Module – 9

Seismic Analysis and Design of Various Geotechnical Structures
Seismic Stability of Finite Soil Slopes
Newmark’s Sliding block analysis (1965) *in Geotechnique*

D. Choudhury, IIT Bombay, India
Newmark’s Method (1965)

\[ FS = \frac{\cos \beta - k_h(t) \sin \beta \tan \phi}{\sin \beta + k_h(t) \csc \beta} \]

\[ k_y = \tan(\phi - \beta) \]

\[ a_{rel}(t) = a_b(t) - a_y = A - a_y \quad t_o \leq t \leq t_o + \Delta t \]

\[ v_{rel}(t) = \int_{t_o}^{t} a_{rel}(t) dt = \left[ -a_y \right]_{t_o}^{t} \]

\[ d_{rel}(t) = \int_{t_o}^{t} v_{rel}(t) dt = \frac{1}{2} \left[ A - a_y \right]_{t_o}^{t} \]

Deepankar Choudhury, IIT Bombay
Variation of pseudo-static factor of safety with horizontal pseudo-static coefficient for block on plane inclined at 20°. For $\phi = 20^0$, block is at the point of failure (FS = 1) under static conditions, so the yield coefficient is zero. For $\phi = 30^0$, and $\phi = 40^0$, yield coefficients are 0.17 and 0.36 respectively.
FIGURE 9.19 Diagram illustrating the Newmark method. (a) Acceleration versus time; (b) velocity versus time for the darkened portions of the acceleration pulses; (c) the corresponding downslope displacement versus time in response to the velocity pulses. (After Wilson and Keefer 1985.)
Model slope with circular failure surface and vertical slices and forces acting on $i^{th}$ slice

Typical Model Slope Study

The following parameters have been considered:

- Soil friction angle, $\phi = 35^0, 40^0, 45^0$.
- Angle of slope, $\beta = 20^0, 25^0, 30^0$.
- Horizontal seismic acceleration coefficient, $k_h = 0.1, 0.2, 0.3$.
- Vertical seismic acceleration coefficient, $k_v = 0, 0.5k_h, k_h$.
- Unit weight of soil, $\gamma = 20$ kN/m$^3$,
- Height of the slope, $H = 10$ m,

To avoid shear fluidization (Richards et al., 1990) and from stability criteria (Sarma, 1990), the following relation has to be satisfied,

$$\phi > \beta + \tan^{-1} \left[ k_h / (1-k_v) \right]$$

Choudhury et al. (2007)
Typical Results by Choudhury et al. (2007)

For slope angle $\beta = 25^0$
Seismic Stability of Tailing Dam
Introduction

Tailings Dam Incidents Due to Earthquake:

- The number of failures of tailings dams due to the earthquake is second highest.
- It can also be found that the most of dams were constructed by the upstream method of construction.

Animated picture showing dam failure due to seismic excitation. 
(http://www.wise-uranium.org)

Deepankar Choudhury, IIT Bombay, India
Seismic Analysis of Earthen Dam as per IS Code

Seismic Analysis as per IS: 7894 (1975):

- IS: 7894 (1975) basically uses a pseudo-static approach.
- The analysis can be performed by two methods. They are:
  1. Analysis for earthquake condition by circular arc method; and
  2. Analysis for earthquake condition by sliding wedge method.
- As per the analysis for earthquake condition by circular arc method the factor of safety

  $$FS = \sum \frac{C + (N - U) \tan \phi}{\sum W \sin \alpha + \sum W_1 \cos \alpha A_H} - \sum (W_1 \sin \alpha \tan \phi \times A_H)$$

Seismic Design as per IS: 1893 (1984):

- As per IS: 1893 (1984) seismic design procedure is based on the assumption that the portion of the dam above the rupture surface is rigid.

Deepankar Choudhury, IIT Bombay, India
The tailings earthen dam located at a site in eastern part of India comes under seismic zone II as per IS: 1893 (2002).

The objective of this analysis is to check the stability of the dam during earthquake events.

The dynamic soil-structure interaction analysis is mainly performed using FLAC$^{3D}$.

The tailings earthen dam has been analyzed for seven different conditions. Those are:
1. When the water table is at 3 m below the existing ground level,
2. When the water level in the reservoir is up to the top surface of the Pond Tailings portion,
3. When the reservoir is filled with water only (no tailings material),
4. When the Pond Tailings portion is filled with water,
5. When the Pond Tailings portion is filled with slurry,
6. When the reservoir is filled with slurry only, and
7. When the reservoir is empty.

As per UNEP considering 42% solid content slurry density is obtained as 12.5 kN/m³

Deepankar Choudhury, IIT Bombay, India
For seismic analysis the input accelerogram is applied at the base of the dam foundation is an actual earthquake history with peak horizontal acceleration of 0.112g. The vertical acceleration is ½ of the horizontal acceleration as per IS: 1893-1984 (Reaffirmed 2003).

Seismic excitation has been applied in two different combinations.

To preserve the non-reflecting seismic wave properties, the dynamic free-field boundaries are generated by using the ‘apply ff’ command.

Acceleration time history.

## Results of Static and Seismic Analysis

<table>
<thead>
<tr>
<th>For 1st phase of Tailings dam conditions</th>
<th>Maximum displacement (mm)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Under gravity loading only</td>
<td>Under seismic loading</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Positive x-acceleration and positive z-acceleration</td>
</tr>
<tr>
<td>Case-1: Water level is up to the top surface of the pond tailings portion with compacted tailings and pond tailings are in solid form</td>
<td>9.7</td>
<td>12.6</td>
</tr>
<tr>
<td>Case-2: In presence of stored water above compacted tailings portion</td>
<td>9.8</td>
<td>12.2</td>
</tr>
<tr>
<td>Case-3: The pond tailings portion is filled with slurry</td>
<td>9.9</td>
<td>12.7</td>
</tr>
<tr>
<td>Case-4: Reservoir is filled with slurry only</td>
<td>9.9</td>
<td>11.9</td>
</tr>
</tbody>
</table>

Results of Static and Seismic Analysis (Cont.)

For 1st phase dam

Static loading

Contours of displacement (in m) for static analysis (the Pond tailings portion is filled with slurry).

Seismic loading

Surface
Magfac = 1.000e+000
Live mech zones shown

Displacement
Maximum = 1.401e-002
Linestyle

Grids with displacement vectors after 30 sec of earthquake shaking (the Pond tailings portion is filled with slurry).

Results of Seismic Analysis after Applying Seismic Loading (Chakraborty and Choudhury, 2011)

Amplification of the base level input acceleration is about **3.93 times**.

Peak Horizontal Acceleration = 2.50 m/sec²

Peak Horizontal Acceleration = 4.32 m/sec²

Deepankar Choudhury, IIT Bombay, India
Validation of Fundamental Time Period

By FLAC\textsuperscript{3D} analysis

- Fundamental time period ($T_1$) = 0.3 sec (for 1\textsuperscript{st} phase)
- Fundamental time period ($T_2$) = 0.83 sec (for 2\textsuperscript{nd} phase)

As per the formula given in IS-1893-1984 (Reaffirmed 2002):

\[ T = 2.9 H_t \sqrt{\frac{\rho}{G}} \]

For 1\textsuperscript{st} phase dam,

\[ T_1 = 2.9 \times 10 \sqrt{\frac{1900}{1.5 \times 10^7}} = 0.33 \text{ sec} \]

For 2\textsuperscript{nd} phase dam,

\[ T_2 = 2.9 \times 28 \sqrt{\frac{1900}{1.5 \times 10^7}} = 0.9 \text{ sec} \]
Seismic Analysis: As per Seed (1979) and Terzaghi (1950) for Earthquake magnitude of about 6.4, the value of $k_h$ and $k_v$ can be taken as 0.1 and 0.05 respectively. But considering the extreme possible case the value of $k_h$ and $k_v$ are also considered as 0.15 and 0.075 respectively.

Factor of Safety and the Yield Acceleration values for seismic slope stability analysis obtained by using TALREN 4 software package for 2\textsuperscript{nd} phase

<table>
<thead>
<tr>
<th>Tailings dam conditions</th>
<th>Factor of Safety values for $k_h=0.1$ and $k_v=0.05$</th>
<th>Factor of Safety values for $k_h=0.15$ and $k_v=0.075$</th>
<th>Yield Acceleration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water table is at 3 m below the existing ground surface</td>
<td>1.97</td>
<td>1.75</td>
<td>0.47g</td>
</tr>
<tr>
<td>Water level in the reservoir is upto the top surface of the Pond Tailings portion</td>
<td>1.87</td>
<td>1.58</td>
<td>0.31g</td>
</tr>
<tr>
<td>Slurry in the reservoir is upto the top surface of the Pond Tailings portion</td>
<td>1.77</td>
<td>1.46</td>
<td>0.27g</td>
</tr>
</tbody>
</table>

### Results

<table>
<thead>
<tr>
<th>Name of the locations</th>
<th>Maximum value of pore pressure ratio ($r_u$)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1st phase dam</strong></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.67</td>
</tr>
<tr>
<td>2</td>
<td>0.70</td>
</tr>
<tr>
<td>3</td>
<td>0.87</td>
</tr>
<tr>
<td><strong>2nd phase dam</strong></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.69</td>
</tr>
<tr>
<td>2</td>
<td>0.66</td>
</tr>
<tr>
<td>3</td>
<td>0.74</td>
</tr>
<tr>
<td>4</td>
<td>0.75</td>
</tr>
<tr>
<td>5</td>
<td>0.72</td>
</tr>
</tbody>
</table>
Assessment of Liquefaction Potential in FLAC3D

By Chakraborty and Choudhury (2012)

Variation of pore water pressure ratio ($r_u$) at location 1, as the function of dynamic time.

Variation of pore water pressure ratio ($r_u$) at location 2, as the function of dynamic time.

Variation of pore water pressure ratio ($r_u$) at location 3, as the function of dynamic time.

Variation of pore water pressure ratio ($r_u$) at location 4, as the function of dynamic time.
Seismic Slope Stability Analysis of Tailing Dam by Pseudo-Static and Pseudo-Dynamic Methods by Chakraborty and Choudhury (2013)

Seismic Slope Stability Analysis of Tailing Dam by Pseudo-Static and Pseudo-Dynamic Methods by Chakraborty and Choudhury (2013)

Resisting force = \( c(l_{AB} \cos \beta + l_{BC} \cos \alpha) \)
\[ + [(W_1 + W_2) - (Q_{v1}(t) + Q_{v2}(t))] \cos \alpha \]
\[ - (Q_{h1}(t) + Q_{h2}(t)) \sin \alpha \tan \varphi \cos \alpha \]
\[ + [(W_3 + W_4) - (Q_{v3}(t) + Q_{v4}(t))] \cos \beta \]
\[ - (Q_{h3}(t) + Q_{h4}(t)) \sin \beta \tan \varphi \cos \beta \]

Driving force = \[ [(W_1 + W_2) - (Q_{v1}(t) + Q_{v2}(t))] \sin \alpha \cos \alpha + (Q_{h1}(t) + Q_{h2}(t)) \cos^2 \alpha \]
\[ + [(W_3 + W_4) - (Q_{v3}(t) + Q_{v4}(t))] \sin \beta \cos \beta + (Q_{h3}(t) + Q_{h4}(t)) \cos^2 \alpha \]

Seismic Design of Pile Foundation
Introduction

Pile foundations are often used for:

- supporting structures founded on soft soils
- supporting structures in liquefying soils: piles pass through the liquefying layer
Some of the failures of structures supported on piles due to recent earthquakes:

(a) Piled “Million Dollar” bridge after 1964 Alaska earthquake (USA);
(b) Piled “Showa Bridge” after 1964 Niigata earthquake (JAPAN);
(c) Piled tanks after 1995 Kobe earthquake (JAPAN)

[photo courtesy NISEE].
Performances of Pile foundations during some recent Earthquakes:

<table>
<thead>
<tr>
<th>No.</th>
<th>Case history</th>
<th>Earthquake event</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10 storey Hokuriku bldg</td>
<td>Nigata, 1964</td>
<td><strong>R.C.C piles</strong>, 0.40m dia. and 12m length. Lateral spreading reported.</td>
</tr>
<tr>
<td>2</td>
<td>14 storey bldg</td>
<td>Kobe, 1995</td>
<td><strong>R.C.C piles</strong>, 2.5m dia. &amp; 33 m length. Lateral spreading reported.</td>
</tr>
<tr>
<td>3</td>
<td>Hanshin expressway pier</td>
<td>Kobe, 1995</td>
<td><strong>R.C.C piles</strong>, 1.5m dia. &amp; 41m length. Lateral spreading is reported</td>
</tr>
<tr>
<td>4</td>
<td>LPG tank</td>
<td>Kobe, 1995</td>
<td><strong>R.C.C piles</strong>, 1.1m dia. &amp; 27m length. Lateral spreading reported.</td>
</tr>
<tr>
<td>5</td>
<td>Kobe Shimim hospital</td>
<td>Kobe, 1995</td>
<td><strong>Steel tubular piles</strong>, 30m length &amp; 0.66 dia.</td>
</tr>
</tbody>
</table>
### Performances of Pile foundations during some recent Earthquakes:

<table>
<thead>
<tr>
<th>Poor Performance</th>
<th>[Madabhushi et al.(2010)]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>No.</strong></td>
<td><strong>Case history</strong></td>
</tr>
<tr>
<td>1</td>
<td>Showa bridge</td>
</tr>
<tr>
<td>2</td>
<td>NHK building</td>
</tr>
<tr>
<td>3</td>
<td>NFCH building</td>
</tr>
<tr>
<td>4</td>
<td>Yachiyo bridge</td>
</tr>
<tr>
<td>5</td>
<td>Four storey fire house</td>
</tr>
</tbody>
</table>
Performances of Pile foundations during some recent Earthquakes:

<table>
<thead>
<tr>
<th>No.</th>
<th>Case history</th>
<th>Earthquake event</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>Three storey building, Fukae Kobe, 1995</td>
<td>Pre-stressed concrete piles, 0.40m dia. &amp; 20m pile length. Lateral spreading reported.</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Elevated port liner railway Kobe, 1995</td>
<td>R.C.C piles of 0.60m dia. and 12m pile length. Lateral spreading reported.</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>LPG tank 106,107 Kobe, 1995</td>
<td>R.C.C hollow piles of 0.30m dia. and 20m pile length. Lateral spreading reported.</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Harbour Masters building, Kandla port Bhuj, 2001</td>
<td>R.C.C piles of 0.40m dia. and 25m pile length. Lateral spreading reported.</td>
<td></td>
</tr>
</tbody>
</table>
Piles in liquefying soil under lateral loads:

**Force method**

JRA (1996): Idealisation for pile design in liquefying soils
Seismic analysis of pile foundations in liquefying soils: Winkler type models

For the seismic analysis of piles in liquefying soil, Winkler type models have been developed by Kagawa (1992); Yao and Nogami (1994), Fuji et al. (1998) and Liyanapathirana and Poulos (2005).

For piles in non-liquefying soils, Abghari and Chai (1995) and Tabesh and Poulos (2001) had developed pseudo-static approaches.

Liyanapathirana and Poulos (2005) developed pseudo-static method which has two solution stages:

Carry out Ground response analysis

Pile is analysed as nonlinear beam on elastic foundation- considering kinematic and inertial Interactions.
➢ Soil Liquefies
➢ Looses its Shear Strength
➢ Starts flowing and dragging with it any non-liquefiable crust above it.
Concept of pile failure as per Ishihara (1997):

**Top down effect**: At the onset of shaking, inertia forces are transferred to the top of the pile and then to soil.

**Bottom up effect**: Seismic motion had already passed the peak and shaking may still be persistent with lesser intensity and therefore the inertia force transmitted from the superstructure will be insignificant.

- Under such a loading condition, the maximum bending moment induced by the pile may not occur near the pile head but at a lower portion at some depth and this is referred as bottom-up effect.
Piles in liquefying soil under lateral loads:

Force method

JRA (1996): Idealisation for pile design in liquefying soils
Prior to the development of pore water pressure, the inertia force from the superstructure may dominate.

- Kinematic forces from the liquefied soil start acting with increasing pore pressure.
- Towards the end of shaking, kinematic forces would dominate and have a significant effect on pile performance particularly when permanent displacements occur in laterally spreading soil.

[see Choudhury et al., 2009, Proc. of National Academy of Sciences, India, Springer, Sec. A]
Case-Specific Design of Pile Foundations under Earthquake Conditions
## Typical Bore hole data for MBH# 1: Mangalwadi site, Mumbai

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Stratum</th>
<th>Layer thickness (m)</th>
<th>Depth below GL (m)</th>
<th>SPT ‘N’ value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Filled up soil</td>
<td>1.5</td>
<td>1.5</td>
<td>10</td>
</tr>
<tr>
<td>2</td>
<td>Yellowish loose sand</td>
<td>1.5</td>
<td>3.0</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.5</td>
<td>4.5</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.5</td>
<td>6.0</td>
<td>16</td>
</tr>
<tr>
<td>3</td>
<td>Black clayey soil</td>
<td>2.0</td>
<td>8.0</td>
<td>20</td>
</tr>
<tr>
<td>4</td>
<td>Yellowish clayey soil</td>
<td>1.8</td>
<td>9.8</td>
<td>25</td>
</tr>
<tr>
<td>5</td>
<td>Greyish hard rock</td>
<td>-</td>
<td>&gt;9.8</td>
<td>-</td>
</tr>
</tbody>
</table>

Equivalent linear ground response Analysis

Typical Modulus reduction curve for soil

\[ G \text{ (in kPa)} = 12000 \times N^{0.80} \]  
(Ohasaki & Iwasaki, 1973)
Typical Damping curve for soil
## Typical Input Parameters for DEEPSOILv3.5 (for site MBH#1)

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Layer</th>
<th>Thickness (m)</th>
<th>Unit wt. (kN/m³)</th>
<th>Vs (m/s)</th>
<th>Damping ratio (%)</th>
<th>Ref. strain(%)</th>
<th>Ref. Stress (MPa)</th>
<th>β</th>
<th>s</th>
<th>b</th>
<th>d</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>back-fill</td>
<td>1.5</td>
<td>16</td>
<td>203</td>
<td>0.5</td>
<td>0.0183</td>
<td>0.01380</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>loose-sand</td>
<td>1.5</td>
<td>17</td>
<td>218</td>
<td>0.5</td>
<td>0.0326</td>
<td>0.02857</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>loose-sand</td>
<td>1.5</td>
<td>17</td>
<td>226</td>
<td>0.5</td>
<td>0.0433</td>
<td>0.04000</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>loose-sand</td>
<td>1.5</td>
<td>18</td>
<td>245</td>
<td>0.5</td>
<td>0.0431</td>
<td>0.04700</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>clay</td>
<td>2.0</td>
<td>18</td>
<td>268</td>
<td>0.5</td>
<td>0.0812</td>
<td>0.10700</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>clay</td>
<td>1.8</td>
<td>18</td>
<td>293</td>
<td>0.5</td>
<td>0.0733</td>
<td>0.11700</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>
Typical Snapshot of ground response analysis in DEEPSOIL
Typical Input for ground response analysis in DEEPSOIL
Input Earthquake Motions considered in present study

Acceleration-time history of 2001 Bhuj earthquake
[Govindaraju et al. (2004)]

Acceleration-time history of 1989 Loma Prieta earthquake
[Hashash et al. (2008)]

Similarly 1989 Loma Gilory earthquake (MHA = 0.442g and Tm = 0.391s, and 1995 Kobe earthquake (MHA = 0.834g and Tm = 0.641s) motions are used
Results: Influence of local soil sites on the ground response

MHA_{GL} = 0.251g;  
MHA_{GL}/MHA_{base} = 2.37

MHA_{GL} = 0.278g;  
MHA_{GL}/MHA_{base} = 2.62

MHA_{GL} = 0.364g;  
MHA_{GL}/MHA_{base} = 3.44

2001 Bhuj earthquake Input motion at bed rock level
Typical Results

Influence of input motion on ground response

Response Spectrum for 5% damping at GL for MBH#1

Fourier amplitude ratio (FAR) vs. Frequency (Hz) at GL for MBH#1
Amplification of acceleration vs. depth (m)
Typical Results

Acceleration time history at GL

MBH#1, MHA = 0.106g

WBH#1, MHA = 0.106g

BBH#1, MHA = 0.106g

MHA_{GL} / MHA_{BEDROCK} = 2.22;

MHA_{GL} = 0.235g

MHA_{GL} / MHA_{BEDROCK} = 2.31;

MHA_{GL} = 0.245g

MHA_{GL} / MHA_{BEDROCK} = 2.53;

MHA_{GL} = 0.268g
Model considered for single Pile passing through liquefied layer

Soil-pile analysis considering ground deformations using finite difference technique
Governing Equations for solving the basic differential equation of laterally loaded pile in liquefied zone is given below:

\[ EI \frac{d^4 y}{dz^4} = - k_h D (y - y_g) \]

\( y_g = \) ground displacement
\( D = \) diameter of pile
\( k_h = \) Subgrade modulus

\[ k_h = k_{hn} S_f \]

\( S_f \) is scaling factor varying from 0.001 to 0.01 (Ishihara and Cubrinovski, 1998) as compared to normal soil condition where there is no liquefaction.

Phanikanth et al. (2013), Int. Jl. of Geomech., ASCE

Tokimatsu et al. 1998

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Bending moment in non liquefied and liquefied soil for free headed single pile with floating tip in Mumbai

D. Choudhury, IIT Bombay, India
Response of Laterally Loaded Pile: Liquefying Soils

Pile behaviour is Rigid.
Relative movement between soil and pile is dominant.
Deflection (inertial) is also predominant.

[Phanikanth et al. (2013), Int. Jl. of Geomech., ASCE]
Comparison of Pile Responses in Liquefying and Non-liquefying soils

<table>
<thead>
<tr>
<th>Ground motion</th>
<th>Non-liquefying soil</th>
<th>Liquefying soil</th>
<th>Amplification</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Deflection at top (mm)</td>
<td>Peak bending moment (kNm)</td>
<td>Deflection at top (mm)</td>
</tr>
<tr>
<td>Loma Gilroy(1989)</td>
<td>70.754</td>
<td>32.672</td>
<td>192.163</td>
</tr>
<tr>
<td>Kobe(1995)</td>
<td>144.375</td>
<td>26.274</td>
<td>241.977</td>
</tr>
</tbody>
</table>

[Choudhury et al. (2013)]
Effect of Depth of Liquefying Layer -
Free headed top and floating tip

Pile deflections (Combined) along depth for Bhuj (2001) earthquake motion

[Choudhury et al. (2013)]
Effect of depth of liquefying layer on the pile bending moments (Combined) along depth for Bhuj (2001) motion - Free headed top and floating tip

Moment (Combined) kNm

Distance from top(m) vs Moment (Combined) kNm

$L=10.0\text{m}; r=0.25\text{m}; E_p=2.74e07\text{ kN/sq.m}; s_f=0.01$

[Choudhury et al. (2013)]