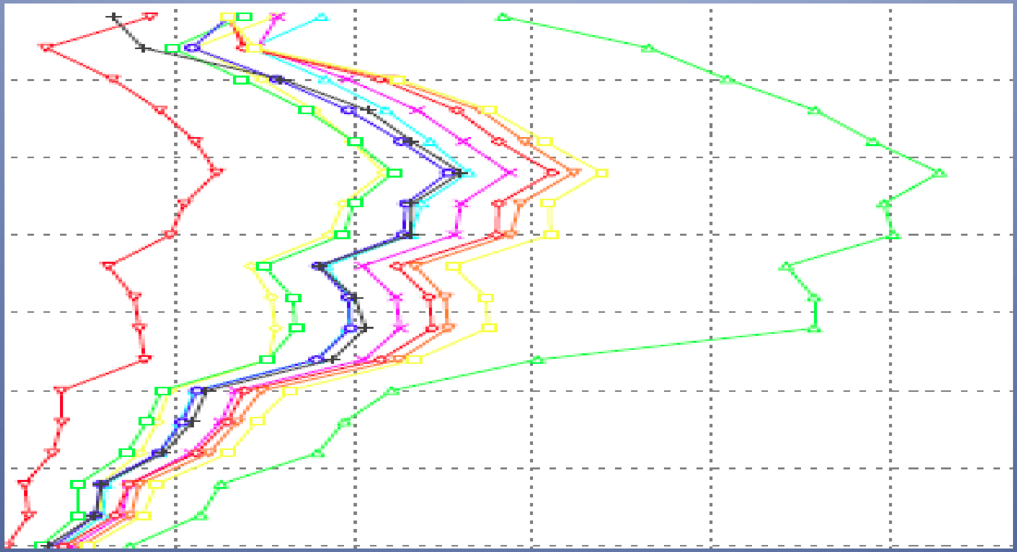
Lateral Squeeze



JOSEPH D. CARTE, P.E.

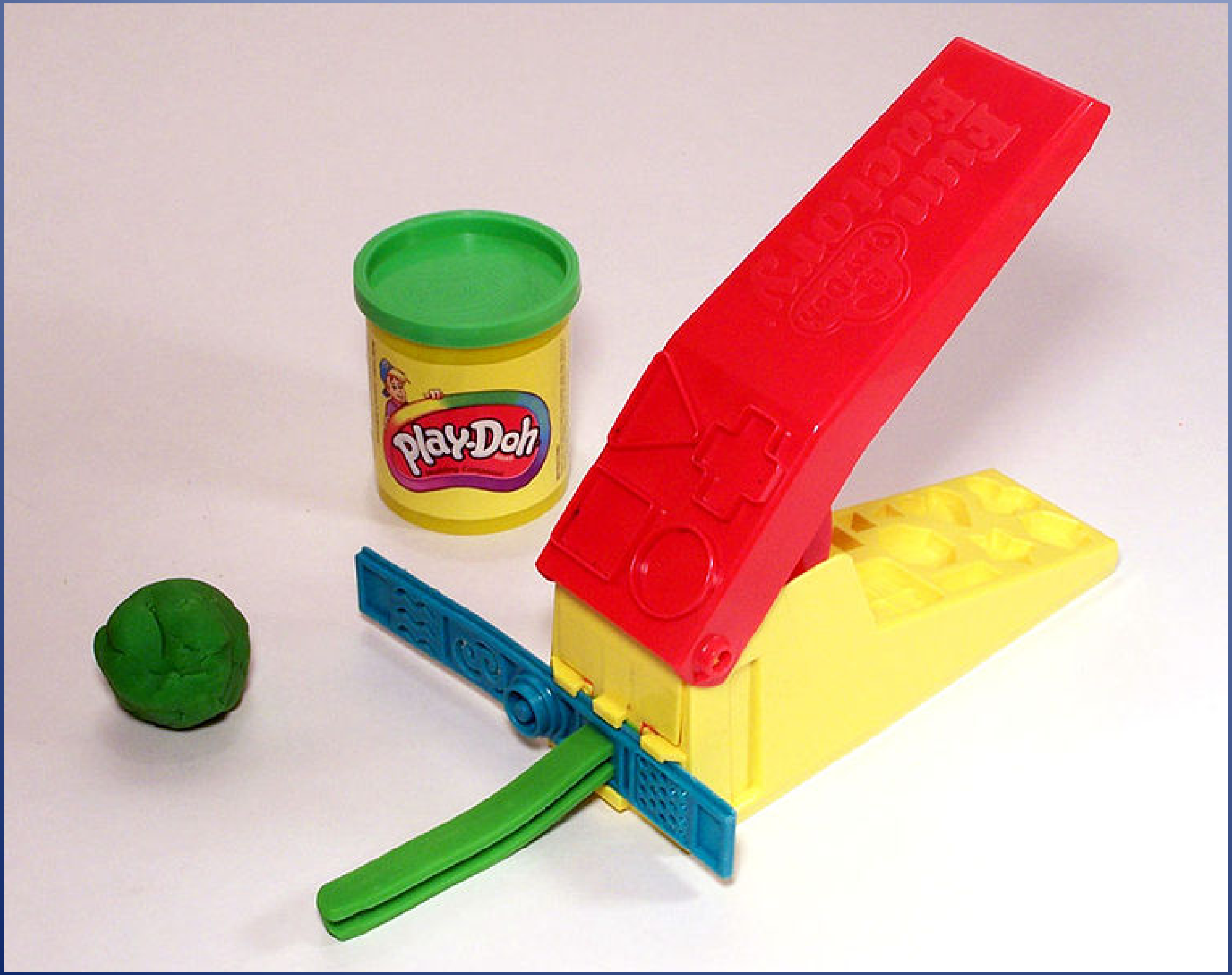
WVDOT

Geohazards 2013

Various Modes of Failure

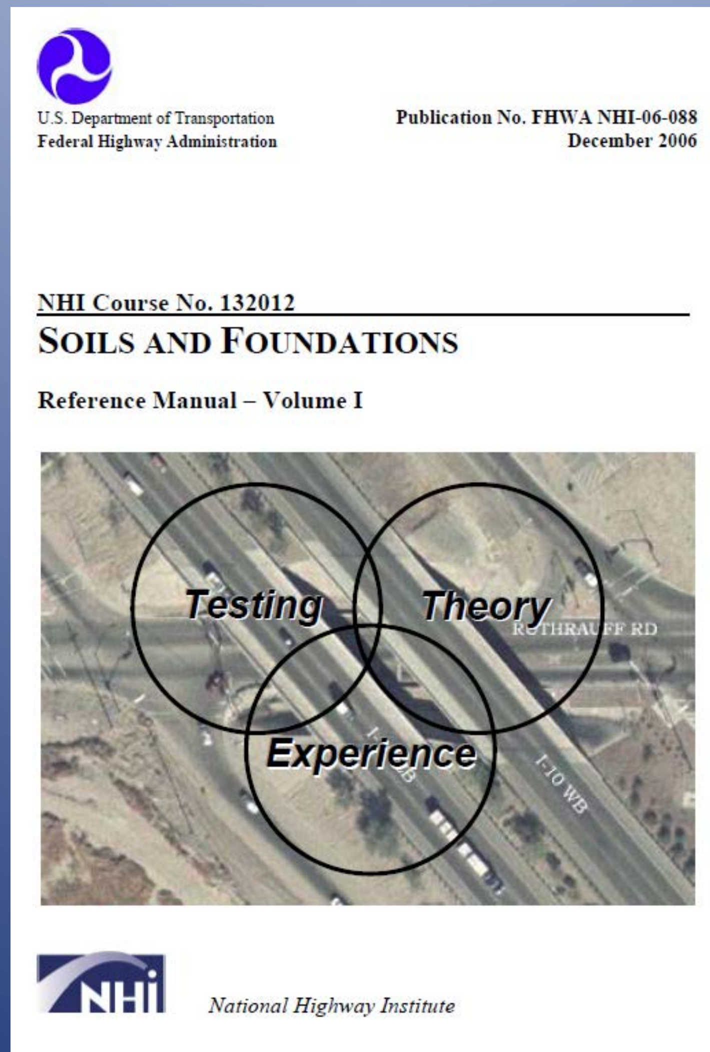
Combination of Failures

Lateral Squeeze



What is Lateral Squeeze?

Lateral Squeeze


U.S. Department of Transportation
Federal Highway Administration

Publication No. FHWA NHI-06-088
December 2006

NHI Course No. 132012

SOILS AND FOUNDATIONS

Reference Manual – Volume I

National Highway Institute

Lateral Squeeze

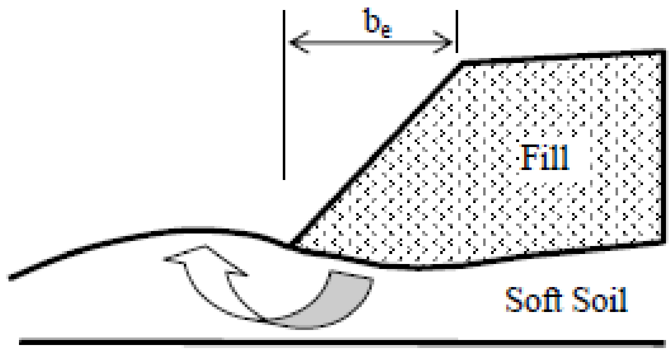


Figure 7-18. Schematic of lateral squeeze phenomenon.

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

EVALUATION OF LATERAL SQUEEZE

FINAL REPORT

TECHNICAL REPORT 91-3

October 1994

Prepared by:
Thomas C. Sandford
Department of Civil and
Environmental Engineering
University of Maine

Lateral Squeeze

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 grater 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) :

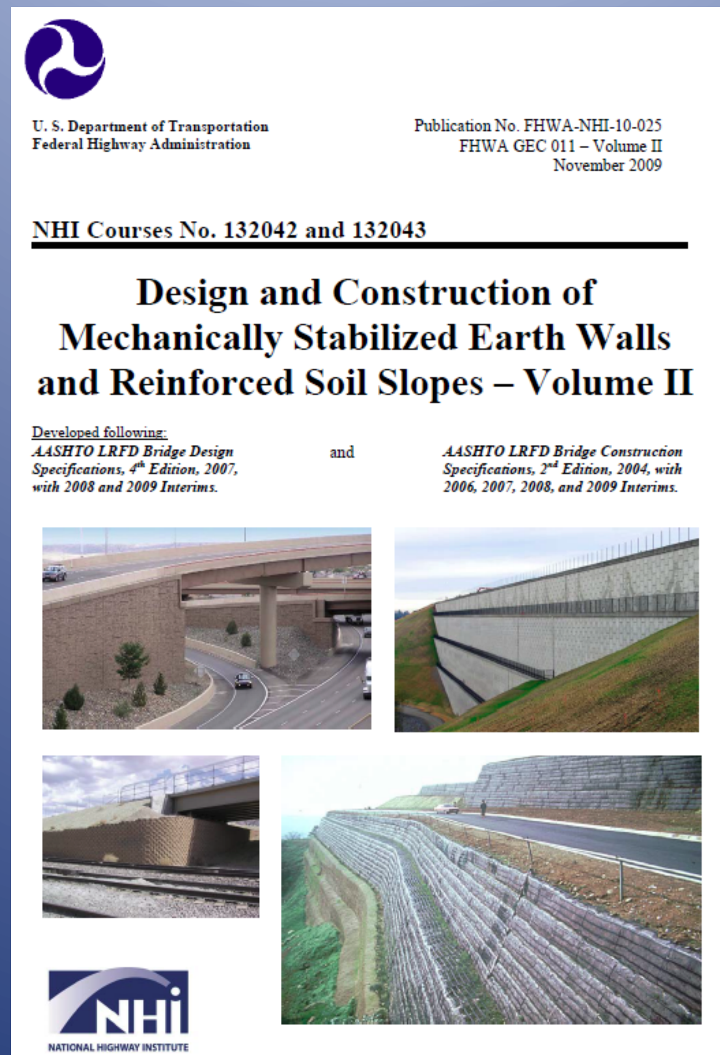
- Movement will occur when embankment stress > 3X the undrained strength of clay

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



U. S. Department of Transportation
Federal Highway Administration

Publication No. FHWA-NHI-10-025
FHWA GEC 011 – Volume II
November 2009

NHI Courses No. 132042 and 132043

Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Volume II

Developed following:

AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007, with 2008 and 2009 Interims.

and

AASHTO LRFD Bridge Construction Specifications, 2nd Edition, 2004, with 2006, 2007, 2008, and 2009 Interims.

NHI
NATIONAL HIGHWAY INSTITUTE

Lateral Squeeze

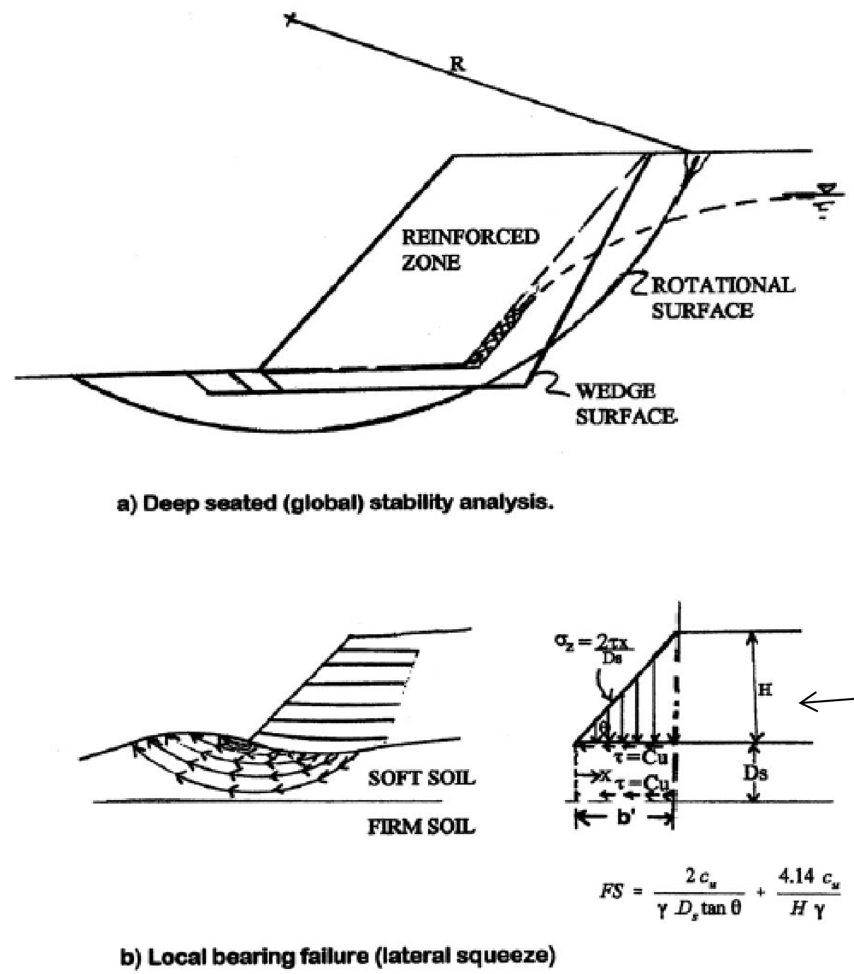


Figure 9-8. Failure through the foundation.

Interesting Model of Squeeze

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):

$$FS_{squeezing} = \frac{2\,c_u}{\gamma\,D_s\,\tan\theta} + \frac{4.14\,c_u}{H\,\gamma} \geq 1.3 \qquad (9\text{-}15)$$

  where:

  θ = angle of slope
  γ = unit weight of soil in slope
  D_s = depth of soft soil beneath slope base of the embankment
  H = height of slope
  c_u = undrained shear strength of soft soil beneath slope

Caution is advised and rigorous analysis (e.g., numerical modeling) should be performed when FS < 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

THRESHOLD EQUATION	(per FHWA Publication No. HI 97-013)				
	Embankment load should not exceed 3 times the undrained shear strength of the foundation soils				
	$\Upsilon_f * H > 3 * c_u =$	5.56	NOT OK		
FACTOR OF SAFETY EQUATION	(per FHWA NHI-10-025 MSE Walls and RSS - Vol II)				
	$FS_{squeezing} = (2*C_u/\Upsilon_f*D_s*\tan\Theta) + (4.14*C_u/H*\Upsilon_f)$				
	$FS_{squeezing} \geq 1.3$ CAUTION ≤ 2.0 OK				
	$\Theta =$	26.6	degrees		
	$\Upsilon_f =$	125	pcf		
	H =	60	feet of embankment		
	$D_s =$	25	feet of soft soil		
	$C_u =$	450	psf		
	$FS_{squeezing} =$	0.82	NOT OK		

Design Solutions

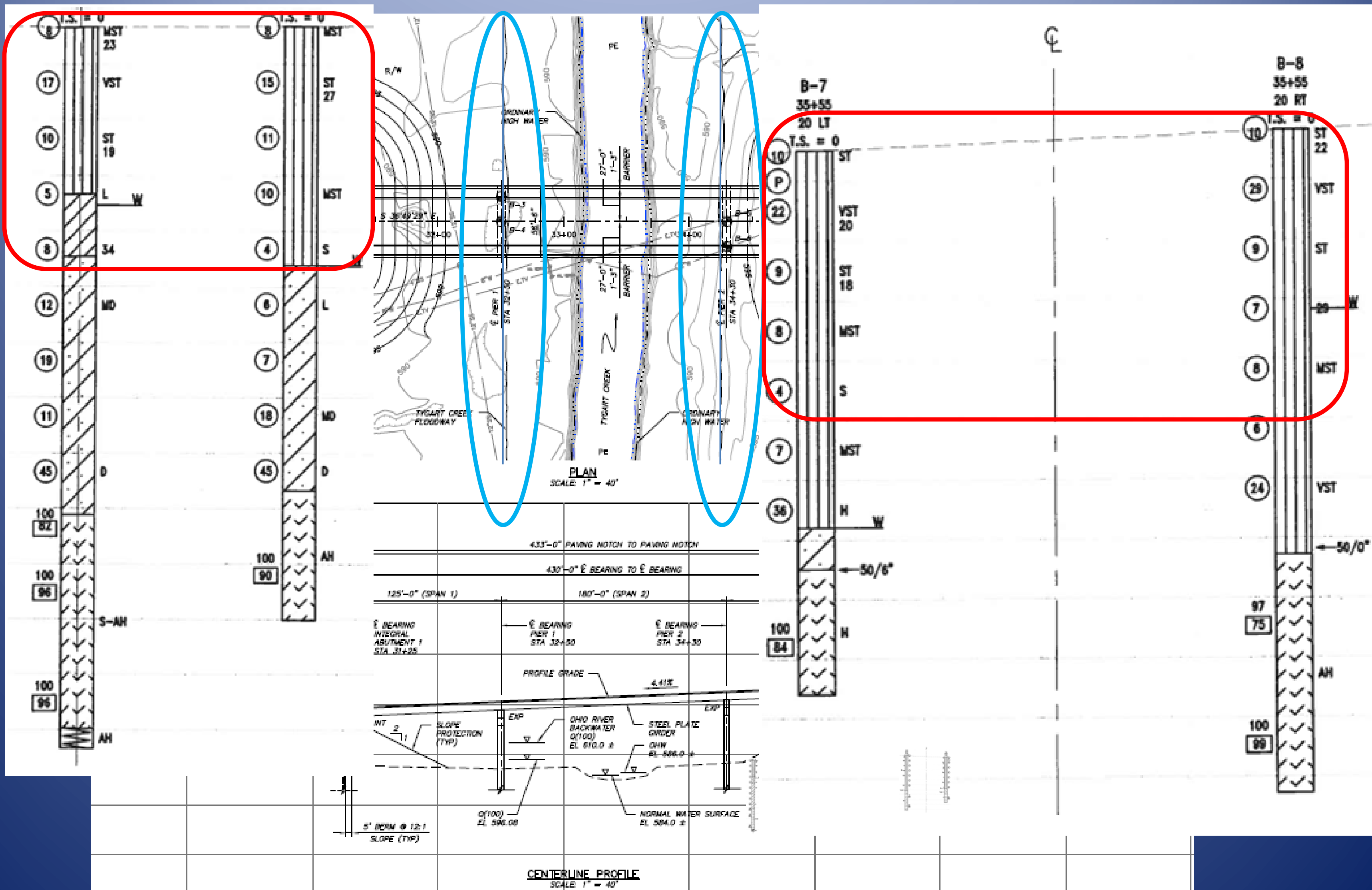
Reduce Settlement

- Reduce Height
- Lightweight Fill

Increase Resistance

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

Our Problem



PLAN
SCALE: 1" = 40'
TYGART CREEK
CENTERLINE PROFILE
SCALE: 1" = 40'
433'-0" PAVING NOTCH TO PAVING NOTCH
430'-0" ℄ BEARING TO ℄ BEARING
125'-0" (SPAN 1)
180'-0" (SPAN 2)
PROFILE GRADE
STEEL PLATE GIRDER
NORMAL WATER SURFACE EL 584.0 ±
B-7
35+55
20 LT
B-8
35+55
20 RT

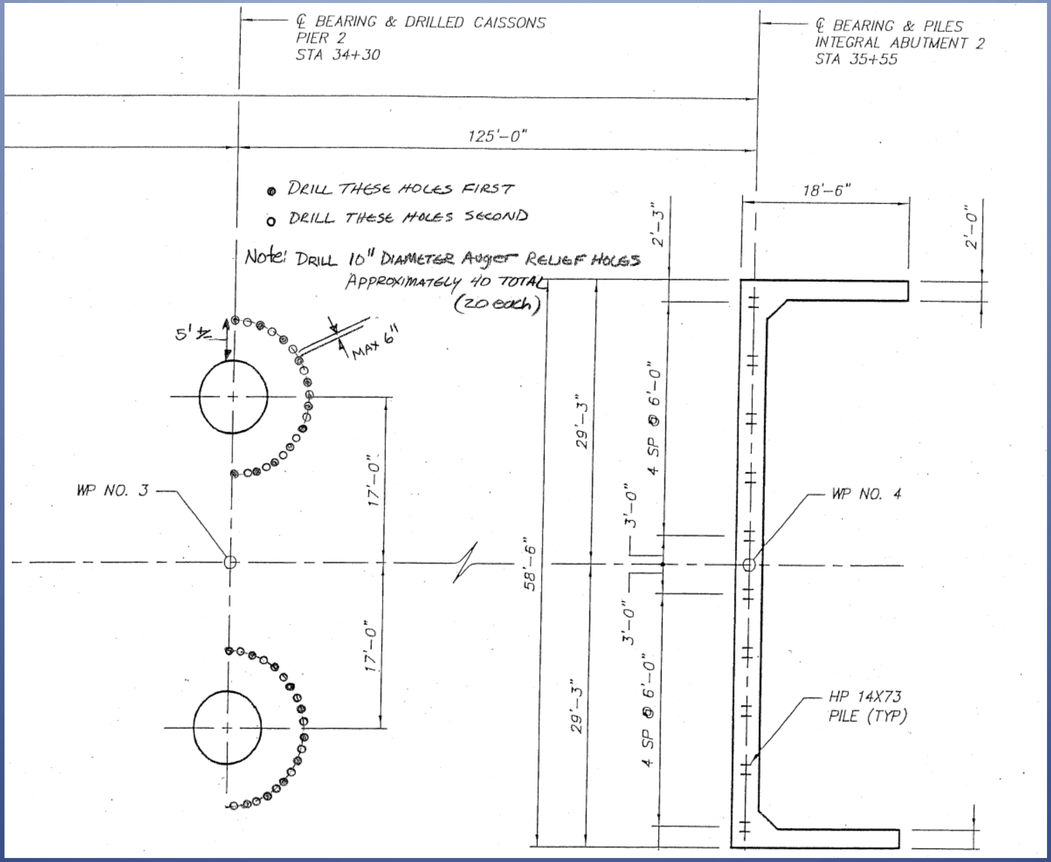
Our Problem


Pier Deflected Approx. 4” at top
And Approx. 1.5” at the Bottom in Nov. 2011.

Our Problem

Our Problem

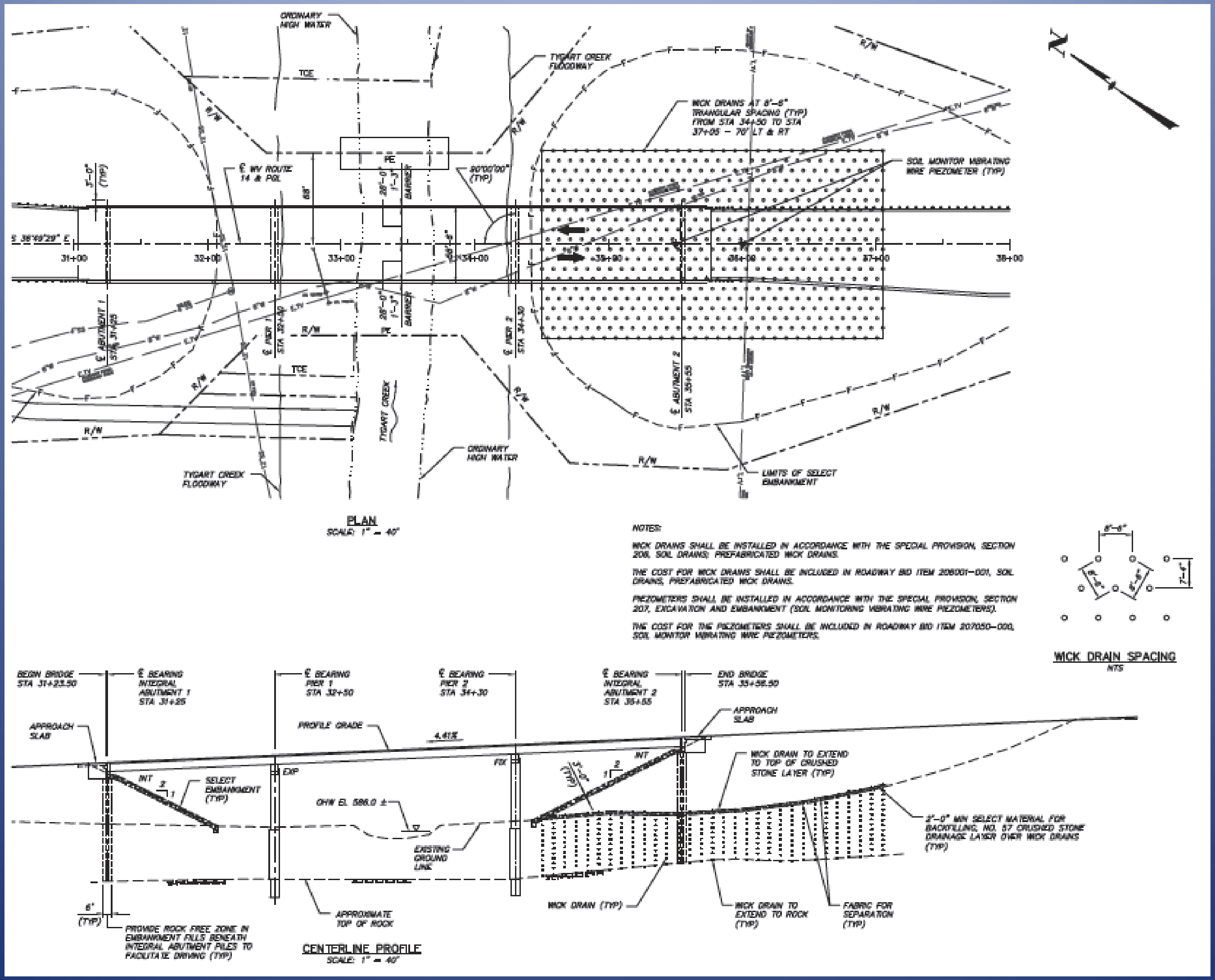
Our Solution



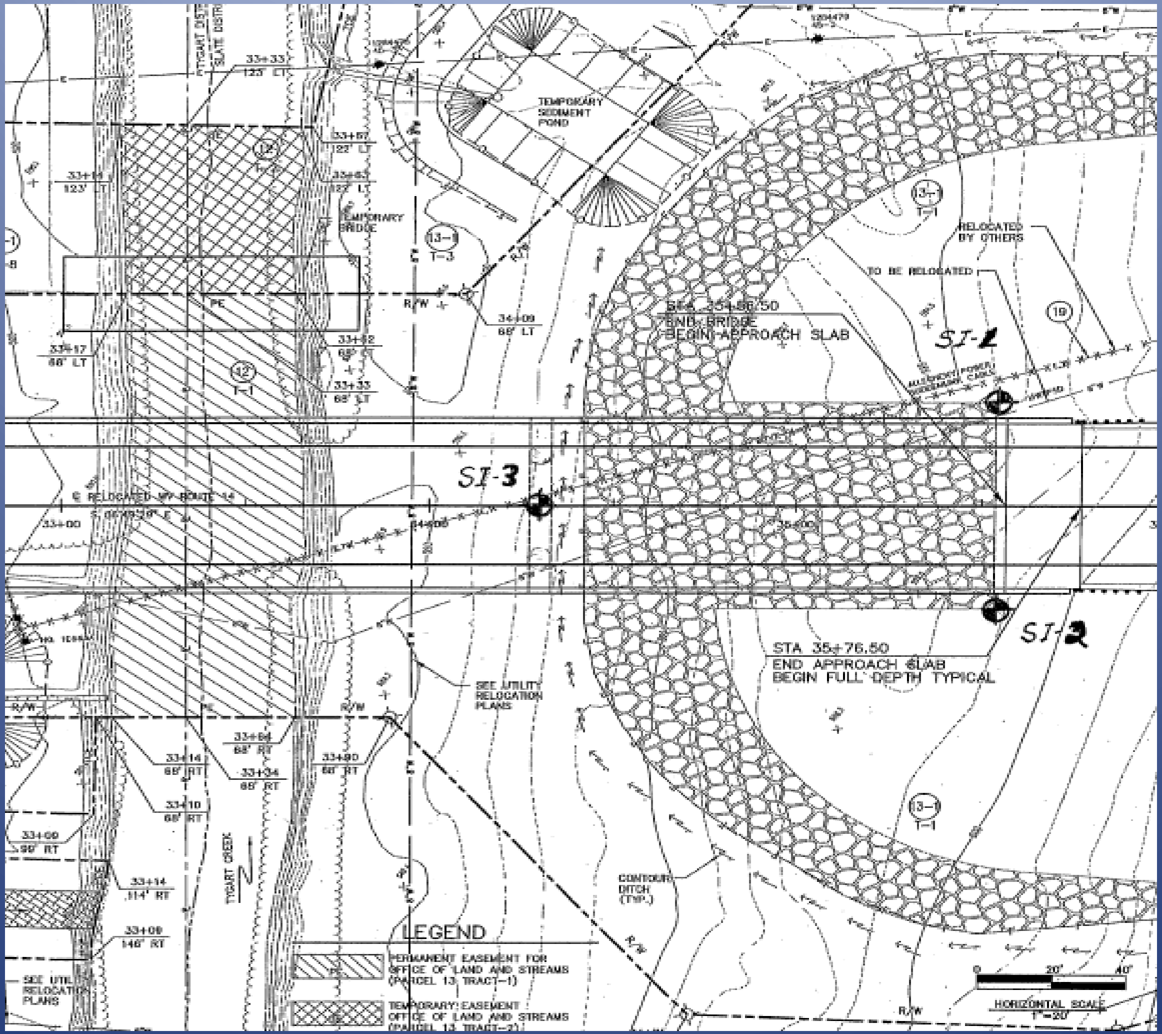
10" Diameter Relief Holes 30' Deep

Our Solution

Wick Drains Helped

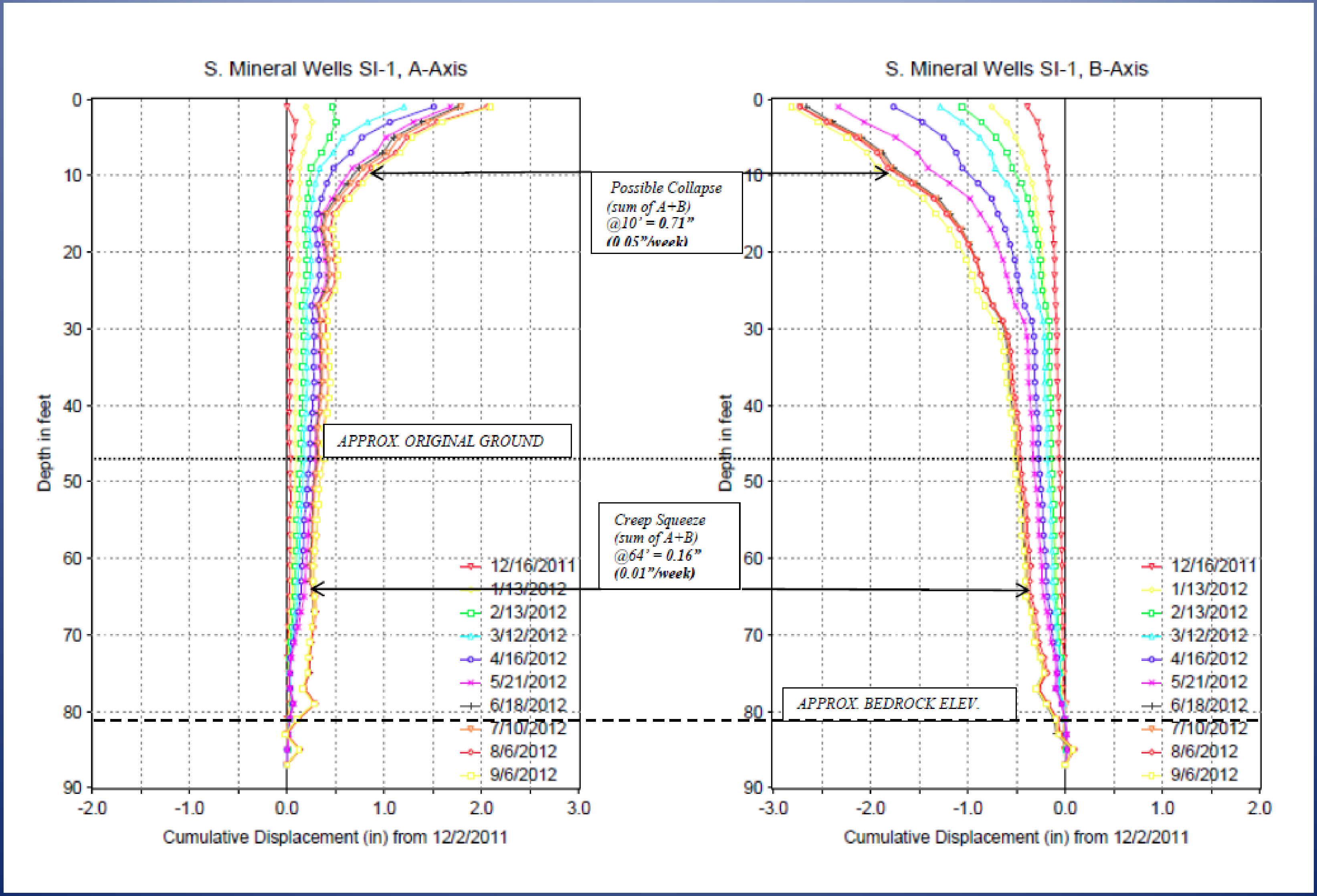


Monitoring



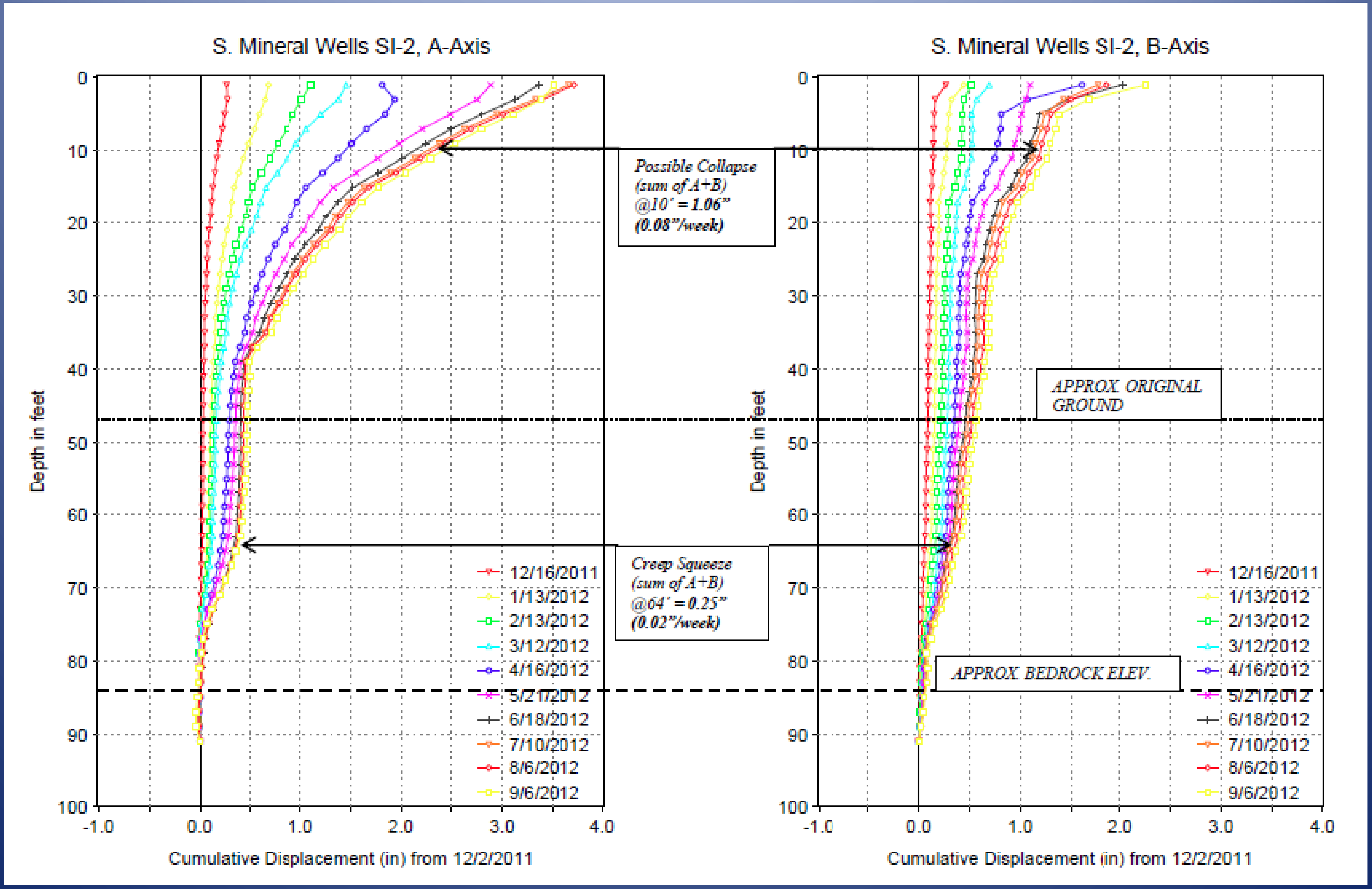
TEMPORARY SEDIMENT POND
TEMPORARY BRIDGE
STA. 35+56.50 END BRIDGE BEGIN APPROACH SLAB
SI-1
SI-3
SI-2
RELOCATED BY OTHERS
TO BE RELOCATED
STA 35+76.50 END APPROACH SLAB BEGIN FULL DEPTH TYPICAL
SEE UTILITY RELOCATION PLANS
CONTOUR DITCH (TYP.)
TYGART CREEK
LEGEND
PERMANENT EASEMENT FOR OFFICE OF LAND AND STREAMS (PARCEL 13 TRACT-1)
TEMPORARY EASEMENT OFFICE OF LAND AND STREAMS (PARCEL 13 TRACT-2)
HORIZONTAL SCALE 1"=20'

Inclinometer Results For SI-1



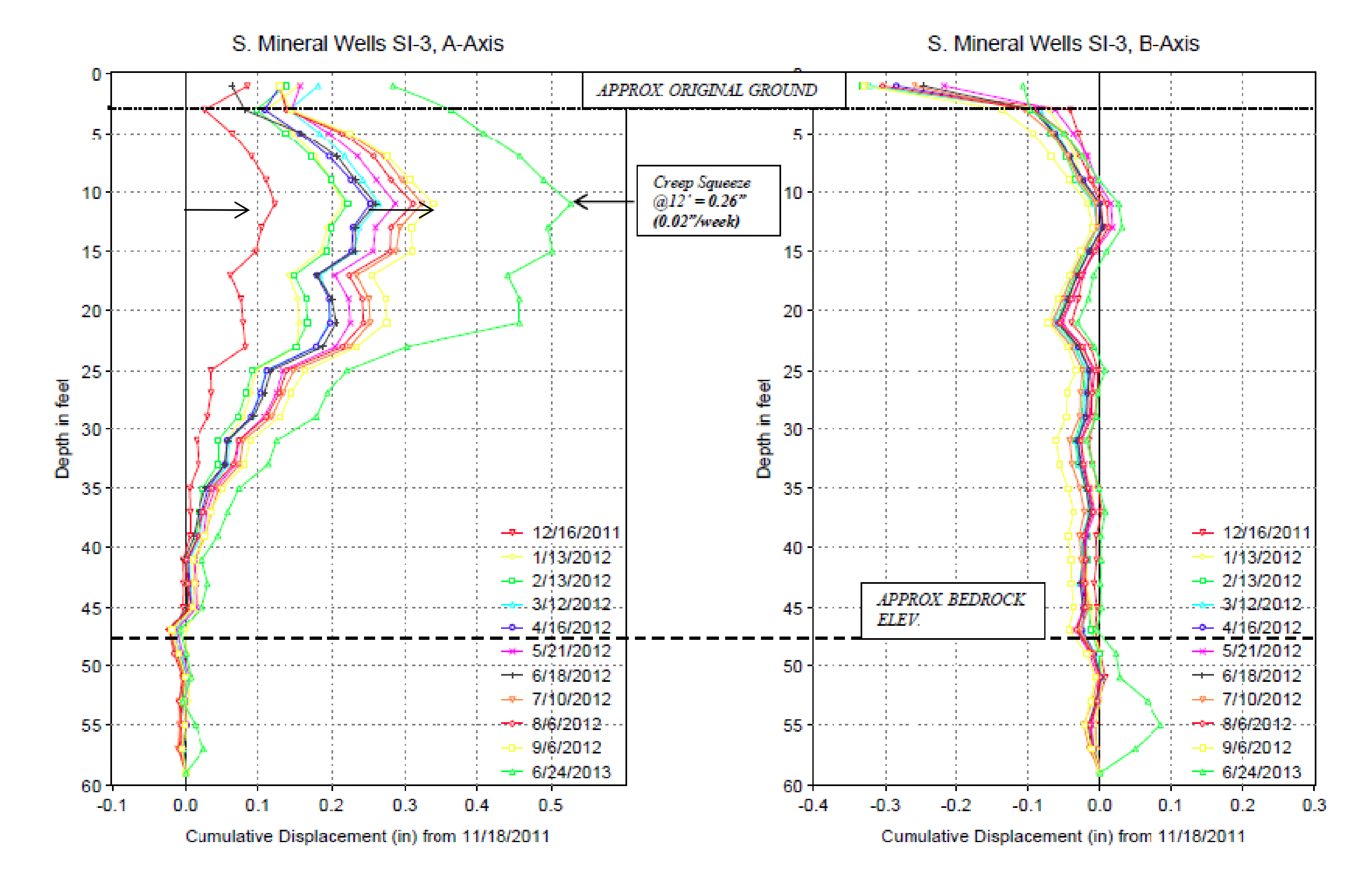
Monitoring SI-2

Inclinometer Results For SI-2

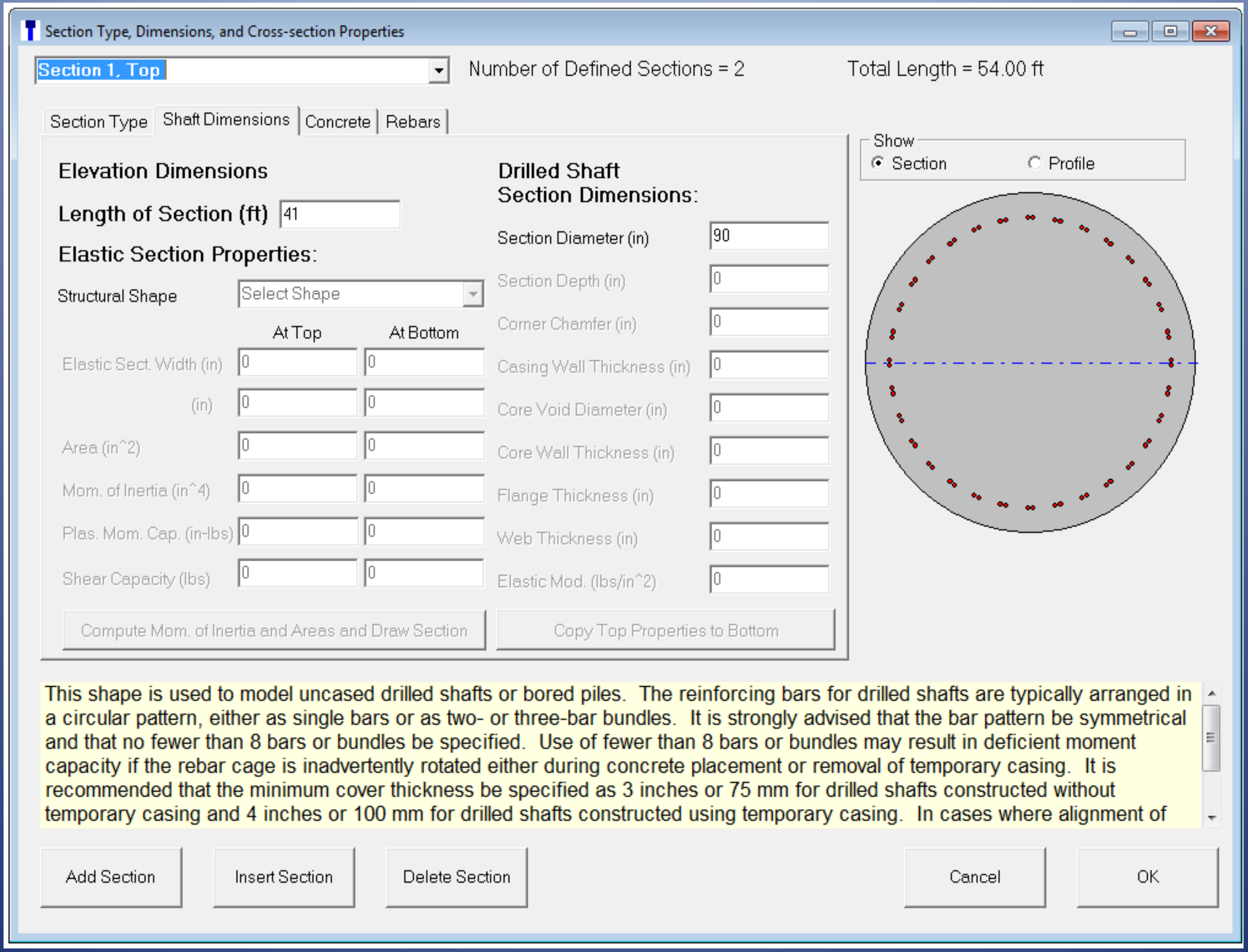


Monitoring SI-3

Inclinometer Results For SI-3

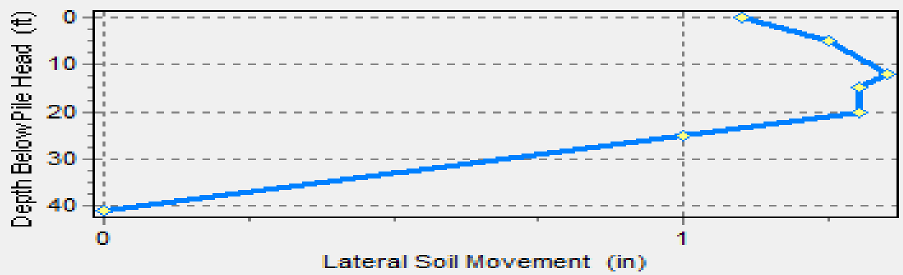


LPile Back Analysis



LPile Back Analysis

Soil Movements



Point	Depth Below Pile Head (ft)	Lateral Soil Movement (in)
1	0	1.1
2	5	1.25
3	12	1.35
4	15	1.3
5	20	1.3

Add Row Insert Row Delete Row

File Name:

Browse View/Edit File Read Values from File

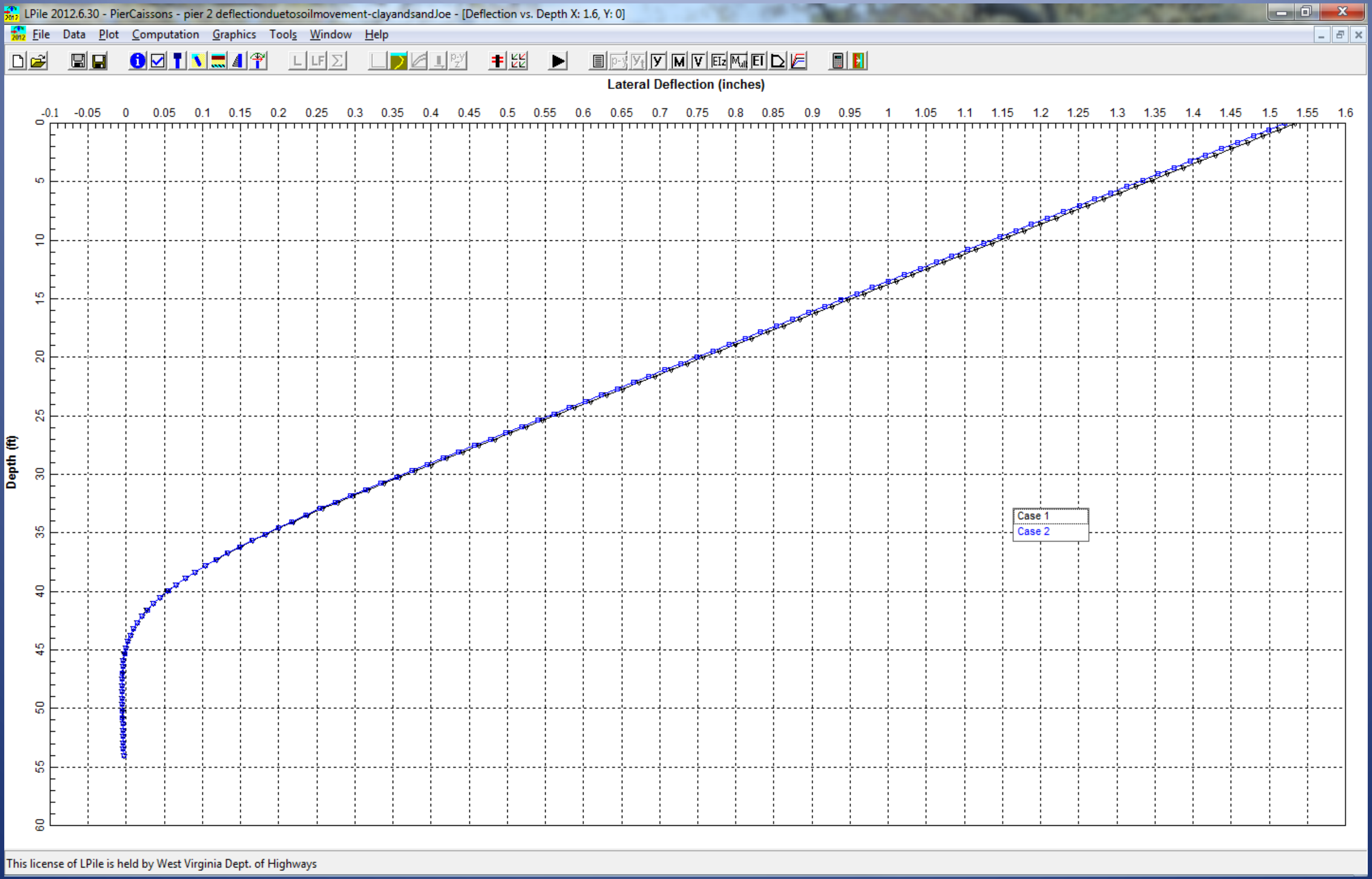
Paste values from Clipboard (text)

The soil movement profile is input as soil movement values versus vertical depth below the ground surface to the tip of the pile.
All soil movement values below the deepest point are assumed equal to zero.

To read a file with soil movement vs. depth data, first specify the filename by using the Browse button,then press the Read Values from File button.
The external file should be a text file with with the data entered one data pair per line, separated by spaces, commas, or tabs.

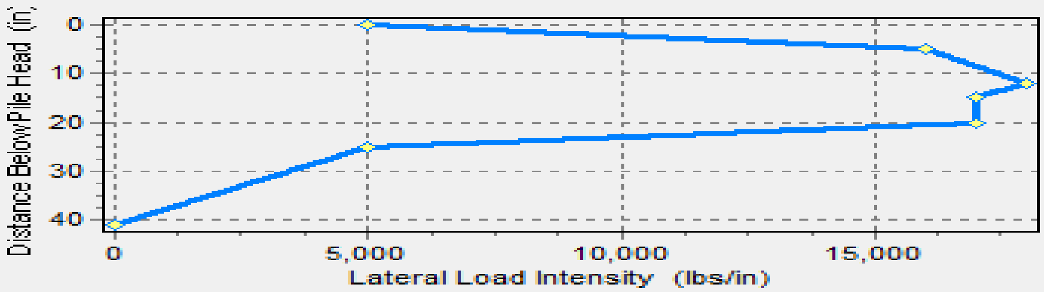
The Peak Movement at the Pier Due to Undrained Squeeze is Estimated to be 1.35”

LPile Back Analysis



LPile Back Analysis

Distributed Lateral Loads



Point	Distance Below Pile Head (in)	Lateral Load Intensity (lbs/in)
1	0	5000
2	5	16000
3	12	18000
4	15	17000
5	20	17000

Add Row | Insert Row | Delete Row

File Name:

Browse | View/Edit File | Read Values from File

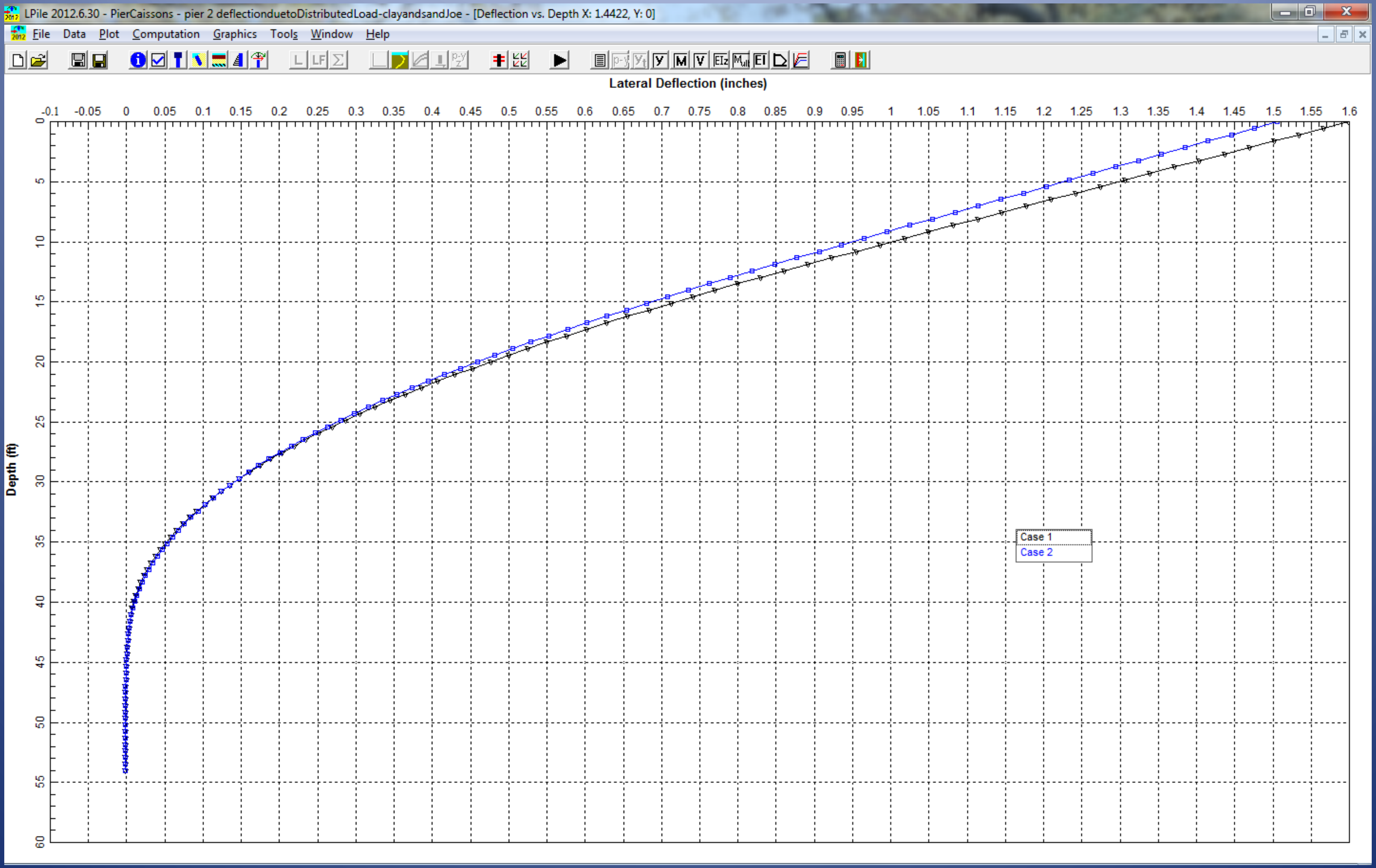
Paste values from Clipboard (text)

Values of distributed lateral load are input as values versus distance below the pile head. Distributed lateral loads are applied perpendicular to the axis of the pile and are input in units of force per unit length of pile.

To read a file of distributed lateral load vs. depth below the pile head, first specify the filename by using the Browse button,then press the Read Values from File button. The external file should be a text file with with the data entered one data pair per line, separated by spaces, commas, or tabs.

The Peak load Equates to a peak stress of 28.8 ksf in the soil

LPile Back Analysis

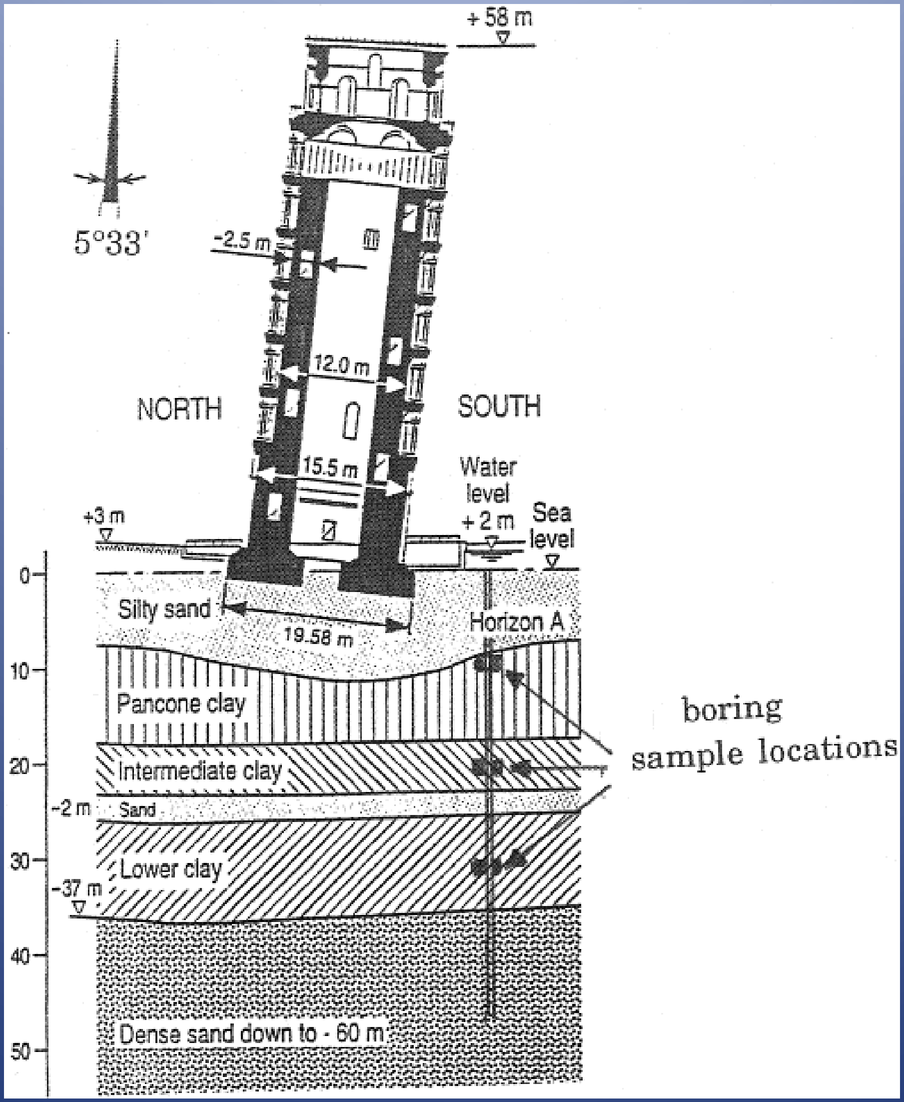


Summary

- 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



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Lateral Squeeze

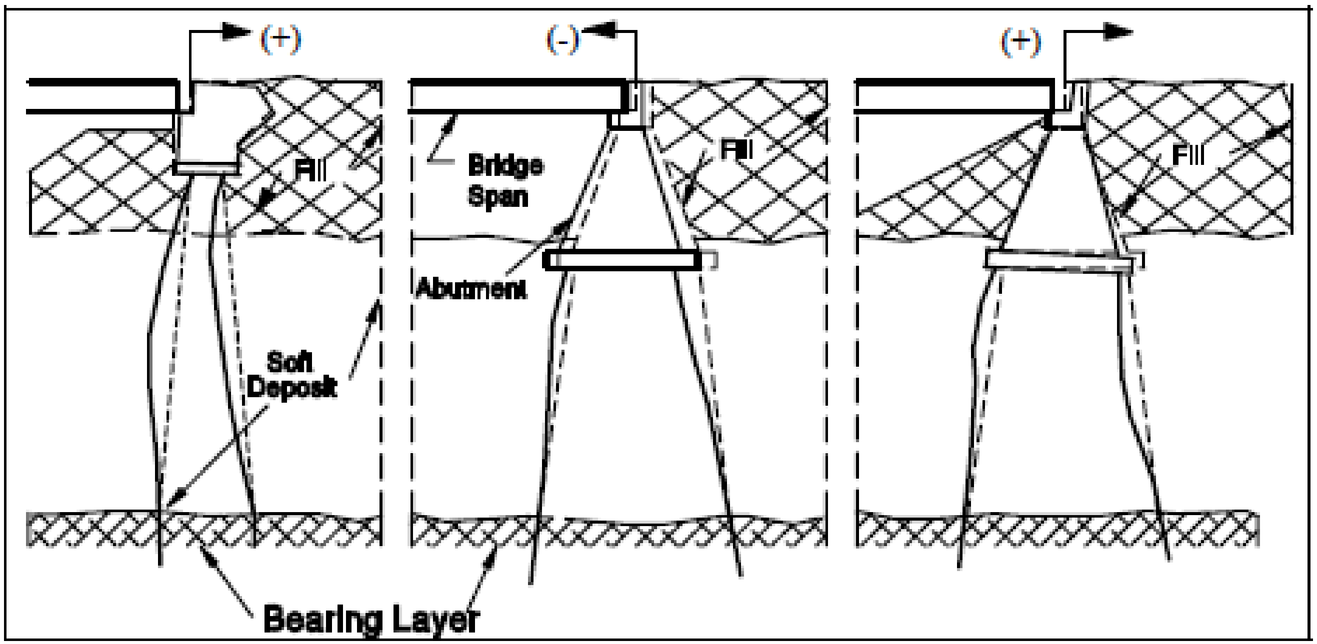

Figure 7-19. Examples of abutment tilting due to lateral squeeze (FHWA, 2006a).

Lateral Squeeze

Estimation of Horizontal Movement

Table 7-5
Summary of abutment movements (Nicu, *et al.*, 1971)

Foundation	Fill Settlement (inches)	Abutment Settlement (inches)	Abutment Tilting (inches)	Ratio of Abutment Tilting to Fill Settlement
Steel H-piles	16	Unknown	3	0.19
Steel H-piles	30	0	3	0.10
Soil bridge	24	24	4	0.17
Cast-in-place pile	12	3.5	2.5	0.19
Soil bridge	12	12	3	0.25
Steel H-piles	48	0	2	0.06
Steel H-piles	30	0	10	0.33
Steel H-piles	5	0.4	0.5 to 1.5	0.1 to 0.3
Timber Piles	36	36	12	0.33

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

SETTLEMENT

LAYER 1

$\sigma_o' = 6(128) = 768$ psf
$\Delta\sigma = 60(130) = 7800$ psf
$\sigma_f' = 8570$ psf

$$\Delta H_1 = \frac{12(12)(0.02)}{1.55} \log \frac{8570}{768} = 1.9 \text{ in.}$$

LAYER 2

$\sigma_o = 6(128) + 6(128-62.4) + 9(122-62.4)$
$= 1698$ psf
$\Delta\sigma = 7800$ psf
$\sigma_f' = 9500$ psf $\quad (P_c' = 5000 \text{ psf})$

$$\Delta H_2 = \frac{18(12)(0.047)}{1.75} \log \frac{5000}{1698} + \frac{18(12)(0.23)}{1.75} \log \frac{9500}{5000}$$

$\Delta H_2 = 10.6$ in.

$\Sigma \Delta H = 12.5$ in

Tilt = 0.25x 12.5” ≈ 3”