Geotechnical Earthquake Engineering

by

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Lecture – 39
Module – 9

Seismic Analysis and Design of Various Geotechnical Structures
Seismic Design of Waterfront Reinforced Soil-Wall
Typical Reinforced Soil-Wall used as Waterfront Retaining Structure during Earthquake (Pseudo-dynamic approach)

Ahmad and Choudhury (2008)

Typical Results for Reinforcement Strength

Ahmad and Choudhury (2008)

Typical Reinforced Soil-Wall used as Waterfront Retaining Structure during Earthquake (External Stability)

Choudhury and Ahmad (2009)

Typical Result for Length of Reinforcement

Seismic Design of Shallow Footings
Figure 2. (a) Geometry of the curved failure surface. (b) Forces considered in the analysis.

Choudhury and Subba Rao (2005)

D. Choudhury, IIT Bombay, India
Design charts given by Choudhury and Subba Rao (2005)

\[ q_{ud} = cN_{cd} + qN_{qd} + 0.5 \gamma BN_{\gamma_d} \]
Design Charts for Seismic Bearing Capacity Factors

\[ q_{ud} = cN_{cd} + qN_{qd} + 0.5\gamma BN_{\gamma d} \]

\[ N_{qd} = \frac{1}{k_h} \left[ \frac{K_{p,qd1} \sin (\alpha_1 - \phi)}{\cos \phi} + \frac{mK_{p,qd2} \sin (\alpha_2 - \phi)}{\cos \phi_2} \right] \frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2} \]

Seismic Bearing Capacity of Shallow Strip Footing Using Pseudo-Dynamic Approach

Seismic Bearing Capacity Factor & Comparison Using Pseudo-dynamic approach

Effect of Soil Amplification on Bearing Capacity Factor

Comparison of present result with other methods

Ghosh and Choudhury (2011) – Pseudo-Dynamic Approach

Seismic Stability of Finite Soil Slopes
CLASSICAL THEORIES in Seismic Slope Stability

- Terzaghi’s method (1950)
- Newmark’s sliding block analysis (1965)
- Seed’s improved procedure for pseudo-static analysis (1966)
- Modified Swedish Circle method (1968)
- Modified Taylor’s method (1969)
Terzaghi’s Wedge Method (1950)

$N$ normal force acting on the slip surface, kN
$T$ shear force acting along the slip surface, kN. The shear force is also known as the resisting force because it resists failure of the wedge. Based on the Mohr-Coulomb failure law, the shear force is equal to the following:

For a total stress analysis: $T = cL + N \tan \phi$, or $T = s_u L$

For an effective stress analysis: $T = c'L + N' \tan \phi'$

where $L$ length of the planar slip surface, m
$c, \phi$ shear strength parameters in terms of a total stress analysis
$s_u$ undrained shear strength of the soil (total stress analysis)
$N$ total normal force acting on the slip surface, kN
$c', \phi'$ shear strength parameters in terms of an effective stress analysis
$N'$ effective normal force acting on the slip surface, kN
Terzaghi’s Wedge Method (1950)

Total stress pseudostatic analysis:

\[
FS = \frac{\text{resisting force}}{\text{driving forces}} = \frac{cL + N \tan \phi}{W \sin \alpha + F_h \cos \alpha} = \frac{cL + (W \cos \alpha - F_h \sin \alpha) \tan \phi}{W \sin \alpha + F_h \cos \alpha}
\]

Effective stress pseudostatic analysis:

\[
FS = \frac{c'L + N' \tan \phi'}{W \sin \alpha + F_h \cos \alpha} = \frac{c'L + (W \cos \alpha - F_h \sin \alpha - uL) \tan \phi'}{W \sin \alpha + F_h \cos \alpha}
\]

\[
FS = \frac{\text{resisting force}}{\text{driving force}} = \frac{cl_{ab} + (W - F_v) \cos \beta - F_h \sin \beta \tan \phi}{(W - F_v) \sin \beta + F_h \cos \beta}
\]
Newmark’s Sliding block analysis (1965) in *Geotechnique*
**Newmark’s Method (1965)**

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

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

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

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

\[ d_{rel}(t) = \int_{t_o}^{t} v_{rel}(t) dt = \frac{1}{2} \left[ -a_y \right]_{t_o}^{t} \]
Newmark’s sliding block analysis (1965)

Variation of pseudo-static factor of safety with horizontal pseudo-static coefficient for block on plane inclined at 20°. For $\phi = 20^\circ$, block is at the point of failure (FS = 1) under static conditions, so the yield coefficient is zero. For $\phi = 30^\circ$, and $\phi = 40^\circ$, 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.)
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**Modified Swedish Circle Method**

\[
F = \sum \left[ \Delta S \cdot R + (N - N') \tan \phi \cdot R \right] \over \sum (k' + T') R
\]

\[
F = \frac{cS + \sum (N - N') \tan \phi}{\sum (k' + T')}
\]
## Method of Slices

<table>
<thead>
<tr>
<th>Type of method of slices</th>
<th>Assumption concerning interslice forces</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary method of slices</td>
<td>Resultant of interslice forces is parallel to average inclination of slice</td>
<td>Fellenius (1936)</td>
</tr>
<tr>
<td>Bishop simplified method</td>
<td>Resultant of interslice forces is horizontal (no interslice shear forces)</td>
<td>Bishop (1955)</td>
</tr>
<tr>
<td>Janbu simplified method</td>
<td>Resultant of interslice forces is horizontal (a correction factor is used to account for interslice shear forces)</td>
<td>Janbu (1968)</td>
</tr>
<tr>
<td>Janbu generalized method</td>
<td>Location of interslice normal force is defined by an assumed line of thrust</td>
<td>Janbu (1957)</td>
</tr>
<tr>
<td>Spencer method</td>
<td>Resultant of interslice forces is of constant slope throughout the sliding mass</td>
<td>Spencer (1967, 1968)</td>
</tr>
<tr>
<td>Morgenstern-Price method</td>
<td>Direction of resultant interslice forces is determined by using a selected function</td>
<td>Morgenstern and Price (1965)</td>
</tr>
</tbody>
</table>
Modified Taylor’s Method

\[ F_c = \frac{c}{\gamma H} \left( \frac{2 \cos (x + \psi) \cot y + 2 \sin (x + \psi)}{\sqrt{1 + \alpha_h^2} \sin x \left[ \frac{1}{2} \csc^2 x (y \csc^2 y - \cot y) + \cot x - \cot \beta \right]} \right) \]
Sarma (1975) in Geotechnique

Model of rigid block on a sloping surface

- Factor of safety and displacement along a failure surface depend on geometry, strength of material, pore-pressure parameters and magnitude of the inertia force.
- Total displacement is proportional to the square of the duration.
- Both the factor of safety and the displacement are unaffected by the inclination of the inertia force.
Sabhahit, Basudhar and Madhav (1996)

Model slope with planar failure surface and horizontal slices
Model reinforced soil slope with circular failure surface and horizontal slices and forces acting on i\textsuperscript{th} slice
Pre and post shaking profiles of slopes 1–4 showing deformed spaghetti strands (near vertical lines shown in soft zone [upper portion] of model). Pretest geometry is shown with dashed line and localized shear displacement surfaces shown with dark solid line (dashed where inferred).
Choudhury, Basu and Bray (2007)

Model slope with circular failure surface and vertical slices
and forces acting on i\textsuperscript{th} slice

Choudhury, Basu and Bray (2007)

\[ FOS = \frac{P_i \tan \phi + cL}{(W + Fv_i)R \sin \theta} \left\{ - \Delta E \left( R \cos \theta - H_i / 3 \right) + E_{i-1}L \sin \theta \right\} \]

\[ \frac{(W + Fv_i)R \sin \theta + Fh_i \left( R \cos \theta - H_i / 2 \right)}{\Delta l} \]
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

<table>
<thead>
<tr>
<th>β (deg.)</th>
<th>φ (deg.)</th>
<th>0</th>
<th>0.1</th>
<th>0.2</th>
<th>0.3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>k_v</td>
<td>k_v</td>
<td>k_v</td>
<td>k_v</td>
</tr>
<tr>
<td>20</td>
<td>35</td>
<td>1.371</td>
<td>1.223</td>
<td>1.234</td>
<td>1.253</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>1.534</td>
<td>1.185</td>
<td>1.189</td>
<td>1.193</td>
</tr>
<tr>
<td></td>
<td>45</td>
<td>1.682</td>
<td>1.415</td>
<td>1.423</td>
<td>1.431</td>
</tr>
<tr>
<td>25</td>
<td>35</td>
<td>1.538</td>
<td>1.265</td>
<td>1.302</td>
<td>1.309</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>1.552</td>
<td>1.311</td>
<td>1.319</td>
<td>1.353</td>
</tr>
<tr>
<td></td>
<td>45</td>
<td>1.946</td>
<td>1.593</td>
<td>1.604</td>
<td>1.615</td>
</tr>
<tr>
<td>30</td>
<td>35</td>
<td>1.543</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>1.590</td>
<td>1.325</td>
<td>1.334</td>
<td>1.341</td>
</tr>
<tr>
<td></td>
<td>45</td>
<td>1.984</td>
<td>1.603</td>
<td>1.615</td>
<td>1.627</td>
</tr>
</tbody>
</table>

(Note: In the above table ‘-’ refers that the results are not valid.)

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

For slope angle $\beta = 25^0$
Comparison of DFS Values [Choudhury et al. (2007)]

<table>
<thead>
<tr>
<th>$\phi$ (deg.)</th>
<th>$k_h$</th>
<th>Newmark (1965)</th>
<th>Present study</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>0.1</td>
<td>1.45</td>
<td>1.31</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>1.15</td>
<td>1.15</td>
</tr>
<tr>
<td>40</td>
<td>0.1</td>
<td>1.74</td>
<td>1.35</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>1.38</td>
<td>1.20</td>
</tr>
<tr>
<td></td>
<td>0.3</td>
<td>1.13</td>
<td>1.04</td>
</tr>
<tr>
<td>45</td>
<td>0.1</td>
<td>2.08</td>
<td>1.63</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>1.64</td>
<td>1.44</td>
</tr>
<tr>
<td></td>
<td>0.3</td>
<td>1.34</td>
<td>1.28</td>
</tr>
</tbody>
</table>

For slope angle $\beta = 20^0$ and $k_v = 0$
Seismic Stability of Tailing Dam
Introduction

- A number of tailings earthen dams have failed during past earthquakes. The failure of tailings dam ultimately results into the release of the stored tailings waste deposit which often fairly dangerous because of the level of toxicity or corrosivity or both to human life and other living beings.

- **Classification of tailings dams:** There are principally two types of tailings dams,
  
  i. **Water retention type dam;** and
  
  ii. **Raised embankment type,** which may be of three types (on the basis of method of construction):
    
    a) **Upstream method of construction;**
    
    b) **Downstream method of construction;** and
    
    c) **Centerline method of construction.**
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)

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Introduction

Tailings dam failure incidents caused by various reasons (ICOLD, 2001)

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Available Methods

- **Pseudo-static method of stability analysis**
  - The analysis is relatively simple and straightforward. Very much popular from 1920s to 1960s.

- **Sliding Block Method for stability**
  - Newmark (1965) proposed based on the deformation, considering the sliding mass as rigid block on inclined plane.

- **Shear Beam Model for stability**
  - Mononobe (1936) first introduced the 1-D ‘shear beam’ model for earth dams.
  - Gazetas (1981) proposed an improved ‘inhomogeneous’ shear beam model.
Available Methods

- **Use of FEM and FDM**
  - Clough and Chopra (1966) first introduced the FEM for 2-D plane-strain analysis.
  - Chopra (1967), Chopra and Perumalswami (1969), Idriss et al. (1973) and others also worked.
  - Makdisi et al. (1982) developed 3-D finite element formulation using prismatic longitudinal.

- **Use of Software Packages**
  - Seid-Karbasi and Byrne (2004) analyzed the Mochikoshi tailings dam using FLAC.
  - Zhu et al. (2005) have presented a 2-D seismic stability for a levee using PLAXIS and TELDYN.
  - Piao et al. (2006) used FLAC to evaluate an innovative remediation design.

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Available Methods

Use of Centrifuge Modeling
- **Arulanandan et al. (1993)** conducted test at 30g to find earthquake effect on dam.
- **Elgamal et al. (2003)** investigated the effect of rigid model container size on earth dam.
- **Adalier and Sharp (2004)** conducted four tests at 100g to study the dynamic behaviour of an embankment founded on liquefied soil layer and the effect of foundation densification.

Other Analytical Methods
- **Nimbalkar and Choudhury (2010)** studied analytically the effect of soil amplification on the seismic behavior of tailings dam.
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
  i. Analysis for earthquake condition by circular arc method; and
  ii. Analysis for earthquake condition by sliding wedge method.
- As per the analysis for earthquake condition by circular arc method the factor of safety

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

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.

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Case Study – by Chakraborty and Choudhury (2011)

- 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$^{3D}$.

<table>
<thead>
<tr>
<th>Soil Designation</th>
<th>Shear strength parameters</th>
<th>Permeability</th>
<th>Dynamic Soil Properties</th>
<th>Unit weight</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$c$ (kPa)</td>
<td>$\phi$ (Degree)</td>
<td>$k$ (m/sec)</td>
<td>$G$ (kPa) ($x 10^6$)</td>
</tr>
<tr>
<td>Impervious clay cover</td>
<td>80</td>
<td>17</td>
<td>$1 \times 10^{-9}$</td>
<td>2.4</td>
</tr>
<tr>
<td>(Region1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Random fill</td>
<td>70</td>
<td>20</td>
<td>$1.5 \times 10^{-9}$</td>
<td>1.5</td>
</tr>
<tr>
<td>(Region2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Drain layer</td>
<td>0</td>
<td>32</td>
<td>$1 \times 10^{-4}$</td>
<td>4.05</td>
</tr>
<tr>
<td>(Region3)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rock toe</td>
<td>0</td>
<td>42</td>
<td>$1 \times 10^{-2}$</td>
<td>4.05</td>
</tr>
<tr>
<td>(Region4)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bed rock layer</td>
<td>300</td>
<td>35</td>
<td>$1 \times 10^{-5}$</td>
<td>$2 \times 10^4$</td>
</tr>
<tr>
<td>(Region5)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pond tailings</td>
<td>12</td>
<td>14.7</td>
<td>$1 \times 10^{-8}$</td>
<td>4.56</td>
</tr>
<tr>
<td>Compacted tailings</td>
<td>15.2</td>
<td>14.7</td>
<td>$1 \times 10^{-8}$</td>
<td>9.54</td>
</tr>
</tbody>
</table>

* The water table is considered at a depth of 3.0 m from the existing ground level.
** The properties of the existing soil layer are assumed same as the properties of the random fill.
The dam is constructed in 2 phases.

In the 1st phase of construction, dam height is upto 10 m above GL.

In the 2nd phase of construction, dam height is upto 28 m above GL.
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
Model Generation and Application of Gravity Loading and Seismic Loading (Cont.)

- 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 $\frac{1}{2}$ 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

<table>
<thead>
<tr>
<th>Surface</th>
<th>Magfac = 1.000e+000</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Live mesh zones shown</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Displacement</th>
<th>Maximum = 1.401e-002</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Linestyle</td>
</tr>
</tbody>
</table>

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

## Results of Static and Seismic Analysis

<table>
<thead>
<tr>
<th>For 2\textsuperscript{nd} phase of Tailings dam conditions</th>
<th>Maximum displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Under gravity loading only</td>
</tr>
<tr>
<td></td>
<td></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>51.5</td>
</tr>
<tr>
<td>Case-2: In presence of stored water above compacted tailings portion</td>
<td>53.9</td>
</tr>
<tr>
<td>Case-3: The pond tailings portion is filled with slurry</td>
<td>54.7</td>
</tr>
<tr>
<td>Case-4: Reservoir is filled with slurry only</td>
<td>54.8</td>
</tr>
</tbody>
</table>

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

Accelration time history at +5 m height of the dam in x-direction for 1st phase dam.

**Peak Horizontal Acceleration = 2.50 m/sec^2**

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

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## Slope Stability Analysis

<table>
<thead>
<tr>
<th>Tailings Dam Conditions</th>
<th>Static FoS</th>
<th>Seismic FoS using TALREN 4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FLAC(^3D)</td>
<td>TALREN 4</td>
</tr>
<tr>
<td></td>
<td>For (k_h = 0.1) and (k_v = 0.05)</td>
<td>For (k_h = 0.15) and (k_v = 0.075)</td>
</tr>
<tr>
<td><strong>For 1(^{st}) phase dam</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>In presence of water in the tailings portion (Case-1 and 2)</td>
<td>3.81</td>
<td>3.92</td>
</tr>
<tr>
<td>In presence of slurry in the tailings portion (Case-3 and 4)</td>
<td>3.74</td>
<td>3.90</td>
</tr>
<tr>
<td><strong>For 2(^{nd}) phase dam</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>In presence of water in the tailings portion (Case-1 and 2)</td>
<td>2.42</td>
<td>2.64</td>
</tr>
<tr>
<td>In presence of slurry in the tailings portion (Case-3 and 4)</td>
<td>2.31</td>
<td>2.62</td>
</tr>
</tbody>
</table>

Validation of Fundamental Time Period

By FLAC$^3$D analysis

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

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

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

For 1$^{\text{st}}$ phase dam,
\[ T_1 = 2.9 \times 10 \sqrt{\frac{1900}{1.5 \times 10^7}} = 0.33 \text{ sec} \]

For 2$^{\text{nd}}$ phase dam,
\[ T_2 = 2.9 \times 28 \sqrt{\frac{1900}{1.5 \times 10^7}} = 0.9 \text{ sec} \]

where
- $T$ = fundamental period of the earth dam in sec,
- $H_t$ = height of the dam above toe of the slopes,
- $\rho$ = mass density of the shell material, and
- $G$ = modulus of rigidity of the shell material.

### Static Analysis:

Factor of Safety values for static slope stability analysis obtained by using three different software packages for the 2nd phase dam

<table>
<thead>
<tr>
<th>Tailings dam conditions</th>
<th>Factor of Safety values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FLAC$^{3D}$</td>
</tr>
<tr>
<td>Water table is at 3m below the existing ground surface</td>
<td>2.73</td>
</tr>
<tr>
<td>Water level in the reservoir is upto the top surface of the Pond Tailings portion</td>
<td>2.42</td>
</tr>
<tr>
<td>Slurry is upto the top surface of the Pond Tailings portion</td>
<td>2.31</td>
</tr>
</tbody>
</table>

Slope Stability Analysis using FLAC\textsuperscript{3D}, TALREN 4 and SLOPE/W

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>

Slope Stability Analysis using FLAC$^3$D, TALREN 4 and SLOPE/W

Seismic Analysis (Cont):

TALRE 4 result when the WT is 3m BGL.

TALREN-4 result when the water level in the reservoir is upto the top surface of the Pond Tailings portion.

Deepankar Choudhury, IIT Bombay, India
Assessment of Liquefaction Potential

By Chakraborty and Choudhury (2012)

- When the water level in the reservoir is up to the top surface of the Pond Tailings portion, the liquefaction potential analysis is carried out in FLAC$^3$D (2006).

- Except tailings portion, all other components of the dam have quite high values of cohesion. So, the tailings portion is only considered for the liquefaction analysis due to composition of loose tailings deposit.

Simulation of Liquefaction conditions in FLAC$^3$D

- The liquefaction model by Byrne (1991) is assigned to dam soils with parameters set to correspond with SPT measurements.

- Command ‘model finn’ is used. This command in FLAC$^3$D incorporates the pore pressure generation effect into the Mohr-Coulomb model.

Because of the presence of some cohesion in the tailings portion, the shear strength of the tailings materials will never become zero. But, there may be significant shear strength loss due to decrease in effective stress during earthquake.

Assessment of Liquefaction Potential by Chakraborty and Choudhury (2012)

### 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>1st phase dam</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>2nd phase dam</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 FLAC$^{3D}$

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)

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

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 \]