ductility and soil type. Charts and equations are provided to compute all the parameters involved in the equivalent formulation. These aids resulted from parametric studies that involved nonlinear static and nonlinear time history analyses of soil-column systems.
Implementation of Direct Displacement Based Design for Pile and Drilled Shaft Bents

by

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In August of 2004, he came to Raleigh to pursue a Master of Science Degree in Structural Engineering. His research interests include but not limited to the seismic analysis and design of buildings and bridges and the analysis of soil-structure interaction problems. Vinicio completed his Ms studies in fall 2005 and is expected to continue his work as a Ph.D. student at NCSU.
Acknowledgments

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1. INTRODUCTION

Column bents are a type of bridge substructure in which piles or columns are extended from the superstructure continuously below grade (Figure 1.1). These structures are commonly used to resist axial and lateral forces produced by dead, live, wind, earthquake and impact loads. Their response is highly dependent on soil-structure interaction phenomena.

Historically, it has been common seismic design practice to simplify the soil-structure interaction problem by considering the piles or shafts within each bent to be fixed at an estimated depth below the ground surface. This simplification is made in an attempt to account for the flexibility that the soil adds to the bent while avoiding difficult soil modeling issues. Once the bridge model is built, Force Based Design is applied in conjunction with Capacity Design Principles. The design objective is the same as for buildings, according to the AASHTO Bridge Design specifications (2004), the structure must have sufficient strength and stiffness to resist a rare earthquake event without collapsing and causing the loss of life. No special attention is given to the damage and performance of the structure under the effects of events of different intensity.

Performance-Based Seismic Engineering (PBSE) has emerged as a promising alternative to the traditional design philosophy. The goal of PBSE is to design a structure that will have predictable levels of performance, when subjected to specified levels of earthquakes, within definable levels of reliability (SEAOC, 2003). The performance is evaluated in terms of damage, which is related to the level of displacement rather than force.
Several procedures have been developed for the application of PBSE, these are generally Displacement-Based in nature. One such procedure called Direct Displacement Based Design (DDBD), was proposed by Priestley (1993) and has been continually developed over the last years.

Figure 1.1 General configuration of pile or drilled shaft bents
1.1 Objectives and Scope of Work

This research attempts to improve existing design practice by introducing PBSE aspects to the design of reinforced concrete pile and drilled shaft bents. In order to do so, the research will focus on: The implementation of the DDBD procedure for the seismic design of column bents, while more fully considering soil-column interaction effects.

1.1.1 Specific objectives

- To develop a simple model to account for soil-pile interaction in the seismic design of reinforced concrete (RC) pile and drilled shaft bents.
- To determine the energy dissipation characteristics of the soil surrounding a column in terms of equivalent viscous damping.
- To produce a simple procedure to find the design strength and stiffness of an RC pile or drilled shaft bent based on information available at the early stages of design, namely, pile diameter, general soil type, general configuration, geometry and elastic design spectra.

1.1.2 Thesis Organization

Chapter 2 presents a literature review including a survey of the current design practice, a description of the DDBD method, and a summary of the available software for analysis.
In Chapter 3, the static and dynamic response of a bent is studied. This chapter also shows the results of several verification analyses that were performed to check the capabilities of the software to be used later in the research.

To implement the DDBD procedure, first a model must be developed to relate local ductility with total displacement ductility. Chapter 4 shows the results of a parametric study that was carried out to develop such a model for RC columns of different diameters and heights, embedded in sands or clays.

Another parametric study that included nonlinear time history analyses (NTHA) of soil-column systems was necessary to determine trends for the equivalent viscous damping associated with the energy dissipated in the RC column and in the soil during the earthquake. The results of these analyses are shown in Chapter 5.

Chapter 6 demonstrates the use of the developed models and the implementation of DDBD. The effect of key design parameters on system response is also evaluated. Chapter 7 includes the conclusions from the study and recommendations for application and future research.
2. LITERATURE REVIEW

2.1 Soil-Structure interaction.

Several interaction modes that exist between a column and the soil around it when subjected to an earthquake action are depicted in Figure 2.1. The most important modes are kinematic and inertial. When a pile is driven in soil, it changes the way the soil would have responded to an earthquake if the pile was not there. This phenomenon is called kinematic interaction and results in internal forces developing along the pile.

![Diagram of Soil-Structure Interaction Modes](Figure 2.1)

Figure 2.1 Soil-Structure interaction modes. Adapted from Maymand P.J. (1998)
This research focuses on inertial interaction effects. However, it is recognized that the pile foundation may also experience significant "kinematic" loads that are imposed by the surrounding soil mass as it deforms relative to the pile during earthquake shaking. Kinematic loads may not be significant in competent soil profiles that experience relatively small strains and deformations during shaking. Large kinematic loads can develop, however, due to lateral spreading of liquefied soils or due to high strain gradients in soft clays, and may be particularly damaging when the soil stratigraphy consists of alternating stiff and soft layers along the embedded column length (Wilson D.W. 1998)

The installation procedure also affects the soil-structure interaction, especially under the influence of axial loads, where driven piles and drilled shafts have different design methodologies for the same soil. The lateral response, however, seems to be less affected since the current practice is to use models that were developed and verified for particular soil types from experiments with both driven piles and drilled shafts (Murchison and O’Neill, 1984). Therefore, in this study no special distinction is made for the design of RC piles and RC drilled shafts and both are referred as RC columns.

2.2 Models for Soil-Column Interaction

Several techniques have been proposed over time to model piles embedded in soils. These fall into two categories: continuum models and equivalent models.
2.2.1 Continuum analysis: Several variations exist, all using primarily the finite element method of analysis (Selby A.R. and Arta M.R., 1997) (Brown D, 1990). It is perhaps the more rigorous approach to model soil-pile systems. Nevertheless the amount of information required in addition to its complexity and computational cost are maybe justified only for analysis of special problems.

2.2.2 Equivalent P-y model. This analysis approach is the more commonly used. The soil is replaced with a series of nonlinear springs closely spaced along the embedded length of the pile as can be seen in Figure 2.2.a. The force-deformation response of the springs has been back-calculated from the results of well instrumented lateral load tests of piles in different soils. In its current state, the method allows for multilayered soils (Geordalis, 1983). There are several available models for lateral response of soil or P-y models, such as the model for soft clays under water by Matlock (1970), for stiff clays by Reese (1975) and the integrated model for clays by O’Neil (1984).

There is software specially developed for the application of this method. Two commercial packages are MultiPier (BSI, 2000) and LPILE (Ensoft, 2004). MultiPier was developed in 1994 at the Bridge Software Institute, and some of its features are a built in interactive pile bent software wizard, models for soil resistance (lateral and axial, single and group) using methods representing the current state of geotechnical engineering practice and nonlinear modeling of the structural elements. MultiPier has been used in this research to perform lateral nonlinear static analysis of soil column systems.
P-y models have also been developed to model the dynamic response of soil-column systems and have been integrated into software packages. OpenSees (McKena F., 2000) was developed at the Pacific Earthquake Research Center and is an object-oriented framework for finite element analysis. The program allows the user to perform Nonlinear Time History Analysis (NTHA) on soil-column models using P-y models for the soil. OpenSees has been used in this research to perform NTHA on soil-column models.

The P-y equivalent model, although simpler and more practical than the continuum analysis, is not yet suitable for design purposes since it requires information that may not be available at the early stages of design, such as the moment-curvature response of the column section.

2.2.3 Equivalent base spring model. The soil and pile beneath the ground surface is replaced by coupled translational and rotational springs. The stiffness of the springs must account for the stiffness of the soil and pile below ground and is obtained on a case by case basis by a substructuring technique. Although the resultant model is smaller than an equivalent P-y model, getting the stiffness matrix to model the soil requires significantly more effort making this procedure less attractive than the P-y method for the analysis of column-soil systems.

2.2.4 Equivalent cantilever model: the piles are considered fully fixed at some depth below ground surface and the soil is ignored, as shown in Figure 2.2.b. The embedded length, also called depth to fixity, $L_f$, is estimated from formulas or from the results of a nonlinear lateral single-pile analysis. The resulting equivalent model is especially convenient for the structural
engineer, who can model a single pile as a cantilever and a pile bent as a frame. In both cases, the piles are fixed at their bases. The result of this simplification is that the structural computations are straightforward and seem to provide all the required information for design. That is, for the application of the code-based force based design procedure, the designer can readily calculate stiffness, fundamental period, and seismic design forces from this model.

Over the last 40 years, several procedures have been proposed for the estimation of \( L_f \) values, such as those proposed by Davisson and Robinson (1965) and Chen (1997). In his paper, Chen, presents a procedure that yields three \( L_f \) values for a pile-soil system: one to match stiffness, and the others to match moment and buckling capacity. The most often used \( L_f \) equations are those proposed by Davison and Robinson in 1965. These equations have been incorporated into the AASHTO LRFD Bridge Design Specifications (2004) and their use is recommended for the assessment of buckling effective length only. For piles in clays, \( L_f \) is evaluated from Equation 2.1, and for piles in sand from Equation 2.2. In both cases, \( L_f \) is measured from the ground level.

In these two equations, \( E_p \) and \( I_{py} \) are the elastic modulus and inertia of the pile, \( E_c \) is the elastic modulus for clays (see Table 2.1 for representative values) and \( n_h \) is the rate by which the soil modulus increases with depth in sands (see Table 2.2 for representative values). Equations 1 and 2 are based on beam on elastic-foundation theory and assume a long, partially embedded pile laterally loaded in a single uniform layer of either clay or sand. The coefficients 1.4 in Equation 2.1 and 1.8 in Equation 2.2 are set so the model can approximately match bending and buckling response simultaneously but will not match lateral stiffness and will not distinguish between a free and pinned head condition.
Values of $L_f$ can also be obtained from charts. Figure 2.3 and Figure 2.4 show the results of a parametric study on columns in sands for pinned and fixed head condition respectively (Budek A.M. 2000). In this charts the depth to fixity normalized with respect to column diameter $D$ is plotted against a nondimensional stiffness value, where $K$ is lateral subgrade modulus of sand, $D^*$ is a reference diameter equal to 1.83 m and $EI_{eff}$ is the flexural stiffness of the column. Figures 2.3 and 2.4 also show the recommendation of Caltrans (1990) to determine $L_f$ for drilled shafts in sands. The $L_f$ value obtained from Figures 2.3 or 2.4 define the length of an equivalent cantilever that has the same lateral stiffness as the pile in soil. However, the moment calculated at the base of this cantilever, does not correspond to the maximum moment in the real column. As an alternative, $L_f$ can be found from the results of single column lateral nonlinear analysis as proposed by Kowalsky et al. (2005). In this case, $L_f$ is chosen to match moment in the pile, and the stiffness and buckling capacity are matched by introducing inertia modifier and length modifier coefficients respectively.

In general, the main drawback of the equivalent cantilever approach is that, for a multiple soil layer profile, the engineer has to determine an equivalent soil layer of either sand or clay.

\[
L_f = 1.4 \left[ \frac{E_p L_p y_y}{E_c} \right]^{0.25} \quad \text{(Clay)} \quad (2.1)
\]

\[
L_f = 1.8 \left[ \frac{E_p L_p y_y}{n_h} \right]^{0.20} \quad \text{(Sand)} \quad (2.2)
\]
Figure 2.2 Soil-pile models. (a) p-y equivalent model (b) Equivalent cantilever model

Table 2.1. Representative $E_c$ values for clays after Y. Chen (1997)

<table>
<thead>
<tr>
<th>Clay type</th>
<th>$s_u$ (tsf)</th>
<th>$E_c$ (tsf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft</td>
<td>0.25</td>
<td>16.75</td>
</tr>
<tr>
<td>Medium</td>
<td>0.47</td>
<td>31.4</td>
</tr>
<tr>
<td>Stiff</td>
<td>0.81</td>
<td>54.4</td>
</tr>
<tr>
<td>Very stiff</td>
<td>1.47</td>
<td>98.5</td>
</tr>
</tbody>
</table>

Table 2.2. Representative $n_h$ values for sands after Y. Chen (1997)

<table>
<thead>
<tr>
<th>Sand type</th>
<th>Saturation condition</th>
<th>$n_h$ (tsf/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose</td>
<td>Moist/dry</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>Submerged</td>
<td>15</td>
</tr>
<tr>
<td>Medium</td>
<td>Moist/dry</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>Submerged</td>
<td>40</td>
</tr>
<tr>
<td>Dense</td>
<td>Moist/dry</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>Submerged</td>
<td>100</td>
</tr>
</tbody>
</table>
Figure 2.3 Depth to fixity for free head columns in sand. Budek et al (2000)

Figure 2.4 Depth to fixity for fixed head columns in sand. Budek et al (2000)
2.3. Nonlinear response of column bents under lateral loads

Column bents can be visualized as plane structures. Usually, a line of two or more columns is attached to a cap-beam. The section, type and length of the columns and the spacing between them are kept constant within the bent. The in-plane response can be characterized for the development of maximum moments at the connection with the cap-beam or superstructure. If the imposed curvature reaches the yield limit, a plastic hinge will develop at this location. The out-of-plane response includes maximum moments developing at some depth under ground, and possibly plastic hinges if the imposed curvature reaches the yield limit. The response in both directions is highly dependent on the above ground length and the flexural stiffness and strength of the column, and the stiffness and strength of the soil (Figure 2.5).

If the flexural stiffness of the cap-beam is very large compared to the stiffness of the columns, the in-plane response of the bent can be estimated by looking at the response of a single column in which, the rotation of the top is restrained (Fixed-head column). The out-of-plane response however, depends on the type of connection with the superstructure. If the sub-superstructure connection is rigid, the response is similar to that of a fixed head column. However, the usual case is to have a pinned connection between the superstructure and the cap beam and therefore the out-of-plane response can be assessed by analyzing a single pile or column allowing free rotation at the top (Pinned-head column). Therefore, this research focuses on the assessment, characterization, and modeling of the nonlinear static and dynamic response of single columns with both fixed and pinned head conditions.
In 2000, Budek et al investigated the response of drilled shafts in cohesionless soils. They conducted a series of nonlinear analysis using the equivalent P-y model to study the force-deformation response, moment pattern, and ductility demand on drilled shafts under a lateral load applied at the top. In their study, shafts are modeled as a series of nonlinear elements and soil is modeled with elastic, bi-linear and nonlinear springs. Figure 2.6 shows

**Figure 2.5 Plastic hinge locations**
the moment pattern of fixed-head columns when a lateral load has been applied at the top and
the column has reached its ultimate displacement capacity. Figure 2.7 shows the moment
pattern of pinned-head columns. In these figures, K is the lateral subgrade modulus of sand.

Figure 2.6 Moment Patterns at ultimate yield displacement in a fixed-head column in sand (Budek et al, 2000).
Figure 2.7 Moment Patterns at ultimate yield displacement in a pinned-head column in sand (Budek et al., 2000).

Figure 2.8 Underground Plastic hinge for pinned-head columns in sand. (Budek et al, 2000)
Figure 2.8 shows values of plastic hinge length plotted against a nondimensional stiffness value for pinned head columns. The nondimensional stiffness was described in section 2.2.4. The plastic hinge length multiplied by the curvature ductility demand yields the total plastic rotation at the hinge. The underground plastic hinge lengths are usually larger than the length for the hinges developed at the top of a fixed head column. In the case of an underground hinge, the plasticity spreads at both sides of the hinge, and the amount of spreading is increased by the presence of the soil.

For plastic hinges forming at the top of fixed-head columns, the plastic hinge length can be calculated from Equation 2.3 (Priestley M.J.N., 1996). In this equation $L$ is the distance from the critical section to the point of inflection in meters, $f_y$ is the yield strength in MPa, and $d_{bl}$ is the longitudinal bar diameter in meters.

\[ l_p = 0.08L + 0.022f_y d_{bl} \quad (2.3) \]

The first term in Equation 2.3 represents the spread of plasticity resulting from variation in curvature with distance from the critical section, and assumes a linear variation in moment with distance. The second term represents the increase in effective plastic hinge length associated with strain penetration into the supporting member.
2.4 Ductility Models

In 2002, Chai Y.H. proposed an analytic model to assess the local ductility demand of a yielding pile using the equivalent cantilever model shown in Figure 2.9. This model is applicable to single column bents and uses a point of fixity that matches the stiffness of the soil-column system. The depth to the point at which the maximum underground moment occurs is determined separately. The total top displacement is then found as the sum of the elastic displacement $\Delta_y$ (Equation 2.4) plus the plastic displacement $\Delta_p$ (Equation 2.5).

Figure 2.9 Ductility Model free head columns (Chai Y.H., 2002)
In Equations 2.4-2.5, $\Delta_y$ is the yield elastic displacement, $V$ is the yield lateral force, $L_f$ is the depth to fixity, $L_a$ is the height of the column, $L_m$ is the depth to the point of maximum moment, $EI$ is the product of the elastic modulus and inertia of the column, $\theta_p$ is the plastic rotation at the plastic hinge, $L_p$ is the plastic hinge length and $\phi_u$ and $\phi_y$ are the ultimate and yield curvatures. This model allows the estimation of lateral strength and the assessment of the local curvature ductility demand in the column for any value of displacement ductility demand at the top of the pile. The elastic stiffness used in this model comes from the elastic Winkler model developed by Poulos and Davis (1980).

2.5 Special effects of earthquake loading.

So far we have covered the nonlinear response of column-soil systems under the effect of lateral load acting at the column top. During an earthquake however, the cyclic nature of the excitation can cause the formation of a gap between the soil and the column. This is due to nonlinear nature of the soil response and to the fact that the soil does not resist tension. As a result of that, the stiffness of the column system is reduced and continually degraded during an earthquake.
Figure 2.9/a shows the results of a lateral cyclic load test performed by Matlock in 1970. In this test a pipe pile was push laterally while embedded in soft clay. Figure 2.9/b. shows the results of a simulation performed using OpenSees with the *pysimple1* element (Boulanger, R.W., 2003).

![Figure 2.9](image)

Figure 2.9  a)Result of Matlock’s experiment in soft clay b) Result of simulation with pysimple1 element

Figure 2.9 show that even though the *pysimple1* element in OpenSees does not model strength degradation, it can reasonably capture the force deformation response of pile-soil systems in soft clay. The *pysimple1* element in OpenSees has been further verified with the results of centrifuge experiments (Boulanger et al, 1999).

### 2.6 Review of the current design practice

The current practice for seismic design of pile and drilled shaft bents includes the application of a forced based approach and requires the following procedure:
1. Obtain an equivalent cantilever model for the pile-soil system. The depth to fixity is chosen from design charts, to match the stiffness of the pile-soil system.

2. Build a bridge model. Once each pile and the surrounding soil have been replaced by a single frame element, a complete model of the bridge is built by adding the other pile and superstructure elements. Boundary conditions, mass, section properties, and loads are then assigned to the model.

3. A modal analysis is performed. The fundamental period is found.

4. The design base shear is found from the appropriate design spectra, using the mass, the period and the force reduction factor $R$. ATC 40 (1996) considers piles/drilled shafts, as structures of limited ductility and recommends $R$ equal 4. AASHTO LRFD Bridge Design Specifications (2004) recommend values of 1.5 for bents classified as critical, 3.5 for essential bents and 5 for other bents.

5. Then an elastic analysis is performed and element internal forces are found for design.

6. The design of the column bent is then performed using special purpose software such as MultiPier (BSI, 2000) or L-pile (Ensoft, 2004). The use of these special codes allows for a more precise determination of the moments that develop along the pile, as well as deflections.
2.7 Direct Displacement Based Design (DDBD)

DDBD works by inverting the traditional seismic design process. For a specified target displacement and earthquake intensity, the method yields the required stiffness and strength. DDBD has been successfully implemented for the design of bridge RC piers (Kowalsky M.J et al 1995) and for RC frames (Priestley M.J.N and Kowalsky M.J., 2000). The procedure is based on the following arguments:

- The yield curvature/displacement can be estimated from the geometry of the section
- A nonlinear system can be substituted by a single degree of freedom linear system that has an effective mass, effective period and equivalent viscous damping.

To apply DDBD for the design of column bents, the following procedure could be followed:

- Gather basic information such as: Length above ground, diameter of the column, weight acting on the bent, soil type and strength.
- Establish the Target Displacement and Displacement Ductility demand.
- Determine the equivalent viscous damping. Due to the soil-structure interaction, the equivalent viscous damping is a combination of viscous damping, hysteretic damping in the column and hysteretic damping in the soil.
- From the displacement design spectra determine required period and then stiffness.
- Find required level of strength.
The procedure presented above is similar to what has been proposed for bridge piers (Kowalsky M.J. et al, 1995). However, piers that are supported on rigid pile caps at soil level are likely to develop plastic hinges above ground only. So, the equivalent damping is a combination of viscous damping and hysteretic damping in the column only (Dwairi H.M., 2005).

This section concludes the literature review, in which the most relevant information about the analysis and design of column bents as well as the implementation of DDBD was summarized. Next, Chapter 3 presents the results of several static and dynamic nonlinear analyses conducted to demonstrate the performance of the P-y equivalent models, and also the capabilities of existing software.
3. MODELING THE SOIL-PILE INTERACTION

3.1 Soil Properties

Throughout this study, the soil used in the analyses has the properties denoted in Table 3.1 for clay and Table 3.2 for sand. The characterization of clay has been done in terms of shear strength \( s_u \), then strains at 50% and 100% of the ultimate shear strength, \( \varepsilon_{50} \) and \( \varepsilon_{100} \), and total unit weight \( w \). The characterization of sand has been done in terms of friction angle \( \phi \), lateral subgrade modulus \( k \), and unit weight \( w \).

Table 3.1 Properties of Clay Soils

<table>
<thead>
<tr>
<th>CLAYS</th>
<th>Su (Kpa)</th>
<th>( \varepsilon_{50} )</th>
<th>( \varepsilon_{100} )</th>
<th>( w ) (kN/m3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay-20</td>
<td>20</td>
<td>0.02</td>
<td>0.06</td>
<td>16</td>
</tr>
<tr>
<td>Clay-40</td>
<td>40</td>
<td>0.015</td>
<td>0.06</td>
<td>17</td>
</tr>
</tbody>
</table>

Table 3.2 Properties of Sand Soils

<table>
<thead>
<tr>
<th>SANDS</th>
<th>( \phi )</th>
<th>k (kN/m3)</th>
<th>( w ) (kN/m3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand-30</td>
<td>30</td>
<td>5500</td>
<td>16.7</td>
</tr>
<tr>
<td>Sand-34</td>
<td>34</td>
<td>16600</td>
<td>17.6</td>
</tr>
<tr>
<td>Sand-37</td>
<td>37</td>
<td>33200</td>
<td>18.5</td>
</tr>
</tbody>
</table>

The properties of the soils were taken from Reese and Impe (2001) as recommended values to use with Matlock’s (1970) P-y model for soft clays and the Reese’s (1974) P-y model for sands.
3.2 Group Effects

When a group of piles is pushed laterally, is it possible that the lateral stiffness of the system be less than the sum of the stiffness of individual soil-pile systems, this is due to the pile-soil-pile interaction that occurs when the separation between piles is small. For pile or column bent design, the group effects vanish for in-line piles at spacing equal to three diameters or more (Schmidt, 1985). In column bents, the spacing between columns is generally larger than three diameters of the column. Therefore the group effects were not included in this study.

3.3 Nonlinear Static Response

This section shows the results of five lateral nonlinear static analyses of a 0.90m diameter RC column embedded in soil. The column is 5.40m high and is embedded 27m. Two analyses were performed for fixed and pinned conditions with the column embedded in clay and two analyses were performed for fixed and pinned conditions with the column embedded in sand. Both MultiPier and OpenSees were used for each analysis. The primary purpose of these analyses was to gain experience in the use and to verify the capabilities of OpenSees.

In both MultiPier and OpenSees the column is modeled as a series of column elements and the soil is modeled as nonlinear springs attached to the connections between the column elements. Both soil and column elements are modeled using nonlinear formulations. So the first step before running the soil-column lateral analysis was to verify that both
programs are accounting for a similar section response for the column. This was done by conducting analyses with both programs in which an increasing moment was applied at the end of a cantilever column element with length equal to one. The rotation of the free node is the curvature that corresponds to the applied moment.

Figure 3.1 shows the moment curvature response obtained by both programs. MultiPier requires the user to input the material properties, reinforcement, and geometry, and defines a fiber section. Thus the response shown in Figure 3.1 is nonlinear and smooth. In OpenSees the user can choose the type of formulation to use for the column elements, the sections can be defined either as fiber sections or the user can input a section response model. For these analyses, once the section response was found by MultiPier, it was converted into a bilinear model and entered in OpenSees.

![M_C RESPONSE 0.90m DIAMETER RC COLUMN](image)

**Figure 3.1** Moment-Curvature response of a 0.90m diameter RC column
Clay was modeled in MultiPier using the built-in P-y model for soft clays under water proposed by Matlock (1970). In OpenSees the soil is modeled using the *pysimple1* developed by Boulanger (2003) element with parameters recommended to match the same clay model used in MultiPier. The sand was modeled in MultiPier using the built-in p-y model for sand proposed by Reese et al (1974). In OpenSees the soil is modeled using the *pysimple1* element and the P-y model for sands recommended by the American Petroleum Institute (API, 1987) which is built-in OpenSees and is similar to the model used in MultiPier.

Figures 3.2 and 3.3 show the lateral force deformation response of the RC column in clay for the fixed head and pinned head conditions. Figures 3.4 and 3.5 show the lateral force deformation response of the RC column in sand for the fixed head and pinned head conditions. The results from each program agree with each other so it was concluded that the modeling techniques used in OpenSees are appropriate. And therefore OpenSees can be used as an analysis tool.

Finally another nonlinear analysis was performed only with MultiPier to compare the lateral response of the RC column when the soil is modeled using the model for soft clays (Matlock 1970) and the integrated model for clays proposed by O’Neill (1984). O’Neill’s model was developed to encompass soft and stiff clays. As it can be observed in Figure 3.6 Matlock’s model is less stiff than the O’Neill’s.

Although it would be beneficial to use O’Neill’s integrated model since it encompass soft and stiff clays, this is not possible because OpenSees only models Clay using Matlock’s model and the analyses have revealed that the results of the application of both P-y models is
different. As a result of this, the present study is constrained to the implementation of DDBD for column bents in sand and soft clay only.

Figure 3.2 Lateral response of a fixed head RC column in clay

Figure 3.3 Lateral response of a pinned head RC column in clay
Figure 3.4 Lateral response of a fixed head RC column in sand

Figure 3.5 Lateral response of a pinned head RC column in sand
3.4 Nonlinear Time History Analysis (NLTH) using OpenSees

In this section, the results of two NLTH analyses are presented. The analyses consisted of the application of an earthquake acceleration record to a fixed head RC column with the soft clay model. The RC column is 0.60m in diameter and 3.6m high. Figure 3.7 shows two cycles of the moment curvature response of the column. This was achieved by using the Hysteretic Bilinear (McKenna F. et al, 2004) model built-in to OpenSees with pinching coefficients of 0.7 for curvature and 0.2 for moment. The soil was modeled using the pysimple1 elements described previously with the referenced parameters to match Matlock’s p-y model for soft clays.
In both analyses the acceleration record shown in Figure 3.8 was applied without amplification first and then with an amplification factor of 6 for the second analysis. This accelerogram was measured during the Imperial Valley earthquake in 1979 at the Station 5053.

Figure 3.7 Hysteretic moment curvature response of a 0.60m diameter RC column

Figure 3.8 Acceleration record. Imperial Valley Earthquake, station 5053, 1979
Figure 3.9  a) Displacement history  b) Shear history  c) Moment history of a 0.6 m diameter RC column
The results in terms of displacement history, shear force history and moment history are shown in Figure 3.9 for both amplification values. The hysteretic force-deformation response for both analyses is also shown in Figure 3.10. The small loops in Figure 3.10.b correspond to the energy dissipated by the hysteretic behavior of the soil only, since at this level of demand the column has remained elastic. The big loops in Figure 3.10.a represent the energy that has been dissipated by the soil and the column after both have yielded.

In general the results of these analyses demonstrate the capabilities of OpenSees to simulate the behavior of column-soil systems. The OpenSees code used for these analyses can be found in appendix 1.

Figure 3.10 Hysteretic force deformation response of a 0.6m D RC column in clay
4. DISPLACEMENT AND DUCTILITY MODELS

4.1 Definition of the equivalent model

As it was discussed in section 2.7, the implementation of DDBD for column bents requires the development of a model to predict the target displacement and ductility demand of columns embedded in soil. This model should account for both pinned and fixed head conditions, different types of soil, and different column heights and diameters. The location of plastic hinges must also be determined to assess the plastic deformation.

In section 2.2 several modeling approaches were described and discussed. It was explained that the equivalent cantilever models are perhaps the most promising for the implementation of DDBD since the formulation is simple and, if defined appropriately, can produce all the parameters required for the computation of seismic forces.

In this study, a variation of the equivalent cantilever model proposed by Chai (2002) (Figure 2.9) is presented. The idea behind this new model is to have a column, fixed at the base, with a length such that the base of the column coincides with the location of the underground hinge in the real column-soil system. In this manner the plastic displacement at the top of the equivalent and the soil-column system are the same (Figure 4.1). Also, the equivalent stiffness must be equal to the secant stiffness of the real column-soil system at yielding. This is attained by introducing a yield displacement coefficient. It is also assumed
that the force-deformation response of the system is bilinear and that the plastic hinge length varies linearly with the displacement ductility demand. (Figure 4.2)

In most column bents, each column exhibits free head displacement when pushed out of the bent’s plane and fixed head displacement when pushed along the plane of the bent, this is due to the flexibility of the super-to-substructure connection and to the high stiffness of the cap beam. It has been observed that the depth to the point of maximum moment and consequently the location of the underground plastic hinge goes deeper if the head restraints are changed from pinned to fixed (Budek A.M. et al, 2000). Thus, two lengths could be specified for the same column in order to match in plane and out of plane behavior. Nevertheless, it has also been observed that in a fixed head column the plastic displacement caused by an underground hinge is negligible when compared to the plastic displacement caused by the hinge at the column’s top. Therefore, the location of the underground hinge is not very important for fixed head columns, and the length used for the pinned head case could be used for the analysis in both directions.

The proposed model is aimed to predict the target displacement ($\Delta_D$) and the displacement ductility ($U_D$). To define the equivalent model, the following parameters must be determined (Figure 4.1):

- The equivalent length ($L_e$): The length from the cap beam to the expected location of the underground plastic hinge.

- A stiffness coefficient ($\alpha$): A correction factor used to match the yield displacements of the equivalent and nonlinear soil-column systems.
An initial plastic hinge length ($L_{po}$) and the slope ($S_{lp}$): Parameters that define a linear variation of plastic hinge length with respect to the displacement ductility ($U_D$) demand on the system (Figure 4.2/b). The plastic hinge length ($L_p$) relates plastic rotation to plastic curvature in a plastic hinge. And $U_D$ is the ratio between $\Delta_D$ and the displacement at yielding $\Delta_y$.

**Figure 4.1 Equivalent model for pinned and fixed head columns**
Once \( L_c \) and \( \alpha \) are determined, the yield displacement \( \Delta_y \) is found from Equation 4.1 and 4.2 for the pinned head and fixed head columns respectively. In these equations \( \phi_y \) is the yield curvature for the column section that can be calculated using Equation 4.3 (Kowalsky, 2000) where \( \varepsilon_y \) is the yield strain for the reinforcing steel and \( D \) is the diameter of the column.

\[
\Delta_y = \frac{\phi_y L_c^2}{3} \quad \text{(Pinned head columns)} \tag{4.1}
\]
\[ \Delta_y = \alpha \frac{\phi_y L_c^2}{6} \quad \text{(Fixed head columns)} \]  
\[ \phi_y = 2.45 \frac{\varepsilon_y}{\Delta} \]  

In the proposed model, the target displacement \( \Delta_D \) is defined as:

\[ \Delta_D = \Delta_y + \Delta_p \]  

Where \( \Delta_y \) is the yield displacement and \( \Delta_p \) is the plastic displacement of the column equal to:

\[ \Delta_p = \phi_p L_p L_c \]  

The plastic curvature \( \phi_p \) is related to the curvature ductility \( \mu \phi \) as:

\[ \phi_p = (\mu \phi - 1)\phi_y \]  

If \( L_p \) is defined as a linear function of the displacement ductility \( U_D \) (Figure 4.2.b);

\[ L_p = S_{lp} U_D + L_{po} \]  

And if equations 4.6 and 4.5 are replaced into Equation 4.4, an equation that relates plastic curvature \( \phi_p \) to displacement ductility \( U_D \) is found:

\[ U_D = \frac{\Delta_y + L_{po} L_c \phi_p}{\Delta_y - S_{lp} L_c \phi_p} \]  

Equation 4.8 can also be written in terms of curvature ductility \( \mu \phi \) :

\[ U_D = \frac{\Delta_y + L_{po} L_c (\mu \phi - 1)\phi_y}{\Delta_y - S_{lp} L_c (\mu \phi - 1)\phi_y} \]  

Equation 4.9 a kinematic model for ductility since it yields the displacement ductility as a function of other parameters that define the displacement response without consideration to
the stiffness of the system. For perfect elasto-plastic systems, \( L_{po} \) is constant, therefore \( S_{Lp} \) equals 0 and Equation 4.9 yields a straight line (Figure 4.3). When \( S_{Lp} \) is greater that 0 as for elasto-plastic systems with a non-zero second stiffness, the relation between \( U_D \) and \( \mu_\phi \) is nonlinear.

![Curvature vs. Displacement ductility for systems with bilinear force deformation response](image)

**Figure 4.3. Curvature vs. Displacement ductility for systems with bilinear force deformation response**

Equation 4.9 is fundamental for the application of DDBD since it allows the calculation of \( U_D \) from a chosen damaged based value of \( \mu_\phi \). The value of \( U_D \) is used to calculate the equivalent viscous damping for the system (Chapter 5) and to calculate the target displacement, \( \Delta_D \), as follows:

\[
\Delta_D = (U_D - 1) \Delta_y
\]  

(4.10)
An evaluation of equation 4.9 is presented in Section 4.3 based on the results of detailed nonlinear lateral static analyses.

4.2 Parametric study.

This study aims to provide the parameters required to calculate the target displacement and ductility demand using the equivalent model described in the previous section. A series of parametric analyses have been performed to find trends to predict the location of the underground plastic hinge, yield displacement and plastic hinge length in soil-column systems.

4.2.1 Procedure

Figure 4.4. Bilinear Moment-Curvature diagram for column response
The study consisted in performing a series of nonlinear static analyses. In each analysis an incremental lateral load was applied at the top of a RC column embedded in sand or clay. The column-soil system was modeled using a bilinear moment curvature response for the column elements and a nonlinear P-y model for the soil springs. The length of the column elements was set to one quarter of the diameter of the column. Figure 4.4 depicts the moment curvature response assigned to the column elements. E is the elastic modulus of concrete. The cracked moment of inertia ($I_{cr}$) of the column’s section was taken as 50% of the gross moment of inertia to account for cracking. This reduction factor was taken from recommendations of Caltrans (2004) for concrete columns with 2% reinforcement ratio and subjected to an axial load equivalent to 20% the capacity of the section. The yield curvature ($\phi_y$) is obtained from Equation 4.3. and the yield moment ($M_y$) from Equation 4.11. The curvature ductility demand ($\mu_{\phi}$) at any point after yield can be also computed with equation 4.12 where the maximum moment in the column is $M$.

$$M_y = 0.5EI\phi_y$$  \hspace{1cm} (4.11)

$$\mu_{\phi} = \frac{M - M_y}{rEI} + \frac{\phi_y}{\phi_y}$$  \hspace{1cm} (4.12)

The general configuration of the soil-column model was varied for each analysis and included: pinned or fixed head condition for the column, diameters of the column ranging from 0.3 m to 2.4 m, height of the column ranging from 2 to 10 column diameters and five different soil types. In all cases the embedded length of the column was set long enough so
the displacement at the column tip was negligible. The properties of the soils that were used are summarized in Table 4.1. Table 4.2 shows the parametric matrix used for the analyses.

**Table 4.1. Definition of soil parameters**

<table>
<thead>
<tr>
<th>CLAYS</th>
<th>Su (Kpa)</th>
<th>e&lt;sub&gt;50&lt;/sub&gt;</th>
<th>w (kN/m&lt;sup&gt;3&lt;/sup&gt;)</th>
<th>P-y model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay-20</td>
<td>20</td>
<td>0.02</td>
<td>16</td>
<td>Matlock</td>
</tr>
<tr>
<td>Clay-40</td>
<td>40</td>
<td>0.015</td>
<td>17</td>
<td>Matlock</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SANDS</th>
<th>φ</th>
<th>k (kN/m&lt;sup&gt;3&lt;/sup&gt;)</th>
<th>w (kN/m&lt;sup&gt;3&lt;/sup&gt;)</th>
<th>P-y model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand-30</td>
<td>30</td>
<td>5500</td>
<td>16.7</td>
<td>API</td>
</tr>
<tr>
<td>Sand-34</td>
<td>34</td>
<td>16600</td>
<td>17.6</td>
<td>API</td>
</tr>
<tr>
<td>Sand-37</td>
<td>37</td>
<td>33200</td>
<td>18.5</td>
<td>API</td>
</tr>
</tbody>
</table>

**Table 4.2. Parametric Matrix for Pushover Analyses**

<table>
<thead>
<tr>
<th>HEAD</th>
<th>D (m)</th>
<th>L&lt;sub&gt;ω&lt;/sub&gt;/D</th>
<th>Soils</th>
</tr>
</thead>
<tbody>
<tr>
<td>PINNED</td>
<td>0.3</td>
<td>2</td>
<td>Clay-20</td>
</tr>
<tr>
<td></td>
<td>0.6</td>
<td>4</td>
<td>Clay-40</td>
</tr>
<tr>
<td></td>
<td>0.9</td>
<td>6</td>
<td>Sand-30</td>
</tr>
<tr>
<td></td>
<td>1.2</td>
<td>8</td>
<td>Sand-34</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td>10</td>
<td>Sand-37</td>
</tr>
<tr>
<td></td>
<td>1.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Number of combinations: 350

The program OpenSees was used to perform the analyses. In each analysis the lateral load was applied with small increments until the analysis failed to converge. For each increment in lateral load, the following parameters were recorded: top displacement, applied force at column’s top, internal forces in the elements above ground and internal forces in the
elements below ground in a depth of ten pile diameters. Then, using post-processing, the following parameters were found for each system:

- Yield displacement: as the top displacement at which the maximum moment along the column reached the value of yield moment calculated from Equation 4.11
- The location of the underground point of maximum moment and the location of the point of inflection for fixed-head columns only: by looking at the moment pattern along the column.
- The curvature ductility demand at each point after yield: from Equation 4.12
- The plastic hinge length: from Equation 4.13 where $\Delta$ and $\phi$ are the top displacement and hinge curvature at the point in the force-deformation response at which $L_p$ is being calculated.

$$L_p = \frac{\Delta - \Delta_y}{L_e (\phi - \phi_y)}$$  \hspace{1cm} (4.13)

4.1.2 Results

By performing the analyses it was found that, for each soil type, there is a linear fit between the equivalent length normalized by the diameter of the column ($L_e/D$) and the normalized above-ground-height of the column ($L_a/D$). It is observed that $L_e$ goes deeper for clays than
for sands within the range of soil parameters considered. Figure 4.5 shows the data for columns in sand while Figure 4.6 shows the data for columns in clay. Table 4.3 presents the best fit equations with the corresponding R-squared values.

The correlations for \( L_e \) are good, better for clay than for sand. These correlations are very important since allow the assessment of the underground plastic hinge location very easily based on information known at the early stages of design.

A trend was also identified the parameter \( \alpha \). Figures 4.7 and 4.8 show the data for fixed head columns in clay and sand respectively, while Figures 4.9 and 4.10 show the data for pinned head columns in clay and sand respectively. The trends that were identified are independent for each soil type and are summarized in Table 4.4 for fixed head columns and in Table 4.5 for pinned head tables.

![Figure 4.5. Le/D as a function of La/D for Columns in Sand](image)

Figure 4.5. \( \text{Le/D as a function of La/D for Columns in Sand} \)
Figure 4.6. Le/D as a function of La/D for Columns in Clay

Table 4.3. Trends for Le/D for Columns in soft clay and sand

<table>
<thead>
<tr>
<th>SOIL TYPE</th>
<th>TREND</th>
<th>R-squared</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay-20</td>
<td>$\frac{L_c}{D} = 6.38 + 0.69\left(\frac{L_a}{D}\right)$</td>
<td>0.98</td>
</tr>
<tr>
<td>Clay-40</td>
<td>$\frac{L_c}{D} = 4.96 + 0.71\left(\frac{L_a}{D}\right)$</td>
<td>0.99</td>
</tr>
<tr>
<td>Sand-30</td>
<td>$\frac{L_c}{D} = 4.39 + 0.82\left(\frac{L_a}{D}\right)$</td>
<td>0.84</td>
</tr>
<tr>
<td>Sand-37</td>
<td>$\frac{L_c}{D} = 3.40 + 0.84\left(\frac{L_a}{D}\right)$</td>
<td>0.91</td>
</tr>
</tbody>
</table>
Figure 4.7. $\alpha$ as a function of La/D for Fixed Head Columns in Clay

Figure 4.8. $\alpha$ as a function of La/D for Fixed Head Columns in Sand
Figure 4.9. $\alpha$ as a function of $La/D$ for Pinned Head Columns in Clay

Figure 4.10. $\alpha$ as a function of $La/D$ for Pinned Head Columns in Sand
Table 4.4. Trends for $\alpha$ for fixed head Columns in sand and soft clay

<table>
<thead>
<tr>
<th>SOIL TYPE</th>
<th>TREND</th>
<th>R-squared</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay-20</td>
<td>$\alpha = 2.84 - 0.38 \ln \left( \frac{L_u}{D} \right)$</td>
<td>0.90</td>
</tr>
<tr>
<td>Clay-40</td>
<td>$\alpha = 2.68 - 0.33 \ln \left( \frac{L_u}{D} \right)$</td>
<td>0.90</td>
</tr>
<tr>
<td>Sand-30</td>
<td>$\alpha = 1.88 - 0.16 \ln \left( \frac{L_u}{D} \right)$</td>
<td>0.72</td>
</tr>
<tr>
<td>Sand-37</td>
<td>$\alpha = 1.86 - 0.18 \ln \left( \frac{L_u}{D} \right)$</td>
<td>0.88</td>
</tr>
</tbody>
</table>

Table 4.5. Trends for $\alpha$ for Pinned Head Columns in sand and soft clay

<table>
<thead>
<tr>
<th>SOIL TYPE</th>
<th>TREND</th>
<th>R-squared</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay-20</td>
<td>$\alpha = 5.52 - 1.09 \ln \left( \frac{L_u}{D} \right)$</td>
<td>0.95</td>
</tr>
<tr>
<td>Clay-40</td>
<td>$\alpha = 5.30 - 1.08 \ln \left( \frac{L_u}{D} \right)$</td>
<td>0.96</td>
</tr>
<tr>
<td>Sand-30</td>
<td>$\alpha = 3.56 - 0.67 \ln \left( \frac{L_u}{D} \right)$</td>
<td>0.95</td>
</tr>
<tr>
<td>Sand-37</td>
<td>$\alpha = 3.41 - 0.69 \ln \left( \frac{L_u}{D} \right)$</td>
<td>0.96</td>
</tr>
</tbody>
</table>
The parameter $\alpha$ is larger for pinned head columns, and it is also larger for clays. The parameter $\alpha$ is the ratio between the yield displacement of a cantilever column with length $L_e$ and the yield displacement of the same column embedded in soil. The two yield displacements are different since the rotation at the underground point of maximum moment is not zero for the column-soil system and because the area inside the curvature diagram of the column-soil system from the point of maximum moment up is bigger than the corresponding for the cantilever column. Therefore it can be expected a higher value of $\alpha$ for the pinned head columns, since the underground rotation at the point of maximum moment is less that the corresponding for a fixed head column. $\alpha$ is also higher for clay because sands increase in strength with depth whereas clays do not, therefore less rotation below the point of maximum moment is expected for sands.

The post yielding stage was also examined and linear trends were found for $S_{lp}$. The trends are presented for each of the soil type and head restraint condition. However, no well defined trend was found for $L_{po}$ and due to the small scatter only average values are shown. The data has been plot in Figures 4.11 and 4.12 for fixed head columns in clay and sand respectively and in Figures 4.13 and 4.14 for pinned head columns in clay and sand respectively. Equations for the trends and average values of $L_{po}$ can be found in Table 4.6 and Table 4.7 for fixed and pinned columns respectively.
Figure 4.11. $S_p/D$ as a function of $L_a/D$ for Fixed Head Columns in Clay

Figure 4.12. $S_p/D$ as a function of $L_a/D$ for Fixed Head Columns in Sand
Figure 4.13.  $S_{lp}/D$ as a function of $La/D$ for Free Head Columns in Clay

Figure 4.14.  $S_{lp}/D$ as a function of $La/D$ for Free Head Columns in Sand
Table 4.6. Trends for $S_p/D$ for Fixed Head Columns in soft clay and sand

<table>
<thead>
<tr>
<th>SOIL TYPE</th>
<th>TREND</th>
<th>R-squared</th>
<th>Average $L_{po}/D$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay-20</td>
<td>$S_p/D = 0.0042 \frac{L_p}{D} + 0.18$</td>
<td>0.65</td>
<td>0.08</td>
</tr>
<tr>
<td>Clay-40</td>
<td>$S_p/D = 0.0076 \frac{L_p}{D} + 0.13$</td>
<td>0.87</td>
<td>0.08</td>
</tr>
<tr>
<td>Sand-30</td>
<td>$S_p/D = 0.013 \frac{L_p}{D} + 0.064$</td>
<td>0.79</td>
<td>0.07</td>
</tr>
<tr>
<td>Sand-37</td>
<td>$S_p/D = 0.015 \frac{L_p}{D} + 0.040$</td>
<td>0.81</td>
<td>0.07</td>
</tr>
</tbody>
</table>

Table 4.7. Trends for $S_p/D$ for Pinned Head Columns in soft clay and sand

<table>
<thead>
<tr>
<th>SOIL TYPE</th>
<th>TREND</th>
<th>R-squared</th>
<th>Average $L_{po}/D$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay-20</td>
<td>$S_p/D = 0.0053 \frac{L_p}{D} + 0.55$</td>
<td>0.06</td>
<td>1.9</td>
</tr>
<tr>
<td>Clay-40</td>
<td>$S_p/D = 0.0053 \frac{L_p}{D} + 0.41$</td>
<td>0.31</td>
<td>1.9</td>
</tr>
<tr>
<td>Sand-30</td>
<td>$S_p/D = 0.0102 \frac{L_p}{D} + 0.14$</td>
<td>0.82</td>
<td>1.5</td>
</tr>
<tr>
<td>Sand-37</td>
<td>$S_p/D = 0.0116 \frac{L_p}{D} + 0.10$</td>
<td>0.91</td>
<td>1.5</td>
</tr>
</tbody>
</table>
The trends that were found for $L_{po}$ and $S_{lp}$ define the plastic hinge length so the equivalent model matches the theoretic plastic displacement of the nonlinear soil-column model as it will be shown in the next section. In the literature review were described a few existing models for underground plastic hinge length (Budek et al, 2000) (Chai et al, 2002). In these models the plastic hinge length is considered constant with respect to the displacement ductility. This assumption is theoretical valid for a perfect elastic plastic system, and might be appropriate for columns embedded in stiff soils where the force deformation response follows that pattern.

The application of DDBD as described in Section 2.7 yields a design base shear ($V$) for a column-soil system. To design the column section however, it is necessary to translate $V$ into a design moment for the column ($M_u$). Figure 4.15 shows a force diagram for a pinned head column. The applied force $V$ at the top of the column, is resisted by the soil from the ground down to the point of maximum moment. The maximum moment in the pile $M_u$ is given by Equation 4.14.

$$M_u = V(L_c - \beta(L_c - L_a))$$  \hspace{2cm} (4.14)

The coefficient $\beta$ has been calculated from the results of the parametric study as shown in the following table:
Table 4.8 β values for calculating $M_u$ in pinned-head columns

<table>
<thead>
<tr>
<th>SOIL</th>
<th>β</th>
<th>Standard Deviation σ</th>
<th>β-2σ</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLAY</td>
<td>0.40</td>
<td>0.03</td>
<td>0.33</td>
</tr>
<tr>
<td>SAND</td>
<td>0.32</td>
<td>0.03</td>
<td>0.26</td>
</tr>
</tbody>
</table>

To calculate the maximum moment in fixed head columns once the base shear $V$ is known, it is necessary to know the length from the top of the column to the inflection point ($L_i$), then $M_u$ can be calculated from Equation 4.15. Values of $L_i$ have been found from the parametric analysis and are summarized in Table 4.9

$$M_u = VL_i$$  \hspace{0.5cm} (4.15)
4.3 Evaluation of the proposed models

The applicability of the proposed models is limited to the range of the parameters used in the parametric study. Verification analyses were conducted by comparing with results of nonlinear lateral static analyses. Each of these nonlinear analyses involved the application of an incremental static load at the top of a 0.9 m diameter RC column with 5.4 m in height above ground (Table 4.10). The column was considered embedded in four different soil types and the analyses were performed with pinned and fixed head conditions.

<table>
<thead>
<tr>
<th>Analysis No</th>
<th>Height (m)</th>
<th>Diameter (m)</th>
<th>Head Restraints</th>
<th>EI (kN.m²)</th>
<th>Soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.4</td>
<td>0.9</td>
<td>Fixed</td>
<td>437548</td>
<td>Clay-20</td>
</tr>
<tr>
<td>2</td>
<td>5.4</td>
<td>0.9</td>
<td>Fixed</td>
<td>437548</td>
<td>Clay-40</td>
</tr>
<tr>
<td>3</td>
<td>5.4</td>
<td>0.9</td>
<td>Fixed</td>
<td>437548</td>
<td>Sand-30</td>
</tr>
<tr>
<td>4</td>
<td>5.4</td>
<td>0.9</td>
<td>Fixed</td>
<td>437548</td>
<td>Sand-37</td>
</tr>
<tr>
<td>5</td>
<td>5.4</td>
<td>0.9</td>
<td>Pinned</td>
<td>437548</td>
<td>Clay-20</td>
</tr>
<tr>
<td>6</td>
<td>5.4</td>
<td>0.9</td>
<td>Pinned</td>
<td>437548</td>
<td>Clay-40</td>
</tr>
<tr>
<td>7</td>
<td>5.4</td>
<td>0.9</td>
<td>Pinned</td>
<td>437548</td>
<td>Sand-30</td>
</tr>
<tr>
<td>8</td>
<td>5.4</td>
<td>0.9</td>
<td>Pinned</td>
<td>437548</td>
<td>Sand-37</td>
</tr>
</tbody>
</table>
4.2.1 Results of Nonlinear Analyses

The analyses were performed using OpenSees with models of similar characteristics to those described in Section 3.2. The force deformation response for analyses 1 to 4 is presented in Figure 4.16/a and for analyses 5 to 8 in Figure 4.16/b.

Figure 4.16 makes evident the higher strength and stiffness of the fixed head columns. Figure 4.16 also shows that the yield displacement of the system depends on the strength and stiffness of the soil. With the soil models and strength parameters used in this study, Clay-20 seems to provide the least stiffness while Sand-37 the biggest. The response of the fixed head columns noticeably changes in stiffness, these corresponds to the formation of the top hinge and then the underground hinge. It can also be noticed that the hinges develop at closer lateral displacement as the stiffness of the soil increases. For the fixed head cases, the yield displacement is taken as the lateral displacement that causes the development of the top hinge.

The location of the underground point of maximum moment is presented for the fixed head cases in Figure 4.17/a and for the pinned head cases in Figure 4.17/b. As it was explained in Section 4.1, the location of the maximum underground moment is slightly deeper for fixed head columns. In general the point of maximum moment tends to go deeper as the lateral load increases, then becomes stable after yielding of the column.

Figure 4.18 shows the variation of plastic hinge length with respect to the displacement ductility demand on the system. The plastic hinge length was calculated from Equation 4.6. The values of $L_c$ were taken from Figure 4.17/b. Remember, in Section 4.1 was
agreed that the depth to maximum moment for pinned head columns will be used also for the fixed head case.

![Force-Deformation Response. Fixed Head Columns](image1)

![Force-Deformation Response. Pinned Head Columns](image2)

Figure 4.16. Force-Deformation response a) Fixed Head b) Pinned Head

The relation between curvature ductility demand and displacement ductility demand for the eight analyses is presented in Figure 14.19/a for the fixed head cases and in Figure 4.20/b for
the pinned head cases. It is observed that the relation between curvature and displacement ductility does not depend much on the soil type.

Figure 4.17. Location of point of maximum moment underground a) Fixed Head b) Pinned Head
Figure 4.18. Plastic hinge length  a) Fixed Head b) Pinned Head
Figure 4.19. Displacement Ductility vs Curvature Ductility

- a) Fixed Head
- b) Pinned Head
4.2.2 Results of the application of the proposed model

A summary of the input data, as well as the calculated parameters that define the proposed model for yield displacement and ductility is presented in Table 4.11. Values of $L_e$ and $\alpha$ were calculated from equations in Tables 4.3-4.5. The parameters $S_{lp}$ and $L_{po}$ that define the plastic hinge length were calculated from equations in Tables 4.6 and 4.7. Table 4.11 also shows the predicted values for yield displacement $\Delta_y$, these were calculated with Equations 4.1 and 4.2 for fixed head columns and pinned head columns respectively.

Table 4.11. Summary of input data and results of proposed model.

<table>
<thead>
<tr>
<th>Ana. No.</th>
<th>$L_s$ (m)</th>
<th>$D$ (m)</th>
<th>Head</th>
<th>$EI$ (kN.m²)</th>
<th>Soil</th>
<th>$L_e$ (m)</th>
<th>$\alpha$</th>
<th>$S_{lp}$ (m)</th>
<th>$L_{po}$ (m)</th>
<th>$\Delta_y$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.4</td>
<td>0.9</td>
<td>Fixed</td>
<td>437548</td>
<td>Clay-20</td>
<td>9.5</td>
<td>2.15</td>
<td>0.19</td>
<td>0.07</td>
<td>0.17</td>
</tr>
<tr>
<td>2</td>
<td>5.4</td>
<td>0.9</td>
<td>Fixed</td>
<td>437548</td>
<td>Clay-40</td>
<td>8.3</td>
<td>2.09</td>
<td>0.16</td>
<td>0.07</td>
<td>0.12</td>
</tr>
<tr>
<td>3</td>
<td>5.4</td>
<td>0.9</td>
<td>Fixed</td>
<td>437548</td>
<td>Sand-30</td>
<td>8.4</td>
<td>1.59</td>
<td>0.13</td>
<td>0.06</td>
<td>0.10</td>
</tr>
<tr>
<td>4</td>
<td>5.4</td>
<td>0.9</td>
<td>Fixed</td>
<td>437548</td>
<td>Sand-37</td>
<td>7.6</td>
<td>1.54</td>
<td>0.12</td>
<td>0.06</td>
<td>0.08</td>
</tr>
<tr>
<td>5</td>
<td>5.4</td>
<td>0.9</td>
<td>Pinned</td>
<td>437548</td>
<td>Clay-20</td>
<td>9.5</td>
<td>3.56</td>
<td>0.52</td>
<td>1.71</td>
<td>0.55</td>
</tr>
<tr>
<td>6</td>
<td>5.4</td>
<td>0.9</td>
<td>Pinned</td>
<td>437548</td>
<td>Clay-40</td>
<td>8.3</td>
<td>3.36</td>
<td>0.40</td>
<td>1.71</td>
<td>0.39</td>
</tr>
<tr>
<td>7</td>
<td>5.4</td>
<td>0.9</td>
<td>Pinned</td>
<td>437548</td>
<td>Sand-30</td>
<td>8.4</td>
<td>2.36</td>
<td>0.18</td>
<td>1.35</td>
<td>0.28</td>
</tr>
<tr>
<td>8</td>
<td>5.4</td>
<td>0.9</td>
<td>Pinned</td>
<td>437548</td>
<td>Sand-37</td>
<td>7.6</td>
<td>2.17</td>
<td>0.15</td>
<td>1.35</td>
<td>0.21</td>
</tr>
</tbody>
</table>

The plastic hinge length as a function of ductility is plotted in Figure 4.20 based on the $S_{lp}$ and $L_{po}$ values shown in Table 4.11 for analyses 1, 4, 5 and 8. Also for these analyses, the relation between curvature and displacement ductilities that results from the application of Equation 4.9 is plotted on Figure 4.21.
Figure 4.20. Predicted plastic hinge length
Figure 4.21. Comparison between predicted and calculated displacement ductility
4.2.3 Results from application of other equivalent models

The equations proposed by Davison and Robinson (1976) and presented in Section 2.2.4 were also used to determine $\Delta_y$ for the eight case study analyses. Table 4.12 presents the input data that was used and the predicted $\Delta_y$. $\Delta_y$ was calculated using Equations 4.1 for fixed head and 4.2 for pinned head with $L_e$ equal to “depth to fixity” values obtained from Equations 2.1 and 2.2 plus the above ground height of the columns ($L_a$).

Table 4.12. Depth to fixity and Yield displacement using Davison and Robinson’s model.

<table>
<thead>
<tr>
<th>Ana. No.</th>
<th>Head</th>
<th>Soil</th>
<th>$L_e$ (m)</th>
<th>$\alpha$</th>
<th>$\Delta_y$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Fixed</td>
<td>Clay-20</td>
<td>9.5</td>
<td>2.15</td>
<td>0.17</td>
</tr>
<tr>
<td>2</td>
<td>Fixed</td>
<td>Clay-40</td>
<td>8.3</td>
<td>2.09</td>
<td>0.12</td>
</tr>
<tr>
<td>3</td>
<td>Fixed</td>
<td>Sand-30</td>
<td>8.4</td>
<td>1.59</td>
<td>0.10</td>
</tr>
<tr>
<td>4</td>
<td>Fixed</td>
<td>Sand-37</td>
<td>7.6</td>
<td>1.54</td>
<td>0.08</td>
</tr>
<tr>
<td>5</td>
<td>Pinned</td>
<td>Clay-20</td>
<td>9.5</td>
<td>3.56</td>
<td>0.55</td>
</tr>
<tr>
<td>6</td>
<td>Pinned</td>
<td>Clay-40</td>
<td>8.3</td>
<td>3.36</td>
<td>0.39</td>
</tr>
<tr>
<td>7</td>
<td>Pinned</td>
<td>Sand-30</td>
<td>8.4</td>
<td>2.36</td>
<td>0.28</td>
</tr>
<tr>
<td>8</td>
<td>Pinned</td>
<td>Sand-37</td>
<td>7.6</td>
<td>2.17</td>
<td>0.21</td>
</tr>
</tbody>
</table>

The yield displacement was also predicted for the pinned head analyses in Sand-37 using the charts for “depth to fixity” proposed by Budek et al. (2000) and presented as Figures 2.3 and 2.4. The input data and results are presented in Table 4.13

Table 4.13. Depth to Fixity and Yield Displacement using Budek et al’s model.

<table>
<thead>
<tr>
<th>Ana.No.</th>
<th>Head</th>
<th>Soil</th>
<th>$K$ (kN/m3)</th>
<th>$1000KD^0/\text{D}^*\text{El}_{\text{eff}}$</th>
<th>Depth to fixity (m)</th>
<th>$\Delta_y$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>Fixed</td>
<td>Sand-37</td>
<td>33200</td>
<td>22.0352</td>
<td>3.51</td>
<td>0.07</td>
</tr>
<tr>
<td>8</td>
<td>Pinned</td>
<td>Sand-37</td>
<td>33200</td>
<td>22.0352</td>
<td>3.78</td>
<td>0.14</td>
</tr>
</tbody>
</table>
4.2.4 Comparison with nonlinear analysis results

The predicted values of yield displacement are compared to the results of nonlinear analysis in Table 4.14. It can be observed that the values predicted by the proposed equivalent model were in good agreement with the results of the nonlinear analysis.

The displacement ductility predicted by the new equivalent model for analyses 1, 4, 5 and 8 was also in good agreement with the theoretical values predicted by the nonlinear analyses, as can be observed in Figure 4.21.

Table 4.14. Comparison of results of nonlinear and equivalent models

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Fixed</td>
<td>Clay-20</td>
<td>0.17</td>
<td>----</td>
<td>0.10</td>
<td>0.17</td>
</tr>
<tr>
<td>2</td>
<td>Fixed</td>
<td>Clay-40</td>
<td>0.12</td>
<td>----</td>
<td>0.09</td>
<td>0.12</td>
</tr>
<tr>
<td>3</td>
<td>Fixed</td>
<td>Sand-30</td>
<td>0.10</td>
<td>----</td>
<td>0.09</td>
<td>0.10</td>
</tr>
<tr>
<td>4</td>
<td>Fixed</td>
<td>Sand-37</td>
<td>0.08</td>
<td>0.07</td>
<td>0.07</td>
<td>0.08</td>
</tr>
<tr>
<td>5</td>
<td>Pinned</td>
<td>Clay-20</td>
<td>0.55</td>
<td>----</td>
<td>0.21</td>
<td>0.58</td>
</tr>
<tr>
<td>6</td>
<td>Pinned</td>
<td>Clay-40</td>
<td>0.39</td>
<td>----</td>
<td>0.18</td>
<td>0.39</td>
</tr>
<tr>
<td>7</td>
<td>Pinned</td>
<td>Sand-30</td>
<td>0.28</td>
<td>----</td>
<td>0.19</td>
<td>0.30</td>
</tr>
<tr>
<td>8</td>
<td>Pinned</td>
<td>Sand-37</td>
<td>0.21</td>
<td>0.14</td>
<td>0.13</td>
<td>0.21</td>
</tr>
</tbody>
</table>
4.2.5 Comparison with experimental results

The new equivalent model was used to predict the yield displacement and ductility for a 0.4 m diameter RC column embedded in sand. This column was tested under lateral loading as part of an experimental program conducted by Chai and Hutchison (2002).

The test involved the application of a cyclic lateral load at the top of a 0.4 m diameter RC column embedded in sand with a friction angle of 37°. The column head was free and the above ground height was 6 times the diameter. The input parameters were taken from the testing report (Chai and Hutchison, 2002) and are summarized in Table 4.15. This table also includes the parameters that were calculated for the application of the proposed equivalent model and the predicted yield displacement for the system.

Table 4.15. Input data and model parameters for 0.4 m diameter column in sand.

<table>
<thead>
<tr>
<th>$L_a$ (m)</th>
<th>D (m)</th>
<th>Head</th>
<th>$E_I$ (kN.m$^2$)</th>
<th>Soil</th>
<th>$L_a$ (m)</th>
<th>$\alpha$</th>
<th>$\Delta y$ (m)</th>
<th>$S_{lp}$ (m)</th>
<th>$L_{po}$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.44</td>
<td>0.406</td>
<td>Pinned</td>
<td>16323.42</td>
<td>Sand-37</td>
<td>3.43</td>
<td>2.17</td>
<td>0.11</td>
<td>0.07</td>
<td>0.61</td>
</tr>
</tbody>
</table>

The predicted yield displacement was found to coincide with the value obtained during the test. The relationship between displacement and curvature ductility predicted by the new model was also in good agreement with the experimental values (Figure 4.22)
This concludes the discussion on the development of a new model to predict target displacement and ductility in soil-column systems. Next chapter will present the result of another parametric study conducted to assess the equivalent viscous damping for this type of systems.
5. EQUIVALENT VISCOUS DAMPING

The DDBD method is based on the substitution of a nonlinear system for an equivalent single-degree-of-freedom system with effective mass, effective period and Equivalent Viscous Damping (EVD). The application of DDBD to column bents a simple problem, since a column bent is basically a SDOF structure. The model derived in Chapter 4 yields an equivalent SDOF with known mass, target displacement and ductility demand. The next step is to find the EVD.

In this chapter, the concept of EVD that was presented in Section 2.8 is extended to column bents. To do so, a parametric study has been conducted to identify trends that relate the amount of displacement ductility imposed on a column-soil system to the level of equivalent viscous damping for different types of soil.

In column-soil systems EVD comes from two sources: The hysteretic behavior of soil, and the hysteretic behavior of the plastic hinges. Viscous damping is not considered since it is difficult to justify once the hysteretic behavior takes place in the soil-column system. The hysteretic behavior in the soil can be observed even at small displacements of the column head, while the hysteretic behavior in the column itself is only significant once a plastic hinges develops. The total EVD related to a certain level of ductility in the system is therefore a combination of the EVD generated in the soil and in the column. In this research however, the effort has been put into developing a model for the total EVD in the system rather to study the sources of damping separately.
5.1 Determination of EVD for pinned of fixed head soil-column systems.

To determine EVD for a nonlinear column-soil system it is required to run a series of NTHA. In each analysis, a different earthquake acceleration record is applied, the maximum response is found, then the corresponding EVD is found as the viscous damping of a SDOF system that has the same effective period and reaches the same maximum displacement.

The start point in the evaluation of EVD is to select a set of acceleration records and obtain for each record a set of displacement response spectra curves for several values of viscous damping. Ten records from stations located on soft soils (Table 5.1) were chosen from a data base (Miranda, 2003) and displacement response spectra curves were found for each acceleration record at 26 different levels of viscous damping. From 0 to 60%

Table 5.1. Description of acceleration records used in the study

<table>
<thead>
<tr>
<th>Label</th>
<th>Date</th>
<th>Earthquake Name</th>
<th>Magnitude (Ms)</th>
<th>Station Name</th>
<th>PGA (cm/s²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EQ1</td>
<td>1/17/1994</td>
<td>Northridge</td>
<td>6.8</td>
<td>Santa Monica City Hall</td>
<td>866</td>
</tr>
<tr>
<td>EQ2</td>
<td>10/17/1989</td>
<td>Loma Prieta</td>
<td>7.1</td>
<td>Gilroy 3, Sewage Treatment Plant</td>
<td>532</td>
</tr>
<tr>
<td>EQ3</td>
<td>10/15/1979</td>
<td>Imperial Valley</td>
<td>6.8</td>
<td>Calexico, Fire Station</td>
<td>270</td>
</tr>
<tr>
<td>EQ4</td>
<td>4/24/1984</td>
<td>Morgan Hill</td>
<td>6.1</td>
<td>Gilroy #4, 2905 Anderson Rd</td>
<td>341</td>
</tr>
<tr>
<td>EQ5</td>
<td>10/15/1979</td>
<td>Imperial Valley</td>
<td>6.8</td>
<td>Calexico, Fire Station</td>
<td>270</td>
</tr>
<tr>
<td>EQ6</td>
<td>6/28/1992</td>
<td>San Fernando</td>
<td>6.5</td>
<td>Los Angeles, Hollywood Storage Bldg.</td>
<td>207</td>
</tr>
<tr>
<td>EQ7</td>
<td>10/1/1987</td>
<td>Yermo, Fire Station</td>
<td>7.5</td>
<td>Los Angeles, 116th St School</td>
<td>240</td>
</tr>
<tr>
<td>EQ8</td>
<td>6/28/1992</td>
<td>Landers</td>
<td>7.5</td>
<td>Palm Springs, Airport</td>
<td>87</td>
</tr>
<tr>
<td>EQ9</td>
<td>10/17/1989</td>
<td>Gilroy #2, Hwy 101 Bolsa Road Motel</td>
<td>7.1</td>
<td>Gilroy 2, Hwy 101 Bolsa Road Motel</td>
<td>394</td>
</tr>
<tr>
<td>EQ10</td>
<td>4/24/1984</td>
<td>Morgan Hill</td>
<td>6.1</td>
<td>Gilroy #2, Keystone Rd.</td>
<td>208</td>
</tr>
</tbody>
</table>
5.1.1 Procedure

The detailed procedure used to find EVD as a function of displacement ductility demand for a given pinned-head column-soil system is as follows:

1. A nonlinear model was built in OpenSees with the selected geometry, soil properties, weight, and top restraints. The column was modeled as a series of nonlinear frame elements. The nonlinear response was achieved by assigning to the column elements the Hysteretic Bilinear (McKenna F et al, 2004) section response model built-in to OpenSees with pinching coefficients of 0.7 for curvature and 0.2 for moment. The soil was modeled using the *pysimple1* elements described previously with strength parameters and P-y models as shown in Table 4.1. The weight was considered lumped at the top of the column and equal to 20% of the gross axial capacity of the section. No viscous damping was assigned to the model.

2. Then, the previously discussed set of 10 earthquake acceleration records was applied to the model. For each earthquake an amplification factor that ranged from 1 to 20 with intervals of 0.5 was used. These results in 390 NTHA performed for each column soil system.

3. The analyses were run and the following data was recorded: top displacement, internal forces inside the element at the top of the column, internal forces inside the elements underground at least for a depth equal to 14 diameters of the column.
4. Then, for each analysis, the maximum top displacement ($\Delta_D$) was found from the displacement history, as well as the corresponding value of shear force ($V_u$) at the top of the column.

5. The history of the internal forces of the underground column elements was processed to find the value and location of the maximum moment ($M_u$) that developed at the same time as the maximum top displacement $\Delta_D$.

6. Steps 4 and 5 are repeated until the values of maximum response have been found for all earthquakes and all amplification factors. Not all the analyses converged. In most cases this happened due to the application of an amplification factor that was too big. The chosen values of amplification factors ranging from 1 to 20 were completely arbitrary and purposed obtaining the response at a wide range of ductility values.

7. For each analysis, and once the time history response was reduced to only the maximum response, moment curvature ductility demand ($\mu_\phi$), the secant stiffness ($K$) and effective period ($T_{eff}$) were calculated using Equations 4.12 and 5.1 and 5.2 Respectively.

\[
K_{eff} = \frac{V}{\Delta_D}
\]  

\[
T_{eff} = 2\pi \sqrt{\frac{W}{gK_{eff}}}
\]
Where $EI_{\text{eff}}$ is the product of the elastic modulus times the cracked inertia of the column section. $W$ is the weight acting on the column and $g$ is the acceleration of gravity.

8. The results were then sorted by earthquake record and then by $\mu\phi$ in ascending order. The closest value of $\mu\phi$ that was greater than 1 and the closest value less than 1 were identified. Next, $\Delta_y$ was found for both values of $\mu\phi$ then linear interpolation found $\Delta_y$ corresponding to $\mu\phi$ equal to 1. As expected, the $\Delta_y$ value found was similar for all the earthquake records, for that particular system.

9. Once the $\Delta_y$ was found, the displacement ductility demand ($U_D$) was calculated for each analysis as the ratio of $\Delta_D$ and $\Delta_y$.

10. Then for each analysis, the EVD was obtained by going to the amplified displacement spectra for the corresponding earthquake and by getting the viscous damping that corresponded to a SDOF system of period $T_{\text{eff}}$ that has a maximum displacement response of $\Delta_D$. This procedure requires interpolation of the viscous damping values to match $T_{\text{eff}}$ and $\Delta_D$.

11. The values of EVD were averaged among the 10 earthquakes records used at specific levels of displacement ductility. This step requires interpolation of the values of EVD within each record, since the amplification factors applied yield different values of ductility for each earthquake. Finally a relation of average EVD and ductility demand can be plotted for the soil-column system.
In the case of fixed head columns, the procedure used was similar to the one for pinned head columns with only one difference: since in fixed head columns the maximum moment occurs at the top, there was no need for recording the response of the underground elements, and the maximum moment was found directly from the history of moment for the top column element.

5.2 Parametric study

Once the procedure for finding a relation between EVD and ductility demand for a pinned and fixed soil-column systems was established and tested, the procedure was applied to the same parametric matrix used in Chapter 4. This matrix is presented again as Table 5.2 with the addition of the weight that was used for each system. The purpose of this study was to find a general trend that relates EVD to ductility demand for a wide range of column-soil configurations.

Table 5.2. Parametric matrix for determination of EVD

<table>
<thead>
<tr>
<th>HEAD</th>
<th>D (m)</th>
<th>Weight (kN)</th>
<th>L_a/D</th>
<th>Soils</th>
</tr>
</thead>
<tbody>
<tr>
<td>PINNED</td>
<td>0.3</td>
<td>336</td>
<td>2</td>
<td>Clay-20</td>
</tr>
<tr>
<td></td>
<td>0.6</td>
<td>1345</td>
<td>4</td>
<td>Clay-40</td>
</tr>
<tr>
<td></td>
<td>0.9</td>
<td>3028</td>
<td>6</td>
<td>Sand-30</td>
</tr>
<tr>
<td></td>
<td>1.2</td>
<td>5383</td>
<td>8</td>
<td>Sand-34</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td>8411</td>
<td>10</td>
<td>Sand-37</td>
</tr>
<tr>
<td></td>
<td>1.8</td>
<td>12112</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.4</td>
<td>21530</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Number of combinations: 350
5.2.1 Results

The results of the parametric study have been plotted in Figures 5.1 to 5.4 for fixed head columns and in Figures 5.5 to 5.8 for pinned head columns. Each chart presents the equivalent damping versus the displacement ductility for columns embedded in a specific soil type.

It was found that hyperbolic functions best fit the data for all soil types and head restraints. These equations represent the trend of variation of equivalent damping with respect to displacement ductility and are summarized in Table 5.3. The curve fitting was performed for ductility values greater than one. It can be observed in Figures 5.1-5.8 that the hyperbolic functions does not follow the trend of damping values for ductility less than one, therefore it is proposed to assume a linear variation of damping with respect to ductility from the origin to damping at ductility of one.

All the trends are presented in Figure 5.9 along with the equivalent damping model proposed by Dwairi (2004) for bridge columns supported by stiff footings or pile caps without considering the soil. It is observed that the trends for fixed head columns in sand show less equivalent damping than Dwairi’s model for ductility values greater than approximately 1.5. If required, viscous damping can be added to the equivalent damping obtained from Figure 5.9 following the recommendations of Priestley M.J. and Grant D. (2005).
Figure 5.1 Equivalent Damping for Fixed Head Columns in Clay-20

Figure 5.2 Equivalent Damping for Fixed Head Columns in Clay-40
Figure 5.3 Equivalent Damping for Fixed Head Columns in Sand-30

Figure 5.4 Equivalent Damping for Fixed Head Columns in Sand-37
Figure 5.5 Equivalent Damping for Pinned Head Columns in Clay-20

Figure 5.6 Equivalent Damping for Pinned Head Columns in Clay-40
Figure 5.7 Equivalent Damping for Pinned Head Columns in Sand-30

Figure 5.8 Equivalent Damping for Pinned Head Columns in Sand-37
Table 5.3. Hiperbolic functions for equivalent damping

<table>
<thead>
<tr>
<th>HEAD</th>
<th>SOIL</th>
<th>EQUIVALENT DAMPING %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fixed Head</td>
<td>Clay-20</td>
<td>$ED = 6.7 + 8.1 \frac{U_D - 1}{U_D}$</td>
</tr>
<tr>
<td>Fixed Head</td>
<td>Clay-40</td>
<td>$ED = 5.6 + 8.7 \frac{U_D - 1}{U_D}$</td>
</tr>
<tr>
<td>Fixed Head</td>
<td>Sand-30</td>
<td>$ED = 2.4 + 10.2 \frac{U_D - 1}{U_D}$</td>
</tr>
<tr>
<td>Fixed Head</td>
<td>Sand-37</td>
<td>$ED = 2.0 + 9.6 \frac{U_D - 1}{U_D}$</td>
</tr>
<tr>
<td>Pinned Head</td>
<td>Clay-20</td>
<td>$ED = 15.8 + 9.4 \frac{U_D - 1}{U_D}$</td>
</tr>
<tr>
<td>Pinned Head</td>
<td>Clay-40</td>
<td>$ED = 13.7 + 10.9 \frac{U_D - 1}{U_D}$</td>
</tr>
<tr>
<td>Pinned Head</td>
<td>Sand-30</td>
<td>$ED = 9.4 + 11.2 \frac{U_D - 1}{U_D}$</td>
</tr>
<tr>
<td>Pinned Head</td>
<td>Sand-37</td>
<td>$ED = 8.5 + 10.4 \frac{U_D - 1}{U_D}$</td>
</tr>
</tbody>
</table>
5.3 Evaluation of the proposed relations for EVD

It is observed in Figures 5.1-5.8 that the scatter of equivalent damping is large. Thus it is necessary to evaluate the effectiveness of the proposed relations to predict the maximum displacement of column-soil systems.

The following procedure was applied to each of the points in Figures 5.1 to 5.8 to obtain the maximum displacement using the proposed trends for equivalent damping:
1. Each point in Figure 5.1 to 5.8 corresponds to the average EVD obtained from a set of acceleration records applied to a soil-column system. For each point it is known the set of applied acceleration records, the corresponding amplification factors, the effective period and displacement of the system at that particular level of ductility. So, the first step was to amplify the displacement spectrum of each of the records.

2. Then, equivalent viscous damping was found using the appropriate equation from Table 5.3

3. Then, maximum displacement is found for each acceleration record by going to the corresponding amplified displacement spectra with the period from step 1 and a damping from step 3.

4. Finally, the maximum displacement is averaged for all the records. This averaged displacement is the predicted displacement using the proposed trends for damping. The predicted displacement was then divided by the displacement given by nonlinear time history analysis (step 1) to obtain a coefficient of variation.

This procedure was applied to all the points in Figures 5.1 to 5.8 and the variation between predicted and simulated results is presented in Figure 5.10 for fixed head column and in Figure 5.11 for pinned head columns.

As a result of this process, it was found that even though the scatter was large in terms of equivalent damping, the predicted displacement is within 20% for a confidence interval of 95%.
Figure 5.10 Variation of predicted to measured maximum displacement for fixed head columns

Figure 5.11 Variation of predicted to measured maximum displacement for pinned head columns
A detailed example covering the application of DDBD using the equivalent model for displacement and ductility and the proposed model for damping is presented in the next chapter.
6. DDBD OF COLUMN BENTS

This chapter demonstrates the use of the equivalent model developed in Chapter 4 and the equivalent damping relations developed in Chapter 5 to implement DDBD for column bents in soft soils. DDBD was described in Chapter 2. The application of this method for the seismic design of column bents requires the following procedure:

1) Gather input information:
   a. Above ground height of the bent.
   b. Minimum diameter of the columns. Usually the minimum diameter is dictated by the geotechnical design.
   c. Reactive weights in the in-plane and out-of-plane directions.
   d. Soil stiffness and strength parameters.
   e. Number of columns. Usually from geotechnical design, width of the bridge and column spacing constrains.

2) Define the design objective for the in-plane and out-of-plane displacement.
   a. For each limit state, performance criteria must be specified in terms of curvature demand and/or displacement limits.
   b. For each limit state, an earthquake level is specified in terms of a displacement response spectrum.
3) Establish the Target Displacement and Displacement Ductility demand using the equivalent model (Chapter 4) for in-plane and out-of-plane response.

   a. If performance criteria is given in terms of curvature demand on column section:
      i. Determine yield displacement.
      ii. Find the curvature ductility for the specified curvature limit.
      iii. Determine displacement ductility.
      iv. Find target displacement.

   b. If performance criteria is given in terms of lateral displacement limits:
      i. Determine yield displacement
      ii. Find ductility demand

4) Determine the equivalent viscous damping using charts from Chapter 5 for in-plane and out-of-plane response.

5) From the displacement design spectra determine required effective period for in-plane and out-of-plane response.

6) Find the design base shear for in-plane and out-of-plane response.

7) Use Equations 4.14 and 4.15 to find the maximum moment in the column, and then use an interaction diagram to find the steel ratio for the combination of moment and axial load. Increase the diameter and/or the number columns in the bent until the steel ratio is within acceptable limits.
6.1 Application Example

A column bent is to be designed with the seismic performance objective presented in Table 6.1. The height of the column above ground is 6 m. The design for vertical loads requires the use of one line of three 1.3 m diameter reinforced concrete extended shafts. The compressive strength of the concrete is 28 MPa and the yield strength of the reinforced steel is 400 MPa. The soil in which the bent is embedded has been idealized as sand with a friction angle equal to 37 degrees (Figure 6.1). Each column supports a factored combined dead and live load ($W_z$) of 2000 kN. The reactive weight along the in-plane direction ($W_x$) is 2000 kN per column and the reactive weight along the out-of-plane direction ($W_y$) is 1000 kN per column.

Table 6.1 Seismic Performance Objective

<table>
<thead>
<tr>
<th>LIMIT STATE</th>
<th>PERFORMANCE LEVEL</th>
<th>CURVATURE DEMAND (1/m)</th>
<th>MAX-DISP INPLANE (m)</th>
<th>MAX-DISP OUT-OF-PLANE (m)</th>
<th>SEISMIC HAZARD</th>
</tr>
</thead>
<tbody>
<tr>
<td>SERVICEABILITY</td>
<td></td>
<td>0.01150</td>
<td>--</td>
<td>0.05</td>
<td>Peak Acc. A (g) 0.2</td>
</tr>
<tr>
<td>DAMAGE CONTROL</td>
<td></td>
<td>0.05000</td>
<td></td>
<td>MPA &lt; 0.10 Mn</td>
<td>Soil Coeff. S 2</td>
</tr>
</tbody>
</table>

Two limit states are considered as performance levels: serviceability and damage control. Serviceability implies no repairs are needed after the earthquake. Damage control implies that only repairable damage will take place. The curvature demand was determined from Equation 6.1 and 6.2 for serviceability and damage control limit states respectively (Kowalsky, 2003). In Equations 6.1 and 6.2, $P$ is the axial load acting on the column, $f'_c$ is the compressive strength of the concrete, $A_g$ is the area of the column section, and $D$ is the column diameter.
Figure 6.1 Column bent for application example. Input information

\[
\phi = \left( 0.015 - 0.020 \left( \frac{P}{f_c A_g} \right) \right) \frac{1}{D} \tag{6.1}
\]

\[
\phi = \left( 0.068 - 0.068 \left( \frac{P}{f_c A_g} \right) \right) \frac{1}{D} \tag{6.2}
\]
The performance levels have also been written in terms of displacement limits. For the serviceability limit state a maximum out-of-plane displacement of 0.05 m has been set. Also, for the damage control limit state the maximum displacement in both directions should not cause a P-Δ moment larger than ten percent of the nominal capacity of the column.

The seismic hazard for damage control limit state is the design spectra specified in the AASHTO LRFD Bridge Design Specifications (2004) in which \( A \) is the peak rock acceleration and \( S \) is the soil type coefficient. For this example, \( A \) equals 0.4 g and \( S \) equals 2 which corresponds to a soft soil. For the serviceability limit state, the peak ground acceleration was reduced by 50%.

6.1.1 Equivalent model

As explained in Section 4.1, the soil-column system is replaced by a column of equivalent length fixed at its base (Figure 4.1). The diameter (D) of the column is 1.3 m. The above ground height (\( L_a \)) of the bent is 6 m. The length of the equivalent column \( L_e \) is found from Figure 4.5 or from the equations on Table 4.3. \( L_e \) equals 9.46 m for a \( L_a/D \) ratio 4.62.

6.1.1.1 Yield displacement

The next step is to find the yield displacement of the column. The yield curvature of the section is found from Equation 4.3 and equals 0.00377 1/m. The yield displacement
coefficient \((\alpha)\) is found from Figure 4.8 for fixed head and from Figure 4.10 for pinned head. For pinned head displacement \(\alpha\) equals 2.35 and the yield displacement from Equation 4.1 is 0.26 m. For fixed head displacement \(\alpha\) equals 1.58 and the yield displacement from Equation 4.2 is 0.09 m.

6.1.1.1 Ductility

The ductility demand can be calculated directly if the target displacement and yield displacement are known. In the out-of-plane direction and for the serviceability limit state the ductility demand is 0.19 (Equation 4.10).

If the performance is given in terms of curvature limits, then the curvature ductility demand \((\mu_\phi)\) must be found first (Equation 4.12). The displacement ductility \((U_D)\) is then calculated (Equation 4.9) and the target displacement \((\Delta_D)\) is finally found from Equation 4.10. However, to calculate displacement ductility in terms of curvature ductility, it is necessary to determine the plastic hinge length as a function of displacement ductility as explained in Section 4.1. The parameters \(S_{lp}\) and \(L_{po}\) are found from Figures 4.12 and 4.14 and from Table 4.6. The calculated values of displacement ductility and target displacement are presented in Table 6.2.
Table 6.2. Ductility demand and target displacement based on curvature limits

<table>
<thead>
<tr>
<th>LIMIT STATE</th>
<th>µφ</th>
<th>Uₜ</th>
<th>ΔD (m)</th>
<th>Sᵢₜ (m)</th>
<th>Lₚₒ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN-PLANE SERVICEABILITY</td>
<td>2.84</td>
<td>1.19</td>
<td>0.11</td>
<td>0.14</td>
<td>0.09</td>
</tr>
<tr>
<td>IN-PLANE DAMAGE CONTROL</td>
<td>13.13</td>
<td>4.64</td>
<td>0.41</td>
<td>0.20</td>
<td>1.95</td>
</tr>
<tr>
<td>OUT-OF-PLANE SERVICEABILITY</td>
<td>2.84</td>
<td>1.56</td>
<td>0.41</td>
<td>0.20</td>
<td>1.95</td>
</tr>
<tr>
<td>OUT-OF-PLANE DAMAGE CONTROL</td>
<td>13.13</td>
<td>6.21</td>
<td>1.64</td>
<td>0.20</td>
<td>1.95</td>
</tr>
</tbody>
</table>

The displacement ductility values are used to calculate the equivalent viscous damping (EVD) using Figure 5.9 or Table 5.3. The calculated EVD values are presented in Table 6.3.

Table 6.3. Equivalent Damping

<table>
<thead>
<tr>
<th>EQUIVALENT DAMPING (%)</th>
<th>SERVICEABILITY</th>
<th>DAMAGE CONTROL</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN-PLANE</td>
<td>5.00</td>
<td>9.53</td>
</tr>
<tr>
<td>OUT-OF-PLANE</td>
<td>5.00</td>
<td>17.23</td>
</tr>
</tbody>
</table>

Once the equivalent damping is known, the effective period (Tₑffective) can be found from Equation 6.3. This relation was obtained from the equation of the AASHTO acceleration response spectrum. Results of the application of this equation are shown in Table 6.4. Then the base shear is found as the less value resulting from Equations 6.4 and 6.5. Results are shown in Table 6.5

\[
T_{\text{eff}} = \left( \frac{4\pi^2\Delta_D}{1.2A_s g} \sqrt{\frac{2 + ED}{7}} \right)^{75} \tag{6.3}
\]
With the design base shear force, the flexural strength of the column is found using Equations 4.14 and 4.15 for the pinned head condition and fixed head condition respectively. At this point the P-∆ moment is calculated using Equation 6.9 and 6.10 and the ratio with respect to the design moment in the column is calculated. Since the curvature limit for damage control in the out-of-plane direction yielded a top displacement that caused a P-∆ moment larger than 10% the moment capacity of the column, the target displacement limit was decreased and the process repeated until the P-∆ limit was satisfied. Then the longitudinal steel ratio (\(\rho\)) is found using a uniaxial interaction diagram. Final results are shown in Table 6.6.
\[ M_{P\alpha} = \Delta_D W_z \quad \text{Pinned head} \quad (6.9) \]

\[ M_{P\alpha} = \frac{\Delta_D}{2} W_z \quad \text{Fixed head} \quad (6.10) \]

Table 6.6 shows that the serviceability limit state for the out-of-plane response controls the design. The longitudinal steel ratio (\( \rho \)) is 2.18 and the base shear is 74% of the reactive weight on that direction.

<table>
<thead>
<tr>
<th>UNITS: kN-m</th>
<th>IN-PLANE</th>
<th>OUT-OF-PLANE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SERVICEABILITY DAMAGE CONTROL</td>
<td>SERVICEABILITY DAMAGE CONTROL</td>
</tr>
<tr>
<td>Le=</td>
<td>9.5</td>
<td>9.5</td>
</tr>
<tr>
<td>( \Delta y )=</td>
<td>0.09</td>
<td>0.09</td>
</tr>
<tr>
<td>( \Delta D )=</td>
<td>0.11</td>
<td>0.41</td>
</tr>
<tr>
<td>( U_D )=</td>
<td>1.19</td>
<td>4.64</td>
</tr>
<tr>
<td>ED=</td>
<td>5.0</td>
<td>9.5</td>
</tr>
<tr>
<td>( T_{eff} )=</td>
<td>0.9</td>
<td>1.8</td>
</tr>
<tr>
<td>( V )=</td>
<td>1000.0</td>
<td>1002.8</td>
</tr>
<tr>
<td>( V/W )=</td>
<td>50.0%</td>
<td>50.1%</td>
</tr>
<tr>
<td>( M_D )=</td>
<td>5203.0</td>
<td>5217.4</td>
</tr>
<tr>
<td>( \rho )=</td>
<td>1.52</td>
<td>1.52</td>
</tr>
<tr>
<td>( M_{P\alpha}/M )=</td>
<td>7.9%</td>
<td>10.4%</td>
</tr>
</tbody>
</table>
6.2. Parametric analyses

Example 6.1 was solved for different heights and soil types. The performance objective and loads were kept the same. The three column bent was also considered fully fixed at ground level as if it was supported on a rigid pile cap or footing. For all the analyses the column diameter of 1.4 m was adopted. Three bent heights were used, 6, 9 and 12 meters. The bents were considered embedded in clay-20 or sand-37. These resulted in 6 analyses with soil interaction plus 1 without soil-interaction.

The design process used for the bent fixed at ground level was as follows:

1. The equivalent length $L_e$ was set equal to the above ground height $L_a$.
2. The yield displacement was calculated using Equations 4.1 and 4.2 for the out-of-plane direction and in-plane direction, with the yield displacement coefficient $\alpha$ set to zero.
3. The plastic hinge length was calculated from Equation 2.3 (Paulay and Priestley, 1996)
4. Then the displacement ductility and target displacement was evaluated in both directions for both the serviceability and damage control limit states.
5. The equivalent viscous damping $ED$ was evaluated as a function of the displacement ductility $U_\Delta$ using Equation 6.7 (Dwairi, 2004) with viscous damping $\zeta$ equal to 5%
\[ ED = \zeta + 50 \left( \frac{U_D - 1}{\pi U_D} \right) \] (6.7)

6. Then the effective period, base shear, design moment and reinforced were calculated in a similar way as for the example in Section 6.1

The results of the analyses are shown in Table 6.7 for the bents embedded in clay-20. In Table 6.8 for bents embedded in sand-37 and in Table 6.9 for the bent fixed at ground level. The columns numbered from 1 to 13 contain the following information:

Column 1: Above ground height, \( L_a \)
Column 2: Equivalent length, \( L_e \)
Column 3: Yield displacement, \( \Delta_y \)
Column 4: Target displacement, \( \Delta_D \)
Column 5: Displacement ductility, \( U_D \)
Column 6: Equivalent damping, \( E_D \)
Column 7: Effective Period, \( T \)
Column 8: Base Shear, \( V \)
Column 9: Ratio of base shear and reactive weight, \( V/W \)
Column 10: Design Moment, \( M \)
Column 11: Steel Ratio, \( \rho \)
Column 12: Strength reduction factor, \( R \)
Column 13: Ratio of moment cause by P-Δ effect to design moment.

Tables 6.7-6.9 show the parameters used during the design process as well as design results in both directions and for both limit states. The strength reduction factor R was calculated for comparison with the corresponding values specified for the force-based design method used in the AASHTO LRFD Bridge Design Specifications (2004). When performing the analysis for the damage control limit state, the target displacement was changed iteratively until the P-Δ moment was less or equal to 10% of the obtained design moment for the column.

### Table 6.7 Design results of bents in clay-20

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In-plane Direction -- Serviceability Limit State

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Out-of-plane Direction -- Serviceability Limit State

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In-plane Direction -- Damage Control Limit State

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### Table 6.9 Design results of bents in clay-20

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Out-of-plane Direction -- Damage Control Limit State
From the results show in Tables 6.7-6.9 the following observations can be made: The serviceability displacement limit of 0.05 m for the out-of-plane direction governs the design in all cases. The maximum moment and the corresponding amount of steel required increases with the height of the bent. Also, if the bent is embedded in a softer soil, it requires more reinforcement. For instance, with $L_a$ equal 12 m, a steel ratio of 4% is required for the bent in clay-20, 3.5% is required in the bent in sand-30 and 2.7% is required for the bent fixed at ground level. Clay-20 is the softest soil, and having the bent fixed at ground level can be thought of as having the bent embedded in infinitely stiff soil.
Table 6.9. Design results of bents fixed at ground level

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<td>0.52</td>
<td>4651.59</td>
<td>0.85</td>
<td>1.53</td>
</tr>
<tr>
<td>12 12</td>
<td>0.17</td>
<td>0.30</td>
<td>1.79</td>
<td>12.00</td>
<td>1.54</td>
<td>508.94</td>
<td>0.51</td>
<td>6107.23</td>
<td>1.34</td>
<td>1.41</td>
</tr>
</tbody>
</table>

It was also observed that the softer the soil, the deeper the plastic hinge, the larger the equivalent length, the larger the yield displacement, and therefore the smaller the displacement ductility. A small value of displacement ductility yields a small value of equivalent damping even though the soft soils exhibit higher damping than the stiff soils for the same level of ductility. So, it can be concluded that the softer the soil, the greater the amount of strength required to keep the top displacement within a certain limit, at least when the top displacement is less than the yield displacement.
Also notice was that P-Δ effects are important and could limit the displacement if not considered in the design method. The displacement limits based on limiting P-Δ have controlled the design over the curvature limits for the damage control limit state.

If displacement limits rather than curvature limits control design, a bent fixed at the ground requires less strength than the same bent in soft soil since the ductility demand and therefore the equivalent damping will be greater.

Through all the analyses the force reduction factor R ranged from 1 to 1.43 for the serviceability limit states and from 1.11 to 1.7 for the damage control limit states. In force based design R = 1 would be used for the serviceability limit state and R = 3 (AASHTO LRFD Bridge Design Specifications) for the damage control limit states. Therefore if used, force based design would yield a bent with less strength.

In summary, the examples presented in this Chapter have show the effectiveness of the models developed in this research for the implementation of DDBD for column bents. However it should be noticed that one of the main assumptions made is that the soil is uniform along the columns. The implications of this idealization could be in the scope of work for future research.
7. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Work in this study attempted to implement Direct Displacement Based Design (DDBD) to the seismic design of reinforced concrete column bents embedded in soft soils. DDBD has been successfully used to design bridge columns that are fixed at ground level and without soil interaction. The implementation of DDBD for column bents however, requires the consideration of soil-structure interaction effects—namely added flexibility and damping. The main objective of this research was to develop an equivalent model to assess yield displacement and ductility and to assess the equivalent viscous damping as a function of ductility demand and soil type.

The study includes a literature review of the current practice for design of column bents with an emphasis on the equivalent models that have been developed to simplify the soil-column interaction problem. The review also includes a description of DDBD and performance based design concepts. Then, the static and dynamic response of RC columns embedded in different soil is investigated by performing a series of nonlinear analyses. Several modeling techniques are explored and the P-y method for soil-column interaction is chosen as the best alternative. Two programs are used for performing the analyses: MultiPier for nonlinear static analysis and verification and OpenSees for the parametric nonlinear static and time history analyses.

Once the behavior of soil-column systems is understood, an equivalent model that replaces the nonlinear P-y model is developed. In the equivalent model, the column is
considered fixed at some depth below ground at a point that coincides with the location of the point of maximum moment and the possible formation of an underground plastic hinge. The yield displacement of the column is matched with the yield displacement of the soil-column model by introducing a coefficient. Charts and equations are given to calculate the equivalent length and yield displacement coefficient for columns as a function of above ground height of the column and soil type. These charts and equations are the result of a parametric study that consisted on performing static nonlinear analysis on a soil-column model with different geometric configurations and soil types. This parametric study also resulted in a model that relates curvature ductility to displacement ductility.

A second parametric study is performed to evaluate the equivalent viscous damping in soil-column systems. The study required performing nonlinear time history analyses and resulted in trends that relate equivalent damping to displacement ductility for different types of soils. Both parametric studies consider five types of soils, clay with shear strength of 20 and 40 kPa and sand with a friction angle of 30, 34 and 37 degrees. The diameter of the columns range from 0.3 m to 2.4 m and the above ground height ranges from 2 to 10 column diameters. In all cases the embedded length is long enough such that the tip displacements are negligible. Both studies are performed for pinned head and fixed head conditions. Finally, application examples are presented to demonstrate the use of the proposed equivalent model and damping relations in the DDBD of column bents. Based on the research results, the following conclusions and recommendations are advanced.
**Equivalent model:**

1. For a given soil type, the equivalent length increases with the diameter of the column and with the height of the bent.
2. For a given soil and height of the bent, an increase in diameter causes an increase in yield displacement.
3. The plastic hinge length must increase with displacement ductility to match the results of a nonlinear analysis. This results in a nonlinear relation between curvature ductility and displacement ductility in which the ratio of curvature to displacement ductility increases with displacement ductility.
4. For a given diameter, height of the bent and displacement limit, the soil type influences the response as follows: the softer the soil, the larger the equivalent length, the larger the yield displacement and the smaller the ductility demand.

**Equivalent damping:**

The equivalent damping was obtained for the soil-column system. The equivalent damping results from the energy dissipated in both soil and column when either undergoes hysteretic cycles. The equivalent damping has been obtained as a function of ductility and soil type.

1. For each soil type, a hyperbolic function best fit the damping values with respect to displacement ductility.
2. For a given value of ductility, softer soils yield more damping.
3. The hyperbolic trends are almost parallel to each other. The difference in damping comes from the initial damping at a ductility value of one. This value corresponds to the damping generated by the soil alone.

4. For the same soil and ductility level. The oscillation of a column with a pinned head generates more damping than the same column with a fixed head. This is due to the larger displacement that the pinned head column exercises to reach the same level of ductility.

5. The proposed relations for EVD if used in the framework of DDBD make possible to predict the maximum displacement response within a range of variation of 20% for a 95% confidence interval.

**DDBD for column bents:**

With the developed models for yield displacement, ductility and damping, DDBD can be applied to column bents following the same approach as for bridge columns on pile caps or footings.

1. If displacement limits are set below the yield displacement of the column-soil system (such as those from serviceability limit state), the softer the soil the greater the strength required in the column. The added flexibility of a soft soil increases the yield displacement, lowers the ductility and, even though soft soils yield more damping, the force reduction is less than for the same column in a stiffer soil.

2. If displacement limits (less than the yield displacement), rather than curvature limits control design, a bent fixed at the ground requires less strength than the same bent in
soft soil since the ductility demand and therefore the equivalent damping will be greater in the fixed bent.

3. Curvature limit states are unlikely to control design since the yield displacements, especially for pinned head condition, are large and P-Δ effects are likely to limit the displacement before the bent reaches the desired curvature limit.

4. If displacement limits control design, the calculated force reduction factors in the analyses performed in Chapter 6 are less than what is recommended by AASHTO.

**Future Research should focus on:**

1. Displacement limit for serviceability and damage control limit states. Based on substructure-superstructure interaction and possible damage to connections.

2. Equivalent model and damping for single column bents that are rigidly connected to the superstructure.

3. More realistic soil models—i.e. the effect of soft soils overlying stiffer materials, or soil models without homogeneous strength or elastic characteristics.

4. Improving numerical simulations. Currently OpenSees only models soft clays and sand and does not consider strength degradation.

5. Equivalent damping in elastomeric superstructure to substructure connections.
LIST OF REFERENCES


Caltrans, Seismic Design Criteria, 2004


Ensoft, Inc. 2004 LPILE Manual, Austin, Texas, USA,


APPENDIX 1: Sample TCL code used with OpenSees
SOIL RESPONSE PROCEDURE

# This procedure records the response of a PySimple material to a
# sinusoidal excitation
# Arguments:
#  Soil:
#    soilType = 1 for soft clays, 2 for Sands
#    pult = ultimate capacity of the p-y material (KN)
#    y50= Strain at 50% of pult
#    Cd= drag resistance within a fully formed gap as Cd*pult
# See: PySimple1 documentation
# Analysis:
#    maxdisp = Max displacement (m)
#    cycles= # of cycles

model BasicBuilder -ndm 1 -ndf 1
#Soil:
set soilType 1
set pult 150
set y50 0.065
set Cd 0.10
#Analysis
set maxdisp 1
set cycles 10

# Define nodes
node 1 0.0
node 2 1.0

# Fix node 1
fix 1  1

# Define PySimple1 material
uniaxialMaterial PySimple1 1 $soilType $pult $y50 $Cd
# Define zerolength element
element zeroLength 1 1 2 -mat 1 -dir 1
pattern Plain 1 "Sine 0 [expr 100*$cycles] 100" {
    sp   2 1 $maxdisp
}

# Record nodal displacements
recorder Node -file soilRdisp.out -load -node 2 -dof 1 disp
# Record element force
recorder Element 1 -time -file soilRforce.out forces

system UmfPack
constraints Penalty 1.0e12 1.0e12
integrator LoadControl .1 .1 .1 .1
test NormDispIncr 1.0e-6 10
algorithm Newton
numberer RCM
analysis Static
analyze [expr 1000*$cycles]
#procedure to obtain cyclic moment curvature response of a rc circular column

# sources: MaterialsRConcrete.tcl, RCcircSection.tcl, MomentCurvature.tcl

# units KN-M

# number of cycles
set cycles 2

# axial load
set axial 0

# id - tag for the section that is generated by this procedure
set secid 1

# ri - inner radius of the section
set ri 0

# ro - overall (outer) radius of the section
set ro [expr 1.80/2]

# cover - cover thickness
set cover 0.075

# coreID - material tag for the core patch
set coreID 1

# coverID - material tag for the cover patches
set coverID 2

# steelID - material tag for the reinforcing steel
set steelID 3

# numBars - number of reinforcing bars around the section perimeter
set numBars 52

# barArea - cross-sectional area of each reinforcing bar
set barArea [expr pow(0.036,2) * 3.1416 / 4]

# nfCoreR - number of radial divisions in the core (number of "rings")
set nfCoreR 20

# nfCoreT - number of theta divisions in the core (number of "wedges")
set nfCoreT 52

# nfCoverR - number of radial divisions in the cover
set nfCoverR 3

# nfCoverT - number of theta divisions in the cover
set nfCoverT 52

# material parameters

# fpc concrete cylinder strength
set fpc 35000

# Ec concrete initial modulus
set Ec 27805000

# As area of longitudinal steel
set As 0.05293

# ds diameter of spiral
set ds 0.019

# s center line distance between spirals
set s 0.075

# fyh yield stress of confining steel hoops
set fyh 455100

# fyl yield stress of longitudinal steel
set fyl 455100

# sid steel id
set sid 1

# ccid confined concrete ID
set ccid 2

# cid cover concrete id
set cid 3
puts "Rho= [expr $numBars*$barArea/(pow($ro,2)*3.1416)]"

model basic -ndm 2 -ndf 3

source MaterialsRConcrete.tcl
MaterialsRC $sid $ccid $cid $fpc $Ec $As $ds $s $fyh $fyl

source RCCircSection.tcl
RCCircSection $secid $ri $ro $cover $ccid $cid $sid $numBars $barArea $nfCoreR $nfCoreT $nfCoverR $nfCoverT

source MomentCurvature.tcl

set maxK [expr .002/$ro*36]

MomentCurvature $secid $axial $maxK $cycles

PROCEDURE TO DEFINE MATERIAL PROPERTIES FOR RC SECTIONS

# MATERIALS FOR RC CIRCULAR COLUMNS
# fpc concrete cylinder strength
# Ec concrete initial modulus
# As area of longitudinal steel
# ds diameter of spiral
# s center line distance between spirals
# fyh yield stress of confining steel hoops
# fyl yield stress of longitudinal steel
# ccid confined concrete ID
# cid cover concrete id

proc MaterialsRC {sid ccid cid fpc Ec As ds s fyh fyl} {
  set epco 0.002
  set R 5
  set Ash [expr pow($ds,2)*4/3.1416]
  set sp [expr $s-$ds]
  set rocc [expr 4*$As/3.1416/pow($ds,2)]
  set ros [expr $Ash*3.1416/$s/$ds]
  set ke [expr (1-0.5*$sp/$ds)/(1-$rocc)]
  puts "ke=$ke ros=$ros fyh=$fyh"
  set fpl [expr $ke*$ros*$fyh/2]
  puts "fpl=$fpl"
  set fpcc [expr $fpc*(2.254*pow(1+7.94*$fpl/$fpc,0.5)-2*$fpl/$fpc-1.254)]
  set epcc [expr ($R*($fpcc/$fpc-1)+1)*$epco]
  set Es 200000000
  set hardening 0.01
  set fpcc [expr -.9*$fpcc]
  set fpc [expr -$fpcc]
  set epcc [expr -$epco]
  set fpc [expr -$fpc]
  uniaxialMaterial Concrete01 $ccid $fpcc $epcc $fpcu 0.014
  uniaxialMaterial Concrete01 $ccid $fpc -0.002 0 -0.0064
  uniaxialMaterial Steel01 $sid $fyl $Es $hardening
  puts "Steel"
puts "Fy= $fyl  Es=$Es  r=$hardening"
puts "Confined Concrete"
puts "Fpcc=$fpcc  eFpcc=$epcc"
puts "Cover Concrete"
puts "Fpc= $fpc"
}

RCcircSection Procedure

# Define a procedure which generates a circular reinforced concrete section
# with one layer of steel evenly distributed around the perimeter and a confined core.
#
# Formal arguments
#  id - tag for the section that is generated by this procedure
#  ri - inner radius of the section
#  ro - overall (outer) radius of the section
#  cover - cover thickness
#  coreID - material tag for the core patch
#  coverID - material tag for the cover patches
#  steelID - material tag for the reinforcing steel
#  numBars - number of reinforcing bars around the section perimeter
#  barArea - cross-sectional area of each reinforcing bar
#  nfCoreR - number of radial divisions in the core (number of "rings")
#  nfCoreT - number of theta divisions in the core (number of "wedges")
#  nfCoverR - number of radial divisions in the cover
#  nfCoverT - number of theta divisions in the cover
#
# Notes
#  The center of the reinforcing bars are placed at the inner radius
#  The core concrete ends at the inner radius (same as reinforcing bars)
#  The reinforcing bars are all the same size
#  The center of the section is at (0,0) in the local axis system
#  Zero degrees is along section y-axis
#
proc RCcircSection {id ri ro cover coreID coverID steelID numBars barArea nfCoreR
  nfCoreT nfCoverR nfCoverT} {

  # Define the fiber section
  section fiberSec $id {

    # Core radius
    set rc [expr $ro-$cover]

    # Define the core patch
    patch circ $coreID $nfCoreT $nfCoreR 0 0 $ri $rc 0 360

    # Define the cover patch
    patch circ $coverID $nfCoverT $nfCoverR 0 0 $rc $ro 0 360

    if {$numBars <= 0} {
      return
    }

    # Determine angle increment between bars
    set theta [expr 360.0/$numBars]

    # Define the reinforcing layer
    layer circ $steelID $numBars $barArea 0 0 $rc $theta 360
  }

# proc MomentCurvature
# Arguments
# secTag -- tag identifying section to be analyzed
# axialLoad -- axial load applied to section (negative is compression)
# maxK -- maximum curvature reached during analysis
# numIncr -- number of increments used to reach maxK (default 100)
#
# Sets up a recorder which writes moment-curvature results to file
# section$secTag.out ... the moment is in column 1, and curvature in column 2
proc MomentCurvature {secTag axialLoad maxK cycles} {
    # Define two nodes at (0,0)
    node 1 0.0 0.0
    node 2 0.0 0.0
    # Fix all degrees of freedom except axial and bending at node 2
    fix 1 1 1
    fix 2 0 1
    # Define element
    # tag ndI ndJ secTag
    element zeroLengthSection 1 1 2 $secTag
    # Create recorder
    recorder Node -file M_Canalysis$secTag.out -time -node 2 -dof 3 disp
    # Define constant axial load
    pattern Plain 1 "Constant" {
        load 2 $axialLoad 0.0 0.0
    }
    # Define analysis parameters
    integrator LoadControl 0 1 0 0
    system SparseGeneral -piv;
    test NormUnbalance 1.0e-9 100
    numberer Plain
    constraints Plain
    algorithm Newton
    analysis Static
    # Do one analysis for constant axial load
    analyze 1

    # Define reference moment
    pattern Plain 2 "Sine 0 [expr 100*$cycles] 100" {
        sp 2 3 $maxK
    }
    # Record nodal displacements
    recorder Node -file MCcurvature.out -load -node 2 -dof 3 disp
    # Record element force
    recorder Element 1 -time -file MCmoment.out forces
system UmfPack
constraints Penalty 1.0e12 1.0e12
integrator LoadControl .1 .1 .1 .1
test NormDispIncr 1.0e-6 10
algorithm Newton
numberer RCM
analysis Static

analyze [expr 1000*$cycles]

NONLINEAR STATIC ANALYSIS OF PILE-SOIL SYSTEMS

#########################################################
#SINGLE PILE PUSH GENERATOR  #
#########################################################
wipe

set ana 1
set maxdisp 1.8

set axial 0

#PARAMETERS FOR PUSH ###########################
set nstep 100

#REQUIRED VARIABLES
#FREE LENGTH OF PILE (m)
set FL 12
#FIXHEAD HEAD=1 FREEHEAD HEAD=2
set HEAD 2
#SOIL TYPE FILE
set stype 1

# SET ELEMENT RECORDED BELOW GROUND
set ERBG 40

# PILE PROPERTIES
#PY ELEMENT SEPARATED EACH PYSEP PILE DIAMETERS
set pysep 0.25
#LENGTH BELOW GROUND= LBG PILE DIAMETERS
set LBG 20

model basic -ndm 2 -ndf 3

source MgenRCcir.tcl
source Nodes.tcl
source nFix.tcl
source PyMaterials.tcl
source Pyele.tcl
geomTransf Linear 1
source Pele.tcl

# apply axial load
pattern Plain 1 Linear {
    load $nnode 0 $axial 0
}

puts "axial=$axial"

system SparseGeneral -piv;
constraints Transformation
numberer RCM
test NormDispIncr 1.0e-12 20
algorithm Newton
integrator LoadControl 0.1
analysis Static
analyze 10
loadConst -time 0.0

pattern Plain 2 Linear {
    load $nnode 1 0 0
}

puts "Solving lateral analysis"

integrator DisplacementControl $nnode 1 [ expr $maxdisp/$nstep]
analysis Static
set a Disp
set c C
set e E
set d Force
set n N
recorder Node -file PushD$ana.out -time -node $nnode -dof 1 disp
recorder Element -file PushC$ana.out -time -ele $nele section 1 deformation
recorder Element -file PushF$ana.out -time -ele $nele localForce

set nod [expr ($LBG/$pysep+1)*2-$ERBG*2-1]
set nod [expr int($nod)]
set col [expr $LBG/$pysep*2-$ERBG*2]
set col [expr int($col)]
while {$nod < [expr ($LBG/$pysep+1)*2]} {
    recorder Node -file PushD$ana$n$nod.out -time -node $nod -dof 1 disp
    set nod [expr $nod+2]
    recorder Element -file PushC$ana$c$col.out -time -ele $col section 1 deformation
    set col [expr $col+2]
}
analyze $nstep
print node $nnode
print ele $nele
print node 1

NONLINEAR TIME HISTORY ANALYSIS IF SOIL-COLUMN SYSTEMS

##########################################################
#SINGLE COLUMN NTHA
##########################################################
wipe
#REQUIRED VARIABLES
#QUAKE
set quake EQ1
set duration 1000
set dt 0.01
set ftor 1
#FREE LENGTH OF PILE (D)
set FL 1
#FIXHEAD HEAD=1 FREEHEAD HEAD=2
set HEAD 1
#DIAMETER
set DP 1
#EI
set EI 5000
# WEIGHT
set WEIGHT 500
#SOIL TYPE FILE
set stype Clay1.60
# SET ELEMENT RECORDED BELOW GROUND
set ERBG 10
# PILE PROPERTIES
#PY ELEMENT SEPARATED EACH PYSEP PILE DIAMETERS
set pysep 1
#LENGTH BELOW GROUND= LBG PILE DIAMETERS
set LBG 30

set gnode [expr $LBG/$pysep*2 +1]
set ZT 0

#model basic -ndm 2 -ndf 3
#set nnode 63
#set nele 61
#source MgenRCcirES.tcl
source MgenRCcirNL.tcl
source Nodes.tcl
source nFix.tcl
source PyMaterials.tcl
source Pyele.tcl
geomTransf Linear 1
source Pele.tcl

#assign mass
mass $nnode [expr $WEIGHT/9.81] 1e-10 1e-10
puts "THA is running"
set accelSeries "Path -filePath $quake.txt -dt $qdt -factor $ftor"

# Raleigh Damping
#set xDamp 0.05
set lambda [eigen 1]
set omega [expr pow($lambda,0.5)]
set Tperiod [expr 2*3.1415927/$omega]
puts Period=$Tperiod
set betaKcomm [expr 2*$xDamp/$omega]
#set betaKcomm 0
# Create UniformExcitation load pattern
#  tag dir
pattern UniformExcitation 2 1 -accel $accelSeries
test NormDispIncr 1.0e-8 200
algorithm Newton
integrator Newmark 0.5 0.25 0.0 0.0 0.0 $betaKcomm
system BandGeneral
constraints Plain
numberer RCM

analysis Transient

recorder Node -file Dt$ana.dis -time -node $nnode -dof 1 disp
recorder Node -file Dg$ana.dis -time -node $nzo -dof 1 disp
recorder Element -file F$ana-1.for -time -ele [expr $nele-0] localForce
recorder Element -file F$ana-2.for -time -ele [expr $ezo-0] localForce
recorder Element -file F$ana-3.for -time -ele [expr $ezo-2] localForce
recorder Element -file F$ana-4.for -time -ele [expr $ezo-4] localForce
recorder Element -file F$ana-5.for -time -ele [expr $ezo-6] localForce
recorder Element -file F$ana-6.for -time -ele [expr $ezo-8] localForce
recorder Element -file F$ana-7.for -time -ele [expr $ezo-10] localForce
recorder Element -file F$ana-8.for -time -ele [expr $ezo-12] localForce
recorder Element -file F$ana-10.for -time -ele [expr $ezo-16] localForce

set tFinal [expr $duration * $dt]
set tCurrent [getTime]
set ok 0
# Perform the transient analysis
while {$ok == 0 && $tCurrent < $tFinal} {
    set ok [analyze 1 $dt]
    # if the analysis fails try initial tangent iteration
    if {$ok != 0} {
        puts "regular newton failed .. lets try an initail stiffness for this step"
        test NormDispIncr 1.0e-12 100 0
        algorithm ModifiedNewton -initial
        set ok [analyze 1 .01]
        if {$ok == 0} {puts "that worked .. back to regular newton"
        test NormDispIncr 1.0e-12 10
        algorithm Newton
    }
    set tCurrent [getTime]
}
# Print a message to indicate if analysis succesfull or not
if {$ok == 0} {
    puts "Transient analysis completed SUCCESSFULLY";
} else {
    puts "Transient analysis completed FAILED";
}

PROCEDURE THAT GENERATES SOIL-PILE MODEL

#########################################################################
#SINGLE FILE MODEL GENERATOR #
#RC circular section
#GOOD FOR PUSHOVER AND THA
#########################################################################
# REQUIRED VARIABLES
# FREE LENGTH OF PILE (m)
# FIXHEAD HEAD=1 FREEHEAD HEAD=2
# SOIL TYPE FILE
# AXIAL LOAD
# SECTION TYPE

wipe

# sec 1 for Section with hinges  sec 2 for fiber model
set sec 2
# ground level
set ZT 0

################################################
# DEFINE MATERIAL AND PILE SECTION
################################################
# Diameter of pile
set DP 1.8
# id - tag for the section that is generated by this procedure
set secid 1
# ri - inner radius of the section
set ri 0
# ro - overall (outer) radius of the section
set ro [expr $DP/2]
# cover - cover thickness
set cover 0.075
# coreID - material tag for the core patch
set coreID 1
# coverID - material tag for the cover patches
set coverID 2
# steelID - material tag for the reinforcing steel
set steelID 3
# numBars - number of reinforcing bars around the section perimeter
set numBars 52
# barArea - cross-sectional area of each reinforcing bar
set barArea [expr pow(0.036,2) * 3.1416 / 4]
# nfCoreR - number of radial divisions in the core (number of "rings"
set nfCoreR 20
# nfCoreT - number of theta divisions in the core (number of "wedges"
set nfCoreT 15
# nfCoverR - number of radial divisions in the cover
set nfCoverR 2
# nfCoverT - number of theta divisions in the cover
set nfCoverT 30

# material parameters
# fpc concrete cylinder strength
set fpc 44800
# Ec concrete initial modulus
set Ec 31685000
# As area of longitudinal steel
set As 0.05293
# ds diameter of spiral
set ds 0.019
# s center line distance between spirals
set s 0.075
# fyh yield stress of confining steel hoops
set fyh 455100
# fyl yield stress of longitudinal steel
set fyl 455100
# sid steel id
set sid 100
# ccid confined concrete ID
set ccid 200
# cid cover concrete id
set cid 300

model basic -ndm 2 -ndf 3

source MaterialsRConcrete.tcl
MaterialsRC $sid $ccid $cid  $fpc $Ec $As $ds $s $fyh $fyl

source RCCircSection.tcl
RCCircSection $secid $ri $ro $cover $ccid $cid $sid $numBars $barArea $nfCoreR $nfCoreT $nfCoverR $nfCoverT

set z [expr $ZT-$DP*$LBG]
set nnode 0
set nele 0
set fnode [open Nodes.tcl w]
set fpy [open Pyele.tcl w]
set fpile [open Pele.tcl w]
set ffix [open nFix.tcl w]

set nnode [expr $nnode+1]
puts $fnode "node $nnode 0 $z"
puts $ffix "fix $nnode 0 1 1"

set nnode [expr $nnode+1]
puts $fnode "node $nnode 0 $z"
puts $ffix "fix $nnode 1 1 1"

set npy [expr $nele+1]
puts $fpy "element zeroLength $nele $nnode [expr $nnode-1] -mat $nele -dir 1"
set z [expr $z+$DP*$pysep]
while {$z <= $ZT} {
    set nnode [expr $nnode+1]
    puts $fnode "node $nnode 0 $z"
    set nnode [expr $nnode+1]
    puts $fnode "node $nnode 0 $z"
    set nnode [expr $nnode+1]
    puts $ffix "fix $nnode 0 1 1"
    set nele [expr $nele+1]
    puts $fpy "element zeroLength $nele $nnode [expr $nnode-1] -mat $nele -dir 1"
    set z [expr $z+$DP*$pysep] 
}

set nnode [expr $nnode+1]
puts $fnode "node $nnode 0 $z"

set nele [expr $nele+1]
puts $fpile "element nonlinearBeamColumn $nele [expr $nnode] [expr $nnode-2] 4 $secid 1"
set z [expr $z+$DP*$pysep]

while {$z < $FL} {
    set nnode [expr $nnode+1]
    puts $fnode "node $nnode 0 \$z"
    set nele [expr $nele+1]
    puts $fpile "element nonlinearBeamColumn \$nele [expr $nnode] [expr $nnode-1] 4 \$secid 1"
    set z [expr $z+$DP*$pysep]
}

set nnode [expr $nnode+1]
puts $fnode "node $nnode 0 $FL"
# SETS FREE OR FIX HEAD
if {$HEAD==1} {
    puts $ffix "fix $nnode 0 0 1"
    set nele [expr $nele+1]
    puts $fpile "element nonlinearBeamColumn \$nele [expr $nnode] [expr $nnode-1] 4 \$secid 1"
}

close $fnode
close $fpy
close $fpile
close $ffix

set soil "P1S$stype.txt"

PySimple1Gen $soil "Nodes.tcl" "Pyele.tcl" "Pele.tcl" "PyMaterials.tcl"