Module – 9
Seismic Analysis and Design of Various Geotechnical Structures
Seismic Design of Waterfront Retaining Wall
Applications on Waterfront Retaining Wall / Seawall

-A soil retaining armoring structure, generally massive

-To defend a shoreline against wave attack

-Designed primarily to resist wave action along high value coastal property

(source: www.mojosballs.com/main.htm)

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Available Literature

On Earthquake
Mononobe-Okabe (1926, 1929)
Madhav and Kameswara Rao (1969)
Richards and Elms (1979)
Saran and Prakash (1979)
Prakash (1981)
Nadim and Whitman (1983)
Steedman and Zeng (1990)
Ebeling and Morrison (1992)
Das (1993)
Kramer (1996)
Kumar (2002)
Choudhury and Subba Rao (2005)
Choudhury and Nimbalkar (2006)
And many others…………..

On Tsunami/Hydrodynamics
Westergaard (1933)
Fukui et al. (1962)
Ebeling and Morrison (1992)
Mizutani and Imamura (2001)
CRATER (2006)
And few others……….
Design Solutions for Waterfront Retaining Walls (Sea Walls) subjected to both Earthquake and Tsunami

(1) For Tsunami attacking the wall (passive case)
   (a) Against Sliding mode of failure
   (b) Against Overturning mode of failure

(2) For Tsunami receding away from wall (active case)
   (a) Against Sliding mode of failure
   (b) Against Overturning mode of failure

Case 1(a): Passive Case – Pseudo-static

Combined effects of tsunami and earthquake
On rigid waterfront retaining wall

Case 1(a): Passive Case – Pseudo-static (Results)

Factor of safety against sliding

Factor of safety against overturning

Design solutions for Active Case (pseudo-static) proposed by Choudhury and Ahmad (2007)

Factor of Safety against Sliding Failure:

$$FS_{sliding_r} = \frac{\frac{1}{2} \gamma_w (h/H)^2 + \mu ((1-k_v)(b/H)\gamma_c + \frac{1}{2} K_{ae} \gamma \sin \delta)}{\frac{1}{2} \gamma_{we} (h/H)^2 + \frac{7}{12} k_h \gamma_w (h/H)^2 + \frac{1}{2} K_{ae} \gamma \cos \delta + k_h (b/H) \gamma_c}$$

$$FS_{sliding_f} = \frac{\frac{1}{2} \gamma_w (h/H)^2 + \mu ((1-k_v)(b/H)\gamma_c + \frac{1}{2} K_{ae} \gamma \sin \delta)}{\frac{1}{2} \gamma_{we} (h/H)^2 + \frac{7}{6} k_h \gamma_w (h/H)^2 + \frac{1}{2} K_{ae} \gamma \cos \delta + k_h (b/H) \gamma_c}$$

Factor of Safety against Overturning Failure:

$$FS_{overturning_r} = \frac{\frac{1}{6} \gamma_w (h/H)^3 + \frac{1}{2} (b/H)^2 (1-k_v) \gamma_c + \frac{1}{2} K_{ae} \gamma (b/H) \sin \delta}{\frac{1}{6} \gamma_{we} (h/H)^3 + (2.8/12) k_h \gamma_w (h/H)^3 + \frac{1}{4} K_{ae} \gamma \cos \delta + \frac{1}{2} k_h (b/H) \gamma_c}$$

$$FS_{overturning_f} = \frac{\frac{1}{6} \gamma_w (h/H)^3 + \frac{1}{2} (b/H)^2 (1-k_v) \gamma_c + \frac{1}{2} K_{ae} \gamma (b/H) \sin \delta}{\frac{1}{6} \gamma_{we} (h/H)^3 + \frac{5.6}{12} k_h \gamma_w (h/H)^3 + \frac{1}{4} K_{ae} \gamma \cos \delta + \frac{1}{2} k_h (b/H) \gamma_c}$$

Typical Results by Choudhury and Ahmad (2007)

Factor of Safety against Sliding

Factor of Safety against Overturning

Active Case with Pseudo-dynamic method

Forces acting on typical seawall subjected to earthquake and tsunami (active case)

Comparison of Results

Seismic Design of Reinforced Soil-Wall
A week after the 1995 Kobe Earthquake

GRS RW for a rapid transit at Tanata

The wall survived!

Ref: Tatsuoka (2010)
Typical Design of Earthquake Resistant Reinforced Soil-Wall (Internal Stability)

\[
K = \frac{\sum t_j}{0.5 \gamma H^2} \approx \frac{t_j}{\gamma h_j D_j}
\]

\[
l_{e,j} = \frac{t_j}{2 \sigma_{v,j} C_i \tan \phi}
\]

Nimbalkar et al. (2006)

Typical Design of Earthquake Resistant Reinforced Soil-Wall (Internal Stability)

For $H = 5 \text{ m}$, $\phi = 30^0$

Reinforcement strength and length required as per Nimbalkar et al. (2006)

Comparison of Results

Typical Design of Earthquake Resistant Reinforced Soil-Wall (External Stability)

Figure 1. Two-part wedge mechanism and forces considered in direct sliding analysis under seismic conditions

Figure 2. Forces considered in overturning analysis under seismic conditions

Sliding stability

Overturning stability

Choudhury et al. (2007)

Typical Design of Earthquake Resistant Reinforced Soil-Wall (External Stability)

Length of Reinforcement for Sliding stability
Choudhury et al. (2007)

Length of Reinforcement for Overturning stability

Comparison of Results

Table 1. Typical comparison of present results for required geosynthetic length, $L_{ds}$, with pseudo-static results by Ling and Leshchinsky (1998)

<table>
<thead>
<tr>
<th>$k_r$</th>
<th>Required length of geosynthetic layer, $L_{ds}$</th>
<th>Method proposed by Ling and Leshchinsky (1998)</th>
<th>Present study</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0.651</td>
<td>0.725</td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>0.715</td>
<td>0.963</td>
<td></td>
</tr>
<tr>
<td>0.2</td>
<td>0.828</td>
<td>1.427</td>
<td></td>
</tr>
</tbody>
</table>

Data used: $k_h = 0.2$, $\phi = 30^\circ$, $H/\lambda = 0.167$, $H/\eta = 0.09$, $H = 5$ m, $\beta = 90^\circ$.

Choudhury et al. (2007)

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

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

Choudhury and Ahmad (2009)

Typical Result for Length of Reinforcement

Choudhury and Ahmad (2009)

Reinforcement required for Soil-Wall used as Waterfront Retaining Structure during Earthquake (Pseudo-static approach, Ahmad and Choudhury, 2012)

Table 2

<table>
<thead>
<tr>
<th>Type of failure surface</th>
<th>Backfill soil condition</th>
<th>Pseudo-dynamic method (Ahmad and Choudhury, 2008)</th>
<th>Pseudo-static method (present study)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td>Wet</td>
<td>0.43</td>
<td>0.49</td>
</tr>
<tr>
<td>Polylinear</td>
<td>Wet</td>
<td>0.29</td>
<td>0.42</td>
</tr>
</tbody>
</table>

*Note: Input data: $H=5\,\text{m}; \ h_{wb}/H=h_{wo}/H=0.75; \ \phi=30^\circ; \ \delta=\phi/2; \ k_v=k_h/2; \ r_u=0.20; \ V_s=100\,\text{m/s}; \ V_p=187\,\text{m/s}; \ T=0.3\,\text{s}$—where $V_s$=velocity of the shear wave propagating through the soil medium; $V_p$=velocity of the primary wave propagating through the soil medium; $T$=period of lateral shaking—for more details on these parameters, see Ahmad and Choudhury (2008).

Seismic Design of Shallow Footings
Failure surface under seismic condition

Failure surface under static \((k_h = k_i = 0)\) condition

Choudhury and Subba Rao (2005)
Figure 2. (a) Geometry of the curved failure surface. (b) Forces considered in the analysis.
Design charts given by Choudhury and Subba Rao (2005)

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

<table>
<thead>
<tr>
<th>Case for</th>
<th>$k_h$</th>
<th>Seismic bearing capacity factors obtained by</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Budhu and Al-Karni (1993)</td>
<td>Present study</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$k_r = 0.5k_h$</td>
<td>$k_r = 1.0k_h$</td>
</tr>
<tr>
<td>$N_{cd}$</td>
<td>0.1</td>
<td>19.62</td>
<td>19.62</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>12.76</td>
<td>12.76</td>
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<td>8.80</td>
</tr>
<tr>
<td></td>
<td>0.4</td>
<td>5.40</td>
<td>–</td>
</tr>
<tr>
<td>$N_{qd}$</td>
<td>0.1</td>
<td>12.52</td>
<td>11.85</td>
</tr>
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<td>2.47</td>
</tr>
<tr>
<td></td>
<td>0.4</td>
<td>1.76</td>
<td>–</td>
</tr>
<tr>
<td>$N_{yd}$</td>
<td>0.1</td>
<td>10.21</td>
<td>9.46</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>3.81</td>
<td>2.86</td>
</tr>
<tr>
<td></td>
<td>0.3</td>
<td>1.21</td>
<td>0.59</td>
</tr>
<tr>
<td></td>
<td>0.4</td>
<td>0.32</td>
<td>–</td>
</tr>
</tbody>
</table>
Shallow Strip Footing embedded in Sloping Ground under Seismic Condition

Choudhury and Subba Rao (2006)

Design Equations proposed by Choudhury and Subba Rao (2006)

\[
N_{cd} = \frac{1}{k_d} \left[ \frac{K_{pcd1}}{\cos \phi} \sin(\alpha_1 - \phi) - \frac{mK_{pcd2}}{\cos \phi_2} \sin(\alpha_2 - \phi_2) \right] \left( \frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2} \right) + \frac{\sin \alpha_1 \tan \phi_2 \cos \alpha_2}{\sin(\alpha_1 + \alpha_2) \tan \phi} - \frac{\sin \alpha_2 \cos \alpha_1}{\sin(\alpha_1 + \alpha_2)}
\]

\[
N_{qd} = \frac{1}{k_h} \left[ \frac{K_{pqd1}}{\cos \phi} \sin(\alpha_1 - \phi) - \frac{mK_{pqd2}}{\cos \phi_2} \sin(\alpha_2 - \phi_2) \right] \left( \frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2} \right)^2 - \frac{1}{\left( \frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2} \right)}
\]

\[
N_{cd} = \frac{1}{1 - k_v} \left[ \frac{K_{pcd1}}{\cos \phi} \cos(\alpha_1 - \phi) + \frac{mK_{pcd2}}{\cos \phi_2} \cos(\alpha_2 - \phi_2) \right] \left( \frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2} \right) + \frac{\sin \alpha_1 \tan \phi_2 \sin \alpha_2}{\sin(\alpha_1 + \alpha_2) \tan \phi} + \frac{\sin \alpha_2 \sin \alpha_1}{\sin(\alpha_1 + \alpha_2)}
\]

\[
N_{qd} = \frac{1}{1 - k_v} \left[ \frac{K_{pqd1}}{\cos \phi} \cos(\alpha_1 - \phi) + \frac{mK_{pqd2}}{\cos \phi_2} \cos(\alpha_2 - \phi_2) \right] \left( \frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2} \right)^2 - \frac{1}{\left( \frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2} \right)}
\]
Typical Design Chart for $N_{cd}$

- Present study
- Budhu and Al-Karni (1993)
- Kumar and Rao (2002)
- Soubra (1999)
- Soubra (1997)
- Richards et al. (1993)
- Sarma and Iossifelis (1990)

$\phi = 30^0, k_v = 0.0$
Typical Design Charts for $N_{qd}$ and $N_{yd}$
Design Charts for Seismic Bearing Capacity Factors

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

\[ N_{qd} = \frac{1}{k_h} \left[ \frac{K_pqd_1 \sin (\alpha_1 - \phi) - mK_pqd_2 \sin (\alpha_2 - \phi)}{\cos \phi_2} \right] \left( \frac{1}{\tan \alpha_1} + \frac{1}{\tan \alpha_2} \right) \]

Typical Results to Show Effects of Ground Slope and Embedment
Seismic Bearing Capacity of Shallow Strip Footing Using Pseudo-Dynamic Approach

Model and forces considered by Ghosh and Choudhury (2011)

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)
Pseudo-Static method of Seismic Analysis

\[ F_h = ma = \frac{Wa}{g} = \frac{W a_{\text{max}}}{g} = k_h W \]

where \( F_h \) = horizontal pseudostatic force acting through the centroid of sliding mass, in an out-of-slope direction, lb or kN. For slope stability analysis, slope is usually assumed to have a unit length (i.e., two-dimensional analysis).

\( m \) = total mass of slide material, lb or kg, which is equal to \( W/g \)

\( W \) = total weight of slide material, lb or kN

\( a \) = acceleration, which in this case is the maximum horizontal acceleration at ground surface caused by earthquake (\( a = a_{\text{max}} \)), ft/s² or m/s²

\( a_{\text{max}} \) = maximum horizontal acceleration at ground surface that is induced by the earthquake, ft/s² or m/s². The maximum horizontal acceleration is also commonly referred to as the peak ground acceleration (see Sec. 5.6).

\( a_{\text{max}}/g = k_h \) = seismic coefficient, also known as pseudostatic coefficient (dimensionless)
Pseudo-Static method of Seismic Analysis

The selection of the seismic coefficient $k_h$ takes considerable experience and judgment. Guidelines for the selection of $k_h$ are as follows:

1. *Peak ground acceleration:* The higher the value of the peak ground acceleration $a_{\text{max}}$, the higher the value of $k_h$ that should be used in the Pseudo-static analysis.

2. *Earthquake magnitude:* The higher the magnitude of the earthquake, the longer the ground will shake and consequently the higher the value of $k_h$ that should be used in the pseudo-static analysis.

3. *Maximum value of $k_h$:* When items 1 and 2 as outlined above are considered, keep in mind that the value of $k_h$ should never be greater than the value of $a_{\text{max}}/g$.

4. *Minimum value of $k_h$: Check to determine if there are any agency rules that require a specific seismic coefficient. For example, a common requirement by many local agencies in California is the use of a minimum seismic coefficient $k_h \geq 0.15$ (Division of Mines and Geology 1997).

5. *Size of the sliding mass:* Use a lower seismic coefficient as the size of the slope failure mass increases. The larger the slope failure mass, the less likely that during the earthquake the entire slope mass will be subjected to a destabilizing seismic force acting in the out-of-slope direction. Suggested guidelines are as follows:
Pseudo-Static method of Seismic Analysis

*a. Small slide mass:* Use a value of $k_h = \frac{a_{\text{max}}}{g}$ for a small slope failure mass. Examples would include small rockfalls or surficial stability analyses.

*b. Intermediate slide mass:* Use a value of $k_h = 0.65\frac{a_{\text{max}}}{g}$ for slopes of moderate size (Krinitzsky et al. 1993, Taniguchi and Sasaki 1986). Note that this value of 0.65 was used in the liquefaction analysis.

*c. Large slide mass:* Use the lowest values of $k_h$ for large failure masses, such as large embankments, dams, and landslides. Seed (1979) recommended the following:

\[ k_h = 0.10 \text{ for sites near faults capable of generating magnitude } 6.5 \text{ earthquakes.} \]
\[ k_h = 0.15 \text{ for sites near faults capable of generating magnitude } 8.5 \text{ earthquakes.} \]
The acceptable pseudo-static factor of safety is 1.15 or greater.
Pseudo-Static method of Seismic Analysis

Terzaghi (1950) suggested the following values: $k_h = 0.10$ for “severe” earthquakes, $k_h = 0.20$ for “violent and destructive” earthquakes, and $k_h = 0.50$ for “catastrophic” earthquakes.

Seed and Martin (1966) and Dakoulas and Gazetas (1986), using shear beam models, showed that the value of $k_h$ for earth dams depends on the size of the failure mass. In particular, the value of $k_h$ for a deep failure surface is substantially less than the value of $k_h$ for a failure surface that does not extend far below the dam crest.

Marcuson (1981) suggested that for dams $k_h = 0.33 a_{\text{max}}/g$ to $0.50 a_{\text{max}}/g$, and consider possible amplification or deamplification of the seismic shaking due to the dam configuration.

Hynes-Griffin and Franklin (1984), based on a study of the earthquake records from more than 350 accelerograms, use $k_h = 0.50 a_{\text{max}}/g$ for earth dams. By using this seismic coefficient and having a pseudo-static factor of safety greater than 1.0, it was concluded that earth dams will not be subjected to “dangerously large” earthquake deformations.

Kramer (1996) states that the study on earth dams by Hynes-Griffin and Franklin (1984) would be appropriate for most slopes. Also Kramer indicates that there are no hard and fast rules for the selection of the pseudo-static coefficient for slope design, but that it should be based on the actual anticipated level of acceleration in the failure mass (including any amplification or deamplification effects).
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 $T = s_uL$
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)**

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

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

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

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

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