Various Modes of Failure

- Short Term & Long Term Global Stability
- Rapid Draw Down Stability
- Excessive Settlement
- Bearing Failure
- Lateral Squeeze
Combination of Failures
Lateral Squeeze

What is Lateral Squeeze?
Lateral Squeeze
Lateral Squeeze

Figure 7-18. Schematic of lateral squeeze phenomenon.

The lateral squeeze phenomenon is due to an unbalanced load at the surface of the soft soil. The lateral squeeze behavior may be of two types, (a) short-term undrained deformation that results from a local bearing capacity type of deformation, or (b) long-term drained, creep-type deformation. Creep refers to the slow deformation of soils under sustained loads over extended periods of time and can occur at stresses well below the shear strength of the soil. As discussed in Section 5.4.1, secondary compression is a form of creep deformation while primary consolidation is not.
Lateral Squeeze

This is an extensive report on the subject.
In Sandford’s Report he cites other authors and case histories. In particular he cites LaCroix and Tschebotarioff (1972):

- The **thicker the clay** layer the greater the movement
- Movement greatest for stress > 4X undrained strength
- Most bridge abutments had stabilized in **15 years** although one was still moving after **20 years**

Citing Vanikar (1986):

- Movement will occur when embankment stress > 3X the undrained strength of clay

Citing Tavenas et al. (1979):

- Movement at toe for normally consolidated clay is **1X the settlement for undrained loading** and 0.16X the settlement for consolidation.
- For overconsolidation portion of settlement about **0.16X** the settlement
Lateral Squeeze

This is Another Good Reference Dated 2009
Lateral Squeeze

Interesting Model of Squeeze

Figure 9-8. Failure through the foundation.
Lateral Squeeze

- Local bearing failure at the toe (lateral squeeze) (Figure 9-8b).

  If a weak soil layer exists beneath the embankment to a limited depth $D_s$ which is less than the width of the slope $b'$, the factor of safety against failure by squeezing may be calculated from (Silvestri, 1983):

  $$ F_{S_{squeezing}} = \frac{2c_u}{\gamma D_s \tan \theta} + \frac{4.14c_u}{H \gamma} \geq 1.3 \quad (9-15) $$

  where:
  \[ \begin{align*}
  \theta & = \text{angle of slope} \\
  \gamma & = \text{unit weight of soil in slope} \\
  D_s & = \text{depth of soft soil beneath slope base of the embankment} \\
  H & = \text{height of slope} \\
  c_u & = \text{undrained shear strength of soft soil beneath slope}
  \end{align*} \]

  Caution is advised and rigorous analysis (e.g., numerical modeling) should be performed when $F_S < 2$. This approach is somewhat conservative as it does not provide any influence from the reinforcement. **When the depth of the soft layer, $D_s$, is greater than the slope base width, $b'$, general slope stability will govern design.**
Lateral Squeeze

<table>
<thead>
<tr>
<th>THRESHOLD EQUATION</th>
<th>(per FHWA Publication No. HI 97-013)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embankment load should not exceed 3 times the undrained shear strength of the foundation soils</td>
<td></td>
</tr>
</tbody>
</table>

\[ \gamma_f \cdot H > 3 \cdot c_u = 5.56 \quad \text{NOT OK} \]

<table>
<thead>
<tr>
<th>FACTOR OF SAFETY EQUATION</th>
<th>(per FHWA NHI-10-025 MSE Walls and RSS - Vol II)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FS_{squeezing} = \left( \frac{2 \cdot c_u}{\gamma_f \cdot D_s \cdot \tan \Theta} \right) + \left( \frac{4.14 \cdot c_u}{H \cdot \gamma_f} \right)</td>
<td></td>
</tr>
<tr>
<td>FS_{squeezing} \geq 1.3 \quad \text{CAUTION} \leq 2.0 \quad \text{OK}</td>
<td></td>
</tr>
</tbody>
</table>

\[ \Theta = 26.6 \text{degrees} \]

\[ \gamma_f = 125 \text{pcf} \]

\[ H = 60 \text{feet of embankment} \]

\[ D_s = 25 \text{feet of soft soil} \]

\[ c_u = 450 \text{psf} \]

\[ FS_{squeezing} = 0.82 \quad \text{NOT OK} \]
Design Solutions

Reduce Settlement
- Reduce Height
- Lightweight Fill

Increase Resistance
- Reinforce Soil
- Staged Construction
- *Wick Drains*
- Stone Columns/Deep Foundations
- Remove/Replace Soft Soil
- Flatten Slope
Our Problem

60 Feet of Fill
Pier Deflected Approx. 4” at top
And Approx. 1.5” at the Bottom in Nov. 2011.

Our Problem
Our Problem

Several Inches of Ground loss Beneath the Abutment
Our Problem

Down-drag On Piling
Our Solution

10” Diameter Relief Holes 30’ Deep
Our Solution
Wick Drains Helped
Inclinometer Results For SI-1

S. Mineral Wells SI-1, A-Axis

Possible Collapse
(sum of A+B)
@10' - 0.71"
(0.05"/week)

S. Mineral Wells SI-1, B-Axis

Approx. Original Ground

Creep Squeeze
(sum of A+B)
@54' - 0.16"
(0.01"/week)

Approx. Bedrock Elev.
Monitoring SI-2
Inclinometer Results For SI-2

S. Mineral Wells SI-2, A-Axis

Possible Collapse (sum of A+B)
@10° = 1.06" (0.08"/week)

S. Mineral Wells SI-2, B-Axis

Creep Squeeze (sum of A+B)
@64° = 0.25" (0.02"/week)

APPROX ORIGINAL GROUND

APPROX BEDROCK ELEV.

Cumulative Displacement (in) from 12/2/2011
Monitoring SI-3
Inclinometer Results For SI-3

S. Mineral Wells SI-3, A-Axis

S. Mineral Wells SI-3, B-Axis

Approx Original Ground

Creep Squeeze @ 12° = 0.26" (0.02"/week)

Approx Bedrock Elev.

Depth in feet

Cumulative Displacement (in) from 11/18/2011
LPile Back Analysis

This shape is used to model uncased drilled shafts or bored piles. The reinforcing bars for drilled shafts are typically arranged in a circular pattern, either as single bars or as two- or three-bar bundles. It is strongly advised that the bar pattern be symmetrical and that no fewer than 8 bars or bundles be specified. Use of fewer than 8 bars or bundles may result in deficient moment capacity if the rebar cage is inadvertently rotated either during concrete placement or removal of temporary casing. It is recommended that the minimum cover thickness be specified as 3 inches or 75 mm for drilled shafts constructed without temporary casing and 4 inches or 100 mm for drilled shafts constructed using temporary casing. In cases where alignment of...
LPIle Back Analysis

The Peak Movement at the Pier Due to Undrained Squeeze is Estimated to be 1.35”
LPile Back Analysis
LPile Back Analysis

The Peak load Equates to a peak stress of 28.8 ksf in the soil.
LPile Back Analysis
Summary

- Check for Lateral Squeeze on Embankments Fills Greater than 10′ on Soft Soils Next to Structures
- Design Measures to Avoid the Movement in the First Place
- Design Measures to Minimize the Effects of Displacement
- Squeeze is Usually a Long-Term Problem
- Limited Information, But Some Good Resources Available
- Instrumentation Assists in Analyzing Squeeze
The Bridge
Any Questions
Figure 7-19. Examples of abutment tilting due to lateral squeeze (FHWA, 2006a).
If the consolidation settlement is estimated based on High Quality Shelby tube samples the horizontal tilt at the top of an abutment can be estimated at about 25% of the vertical settlement.
Lateral Squeeze

\[ Tilt = 0.25 \times 12.5'' \approx 3'' \]