NATIONAL ACADEMIES Sciences Engineering Medicine

TRB TRANSPORTATION RESEARCH BOARD

TRB Webinar: Foundations in Shrink Swell Soil—Innovation and State of Practice

March 13, 2023

11:00 AM – 12:30 PM

PDH Certification Information

1.5 Professional Development Hours (PDH) – see follow-up email

You must attend the entire webinar.

Questions? Contact Andie Pitchford at TRBwebinar@nas.edu

The Transportation Research Board has met the standards and requirements of the Registered Continuing Education Program. Credit earned on completion of this program will be reported to RCEP at RCEP.net. A certificate of completion will be issued to each participant. As such, it does not include content that may be deemed or construed to be an approval or endorsement by the RCEP.

ENGINEERING RCEP REGISTERED CONTINUING EDUCATION PROGRAM

Purpose Statement

This webinar will share innovative findings and the state of practice for foundations in shrink swell soil. Presenters will share research findings on the uplift of drilled shafts and pressure on retaining walls due to these soils. Presenters will also discuss the pile load transfer mechanism in unsaturated shrink swell soil and practical aspects of deep foundation design.

Learning Objectives

At the end of this webinar, you will be able to:

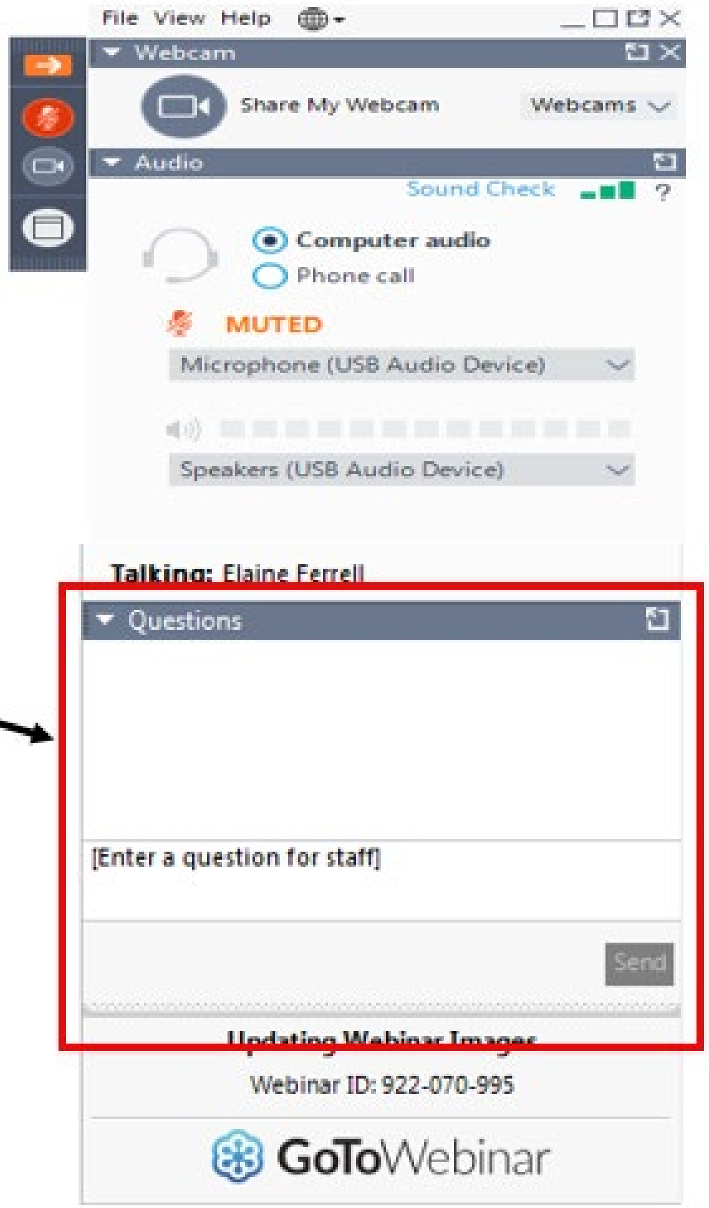
(1) Clarify the nature of uplift loading on drilled shafts

(2) Understand the pile load transfer mechanism in unsaturated shrink swell soil

(3) Use deep foundation design in shrink swell soil

Questions and Answers

- Please type your questions into your webinar control panel
- We will read your questions out loud, and answer as many as time allows



Today's presenters

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Foundations in Shrink Swell Soil—Innovation and State of Practice

Deep Foundation Design Practice

Shailendra Endley, Ph.D., P.E.
Intertek-PSI
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EXPANSIVE SOILS

Expansive soils exhibit unusually large volume changes as a result of moisture variations.

Expansive soils contain clay minerals with a strong affinity for water. Montmorillonite minerals undergo the largest volume changes.

Photo from US Department of Agriculture.

EXPANSIVE SOIL IDENTIFICATION

Different test methods to identify expansive soils:

- Atterberg Limits
- Activity (function of Plasticity Index and clay content)
- Free Swell / Swell Pressure Tests
- Expansion Index Test
- Coefficient of Linear Extensibility (COLE)

Table 1—Determining Degree of Expansion in Soil

Degree of Expansion	LL	PI	τ_{nat}, kPa
High	>60	>35	>383 kPa
Marginal	50–60	25–35	144 to 383 kPa
Low	<50	<25	<144 kPa

From AASHTO T258-81 “Standard Method of Test for Determining Expansive Soils”

EXPANSIVE SOIL DISTRIBUTION

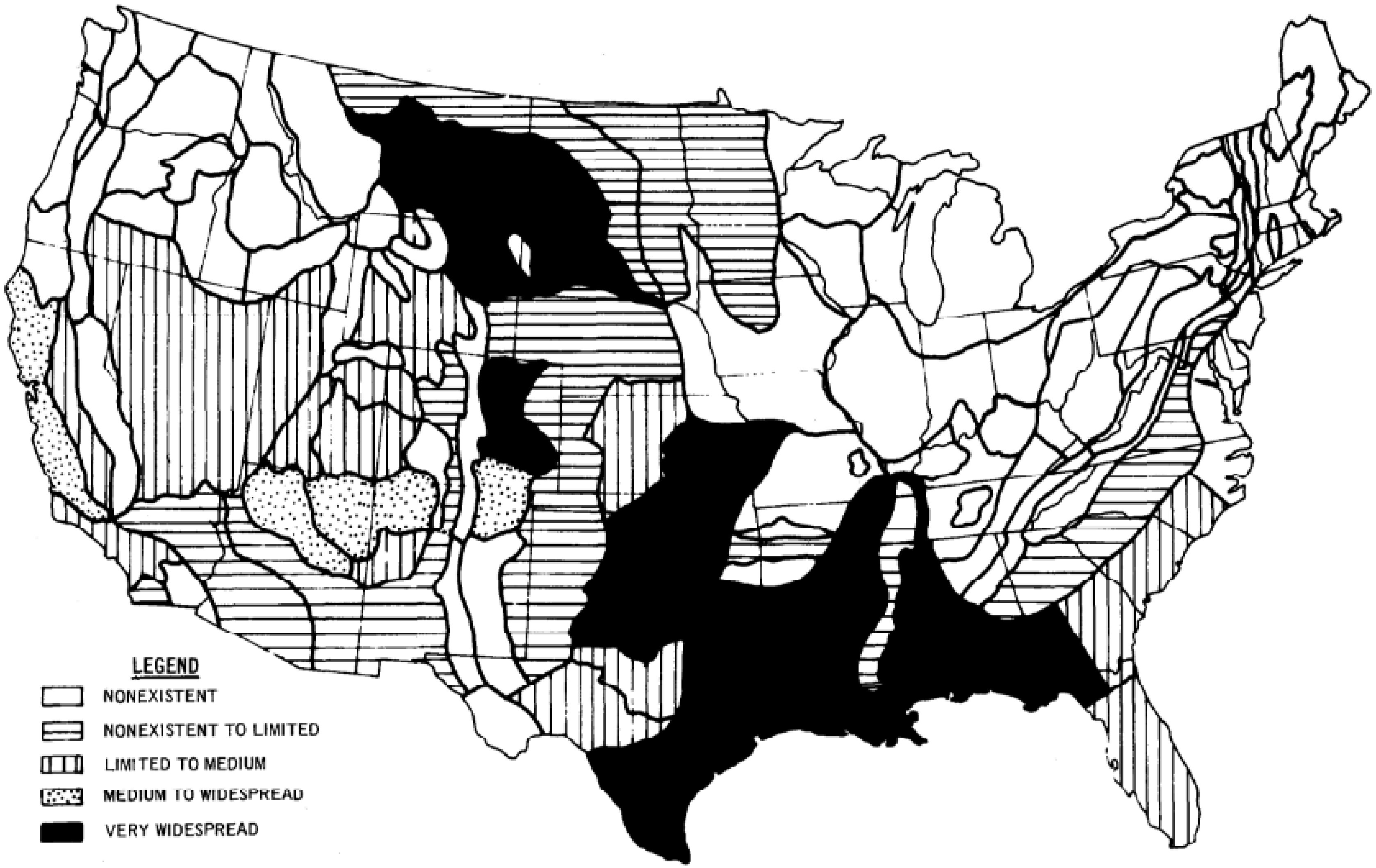


Figure 3. Estimated final adjusted frequency of occurrence rating of high volume change soils, by physiographic unit (from Witczak[9])

Expansive (high volume change) soils are present to some degree in large portion of
the United States.

About 12 States have expansive soils coverage in the medium to very widespread category.

(From: FHWA-RD-76-82, An Occurrence and Distribution Survey of Expansive Materials in the United States by Physiographic Areas, Jan 1976)

EXPANSIVE SOILS AND CLIMATE

- Damage is most prevalent in certain parts of California, Wyoming, Colorado, and Texas.
- Semi-arid climate with periods of intense rainfall followed by long periods of drought.
- Pattern of wet and dry cycles results in periods of extensive near-surface drying and desiccation crack formation.
- During intense precipitation, water enters the deep cracks causing the soil to swell; upon drying, the soil will shrink.

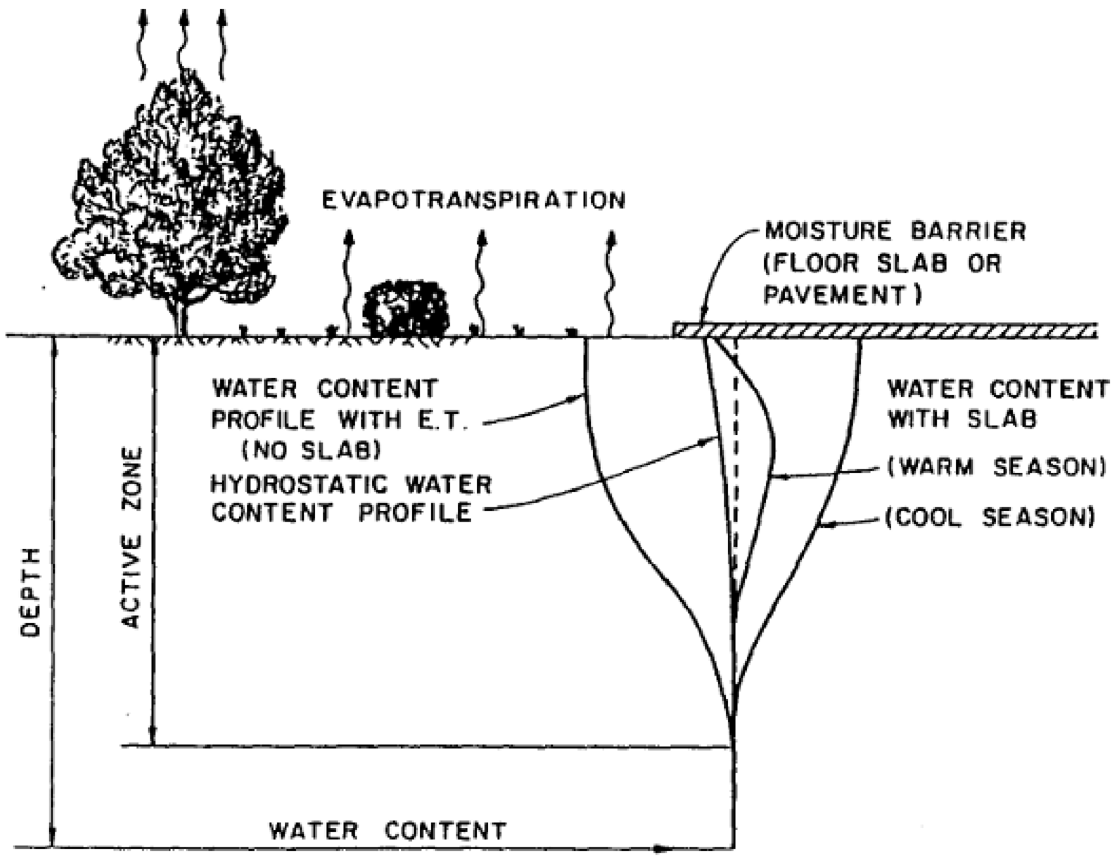
EXPANSIVE SOILS AND ACTIVE ZONE

- "Zone of seasonal moisture fluctuation" is the depth to which climatic and seasonal changes in temperature, precipitation, groundwater levels, etc. can cause fluctuations in soil water content.

- "Active Zone" or "Active Depth" is the zone of soil that contributes to heave due to soil expansion.

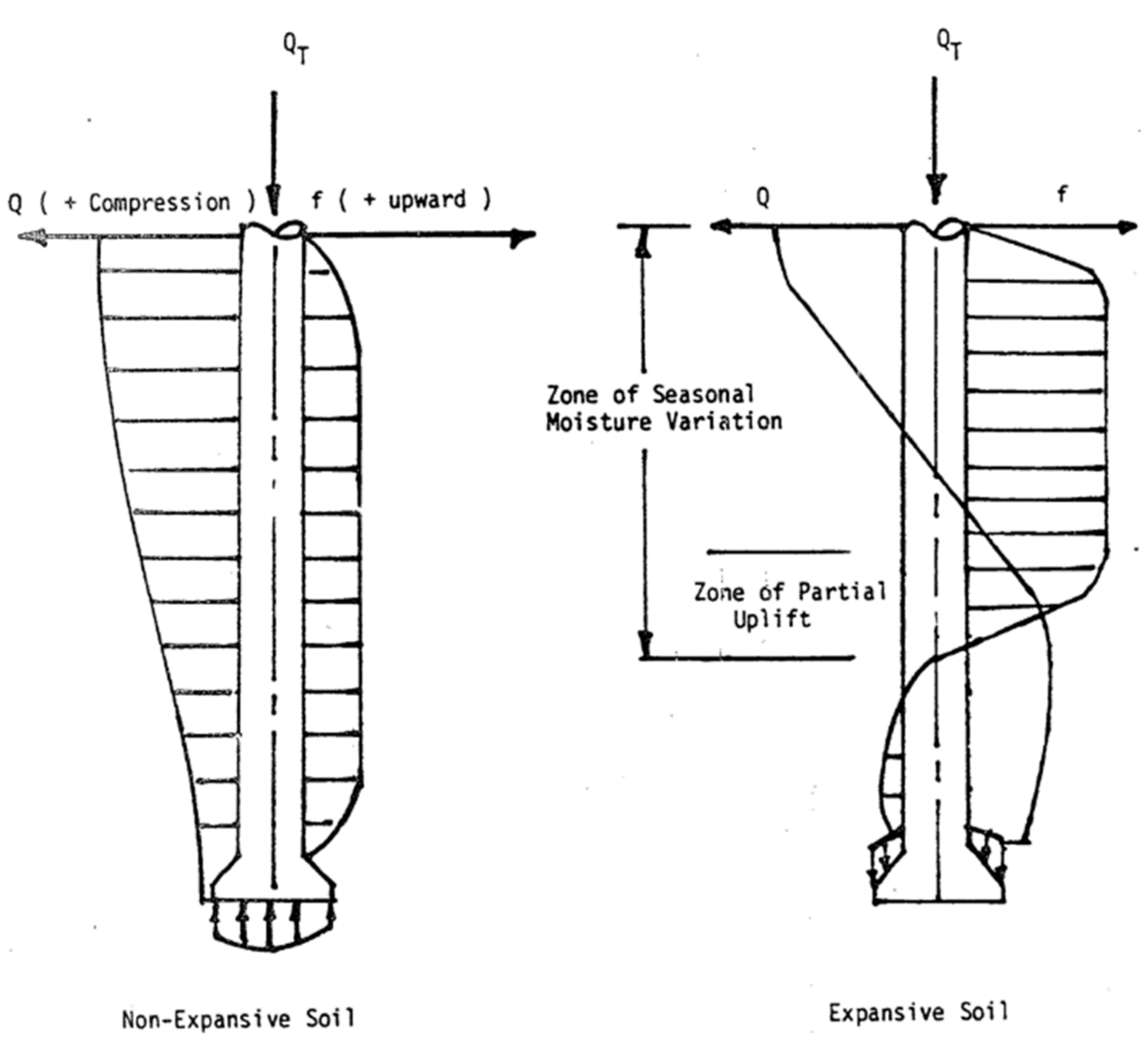
- Active Depth is difficult to calculate and can vary significantly. For example, in Texas it can range from about 5 feet along the Gulf Coast to 30+ feet away from it.

- Impermeable surfaces, drainage, vegetation and irrigation can affect active zone.



From Nelson and Miller (1992) Expansive Soils: Problems and Practice in Foundation and Pavement Engineering.

EXPANSIVE SOIL IMPACTS ON DESIGN OF DEEP FOUNDATIONS



If Uplift force exceeds the compression load (Q_T) near the top, two potential scenarios can develop:

1) Foundation uplift movement may occur, if insufficient uplift capacity, i.e. embedment, is provided below active zone;

2) and/or, foundation tension failure (cracking) may occur if expansive soil uplift exceeds structural (tensile) capacity of foundation.

(From: Design of Drilled Shafts in Expansive Clays, Michael W. O'Neill, 1987)

EXPANSIVE SOILS IN TRANSPORTATION FOUNDATION PRACTICE

State DOT Geotechnical Guidelines for Expansive Soils are limited or non-specific, for example:

Texas DOT Geotechnical Manual (2020):

Shrinkage and swelling to be included in Disregard Depth; no direct guidance in how to evaluate.

Disregard Depth

Disregard surface soil in the design of drilled shafts and piling foundations. The disregarded depth is the amount of surface soil that is not included in the design of the foundation due to potential erosion from scour, future excavation, seasonal soil moisture variation (shrinkage and swelling), lateral migration of waterways, and other factors. Disregard a minimum amount of 5 ft. over non-water crossings and 10 ft. over stream crossings. For abutments, disregard the portion of foundation passing through embankment fills.

Colorado DOT Geotechnical Manual (2021):

Expansive soils and rocks to be considered in the subsurface investigation and in foundation design; but no guidance in how to evaluate either.

Investigate potentially unfavorable conditions such as springs, swamps, bogs, seepage, slide areas, expansive soils or rock, collapsible soils, soft soils, weak soils or rock, compressible soils, liquefiable soils, or other conditions that could affect construction of highway structures or roadbed stability. Conduct field explorations

The foundation types applicable to a given project depend on anticipated loads and scour depths (where applicable), along with consideration of settlement, downdrag, bearing resistance, lateral resistance, seismic hazards, constructability, and other applicable factors.

California CALTRANS Geotechnical Design Reports (2021):

To be included in subsurface conditions evaluation, no specific guidance in design.

Describe project site geology and subsurface conditions, including a general description of regional geology relevant to the project. Present information only, not how it relates to design and construction. A generalized discussion is sufficient. Include:

- Heaving or expansive materials

AASHTO METHODOLOGY

AASHTO LRFD Bridge Design Specifications (9th Edition, 2020) addresses uplift due to expansive soils in:

<u>Section 10.4</u>

Potential for soil swell that may result in uplift of deep foundations should be evaluated based on Table 10.4.6.3-1.

Table 10.4.6.3-1—Method for Identifying Potentially Expansive Soils (Reese and O'Neill, 1988)

Liquid Limit *LL* (%)	Plastic Limit *PL* (%)	Soil Suction (ksf)	Potential Swell (%)	Potential Swell Classification
>60	>35	>8	>1.5	High
50–60	25–35	3–8	0.5–1.5	Marginal
<50	<25	<3	<0.5	Low

<u>Section 10.7</u>

Design considerations for deep foundations are established under **10.7.1.6.3** for driven piles:

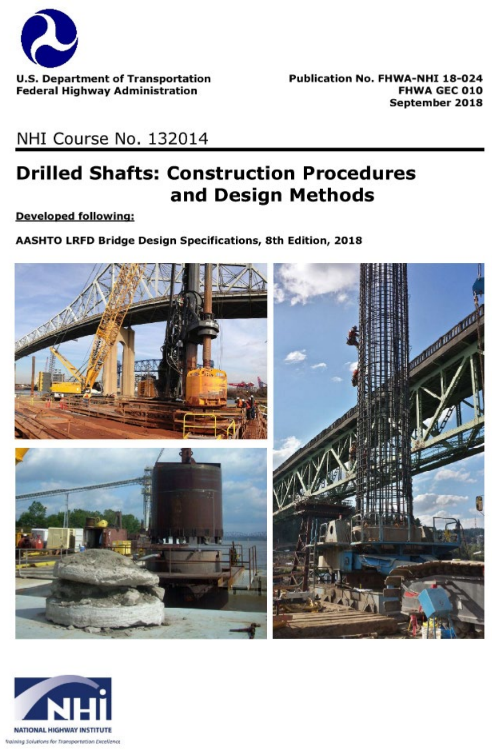
- Piles shall extend to a depth into moisture-stable soils sufficient to provide adequate anchorage to resist uplift.

(also applicable to drilled shafts and micropiles)

FHWA METHODOLOGY

Current Federal Highway Administration Geotechnical Engineering Manuals for Deep Foundations:

Only the Drilled Shaft and Flight Auger Pile GEC's address formally Uplift from Expansive Soils


U.S. Department of Transportation
Federal Highway Administration

Publication No. FHWA-NHI 18-024
FHWA GEC 010
September 2018

NHI Course No. 132014

Drilled Shafts: Construction Procedures and Design Methods

Developed following:

AASHTO LRFD Bridge Design Specifications, 8th Edition, 2018

GEOTECHNICAL ENGINEERING

CIRCULAR (GEC) No. 8

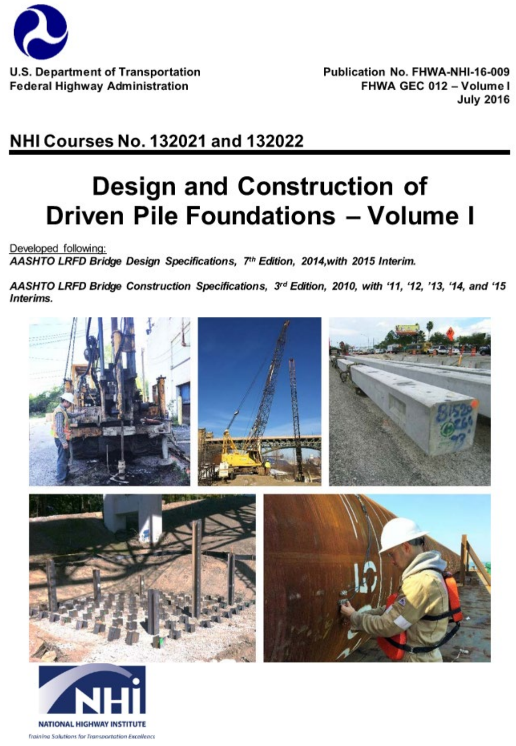
DESIGN AND CONSTRUCTION

OF CONTINUOUS

FLIGHT AUGER PILES

FINAL

April 2007


U.S. Department of Transportation
Federal Highway Administration

Publication No. FHWA-NHI-16-009
FHWA GEC 012 – Volume I
July 2016

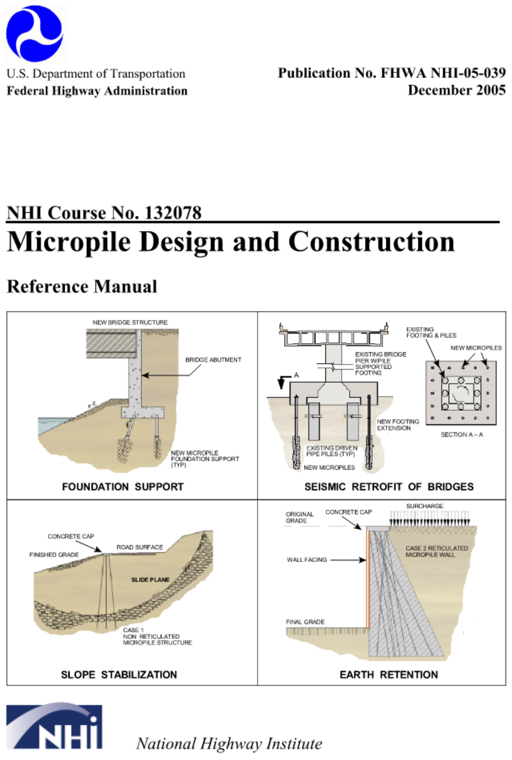
NHI Courses No. 132021 and 132022

Design and Construction of Driven Pile Foundations – Volume I

Developed following:

AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014,with 2015 Interim.

AASHTO LRFD Bridge Construction Specifications, 3rd Edition, 2010, with '11, '12, '13, '14, and '15 Interims.


U.S. Department of Transportation
Federal Highway Administration

Publication No. FHWA NHI-05-039
December 2005

NHI Course No. 132078

Micropile Design and Construction

Reference Manual

National Highway Institute

FHWA METHODOLOGY - CFA PILES

FHWA GEC No. 8 Flight Auger Piles, April 2007, Section 5.5.4, presents the following guidelines for uplift causes by the swelling of expansive soils:

- Uplift force should be considered as a load to the pile
- Load may be determined equal to the ultimate downward side shear values, but opposite direction.

FHWA METHODOLOGY – DRILLED SHAFTS

FHWA-NHI 18-024 (2018) GEC No. 10 presents guidelines for uplift caused by the swelling of expansive soils in Appendix B.6.

The methodology steps include:

1) Identification of expansive geomaterials. The classification method prescribed is the "WES (Waterways Experiment Station) Classification Method", summarized in Table B-2.

TABLE B-2 METHOD OF IDENTIFYING POTENTIALLY EXPANSIVE SOILS (SNETHEN, ET AL., 1977)

Liquid Limit (%)	Plasticity Index	h (tsf) [a]	Potential Swell (%) [b]	Potential Swell Classification
< 50	< 25	< 1.5	< 0.5	low
50 – 60	25 - 35	1.5 – 4.0	0.5 – 1.5	marginal
> 60	> 35	> 4.0	> 1.5	high

[a] in situ total soil suction, which can be measured as prescribed in ASTM D 5298
[b] vertical swell at in situ overburden stress, which can be measured as prescribed in ASTM D 4546

- Explicit consideration given to expansive behavior in sites with surficial geomaterials classifying as "High"
- Explicit consideration not usually necessary for sites with surficial geomaterials classifying as "Low"
- With "Marginal" geomaterials, designers to rely on local experience to decide.

FHWA METHODOLOGY – DRILLED SHAFTS

2) <u>Explicit consideration of expansive geomaterials.</u>

a) Drilled shafts must be designed to extend below depths where substantial volume changes may occur.

Accomplish by extending the shafts to:

- Beyond depths where expansive geomaterials are present, or
- Below the zone of seasonal moisture changes, if the expansive soils are deep.

b) Drilled shafts must be designed to resist uplift forces by swelling soils.

Consider that:

- Uplift forces are counteracted by loading from superstructure.
- Hence heavy structures such as bridges are seldom impacted.
- For lightly-loaded structures, however, shafts must be designed to have sufficient structural capacity to resist uplift forces.

FHWA METHODOLOGY – DRILLED SHAFTS

2) Additional considerations.

   a) Zone of seasonal moisture change.

      - No definitive method exists for determining.

      - Hence, **most effective means** to establish is from **local/regional experience**.

   b) Uplift force from expansive geomaterial. Two approaches possible:

      - Isolation from "active zone", such as using casing or double-casing.

      - Design the drilled shaft to have the required structural capacity using conservative determination of the nominal/ultimate load. The recommended (prudent) approach is to assume that the **full strength of the soil** can be mobilized in uplift.

FHWA METHODOLOGY – DRILLED SHAFTS

2) Additional considerations (continued).

- The use of full strength of soil implies a larger force than used for nominal/ultimate side resistance in compression. (For clays this is “alpha” equal to 1.0.)
- What “full strength” should be used?   GEC 10 (2018) Appendix B.6 does not define it.
- Prior version of GEC 10 (from 2010), Section 13.7.3 did offer additional guidance on limiting values for uplift in expansive clays:

  > ‘*The undrained shear strength should be evaluated at the water content of the soil or rock after it absorbs all the water possible under the overburden pressure corresponding to the depth below finished grade.*’

Note: Undrained strength at a saturated state seems reasonable, but from a practical perspective would require extensive laboratory testing beyond typical standard of practice.

HISTORICAL FHWA METHODOLOGY FOR EXPANSIVE SOIL UPLIFT ON DRILLED SHAFTS

DRILLED SHAFT MANUALS FROM 1988 and 1999 – The effective stress approach correlated to swell pressure.

Geotechnical community has not fully adopted GEC 10 (2018) or even GEC 10 (2010) and still widely use 1988 and 1999 versions. Particularly true in States where DOTs have not adopted LRFD geotechnical design, such as Texas.

- Axial capacity in compression uses ‘alpha’ method to estimate side resistance
- Alpha value of zero (0) may be used up to the full depth of seasonal moisture change because expansive soil can shrink away from the drilled shaft sides during dry weather.

HISTORICAL FHWA METHODOLOGY FOR EXPANSIVE SOIL UPLIFT ON DRILLED SHAFTS

DRILLED SHAFT MANUALS FROM 1988 and 1999 – The effective stress approach correlated to swell pressure

Design for uplift assumes:

- Uplift Load is the accumulation of skin friction on the shaft within the active zone calculated as:

$$f_{max} = \varphi\, \sigma'_{ho} \tan \delta_r \qquad (12.4)$$

where

φ = a correlation coefficient,

σ'_{ho} = the horizontal zero swell pressure in the soil at the depth at which f_{max} is computed, relative to the predicted moisture state at the time of drilled shaft construction, which can be measured in an odometer, and

δ_r = the effective residual angle of interface friction between the soil and the concrete.

- 'Phi' Correlation coefficient ranges from 1.0 to 1.3.

HISTORICAL FHWA METHODOLOGY FOR EXPANSIVE SOIL UPLIFT ON DRILLED SHAFTS

- Swell pressure testing is seldom carried out, and in practice values are obtained by published correlation or local/regional experience.
- In Central and East Texas our practice has adopted expected Swell Pressure values as function of soil PI, and resulting uplift load (<u>to be multiplied by shaft diameter</u>). Note that unit skin friction should not exceed undrained shear strength:

Uplift on Piers Due to Expansive Soils for Houston Area

Active Depth: 7

PI	Swell Press. ksf	Skin Friction ksf	Uplift Load kips	Uplift Load tons
30	2.0	0.60	13	7
35	2.5	0.75	17	8
40	3.0	0.90	20	10
45	3.5	1.05	23	12
50	4.0	1.20	26	13
55	4.5	1.35	30	15
60	5.0	1.50	33	17

a = 1.3

Uplift on Piers Due to Expansive Soils for Dallas & San Antonio and Austin

Active Depth: 10

PI	Swell Press. ksf	Skin Friction ksf	Uplift Load kips	Uplift Load tons
30	3.0	0.90	28	14
35	3.5	1.05	33	17
40	4.0	1.20	38	19
45	4.5	1.35	42	21
50	5.0	1.50	47	24
55	5.5	1.65	52	26
60	6.0	1.80	57	28

a = 1.3

EVOLUTION OF FHWA METHODOLOGY FOR EXPANSIVE SOIL UPLIFT ON DRILLED SHAFTS

Similar to current methodology, the 1988 & 1999 methodology offered suggestions for mitigation, by:

- Isolating the drilled shaft from the expansive soil by the use of casing, or
- (Most common) Designing the drilled shaft length such that the side resistance along the shaft below the active depth is greater than the uplift forces.

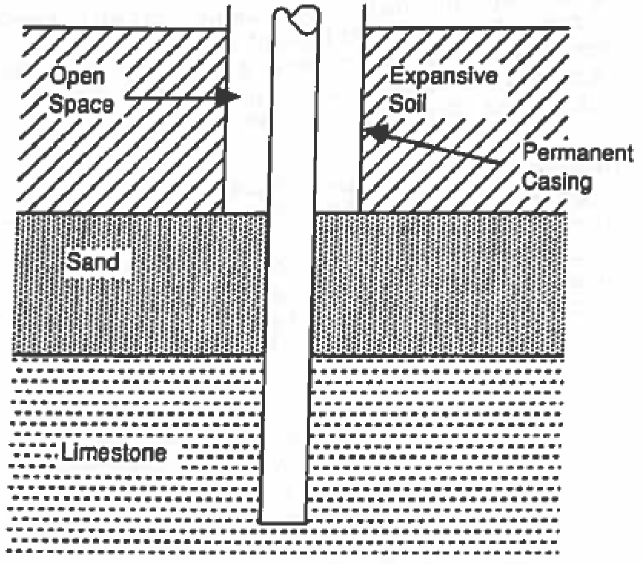


Figure 12.6. Use of a permanent surface casing for design in expansive soil.

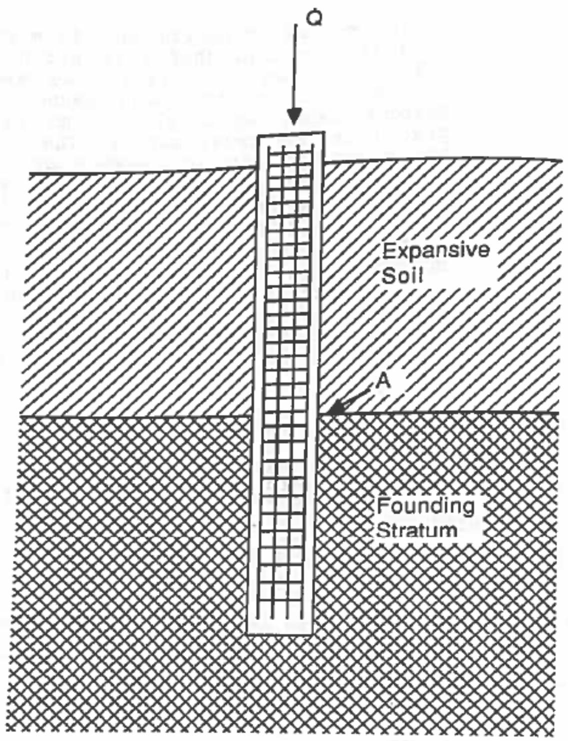


Figure 12.8. Use of rebar cage for design in expansive soil.

OTHER WIDELY USED METHODS (NOT SPECIFIC TO TRANSPORTATION)

Colorado Association of Geotechnical Engineers (CAGE) developed a document, **Drilled Pier Design Criteria for Lightly Loaded Structures in the Denver Metropolitan Area** (December 1999).

The potential uplift force ('U') is calculated also based on swell pressure as:

$$U = \Pi D L_w \propto S_p$$

Where U = pier uplift (kips)

D = pier diameter (feet)

L_w = zone of influence (feet)

$\propto$ = swell pressure coefficient. This is an empirical factor relating swell pressure measured in the laboratory swell-consolidation test to pressure acting along the pier. The swell pressure coefficient that has been frequently used is 0.15 but may vary. This coefficient has been determined from laboratory testing by Chen (1988)[1] and is based on limited data and varies with the engineer's judgement.

S_p = swell pressure determined from load-back during swell-consolidation test (ksf)

Notes:

- The zone of influence (Lw) is not necessarily same as the zone of moisture variation, as the Lw is defined as *"the zone through which instantaneous wetting and uplift occurs".*

- Overall design of a pier/shaft includes calculation of the swelling pressure of the soil, uplift force, the zone of influence depth, and the shear resistance of the portion of pier below zone of influence.

SUMMARY AND FINAL CONSIDERATIONS

1. Uplift forces on deep foundations due to swelling soils can be significant and should be considered for lightly-loaded foundations, or foundation elements with limited structural capacity in uplift.
2. Foundations supporting heavier loads such as for bridges are not likely to be significantly impacted. However, construction sequence should be evaluated as there may be a lag between foundation installation and construction of the superstructure.
3. FHWA’s approach for drilled shaft foundation (current and historical) appears to be reasonable, but more research appears to be warranted.
4. Local/regional experience is very important, particularly with respect to zone of moisture variation depth.
5. Coordination with Structural Engineer is paramount. Discuss the issue ahead of time to ensure adequate reinforcement and depth is provided.

THANK YOU

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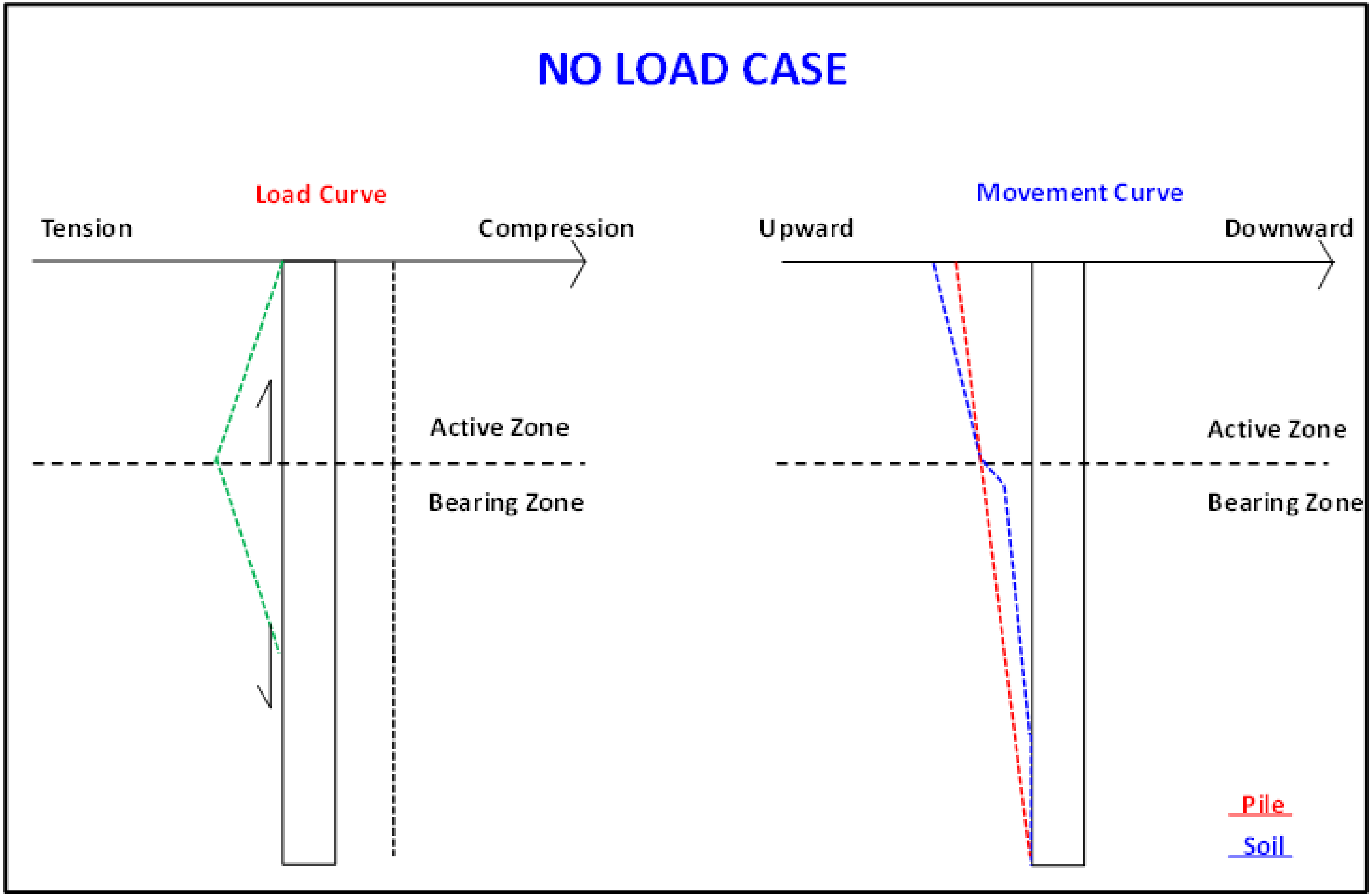
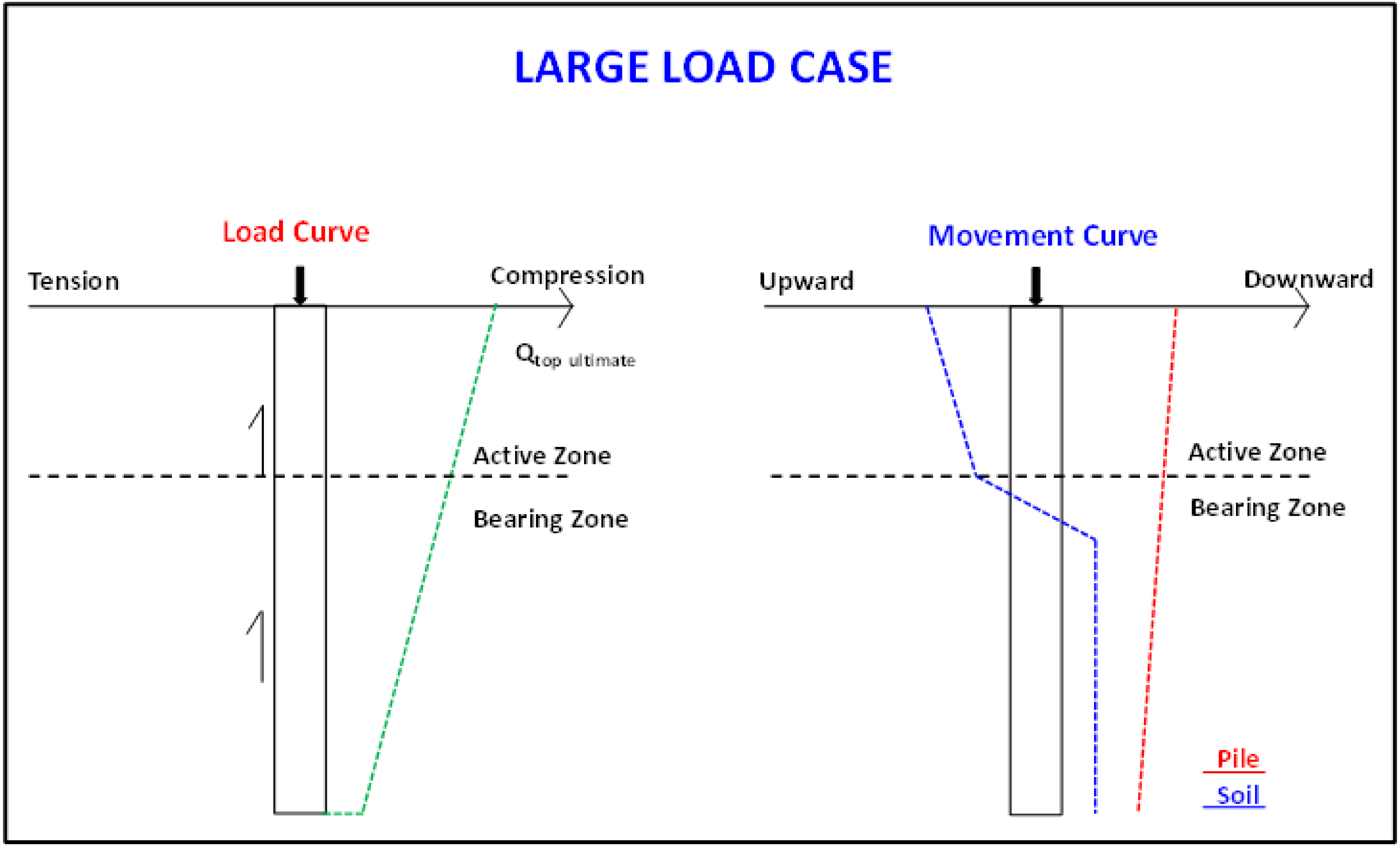
UPLIFT LOAD ON DRILLED SHAFTS IN SHRINK SWELL SOILS

A CERGEP Project

Jerome Sfeir, Jean-Louis Briaud
Texas A&M University

with the help of Tracy Brettmann (A.H. Beck) and Shailendra Endley (Intertek-PSI)


NO LOAD CASE
Load Curve
Tension
Compression
Active Zone
Bearing Zone
Movement Curve
Upward
Downward
Active Zone
Bearing Zone
Pile
Soil

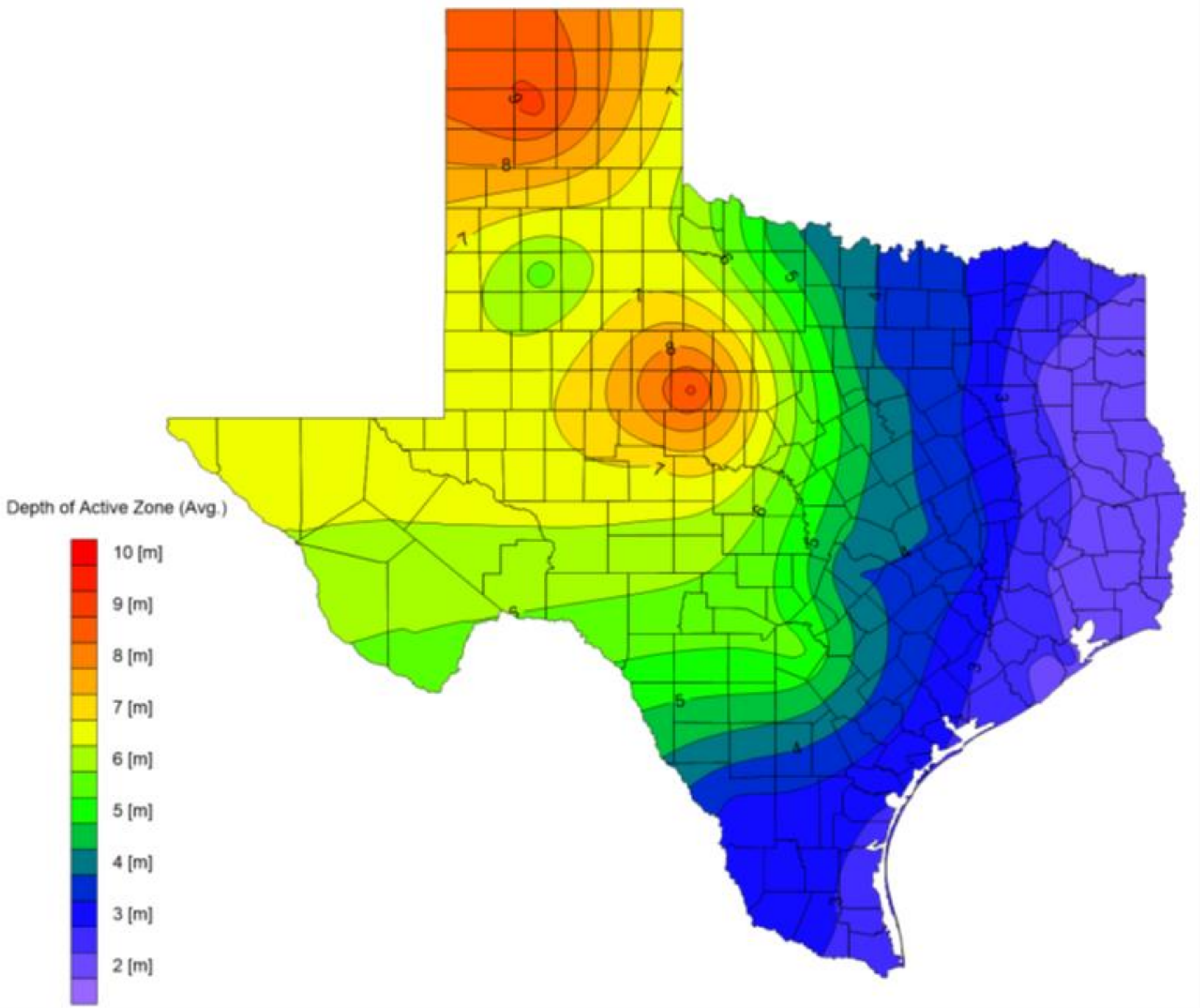

LARGE LOAD CASE
Load Curve
Tension
Compression
Q top ultimate
Active Zone
Bearing Zone
Movement Curve
Upward
Downward
Active Zone
Bearing Zone
Pile
Soil

EXISTING KNOWLEDGE FOR f_{max} UPLIFT

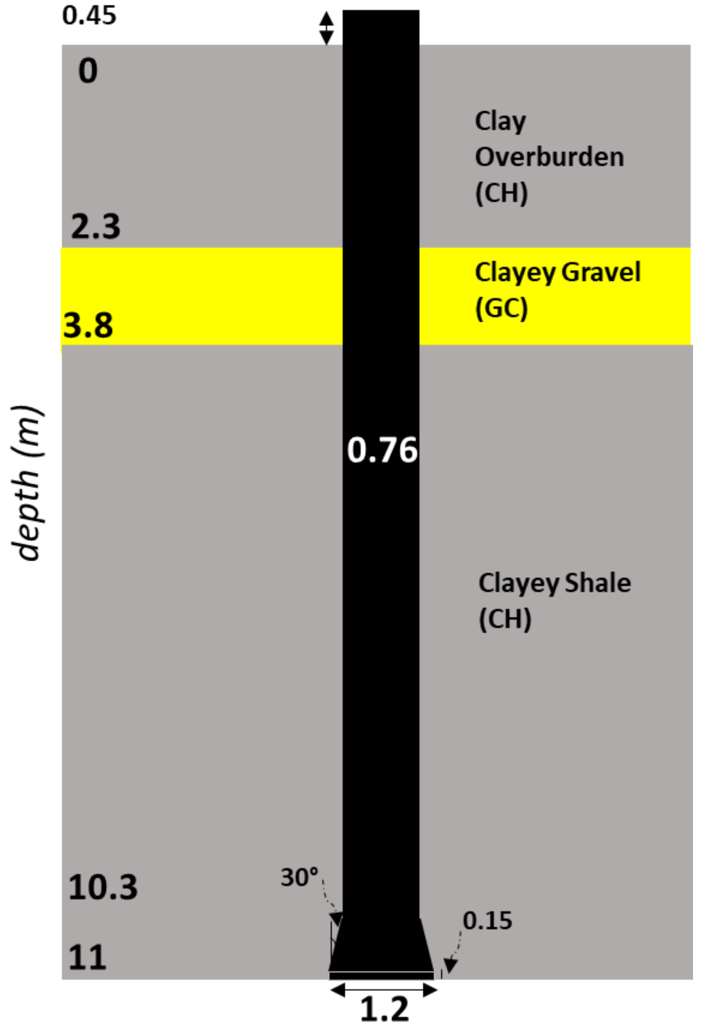
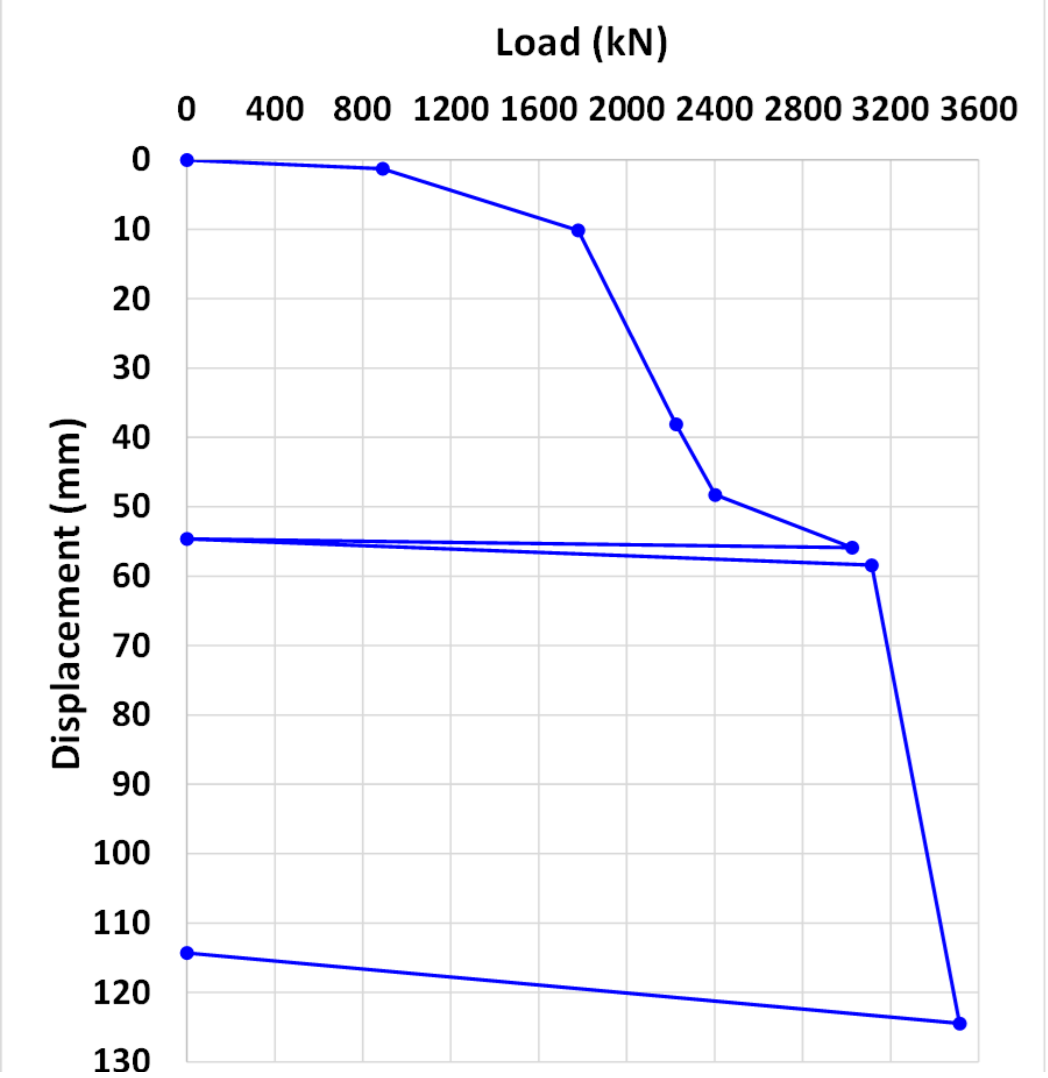
- $f_{max} = \alpha\ s_u$
  - FHWA: $\alpha = 1$ and s_u is undrained shear strength at swell limit
  - FPA (Houston): α = same as for carrying capacity (0.55 for very stiff clays)
  - USACE (Johnson-Stroman): α approaches 1

- $f_{max} = \beta_1\ \sigma'_s \tan\varphi_r$ with $\beta_1 \sim 1.3$ O'Neill (1988)
- $f_{max} = \beta_2\ \sigma'_s$ with $\beta_2 \sim 0.15$ Chen (1988)

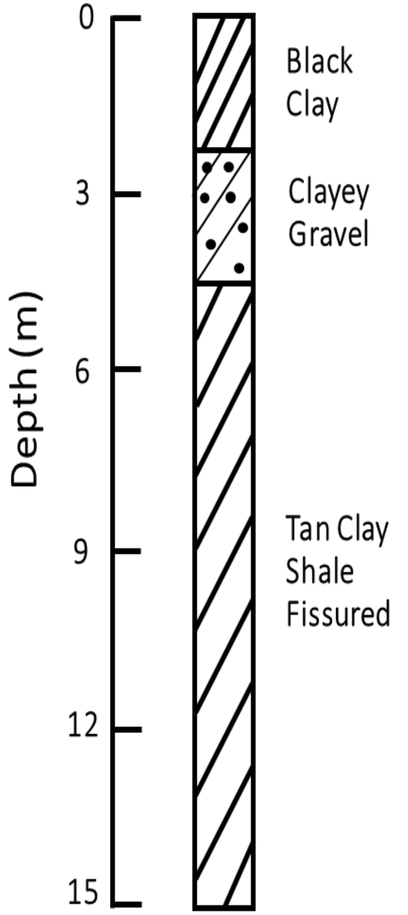
DEPTH OF SHRINK SWELL ZONE

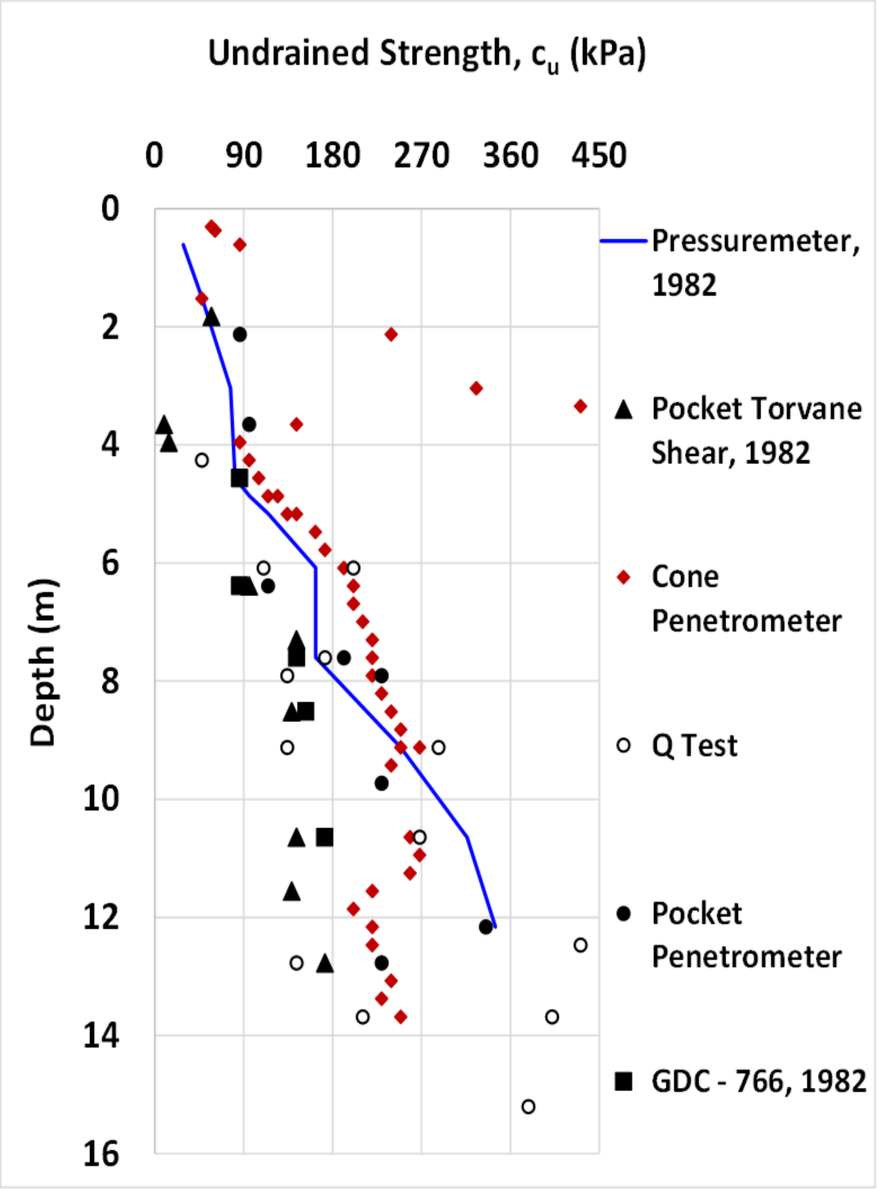


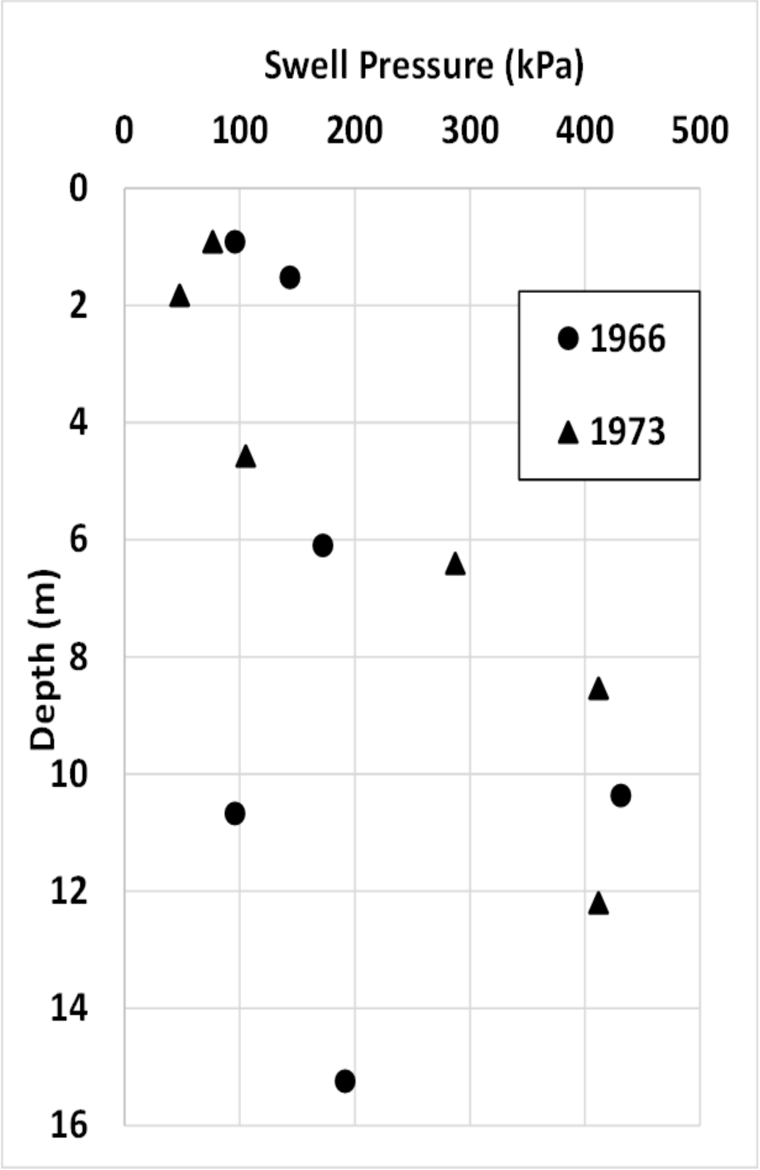
CASE HISTORY – JOHNSON STROMANN - 1982

a) PILE LAFB-2 AND STRATIGRAPHY	b) LOAD TEST CURVE
	

CASE HISTORY – JOHNSON STROMANN - 1982





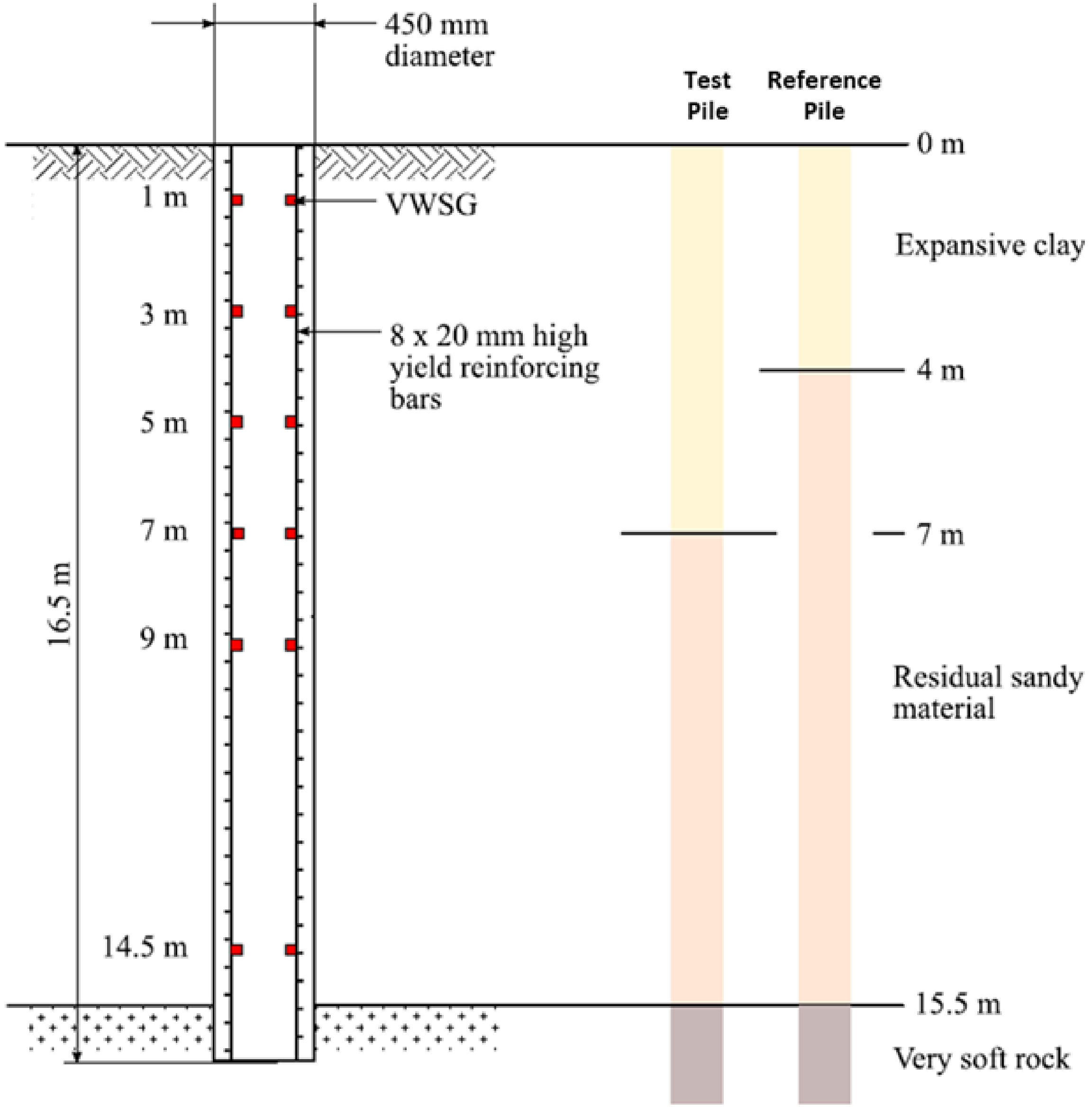


CASE HISTORY – JOHNSON STROMANN - 1982

- $f_{max} = \alpha\ s_u$
  - Back-calculated $\alpha = 0.59$

- $f_{max} = \beta_2\ \sigma'_s$ with $\beta_2 \sim 0.15$ Chen (1988)
  - Back-calculated $\beta_2 = 0.56$

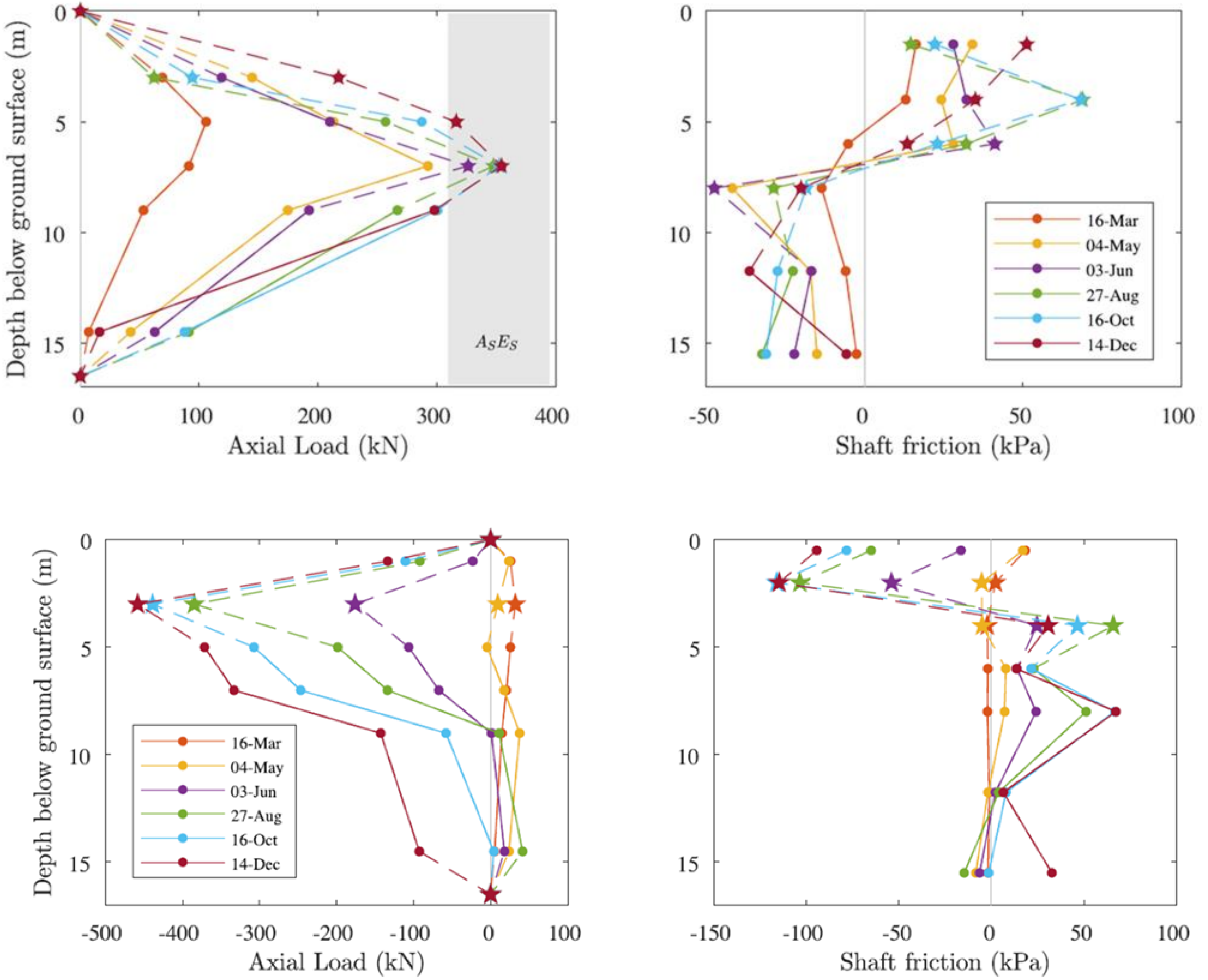
CASE HISTORY – DA SILVA et al. - 2022



SWELLING
TENSION

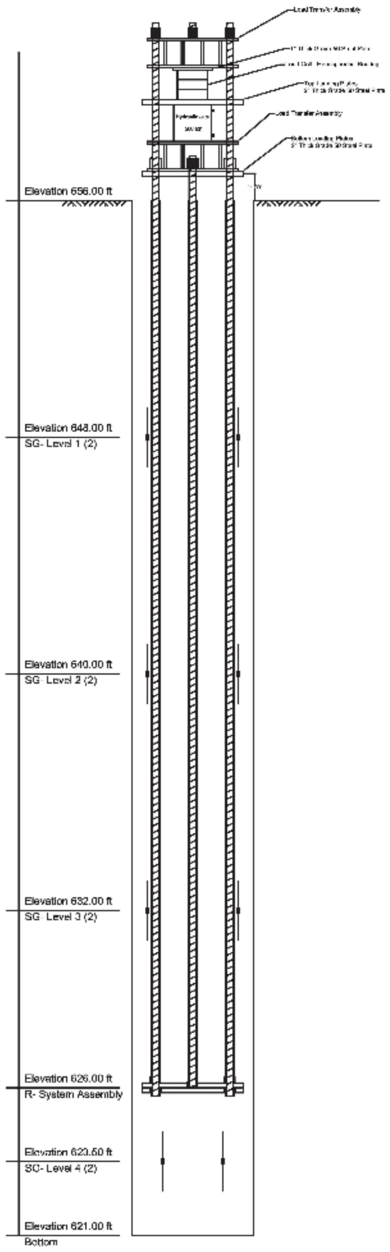
CASE HISTORY –
DA SILVA et al. -
2022

SHRINKING
COMPRESION

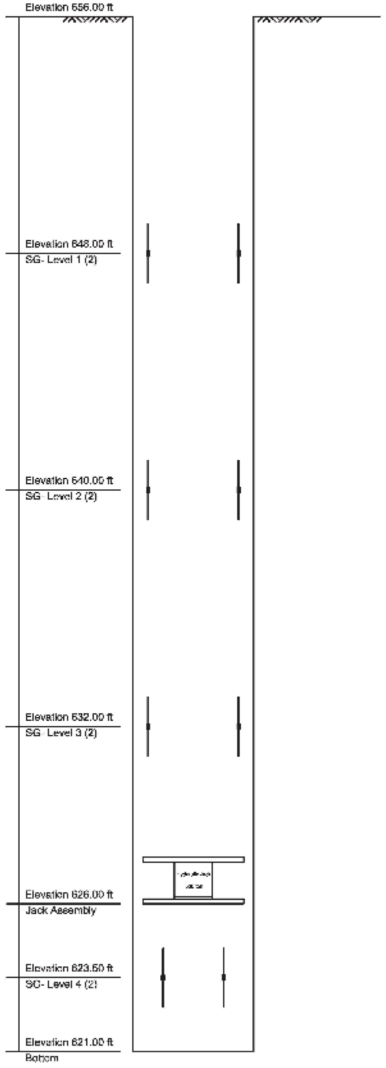


CASE HISTORY – DA SILVA et al. - 2022

- $f_{max} = \alpha\ s_u$
  - Back-calculated swelling $\alpha = 0.34$
  - Back calculated shrinking $\alpha = 0.58$



Top-Loaded Bi-Directional Test (TLBT)



Top-Loaded Bi-Directional Test (TLBT)

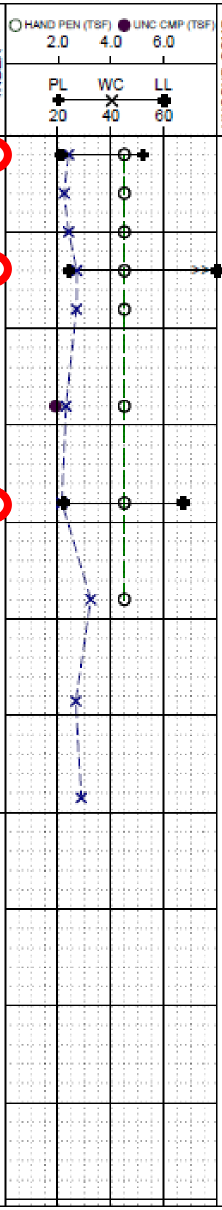
Proposed Office and Warehouse Development
9014 Green Road
Project No. 0312-1591

BORING B-9

LOCATION: See Boring Location Plan

Elevation: 658.0

SOIL DESCRIPTION	MOISTURE CONTENT	% RETAINED #4	% PASSING #200	SPT (N) & TCP (T) VALUES	% REC	%RQD	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	UNCONF. COMP. (TSF)	UNIT DRY WT. (LB/CU FT)
FAT CLAY (CH), very stiff, dark brown	24	12	69				52	21	31		
	22										
- light brown below 4'	24										
	27		94				85	24	61		
	27										
	23									1.9	101
	21		100				67	22	45		
	32										
	27			31							
- hard below 33'	29			34							
Boring Terminated at 35'											



COMPLETION DEPTH: 35.0 Feet
DATE: 12/16/17-12/16/17

DEPTH TO GROUND WATER
SEEPAGE (ft.): 22
END OF DRILLING (ft.): 22
DELAYED WATER LEVEL (FT):

psi Information To Build On
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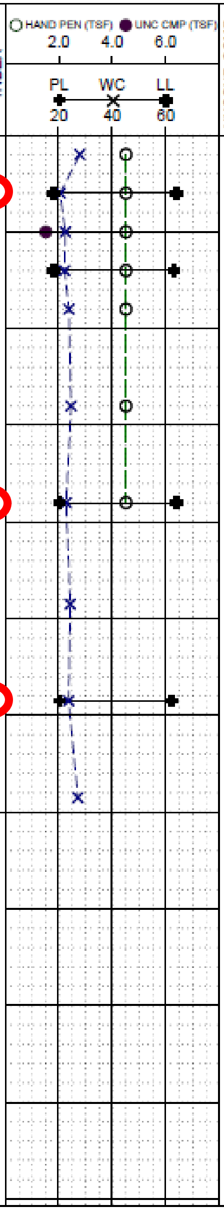
BORING B-12

LOCATION: See Boring Location Plan

Elevation: 658.0

SOIL DESCRIPTION	MOISTURE CONTENT	% RETAINED #4	% PASSING #200	SPT (N) & TCP (T) VALUES	% REC	%RQD	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	UNCONF. COMP. (TSF)	UNIT DRY WT. (LB/CU FT)
FAT CLAY (CH), very stiff, dark brown	28										
	20	5	82				64	18	46		
	22									1.5	101
- light brown below 6'	22	2	82				63	18	45		
	24										
	24										
	23		100				64	20	44		
- hard below 23'	24			40							
	24		100	30			62	20	42		
	27			50/3"							
Boring Terminated at 35'											



COMPLETION DEPTH: 35.0 Feet
DATE: 12/14/17-12/14/17

DEPTH TO GROUND WATER
SEEPAGE (ft.): 22
END OF DRILLING (ft.): 22
DELAYED WATER LEVEL (FT):

psi Information To Build On
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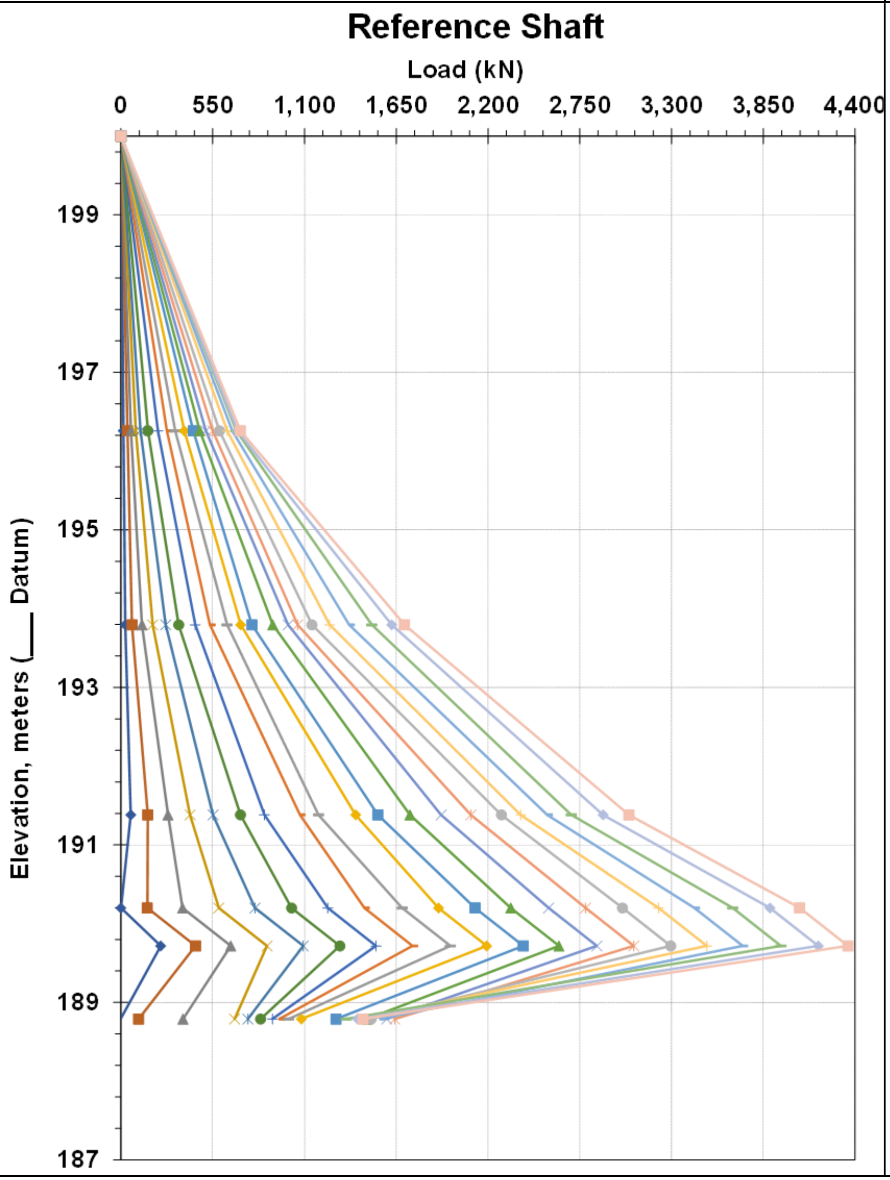
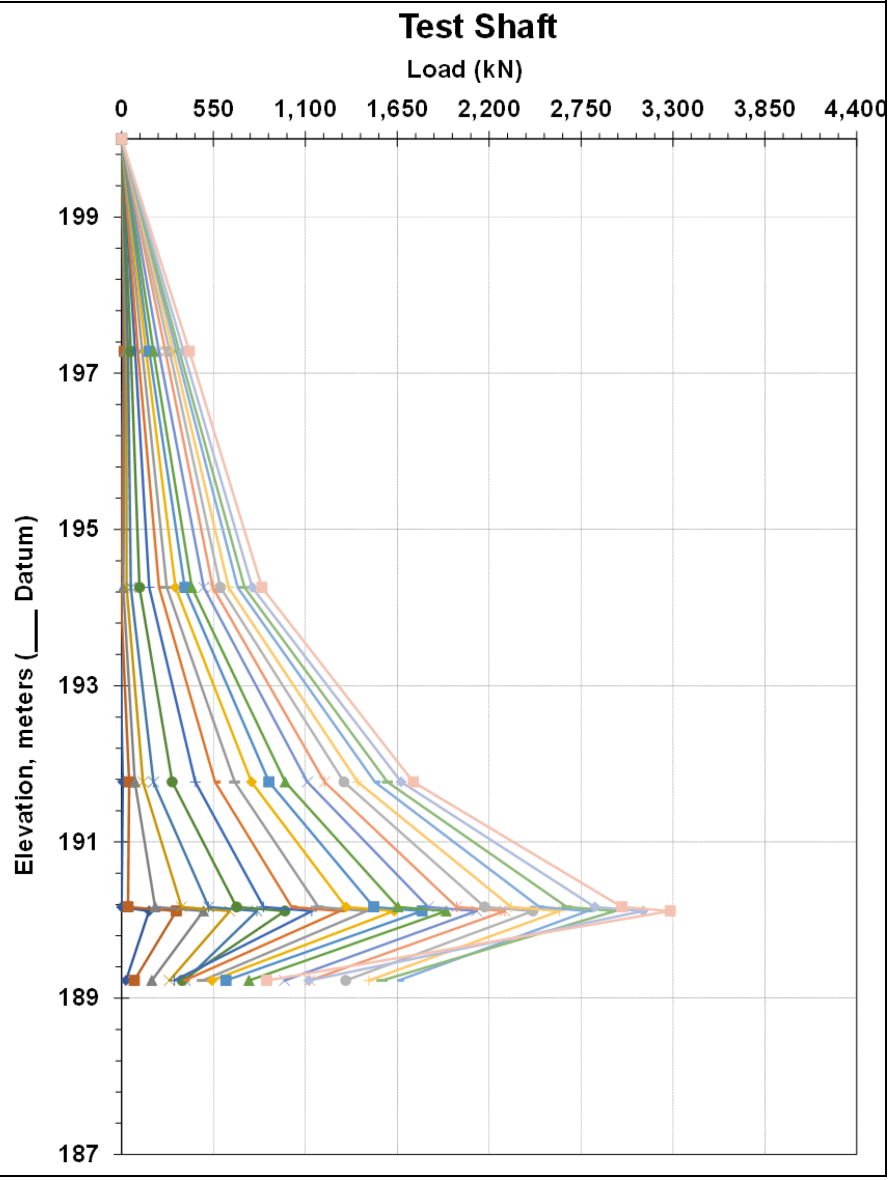
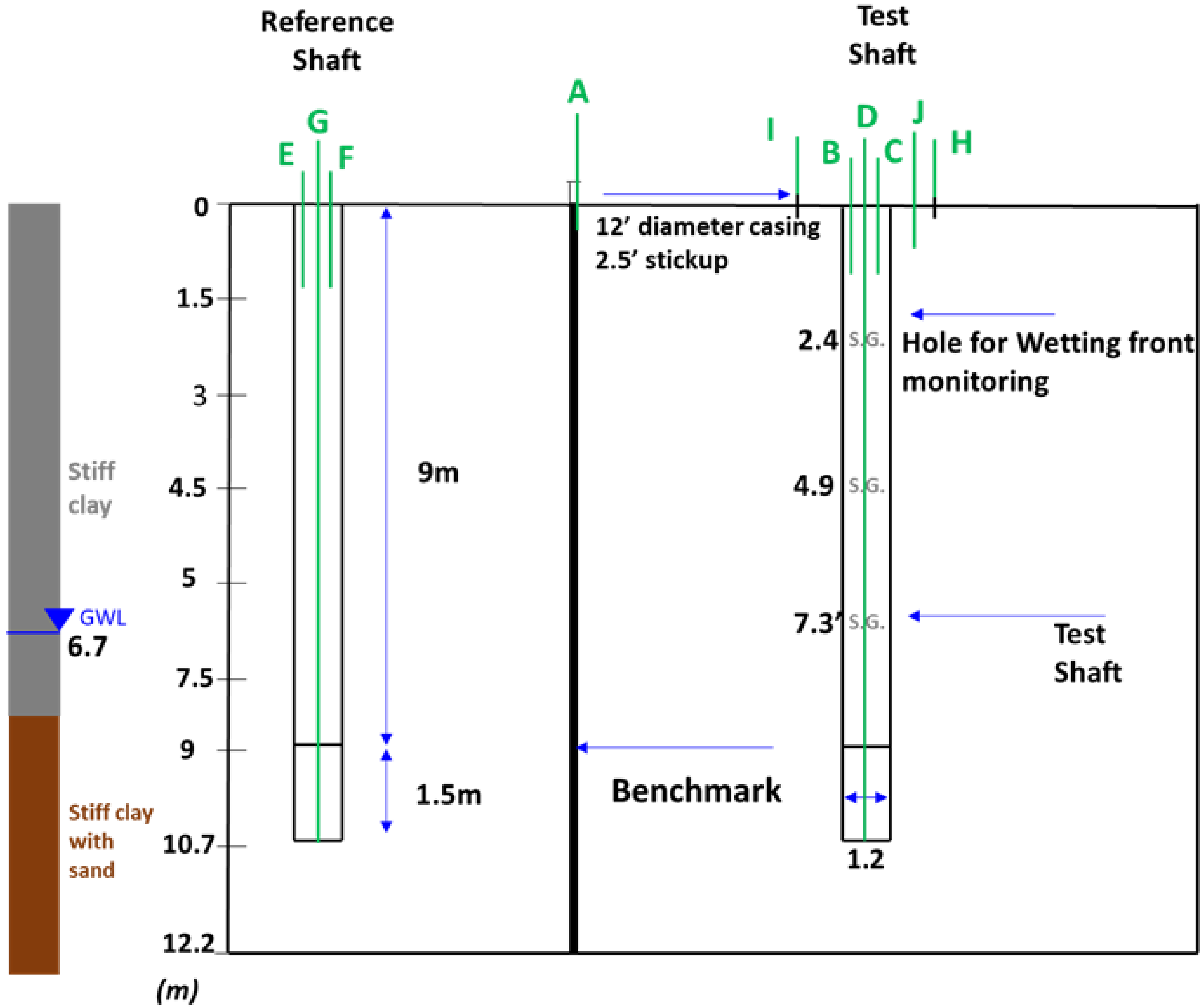
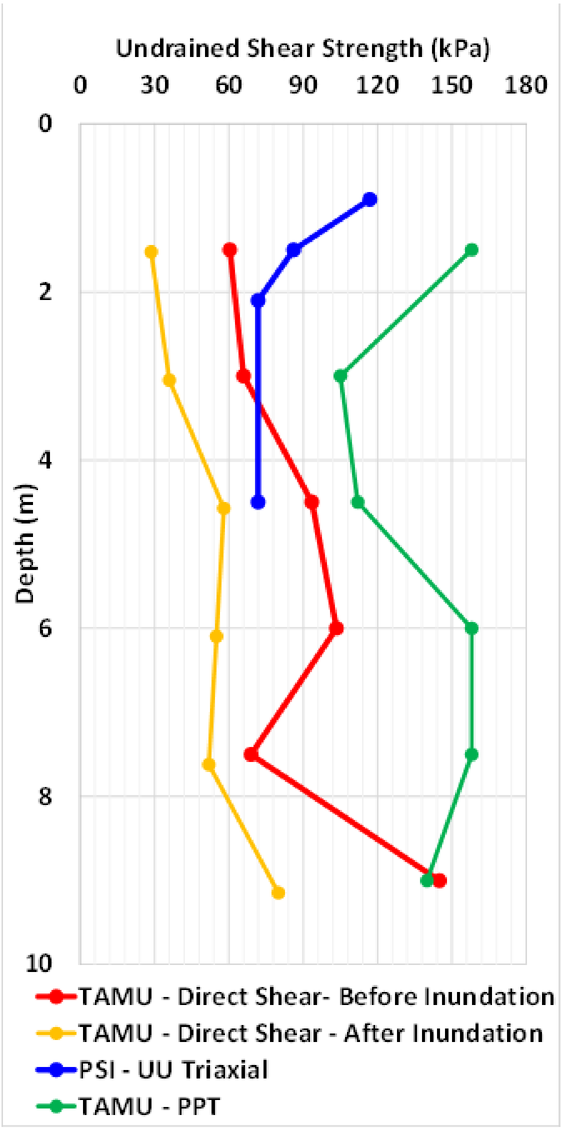
CASE HISTORY – CERGEP - 2023

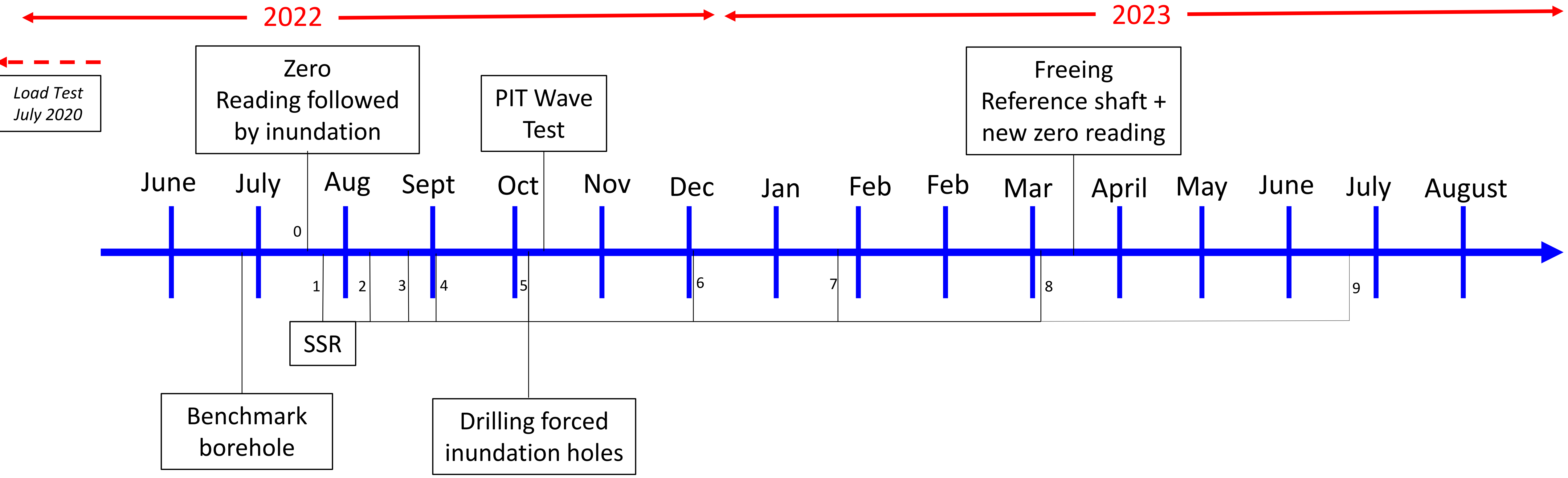
GRL BDLT – Reference Pile	RBM TLBT – Test Pile
	

CASE HISTORY – CERGEP - 2023





Sequence of Events



SSR – Surveying and Strain Readings									
SSR 0	SSR 1	SSR 2	SSR 3	SSR 4	SSR 5	SSR 6	SSR 7	SSR 8	SSR 9
Week 0	Week 1	Week 2	Week 4	Week 10	Week 11	Week 21	Week 27	Week 33	Week 48
July 20, 2022	July 27, 2022	August 3, 2022	August 14,2022	September 1, 2022	October 7, 2022	December 1, 2022	January 25, 2023	March 9, 2023	June 20, 2023

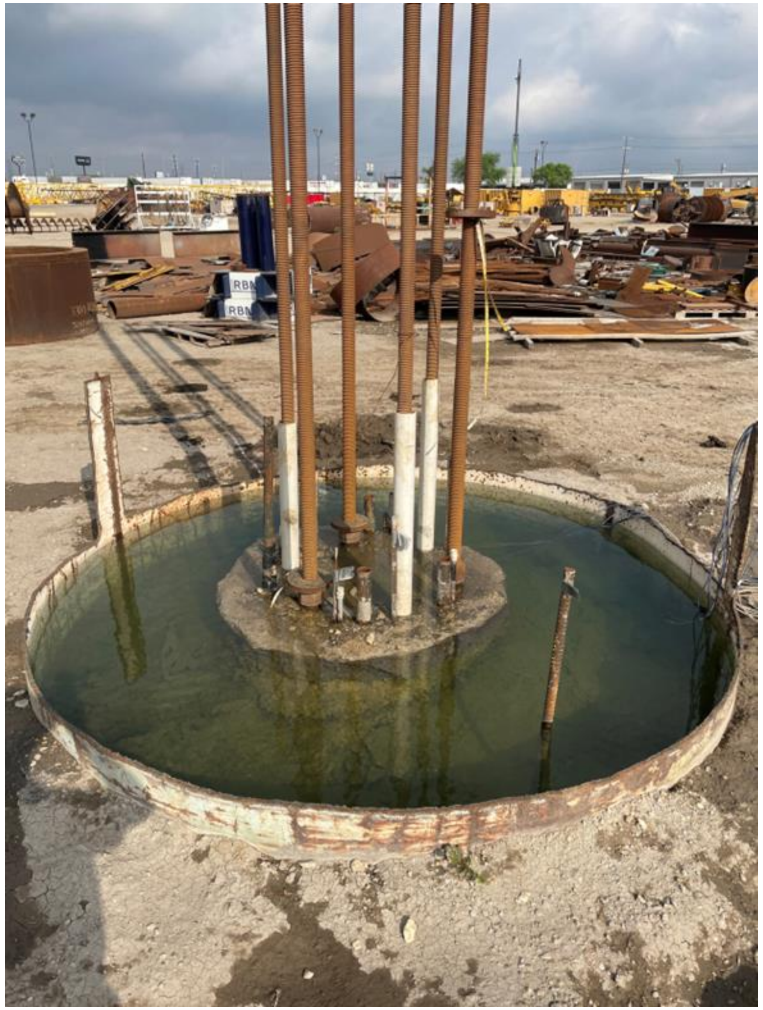
Zero Readings

Surveying

Testing Area Final set up

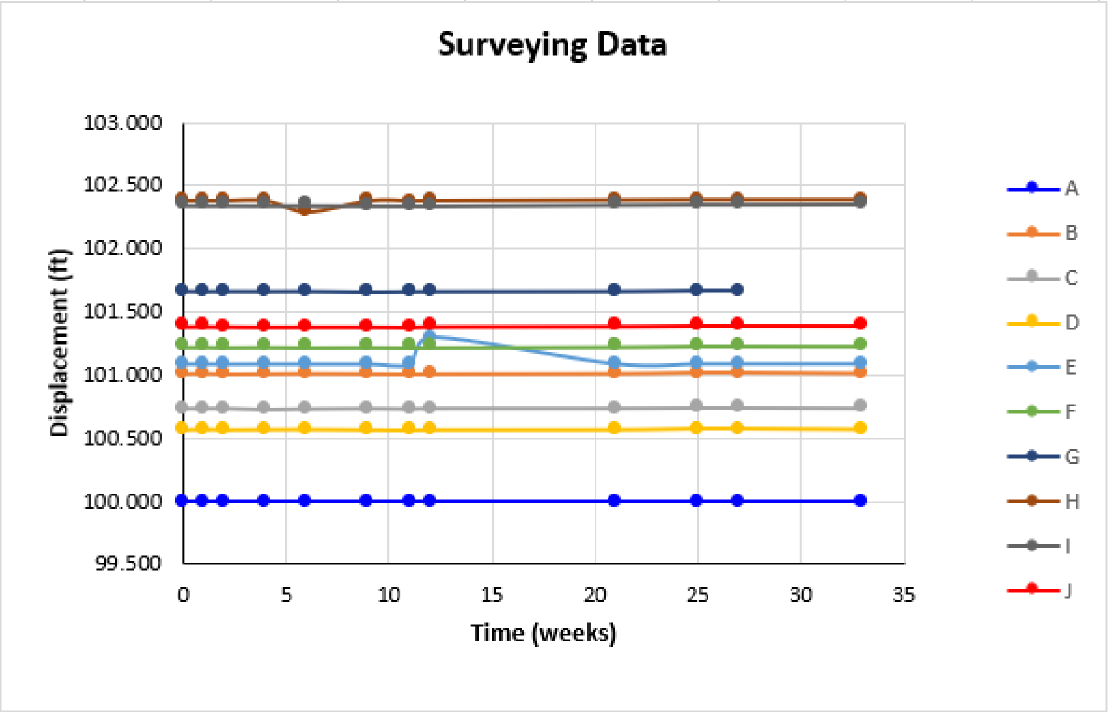
Inundation Set up

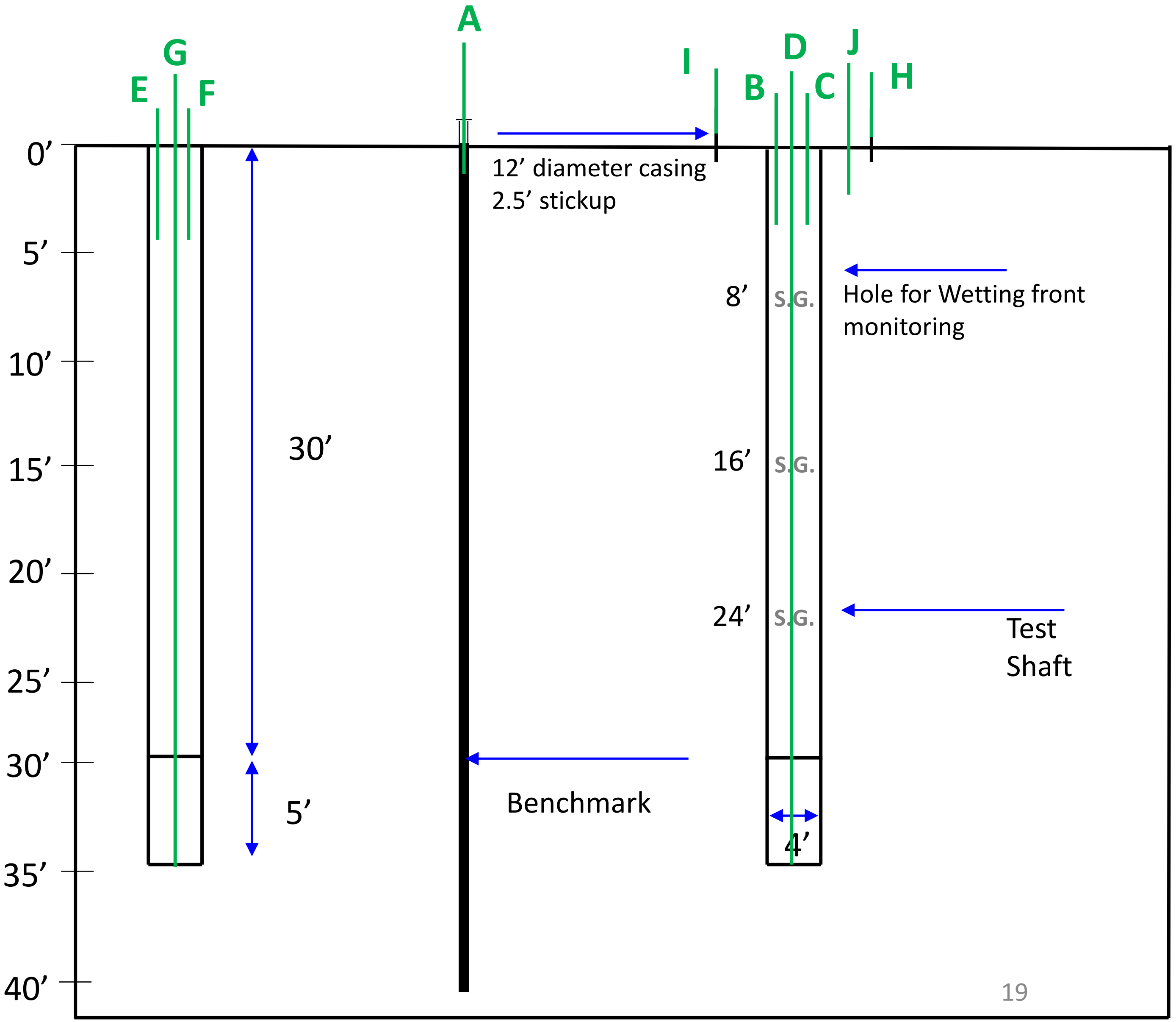
Test Shaft Pre-Inundation



Inundation

Surveying Data as of March 2023

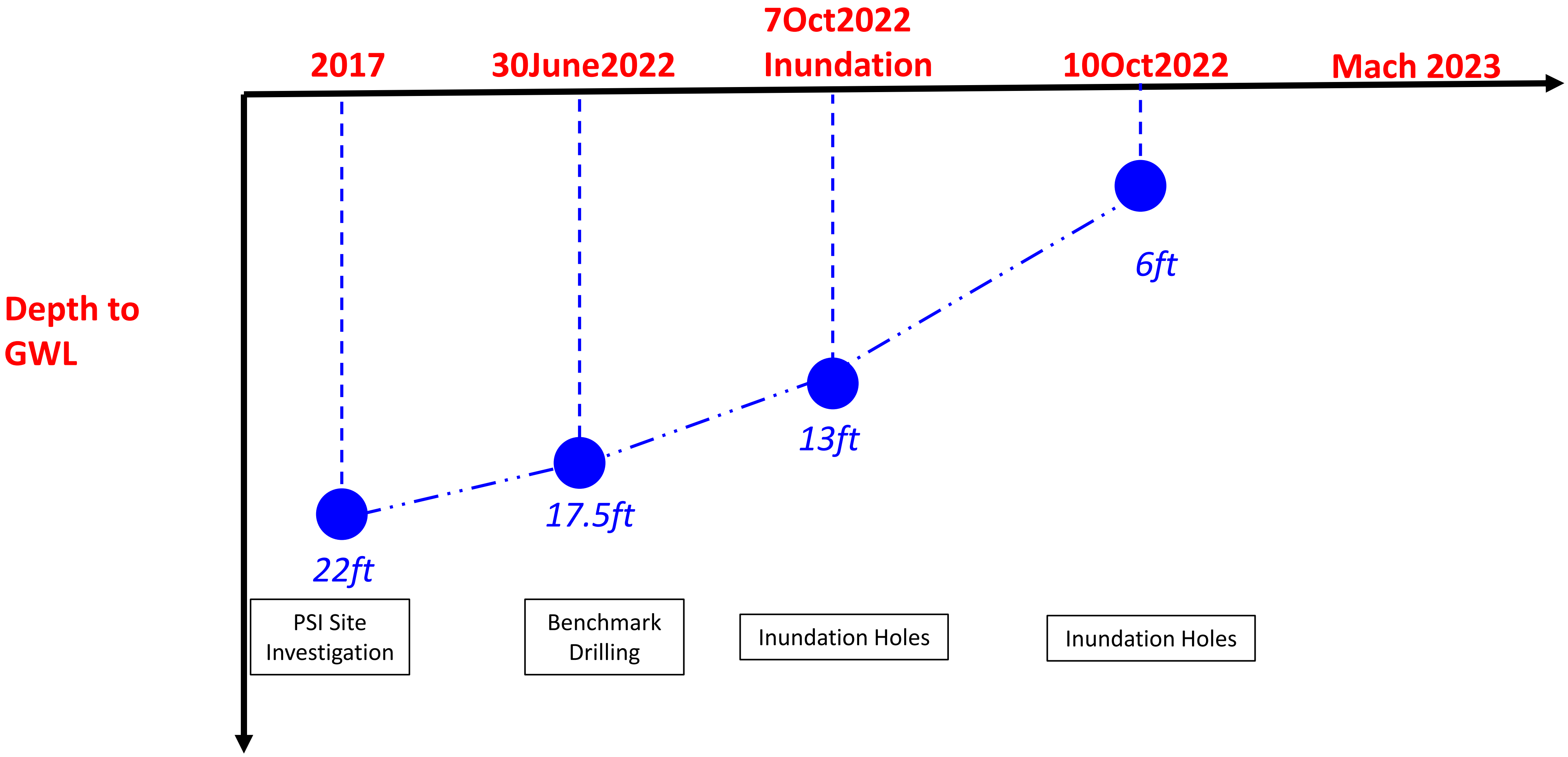




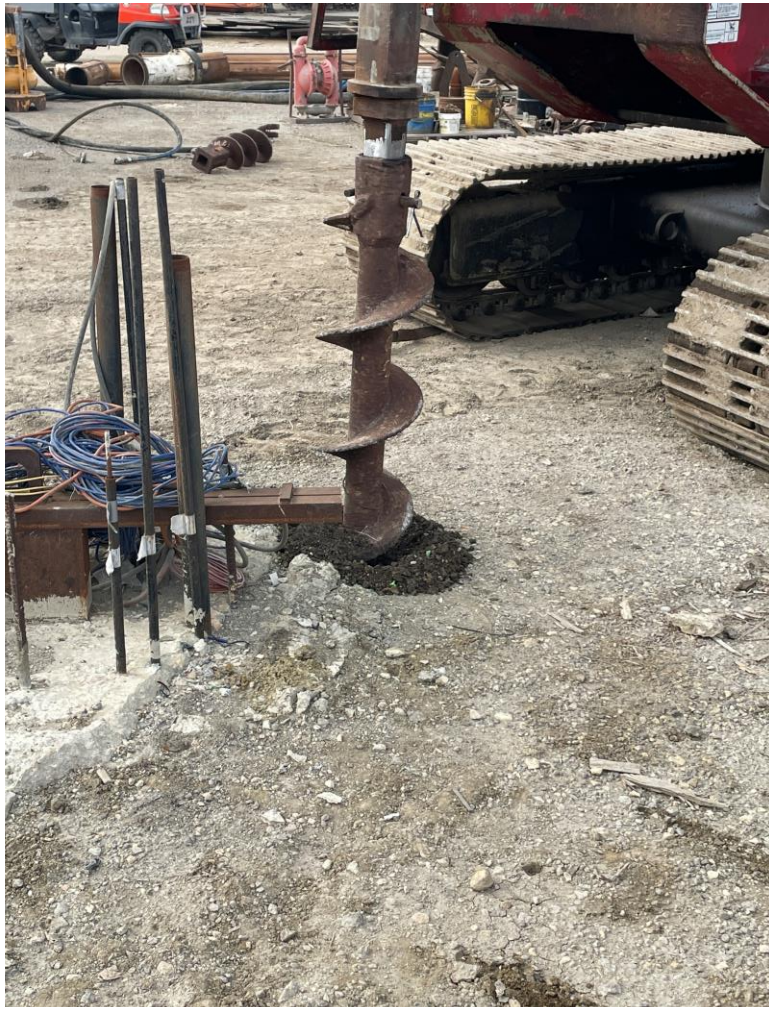
Surveying Reading spots:

A- Benchmark
B,C- Test Shaft Head
D- Test Shaft Bottom
E,F – Reference shaft head
G- Reference Shaft Bottom
H,I- Inundation Casing Head
J- Test Soil free swell

Ground Water Level Variations



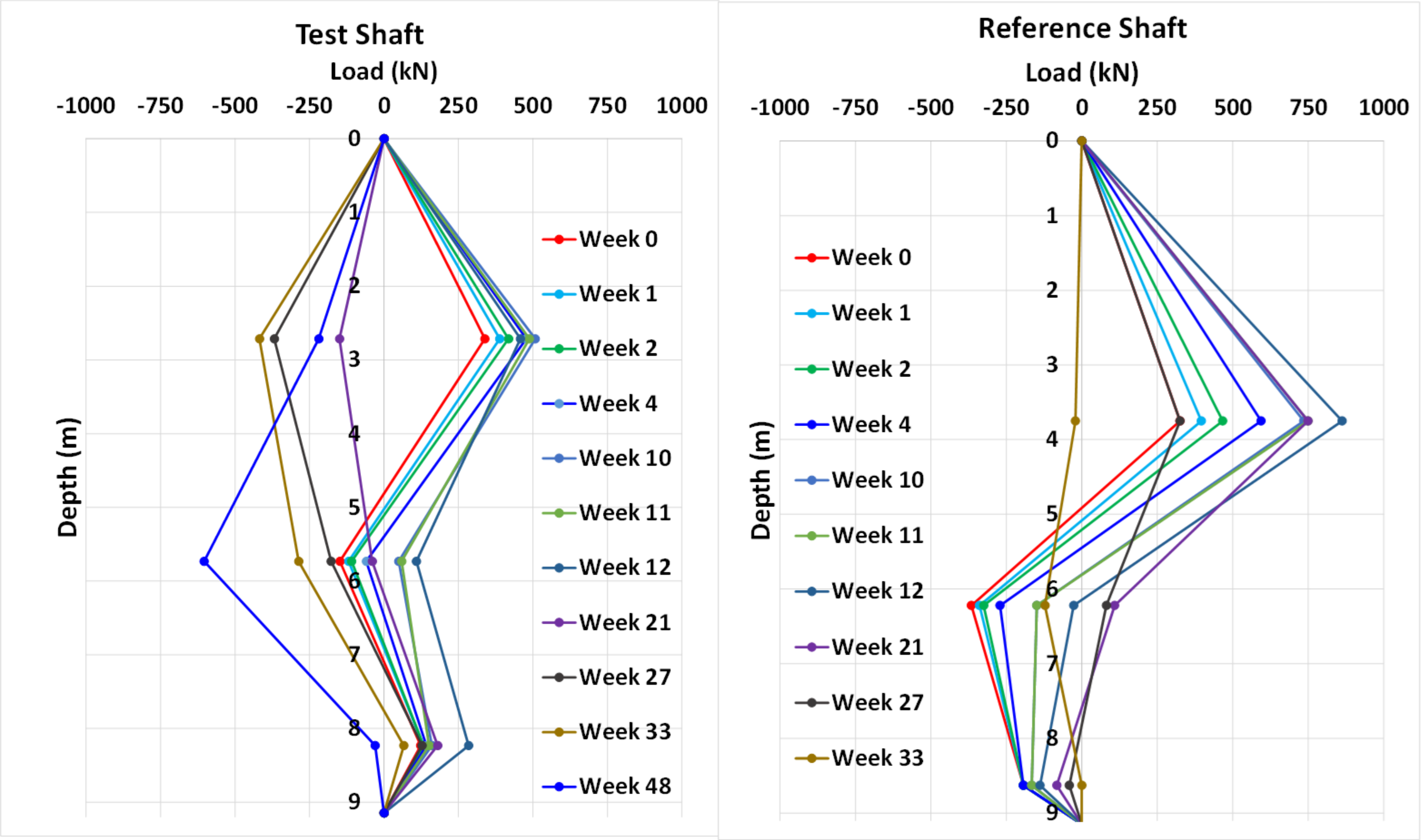
Freeing the Reference Shaft down to 21 ft



Freeing Reference Shaft

Free Reference Shaft

CASE HISTORY – CERGEP - 2023



Load Distribution in the Test & Reference shaft

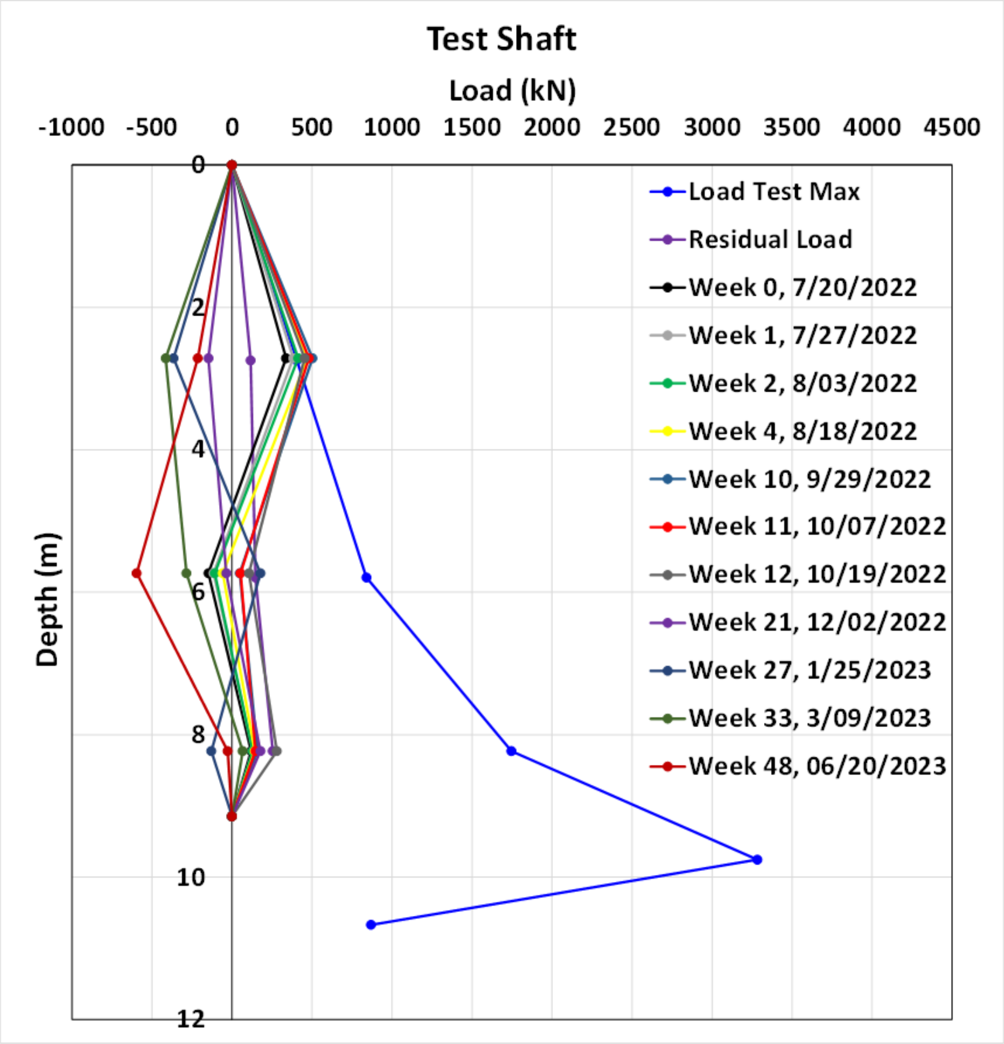
Load Test : 7/29/2020

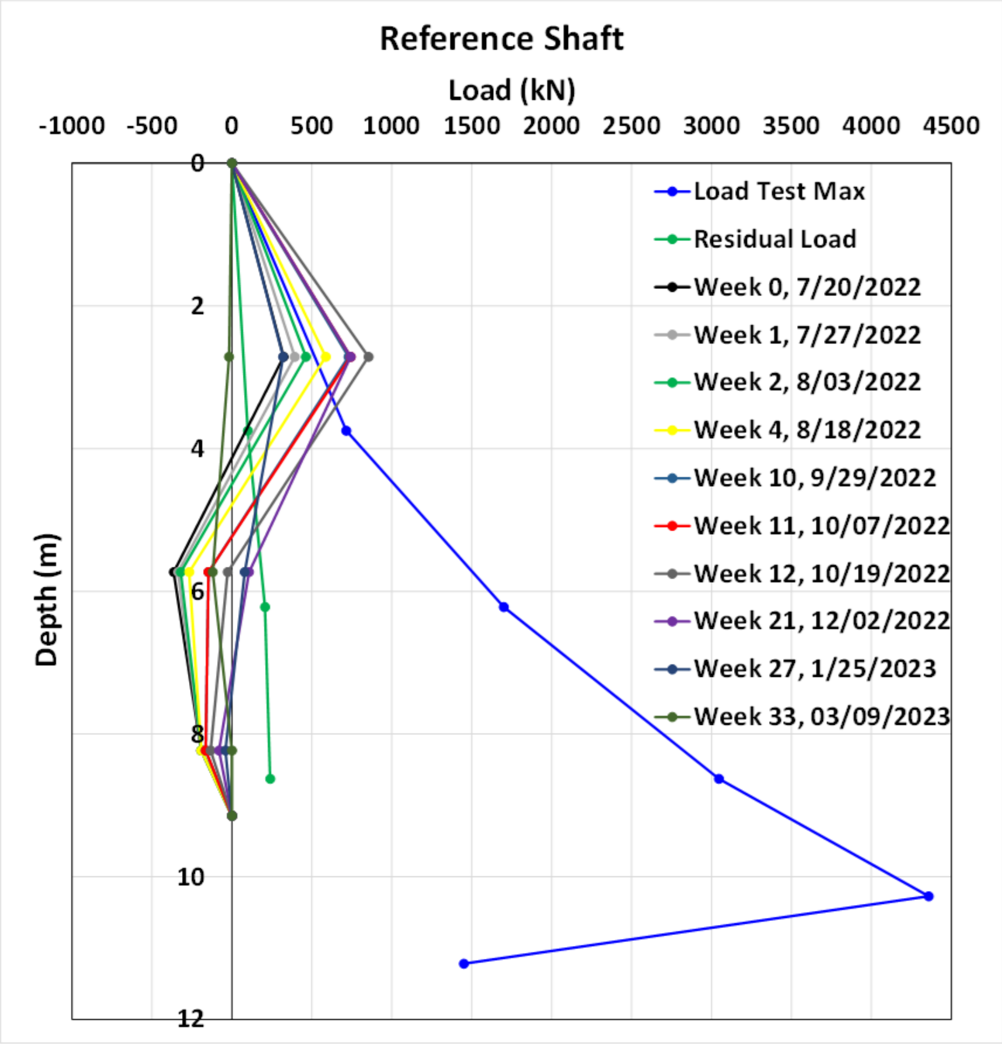
Week 0 : 7/20/2022

Week 33 : 3/09/2023

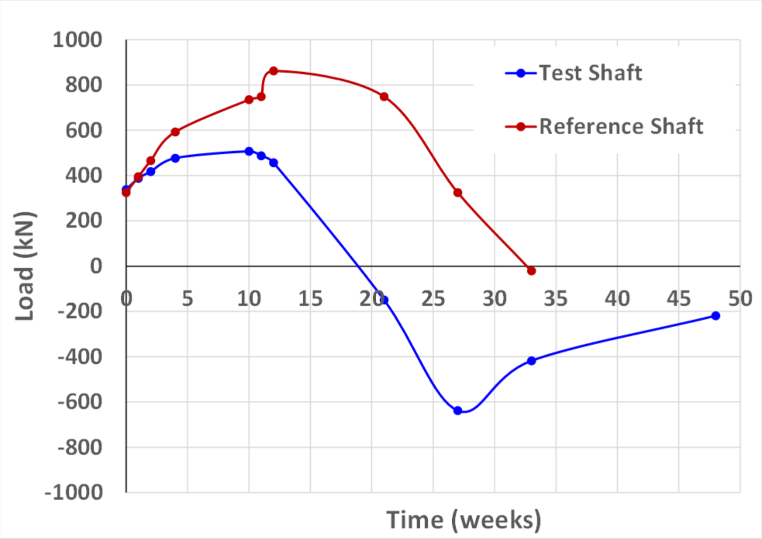
Week 48: 6/20/2023

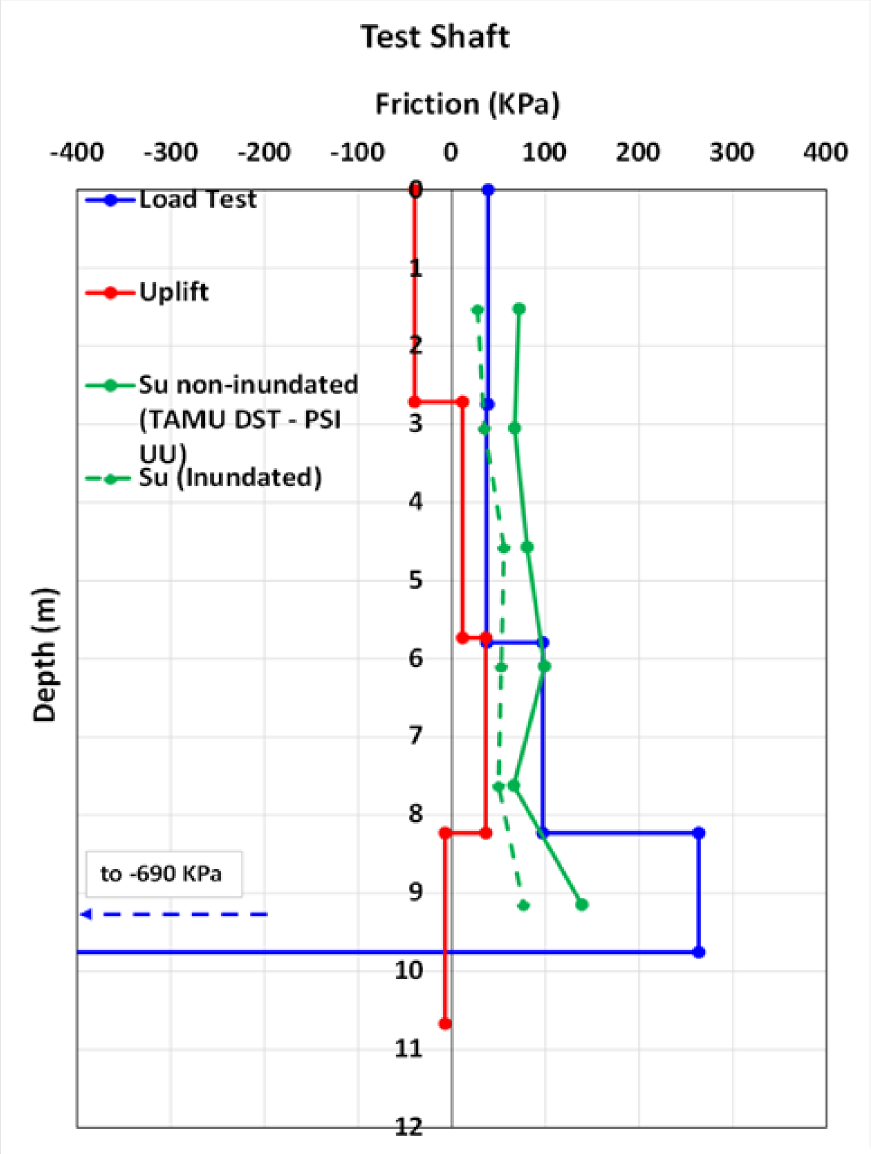
Excavation and New Zero Readings : 3/16/2023

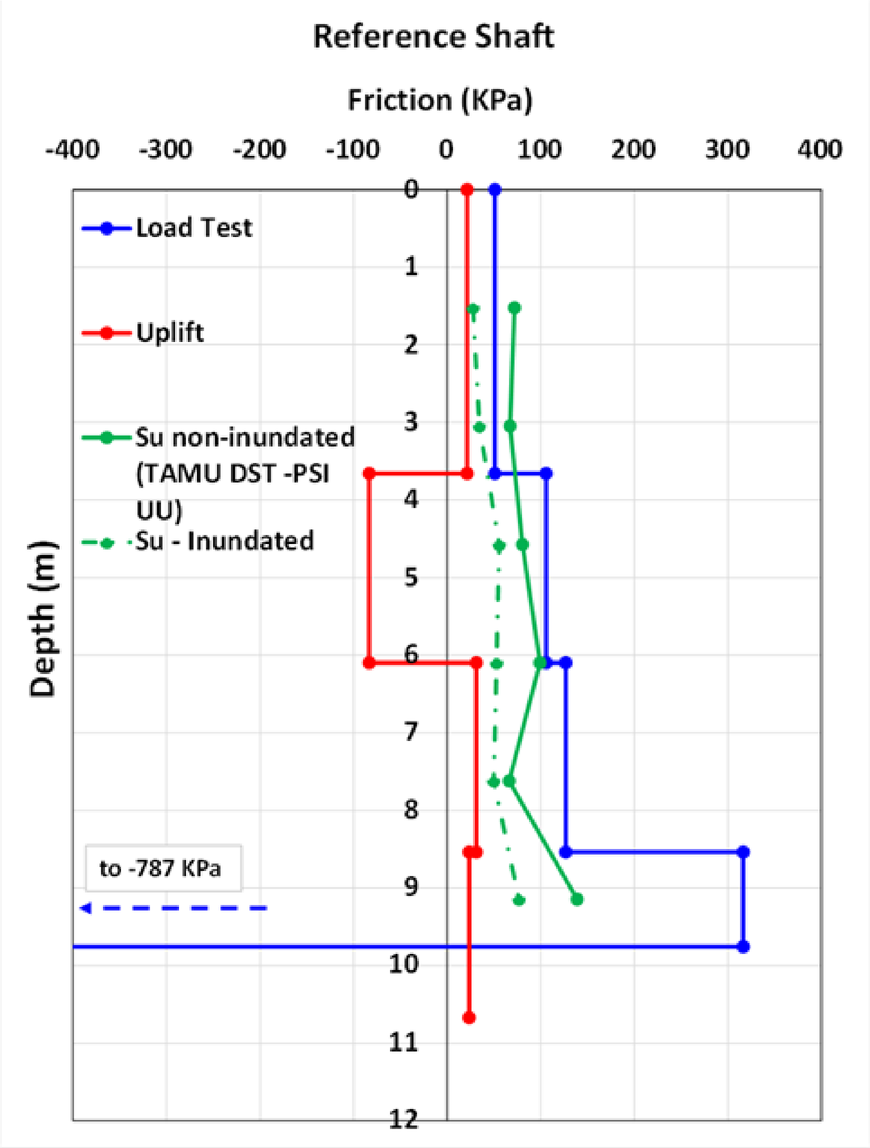




CASE HISTORY – CERGEP - 2023







CASE HISTORY – CERGEP - 2023

- Inundated s_u = 0.5 intact s_u
- Depth of active zone = 3 m
- $f_{max} = \alpha\ s_u$
  - Back-calculated swelling α = 0.51 when using the intact s_u value
  - Back calculated swelling α = 0.89 when using the inundated s_u value

CONCLUSIONS

- $\alpha = 0.5$ for swelling uplift friction when using the not-inundated undrained shear strength
- $\alpha = 1.0$ for swelling uplift friction when using the undrained shear strength where the sample is inundated for 24 hours before shearing
- $\alpha = 1.0$ for shrinking downdrag friction when using the not-inundated undrained shear strength
- β effective stress approach not reliable

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Upcoming events for you

March 21, 2024

TRB Webinar: PFAS Source Differentiation at Airports

June 23-26, 2024

2nd International Roadside Safety Conference

https://www.nationalacademies.org/trb/events

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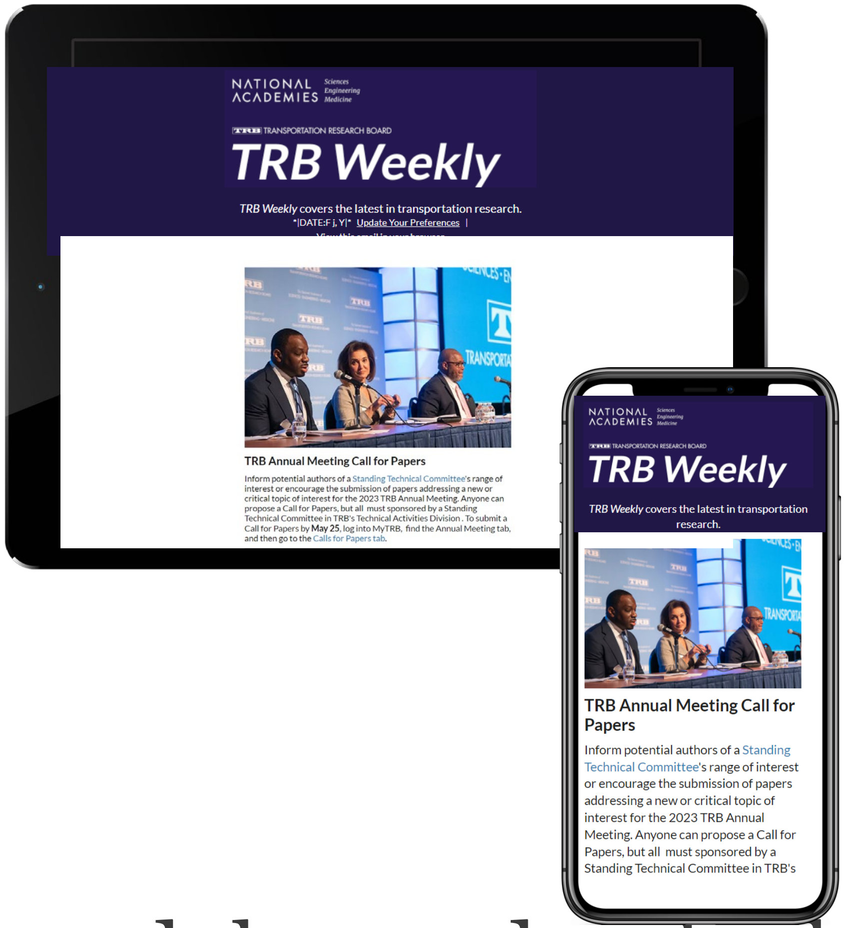

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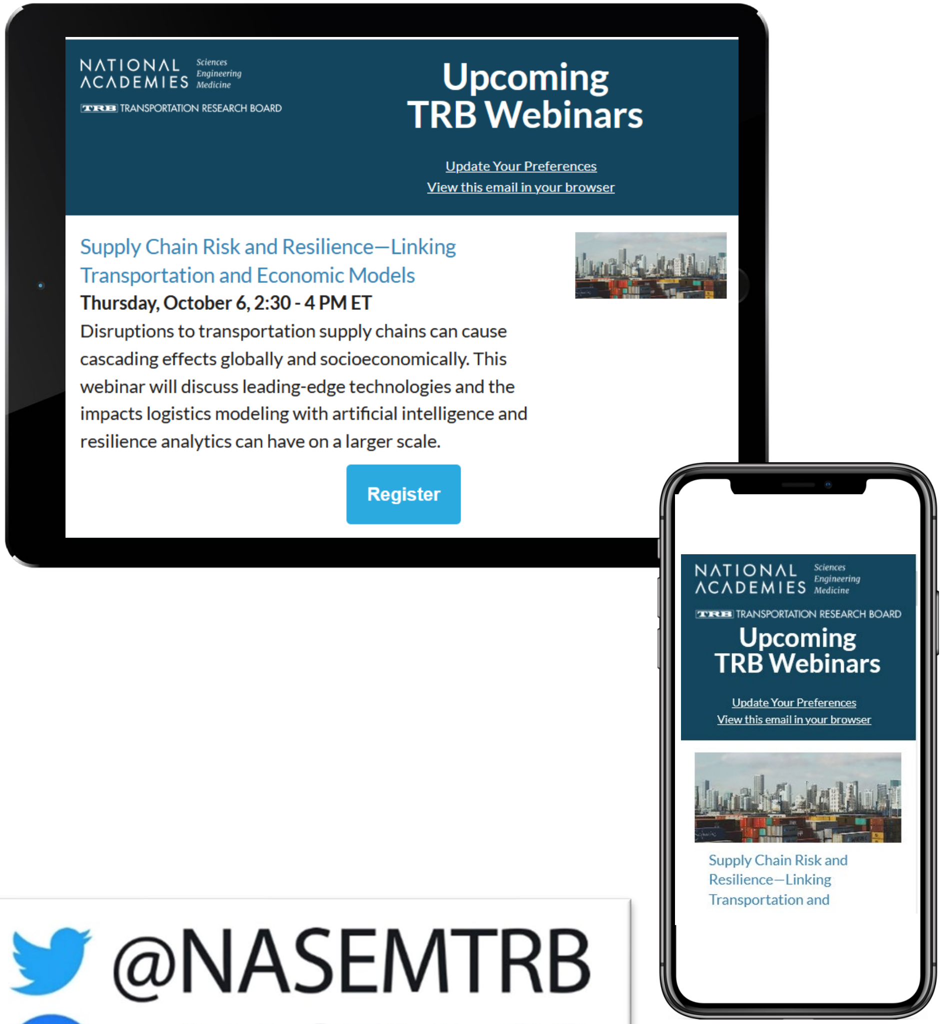

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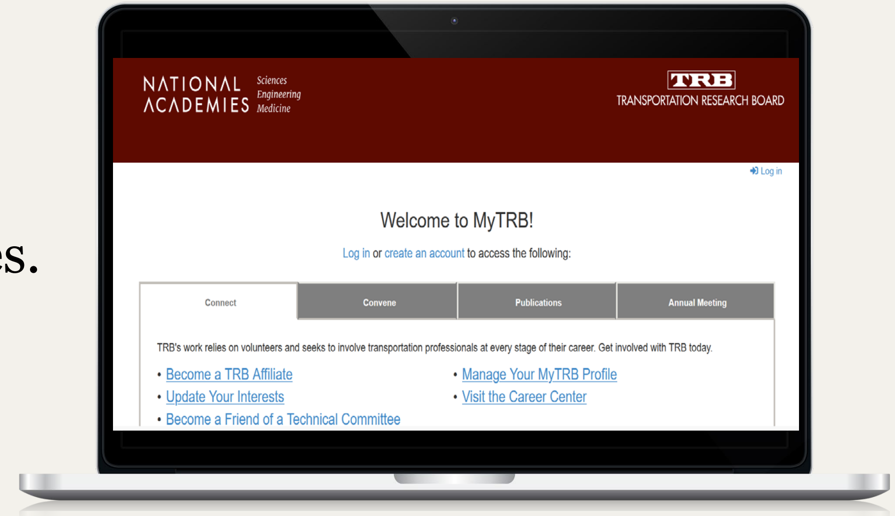

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