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GEOTECHNICAL PRACTICE AND RESEARCH

Revision of the CLM Spreadsheet for Lateral
Load Analyses of Deep Foundations

by

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Introduction

The first version of the Characteristic Load Method (CLM) spreadsheet (Brettmann and Duncan, 1996) was based on the CLM method (Duncan, et al., 1994) for analysis of single piles, and the Group Amplification Method (Ooi and Duncan, 1994) for analysis of pile groups. That first version of the CLM spreadsheet was found to produce quite accurate results for single piles, but was found in many cases to overestimate deflections and bending moments for pile groups. The revised spreadsheet described in this report uses the same method of analysis as the original for single piles, but uses an improved method of analysis for pile groups.

Background

The characteristic load method (CLM) of analysis of laterally loaded piles (Duncan et al., 1994) was developed by performing nonlinear p-y analyses for a wide range of free-head and fixed-head piles and drilled shafts in clay and sand. The results of the analyses were used to develop nonlinear relationships between dimensionless measures of load and deflection. These relationships were found to be capable of representing the nonlinear behavior of single piles and drilled shafts quite accurately, producing essentially the same values of deflection and maximum moment as p-y analysis computer programs like COM624 and LpilePlus3.0. The principal limitation of the CLM method is that it is applicable only to uniform soil conditions.

The Group Amplification Method was developed by Ooi and Duncan (1994), to extend use of the CLM method to groups of piles and drilled shafts. Values of group amplification factors for deflection and moment were computed using the method developed by Focht and Koch (1973).

The original version of the CLM spreadsheet (Brettmann and Duncan, 1996) used the CLM method to calculate deflections and bending moments in single piles, and the Group Amplification Method to calculate deflections and moments for piles in pile groups. It was found that the original version of the spreadsheet resulted in accurate values of moment and deflection for single piles, but often over-estimated deflections and bending moments for the piles in pile groups, as judged by comparison with p-y analysis programs and the results of field load tests.
The revised spreadsheet uses p-y multipliers as the basis for improving the accuracy of pile group analyses. The p-y multiplier values used are those recommended by Mokwa and Duncan (2001) based on their field tests and review of recent literature.

**p-y Multipliers**

Piles or drilled shafts that are spaced close together respond to lateral loading differently than single isolated piles. Pile-soil-pile interaction or “shadowing” occurs when piles are close enough so that they transfer load to the same zone within the soil around the piles. As a result, the ground is more severely loaded, and pile deflections and moments are larger.

This phenomenon can be accounted for by reducing the soil resistance in p-y analyses (p is the soil resistance per unit length of pile, and y is the pile deflection). Multiplying the p-values by factors smaller than 1.0 reduces the resistance of the soil, and, by using appropriate values of p-multipliers, can account for “shadowing” within pile groups.

Mokwa and Duncan (2001) reviewed 37 experimental test results in which the effects of pile-soil-pile interaction were measured. These studies included full-scale lateral load tests, centrifuge tests and laboratory tests. They used these results, together with data from their tests at the Virginia Tech field test site, to develop p-y multipliers for use in design of pile groups. Their recommended values are shown in Figure 1, which shows the variation of p-multiplier values with pile spacing and row. These p-multipliers have been incorporated in the revised version of the CLM spreadsheet (CLM2.0) to account for pile group effects.

Figure 1 illustrates the values of the p-multipliers \( f_m \) decrease as the number of rows of piles increases, and as the spacing of the rows decreases. (The direction of load application is perpendicular to the axis of the rows.) The values of \( f_m \) shown in Figure 1 are proportional to the load carried by each row of piles, since the deflection of all piles is the same. The chart shows that the leading row (the one at the front of the pile group) takes the most load. The load drops off in the first, second, and third trailing rows, and remains constant for rows further back. The values of \( f_m \) decrease as the spacing of the rows decreases, indicating less efficient transfer of load to the ground. For piles spaced more than six diameters apart, the values of \( f_m \) are equal to 1.0, indicating that group interaction effects are negligible.
Notes:

1. The term row used in this chart refers to rows of piles aligned perpendicular to the direction of applied load.
2. Use the $f_m$ values for the 3rd trailing row for all rows beyond the third trailing row.
3. Bending moments and shear forces computed for the leading row corner piles should be adjusted as follows:

<table>
<thead>
<tr>
<th>Side-by-side spacing (within rows), expressed in pile diameters</th>
<th>Multiplier factor for corner piles (outside piles in the leading row)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3D</td>
<td>1.00</td>
</tr>
<tr>
<td>2D</td>
<td>1.20</td>
</tr>
<tr>
<td>1D</td>
<td>1.60</td>
</tr>
</tbody>
</table>

Figure 1 – p-multiplier design curves (after Mokwa and Duncan, 2001)
The experiments from which values of $f_m$ were determined showed that the maximum bending moments in the piles in a group are nearly the same except for the corner piles (the outside piles in the leading row). For pile spacings smaller than three diameters (3D), the moments in the corner piles are larger, as shown in Figure 1. CLM2.0 computes values of maximum moment that are applicable to all except the corner piles. For piles spaced closer than 3D, the maximum moments in the corner piles should be increased by multiplying the moments computed by the spreadsheet by the corner pile factors shown in Figure 1.

**Method of Including p-y Multipliers in CLM2.0**

Because the value of $f_m$ is a constant for each row, independent of depth, the value of $f_m$ can be thought of as an efficiency factor for the piles in each row. The efficiency factor for the pile group ($F_m$) is the average of the efficiency factors for the rows in the group.

Pile group efficiency factors were incorporated in the Characteristic Load Method of analysis by reducing the soil resistance in the expressions for characteristic load ($P_c$) and characteristic moment ($M_c$). The undrained strength of clay ($S_u$), and the coefficient of passive earth pressure for sand ($K_p$) in those expressions were multiplied by $F_m$, leading to the following expressions for $P_c$ and $M_c$.

For clay

$$P_c = 7.34D^2\left(\frac{E_pR_t}{\mu_p}\right)\left(\frac{S_uF_m}{E_p R_t}\right)^{0.68}$$

(1)

$$M_c = 3.86D^2\left(\frac{E_pR_t}{\mu_p}\right)\left(\frac{S_uF_m}{E_p R_t}\right)^{0.46}$$

(2)

For sand

$$P_c = 1.57D^2\left(\frac{E_pR_t}{\mu_p}\right)\left(\frac{\gamma'D\phi'K_pF_m}{E_p R_t}\right)^{0.57}$$

(3)

$$M_c = 1.33D^2\left(\frac{E_pR_t}{\mu_p}\right)\left(\frac{\gamma'D\phi'K_pF_m}{E_p R_t}\right)^{0.40}$$

(4)

where, $P_c =$ characteristic load (force units, F), $M_c =$ characteristic moment (force times length, FL), $D =$ pile or shaft width or diameter (L), $E_p =$ pile or drilled shaft modulus of elasticity ($F/L^2$), $R_t =$ moment of inertia ratio (dimensionless), $S_u =$ undrained shear strength for clay soil ($F/L^2$), $F_m =$ pile or drilled shaft group efficiency based on pile spacing (dimensionless), $\gamma' =$
effective unit weight for sand \((F/L^3)\), \(\phi'\) = effective stress friction angle of sand (degrees), \(K_p\) = Rankine coefficient of passive earth pressure of sand (dimensionless). Any consistent set of units may be used.

The value of \(R_I\) is the ratio of the moment of inertia of the pile to the moment of inertia of a solid circular section of the same width, or diameter.

**Soil Properties for CLM Analyses**

The principal limitation of the CLM method is that it is applicable only to uniform soil conditions. For cases where soil conditions vary with depth, it is necessary to use an equivalent uniform soil profile. It has been found that only the soil within eight pile diameters of the ground surface is important for lateral load conditions, because almost all lateral load is transferred from the piles to the soil within eight diameters below the ground surface. Suitable soil properties for CLM analyses can be established by averaging soil properties within the upper eight diameters.

**Comparisons of CLM2.0 with Field Load Tests and Other Software**

The accuracy of CLM2.0 was examined by comparing results computed using it with the results of the following field load tests:

1. **Brown, et al. (1988)** conducted full-scale lateral load tests to determine the variation of soil resistance within a pile group. Ten steel pipe piles, consisting of one 3-by-3 group spaced 3 diameters center-to-center (c/c) and one isolated pile were subjected to two-way cyclic loading. The 10.75 inches in outside diameter (O.D.) and 10.00 inch in inner diameter (I.D.) piles were tested in submerged dense sand that was placed and compacted around the piles.

2. **Kim et al. (1979)** performed full-scale lateral load tests to relate the behavior of single piles to the behavior of pile groups in cohesive soils. Twenty 10BP42 steel piles, consisting of three six-pile groups and two isolated single piles were driven 40 feet to refusal into limestone. Two of the pile groups consisted of vertical piles, with one group spaced at 4-ft c/c and the other group spaced at 3-ft c/c. The third group consisted of 4 battered piles. This group was not used for comparison, because CLM2.0 does not account for battered piles.

3. **Rollins, et al. (1998)** performed full-scale lateral load tests to determine pile-soil-pile interaction within a group. Ten steel pipe piles, consisting of one 3-by-3 group spaced 3
diameters center-to-center (c/c) and one isolated pile were driven into soft to medium stiff clays and silts underlain by sand. The piles were 12.75 inches in outside diameter (O.D.) and 12.00 inches in inner diameter (I.D.).

**Single Piles.** Comparisons of results computed using CLM2.0 for single piles with the results of field load tests and results computed using LpilePlus 3.0© (Reese and Wang, 1997) are shown in Figures 2, 3, and 4. The actual soil profile and the averaged soil profile used in CLM2.0 are shown at the top of the figures. Tabulated results are shown in the middle of each figure, and variations of deflection with applied load are shown at the bottom.

Because the load was applied above the ground surface for the load test shown in Figure 2, the “flagpole” analysis feature of CLM2.0 was used to compute the results shown in Figure 2. It can be seen that the results computed using CLM2.0 are in close agreement with the results computed using LpilePlus 3.0©, and that the measured deflections are about 30% smaller than the computed deflections.

In the load test shown in Figure 3, the single pile was attached to a “cap” to restrain rotation. Analyses performed using LpilePlus 3.0© and CLM2.0 were performed assuming fixed-head conditions at the top of the pile. It can be seen that these analyses resulted in deflections that were smaller than those measured, indicating that the cap probably provided only partial restraint against rotation at the top of the pile.

In the case shown in Figure 4, the deflections computed using LpilePlus 3.0© and CLM2.0 are in close agreement with the field measurements. As in the case shown in Figure 2, the load was applied above the ground surface, and the flagpole feature in CLM2.0 was used to represent this loading condition.

**Pile Groups.** Comparisons of results computed using CLM2.0 pile groups with the results of field load tests and results computed using Group4.0© (Reese et al., 1996) are shown in Figures 5, 6, 7, and 8.

In the case shown in Figure 5, the piles were pinned to the cap, and the CLM2.0 and Group4.0© analyses were performed assuming free head conditions. It can be seen that the computed results are in close agreement with the field measurements.
Figure 2 – Single Pile Comparison (load test data from Brown et al., 1988)
SOIL PROFILES

Lpyle Plus 3.0 Analysis

CLM Analysis

### SOIL PROFILES

- **CAP**
  - Width = 3.5’
  - Length = 3.5’
  - Thickness = 4’

10BP42 pile

<table>
<thead>
<tr>
<th>Load per Pile (kips)</th>
<th>Measured (inches)</th>
<th>Lpyle Plus 3.0 (inches)</th>
<th>CLM - Fixed-head (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.3</td>
<td>0.18</td>
<td>0.04</td>
<td>0.05</td>
</tr>
<tr>
<td>16.7</td>
<td>0.39</td>
<td>0.18</td>
<td>0.17</td>
</tr>
</tbody>
</table>

**Figure 3 – Single Pile Comparison (load test data from Kim et al., 1979)**
Figure 4 – Single Pile Comparison (load test data from Rollins et al., 1998)
Figure 5 – Pile Group Comparison (load test data from Brown et al., 1988)
Figure 6 – Pile Group Comparison (load test data from Kim et al., 1979)
### SOIL PROFILES

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Soil Type</th>
<th>( \gamma_m ) (pcf)</th>
<th>( S_u ) (psf)</th>
<th>( \phi_u ) (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Silty Clay</td>
<td>120</td>
<td>1000</td>
<td>0</td>
</tr>
<tr>
<td>7</td>
<td>Sandy Clay</td>
<td>115</td>
<td>1400</td>
<td>0</td>
</tr>
<tr>
<td>13</td>
<td>Clay</td>
<td>110</td>
<td>500</td>
<td>0</td>
</tr>
<tr>
<td>24</td>
<td>Clay Loam with Limestone Gravel Layers</td>
<td>115</td>
<td>2000</td>
<td>0</td>
</tr>
<tr>
<td>35</td>
<td>Limestone</td>
<td>150</td>
<td>5000</td>
<td>0</td>
</tr>
<tr>
<td>40</td>
<td>Clay</td>
<td>117</td>
<td>1200</td>
<td>0</td>
</tr>
</tbody>
</table>

#### Load per Pile and Deflection

<table>
<thead>
<tr>
<th>Load per Pile (kips)</th>
<th>Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Measured</td>
</tr>
<tr>
<td>8.3</td>
<td>0.1</td>
</tr>
<tr>
<td>16.7</td>
<td>0.24</td>
</tr>
</tbody>
</table>

**Figure 7 – Pile Group Comparison**

(load test data from Kim et al., 1979)
SOIL PROFILES

Group 4.0 Analysis

CLM Analysis

SOIL Profiles

Load per Pile (kips) | Deflection
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Measured (inches)</td>
<td>Group 4.0 (inches)</td>
<td>CLM - Free-head (inches)</td>
</tr>
<tr>
<td>14</td>
<td>0.79</td>
<td>0.66</td>
<td>0.50</td>
</tr>
<tr>
<td>20</td>
<td>1.57</td>
<td>1.12</td>
<td>0.97</td>
</tr>
<tr>
<td>23</td>
<td>2.36</td>
<td>1.44</td>
<td>1.25</td>
</tr>
</tbody>
</table>

Figure 8 – Pile Group Comparison (load test data from Rollins et al., 1998)
In the case shown in Figure 6, it was assumed that the cap provided full restraint against rotation at the tops of the piles, and the CLM2.0 analyses were performed assuming fixed-head conditions. The results computed using CLM2.0 and Group4.0© agree well with the measured results for loads up to 8 kips per pile, but the computed deflections are larger than those measured for higher loads.

In the case shown in Figure 7, it was assumed that the cap provided complete restraint against rotation. The computed and measured results are in reasonable agreement overall, but the computed deflections are smaller than those measured for small loads, and larger than those measured for higher loads, indicating that the overall agreement may be somewhat fortuitous.

In the case shown in Figure 8, the soil profile contains sand, silt, and clay at shallow depth, making it difficult to determine what is a suitable average soil profile. It can be noted that for this case, the results computed using Group4.0© are in better agreement with the field measurements than those computed using CLM2.0. This case shows that the results computed using CLM2.0 are affected by the difficulty involved in representing the soil conditions around the top of the pile as uniform.

**Using CLM2.0 – A Guide**

The CLM2.0 workbook contains two spreadsheets, one for clay and one for sand. Examples are shown in Figures 9 and 10. Soil and pile properties are input in the block at the upper left in the sheets. Only the values shown in red on the spreadsheet can be changed. The explanatory text and the computed values shown in black are locked, and cannot be changed by the user. (Note that it is not possible to distinguish the “red” cells in Figures 9 and 10, which are shown entirely in black and white. The red cells are visible on the spreadsheet when it is opened in Excel.)

**Units.** The user enters the units of force and length to be used in the analysis. All of the soil and pile properties must be entered in consistent units, and the computed deflections and moments are also expressed in consistent units. For example, if the force unit is kips and the
Figure 9 – CLM2.0 Spreadsheet for Clay
Figure 10 – CLM2.0 Spreadsheet for Sand
length unit is inches, the input shear strength must be expressed in kips per square inch, the pile
diameter or width must be expressed in inches, the pile moment of inertia must be expressed in
inches to the fourth power, and the pile length must be expressed in inches. With these input
units, the computed values of pile deflection are expressed in inches, and bending moments are
expressed in inch-kips. Any system of units can be used provided they are used consistently.

**Soil Strength.** In the clay spreadsheet the soil strength is characterized by undrained shear
strength, $S_u$, and $\phi_u = 0$. In the sand spreadsheet the soil strength is characterized by a friction
angle, $\phi'$, and $c' = 0$.

**Unit weight.** The unit weight of sand is also required. This is the moist unit weight above
the water table, and the buoyant unit weight below the water table. Unit weight is not required
for clay.

**Pile Properties.** The outside diameter of the pile (or the width of the pile for non-circular
sections) is entered by the user.

For circular pipe piles the user also enters the inside diameter, and the spreadsheet computes
the moment of inertia.

For drilled shafts the user enters zero for inside diameter. In this case the spreadsheet
computes the moment of inertia, but the value is not displayed below the space for inside
diameter. Instead, the value computed by the program is displayed in the top right section
opposite $I_{\text{circ}}$.

For H-piles and other non-circular sections, the user leaves the inside diameter blank, and
enters the moment of inertia value.

$R_{cr}$ is the ratio of the $EI$ after cracking to the $EI$ before cracking. If a value smaller than 1.0 is
entered, the spreadsheet reduces the moment of inertia used in computing deflections and
moments. If it is likely that a reinforced concrete pile or drilled shaft will crack under the
imposed loads, the user should calculate or estimate the value of $R_{cr}$, and enter it.

$E_p$ is the modulus of elasticity of the pile material. For steel $E_p = 29,000$ ksi. The stiffness of
reinforced concrete piles and drilled shafts depends on a number of factors including the amount
of reinforcing, whether or not cracking will occur, and the magnitude of creep deflections under
sustained loads. In many cases it is reasonable to simplify the analyses by using the modulus of
elasticity for the concrete and the moment of inertia of the uncracked section. This simple approximation ignores the compensating factors of the contribution of the steel to the moment of inertia, and cracking of the concrete. If no data are available, the modulus of elasticity of the concrete can be estimated using the following approximate correlation with compressive strength:

<table>
<thead>
<tr>
<th>( f'_c ) (psi)</th>
<th>3,000</th>
<th>4,000</th>
<th>5,000</th>
<th>6,000</th>
</tr>
</thead>
<tbody>
<tr>
<td>( E_c ) (psi)</td>
<td>3,000,000</td>
<td>3,600,000</td>
<td>4,000,000</td>
<td>4,400,000</td>
</tr>
</tbody>
</table>

Pile length is needed to determine if the CLM method is applicable to the piles analyzed. One of the simplifications used in deriving the CLM method is that the piles are infinitely long. This simplifies the theory by eliminating the need to examine the boundary condition at the bottom of the pile. While no real pile is infinitely long, the theory has been found to be accurate for real piles provided they exceed a certain length, expressed as a minimum number of pile diameters. CLM2.0 computes \( L/D \) (the ratio of length to diameter) and displays this in the box at the upper right in the sheet. CLM2.0 also computes the ratio \( E_pR_s/S_u \) for clay and the ratio \( E_pR_s/\gamma D \phi K_P \) for sand. These ratios determine the minimum length for which the CLM method is applicable, as shown at the bottom of the box. Provided the value of \( L/D \) for the pile is larger than the minimum corresponding to the value of \( E_pR_s/S_u \) or \( E_pR_s/\gamma D \phi K_P \) for the pile, the assumption that the pile is infinitely long will not introduce much inaccuracy. If the value of \( L/D \) for the pile is not larger than the minimum value shown, the computed deflections will tend to be too small, and the computed moments will tend to be too large.

**Pile Group Geometry.** The number of rows and the spacing between rows (expressed in pile diameters) are entered in the box at the upper left. These characterize the pile group for analysis. The number of piles per row is not needed.

**Load Per Pile and Computed Results.** The lower part of the spreadsheets contains four tables where the user enters applied loads and applied moments, and where results are displayed. The loads are specified per pile. Thus, if a four-pile group carries a horizontal load of 50 kips, the load would be entered as 12.5 kips per pile. For convenience, each table has 10 rows, making it possible to compute and view results for 10 load cases at the same time.
The upper two tables show results for single piles and pile groups where the horizontal load is applied at the ground surface. The table on the left gives results for free-head conditions, and the table at the right gives results for fixed-head conditions.

The table at the center of the page is for piles loaded by moment only, with zero horizontal load at the ground surface. This loading condition is unusual, and not frequently encountered.

The table at the bottom of the page is for single piles subjected to both load and moment at the ground surface, which is often called the “flagpole” loading condition. These results are computed using the nonlinear superposition technique developed by Duncan et al. (1994).
References


