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Presented to: Geo-Omaha 2020

Geo-Institute Omaha, Nebraska
February 21, 2020

• Ramp ES
Design & Construction
• Subsurface Conditions
• PVD Evaluation
• Undrained Stability Analyses
• Summary

Unsaturated Stability Analysis

➢ >115,000 CY embankment, N of US 224 ➢ 3,700 lineal ft. on new alignment

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Ramp ES Failure ~ 300 ft.

CR 97
• 35 to 60 ft deep

• 6 ft c/c spacing
• 325,000 lineal ft. installed
• Rate of fill < 1 ft/day to 5 ft/wk.
  • Piezometers to monitor pore water pressure: ➢Read after every 1 ft of new embankment
  • \( \Delta u < 7 \) psi above baseline pore water pressure of 8 psi
  • Settlement monuments and platforms read weekly
  • Expected 5 ft settlement in places
6/16/07 filling starts

7/13/07-8ft,
cracks, <7 psf

\(\Delta u \sim 8 \text{ to } 14 \text{ psi} \quad \Delta u \sim\)

75\%*\(\Delta \sigma; \) 25%

Consolidation
July 13, 2007 – Tension cracks; work stopped for geotechnical study
• Fill = 8 ft (2.4 m) of 30 ft (9.2 m) or 32 ft (9.8 m) – 25% = 8’/32’
• July 24, 2007 – Filling continues with added monitoring

8/1/07: Work Stopped (1.5 months).
• ODOT performs geotechnical evaluation

• July 24, 2007 – Filling continues with added monitoring - Survey pins installed

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• July 30, 2007 – more cracking after fill reaches 13 ft
• August 1\textsuperscript{st}, 2007 (1.5 months) – pore-water pressure > 7 psi above baseline - \textit{work stopped}
  - survey pins show uplift and toe rotation
• August 2\textsuperscript{nd}, 2007 – cracking from STA 202 to 205
• August 6\textsuperscript{th}, 2007 – 1.5 ft high scarp at ramp centerline and displacement near R/W fence along STA 202 to 205
• Potential contract delays were $250,000/month
• Possible remedial measures:
  • Lightweight fills,
  • Deep soil/concrete columns,
  • Geogrid reinforced granular embankment, and
• Pile supported bridge

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Three Causes of Failure.

Stratigraphy - Ch - Embankment
Strength

Ramp ES Design & Construction

• Subsurface Conditions

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Unsaturated Stability Analysis

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c = 1,500 psf & \( \varphi = 0^\circ \)
$S_u = 375 \text{ psf}$

$S_u = 250 \text{ psf}$

$S_u = 500 \text{ psf}$

$S_u = 500 \text{ psf}$
Boring ES-8A

Boring ES-8C

Fence 1

2

c = 1,500 psf & φ = 0°

Scarp

Su = 250 psf

• Thicker weak soil layer

Su = 750 psf

Su = 250 psf

Su = 200 psf Su = 1,000 psf
Weakest Station

Weakest 201+61 soil soil is at deeper (S\textsubscript{Station u} > 250 (El. 203+58 psf/12 964 ft kPa);
to (S\textsubscript{u} 954 of ft)
200-250 psf/9.6-12 kPa) at

- Ramp ES Design & Construction
- Subsurface Conditions
- PVD

Evaluation
- Undrained Stability Analyses
- Summary
- Unsaturated
Stability Analysis

• Three Causes of Failure

Stratigraphy -

Embankment Strength

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• Sta. 203+58
• Design FS: $FS_{\text{undrained}} > 1.3$ and $FS_{\text{drained}} > 1.5$
• Predicted strength increase due to consolidation

Fill Height (ft) Design EOC$^1$ EOP$^2$ EOC EOP Value

<table>
<thead>
<tr>
<th>Fill Height (ft)</th>
<th>Design EOC$^1$ EOP$^2$ EOC EOP Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>15 ft</td>
<td>15 ft 30 ft 30 ft</td>
</tr>
</tbody>
</table>

Depth Soil Type Undrained Shear Strength, $s_u$ (psf)

<table>
<thead>
<tr>
<th>Depth</th>
<th>Soil Type</th>
<th>Undrained</th>
<th>Shear Strength, $s_u$ (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-8</td>
<td>A-6b</td>
<td>750</td>
<td>750 2592 2592 4435 250</td>
</tr>
<tr>
<td>8-13</td>
<td>Peat</td>
<td>250</td>
<td>250 824 824 1398 13-18 A-5</td>
</tr>
<tr>
<td>13-18</td>
<td>A-5</td>
<td>755</td>
<td>824 1398 1398 250 250 824</td>
</tr>
<tr>
<td>18-28</td>
<td>A-7-6</td>
<td>1,000</td>
<td>477 755 28-32 A-6b 1,000</td>
</tr>
<tr>
<td>28-32</td>
<td>A-6b</td>
<td>1,000</td>
<td>2099 3197 Factor of Safety</td>
</tr>
</tbody>
</table>

1 EOC = End of construction 2 EOP = End of primary consolidation

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6/16/07 filling starts
7/13/07-8ft, cracks, <7 psf

• Not dissipating

Δu ~ 8 to 14 psi Δu ~ 75%*Δσ; 25%

Consolidation
8/1/07: Work Stopped (1.5 months).

- **Inverse analysis easier than design!!**
- Design analysis:
  - Use $C_h \text{ field } = 200 \text{ settlements ft}^2/\text{year}$
  - $C_h$ estimates Asaoka (1978) $C_h$;
  - Low $C_{smearing}$, $h = C68_h$ due to clogging, 91 to: $ft^2/\text{year}$

or drain discharge capacity
4) $S_{1+j}^{\text{EOP Settlement}} \sim 4 \text{ ft} \ 3 \beta = 0.9$

$S_j (\text{ft})$ Observed settlements at STA 204+00 with 6-ft drain spacing.

- **Typical Design Ratio:** - $C_h/C_v = 1.5$ to $4.0$

- **Design Analysis:** - $C_h = 200 \text{ ft}^2/\text{year}$ -
  Design $C_h/C_v = 7.5$ to $10.4$

- **Inverse analysis:** - Asaoka
  $C_h = 68 \text{ ft}^2/\text{year}$ -
  Asaoka $C_h/C_v = 2.6$ to $3.6$ -
  Typical $C_h/C_v =$
1.5 to 4.0

• **Lesson:** \( C_h = \) 
\[(1.5 \text{ to } 4.0) \times C_v\]

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• **Inverse analysis:** - Asaoka
\[ C_h = 68 \text{ ft}^2/\text{year} - 90\% \text{ at } \sim 5 \text{ months} - 55\% \text{ at } \sim 1.5 \]

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• **Ramp ES Design &**
Construction
• Subsurface Conditions
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Unsaturated Stability Analysis

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• Three Causes of
Failure

Stratigraphy -

Ch -

Embankment

Strength

• Easier than design!!
• STA 202+00 to 205+00
Subsurface Conditions – STA 203+58

Fence 2
\[ c = 1,500 \text{ psf} \& \phi = 0^\circ \]
1

Scarp
\[ S_u = 250 \text{ psf} \]
• Boring ES-8C at STA 203+58

• • • Design Failed Design at FSH=13

undrained H=32 ft ~ 1.6 ft so FS(30 ft + at H=15 ft = ok

undrained 2 ft)

~ 1.0

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$S_u = 750 \text{ psf}$

$S_u = 250 \text{ psf}$

$S_u = 200 \text{ psf} \quad S_u = 1,000 \text{ psf}$

• Check design FS_{undrained} ~ 1.6 at H=15 ft

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$FS^*_{c \ u \ avg}$

Fill $*$

Fill

c
\[ 5.14N \, S_{HN} \]

\[ \gamma = \]

c' = 300 psf & \varphi' = 33^\circ

Fence Scarp

\[
750, \quad \text{avg} \quad (8) \; 250 \; (10) \; 200 \; \quad (60) \quad (8 \; 10 \; 60)
\]

\[
230.8 \, S = psf \, ft + psf \, ft + psf \, ft + ft + ft +
\]
\[
\text{ft} \cdot \text{ft} = \text{psf}
\]

21

\[ S_u = 750 \text{ psf} \]
\[ S_u = 250 \text{ psf} \]
\[ S_u = 250 \text{ psf} \]
\[ S_u = 200 \text{ psf} \]
\[ S_u = 1,000 \text{ psf} \]

- **Check design** \( FS_{undrained} \sim 1.6 \) at

\[ \text{H=15 ft} \quad FS = N_c \ast S_{u, \text{avg}} \]
\( \gamma \)  

\( H \)

\( Fill \quad FS = 5.14 \times 230.8 \)

\( 130 \quad pcf \times 15 \)
0.61 \text{psf}\n\text{ft} = \text{FS undrained} \approx 1.6 \text{ at H=15 ft}\n\text{FS} = N_c \ast S_{u, \text{avg}}

• Check design FS_{undrained} \approx 1.6 at
\[ \gamma \quad Fill \quad * \]

\[ H \]

\[ Fill \quad FS \quad = \quad 5.14 \times 230.8 \]

\[ 130 \quad pcf \quad * 8 \]
1.14 psf ft

Bearing Capacity Factor of Safety
7/7/07 – tension cracks first observed
2.4m/8 ft 1.14
8/1/07 – tension cracks restart 4.0m/13 ft 0.71
Stage 1 Fill Height 4.6 m/15 ft 0.61 Full
Embankment Height 9.2 m/30 ft 0.31

\[ H = c \gamma \times 375 \times 130 \]

\[ N = psf \times pcf \times 5.14 = 14.8 \text{ ft} \]

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Embankment Fill Condition
Fill Height ($H_{Fill}$) (m/ft)

- Why failure at 13 ft?!?! _

Staged and controlled construction

- Compacted _

$F_{Sundrained} \sim 1.6 > 1.3$ fill

strength - $c = 1,500$ psf & $\varphi =$
\[ FS = \sum \left( c \sum^+ \left( \sigma W^* \tan \sin \alpha \varphi \right) \right) \]
• GB-6 – inactive • $c = 1,500$ psf & $\varphi = 0^0$

• PI = 10; $\varphi' = 33^0$ and $c'=0$

• $c = 1,500$ psf & $\varphi = 0^0$

Compacted fill strength - $c = 1,500$ psf & $\varphi = 0^0$

H

Crack
\[ \gamma = \tan 2^\circ \left( \frac{c}{45} \right) \]

Fill

\[ 0 \]

Fill

\[ \phi^2 \]
2* 2*1,500 H 23.0 130

Fill crack

\[ \gamma = \frac{c}{psf} \frac{ft}{pcf} \]

• Failed at 13 ft....................

• No embankment strength!

• Bearing Capacity Analysis ~
field condition

Chirapuntu and Duncan (1976) - strain incompatibility - strength mobilized in stiff embankment<foundation

\[ H_{\text{Crack}} = 5.1 \times C_{\text{Foundation Fill}} \times \begin{vmatrix} 0.75 & 0.25 \\ 0.75 & 0.25 \end{vmatrix} K_{\text{Foundation}} \]
\[ K_{\text{Fill}} \]

\[
\begin{bmatrix}
\text{W} \\
\text{D}
\end{bmatrix}
\begin{bmatrix}
\text{H}^{\text{Crack}} = 5.1^* \\
348.5 \text{pcf psf} \\
130 \text{pcf psf} \\
120
\end{bmatrix}
\begin{bmatrix}
\text{H}^{\text{Crack}} = 15.5 \\
150 \text{ft} \\
120 \text{ft} \\
90.5 \text{ft}
\end{bmatrix}
\begin{bmatrix}
\text{H}^{\text{Crack}} = 27.9 \\
150 \text{ft}
\end{bmatrix}
\]

\[ K_{\text{Fill}} = 15.5 \]
\[ S_u/\sigma'_p = 0.22 \]

TC: \( S_u/\sigma'_p \sim 0.32 \)
DSS: \( S_u/\sigma'_p \sim 0.00061*(\text{PI}) + 0.22 \)
TE: \( S_u/\sigma'_p \sim 0.00117*(\text{PI}) + 0.13 \)
• Terzaghi, Peck, and Mesri (1996)

• Compacted fill strength - drained

strength - GB-6 - PI = 10; $\phi' = 33^0 - c'$

from inverse analysis - $c' = 300$ psf

• GB-6 - Clays $c' = 300-550$ psf
• New cross-section at changes in soil stratigraphy - review borings carefully
• Big impact of c or $S_u$ on FS
• If $S_u$, use tension crack
• Undrained Bearing Capacity Analysis
• Shear strength correlations – - Planning level?? - Use Atterberg Limits not SPT - CPT, Vane Shear, etc.

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• Subsurface Conditions
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• Unsaturated Stability Analysis

3 Stiff Reasons: embankment
stratigraphy, over soft foundation $C_h$, with and $Su$, fill include strength
tension crack
• Depth of tension crack Chirapantu and Duncan (1976) or:

$H = \gamma \left[ 2c \tan\left(45 - \frac{\Phi}{2}\right) \right]$  

• Don’t use $N$ to estimate $Su$, use $PI$ and undrained strength ratio
• Undrained bearing capacity analysis to guide short-term design
• Reasonable range of $C_h/C_v = 1.5$ to 4.0
• Asaoka consolidation (1978) occurring method to estimate predicted mobilized. If not, modify $C_h$ to verify fill placement

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Fill Crack

Fill Fill

• Ramp ES Design & Construction
• Subsurface Conditions
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• Undrained Stability
Analyses

• Summary

• Unsaturated Stability Analysis

Fredlund et al. (1978)
\[ \rho_0 \gamma_0 \gamma_0 = 14.4 \text{ kPa} \]

\[ = 33^0 \]

\[ = \text{pore water pressure} = \text{suction pressure} = \text{strength due to suction} \]
• **Desiccation Crack Depth** - Lu and Likos (2004) - Steady-state

- 6.6 ft (2.0 m)

Field ~ 0.3 to 0.6 m

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• **Strain Incompatability Crack Depth** - Lu and Likos (2004) - Steady-state
- 0.9 to 3.4 m - Sand Blanket not Cracked

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• Final Fill = 32 ft (9.8 m)
• No Embankment Strength
• Fill = 8 ft (2.4 m)

• Total Crack Depth ~ 1.2 to 4.0 m
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• 3 Stiff Reasons: embankment stratigraphy, over soft foundation $C_h$, with and Su, fill include strength tension crack

• Depth of tension crack Chirapanttu and Duncan (1976) or:

$$H = \gamma \left(0.2c \tan \left(45 - \frac{\phi}{2}\right)\right)$$
• Don’t use N to estimate Su, use PI and undrained strength ratio
• Undrained bearing capacity analysis to guide short-term design
• Reasonable range of $C_h/C_v = 1.5$ to $4.0$
• Asaoka consolidation (1978) occurring method to as estimate predicted. mobilized If not, modify $C_h$ to verify fill placement

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Fill Crack

Fill Fill

• ODOT for supplying data and project information
• Luke Wysocki – ODOT Project
Engineer

• Mike Currier - ODOT/Shelly & Sands

• Beth Wilson – ODOT District 3 Construction
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