CELEBRATING 50 DFI YEARS

50th Annual Conference on Deep Foundations

Nashville

October 20 - 23, 2025



2025

PRESENTATION PROCEEDINGS

OPENING PLENARY

Gold Medal Winners

Foundation Optimization Through Prediction and Performance Monitoring

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Senior Vice President
GEI Consultants., Inc.
Woburn, MA

The opinions expressed in this presentation and on the following slides are solely those of the presenter(s) and not necessarily those of DFI. DFI does not guarantee the accuracy or reliability of the information nor endorse the recommendations offered by the presenter(s).

GEI Consultants

Outline

- Risk Reduction and Observational Method
- Deep Foundations on Hardpan in Midwest
- Epic Earth Retention System

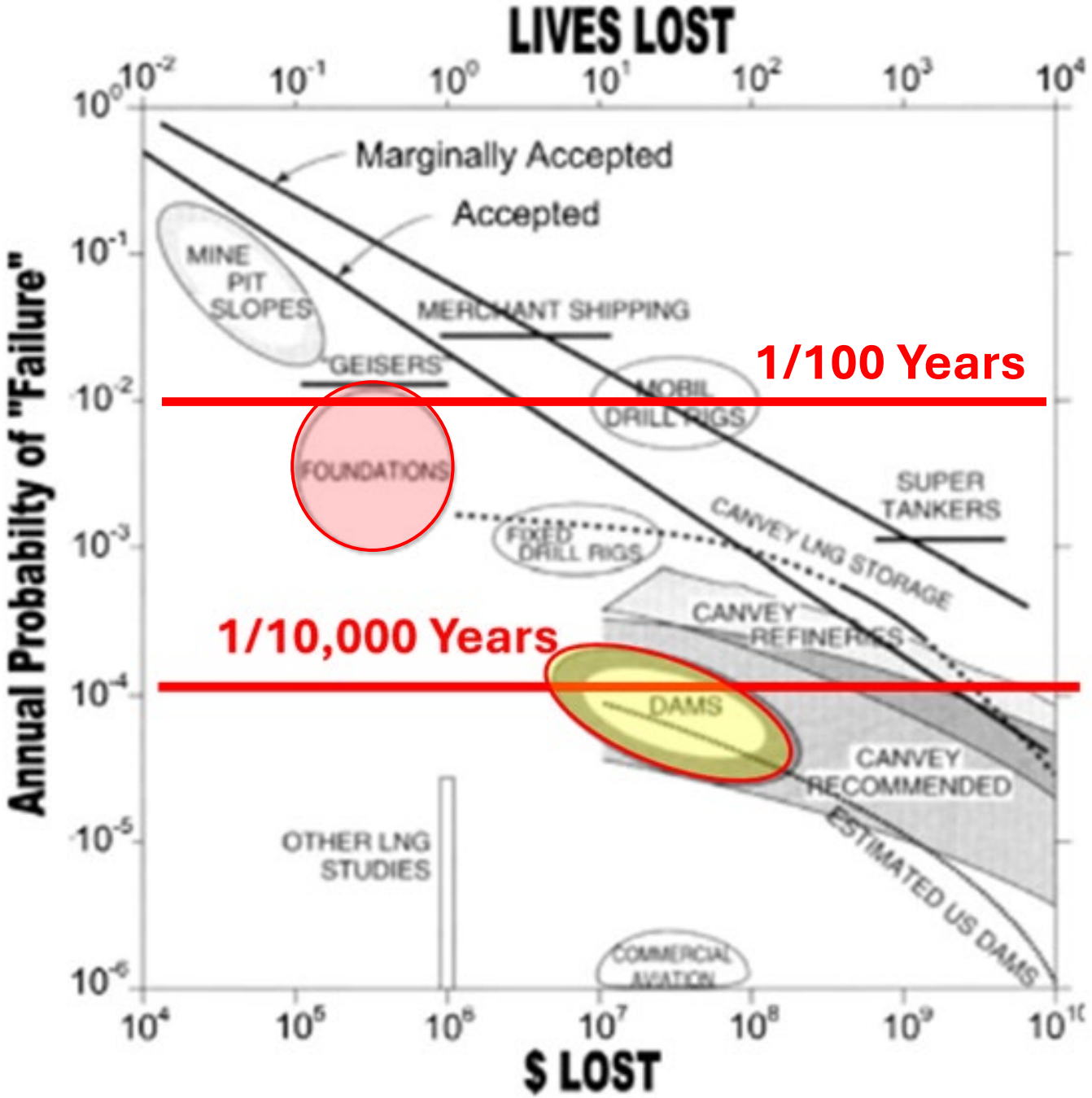
Risk Management for Subsurface Conditions
Hatem, 1998

1.0.3 ASFE Experience

No group of design professionals has demonstrated more success in minimizing risk exposures than geotechnical engineers. In 1970, so many of these engineers were being faced with meritless claims that professional liability insurance was all but unavailable to them as a profession. By 1980, 10 years after the formation of the Association of Soil and Foundation Engineers (ASFE), members had the best professional liability record of all design professionals.

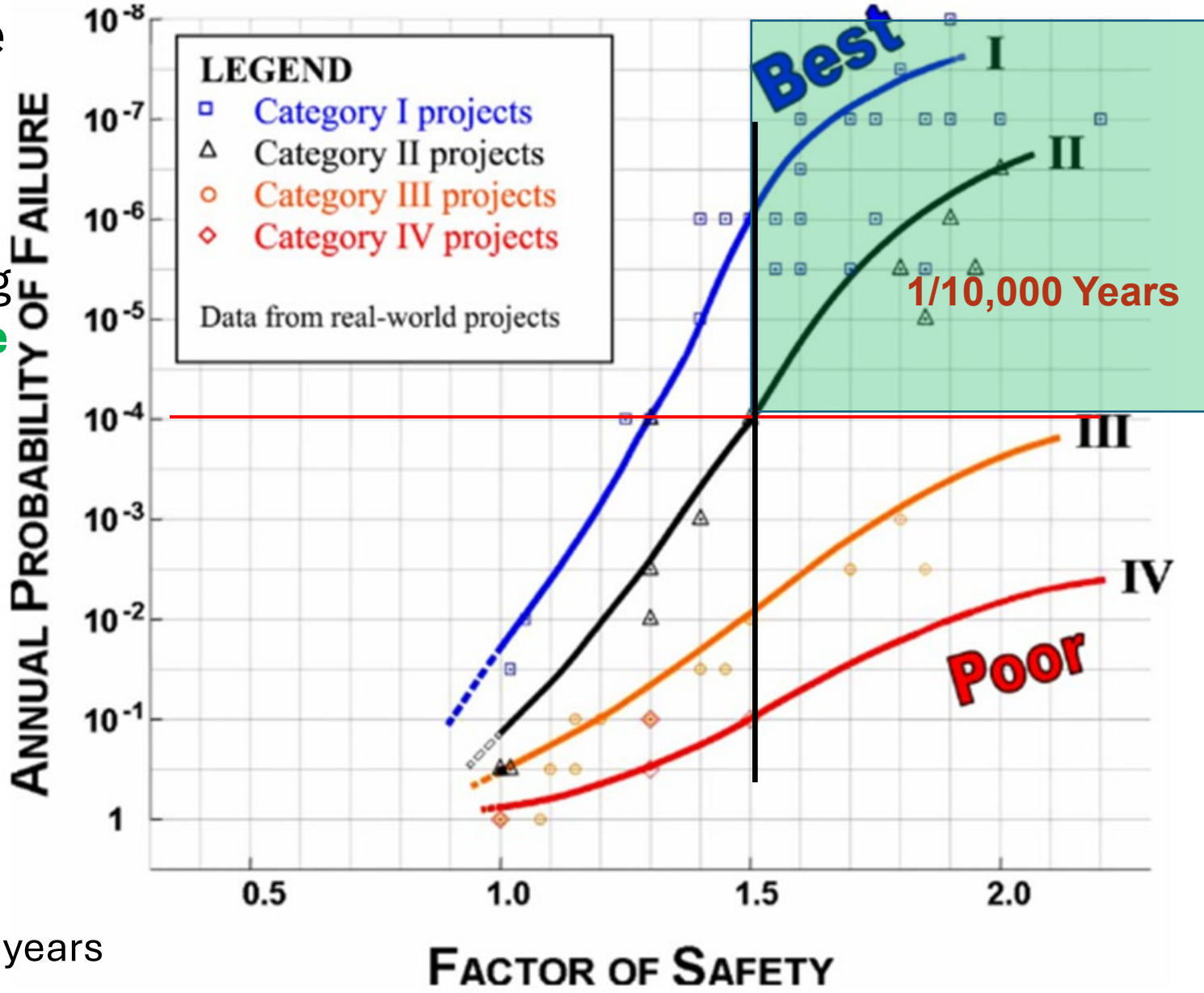
Risk vs Cost Comparisons
Silva et al. (2008)

Historically, foundations are riskier than dams.



Risk Informed Decision Making – Silva et al., 2008

1. **Category I** –state-of-the-practice engineering. Generally, these facilities have high failure consequences; **Best Practice**
2. **Category II** –standard engineering practice. **Above Average Practice**
3. **Category III** – facilities without site-specific design and substandard construction or operation. **Average Practice**
4. **Category IV** –little or no engineering; **Poor Practice**



75 projects over 40 years

Industry Has Generally Accepted Certain Risk-Reduction Guidelines: Factors of Safety and Movement

Safety Factors:

$$\mathbf{FS = \frac{\sum F\ resisting\ forces}{\sum F\ driving\ forces}}$$

- Static conditions – FS ≥ 1.5
- Flood conditions – FS ≥ 1.4
- Post Earthquake - FS ≥ 1.1 to 1.2
- Foundation bearing capacity - FS of 2.0 to 3.0

Movements:

- Foundation Settlement – Allowable 1 to 2 inches
- Differential Settlement – Maximum Distortion $\Delta s/L < 600$ frames with diagonals, and $\Delta s/L < 500$ for buildings
- Lateral Drift - ¼ to ½ inches at foundation level

Pre-Construction Prediction

Field Testing for Strength and Modulus

Good Parameters Make for Better Predictions

For larger projects, do the expensive testing:

Pressuremeter
Dilatometer
Vane Shear
Seismic Cone

Triaxial and DSS
Consolidation

FLAC
PLAXIS

Field Exploration

- SPT Borings and HQ Coring
- sCPTu Soundings
- Pressuremeter or Dilatometer Tests
- Field Vane Shear Testing
- Fixed Piston Tube Sampling



Laboratory Testing

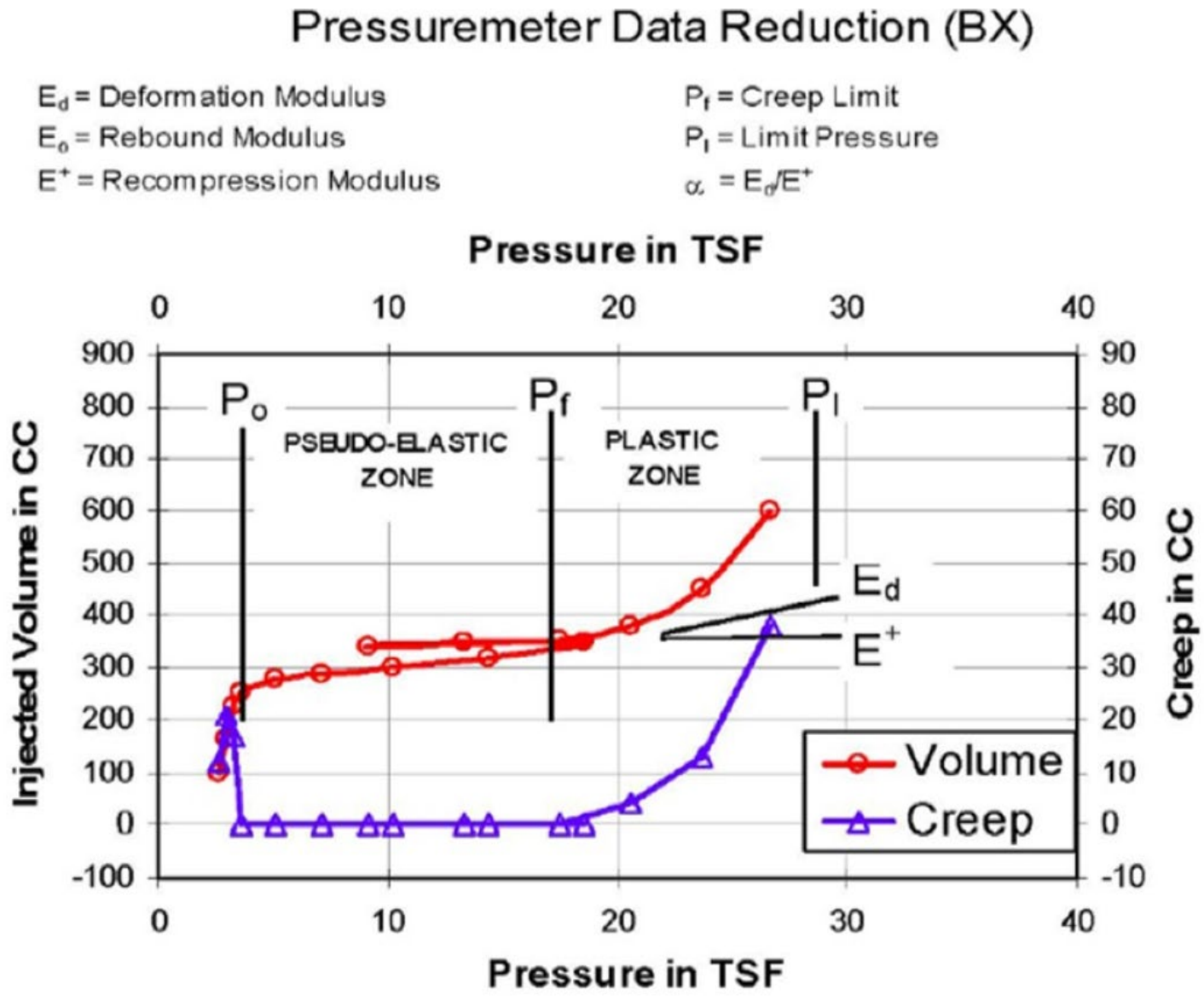
- Water Contents and Void Ratio
- Consolidation Testing
- Vertical X-ray of tube samples
- UU, CIU, CAU Triaxial and DSS Tests
- Plaxis or FLAC Model Development



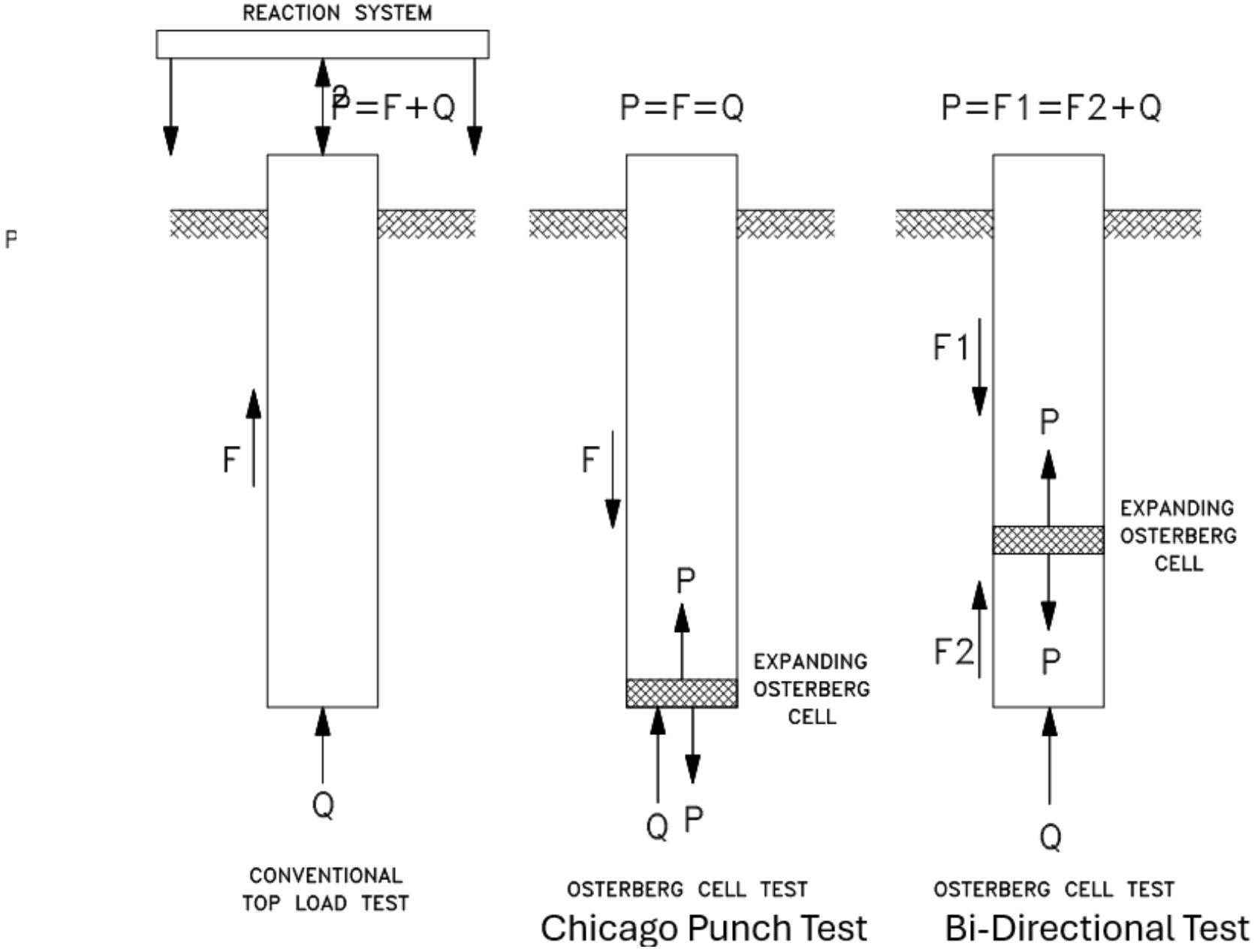
Pre-Construction Testing for Design - Pressuremeter



Baker, 2009 ASCE Terzaghi Lecture



Pre-Construction Testing for Design – Piles and Shafts



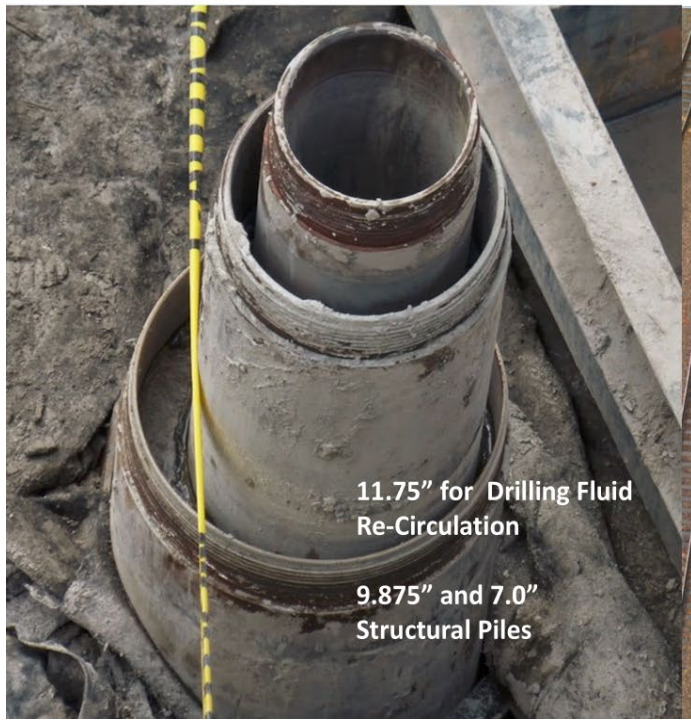
Pre-Construction Testing for Design – Static Testing

1,000 Kip Auger Cast Pile (ACP) Supporting Residential Tower



2,000 Kip Load Test on 30-inch ACP

600 Kip Micropile Supporting Podium that Spans Metra and Amtrak Rails into Union Station



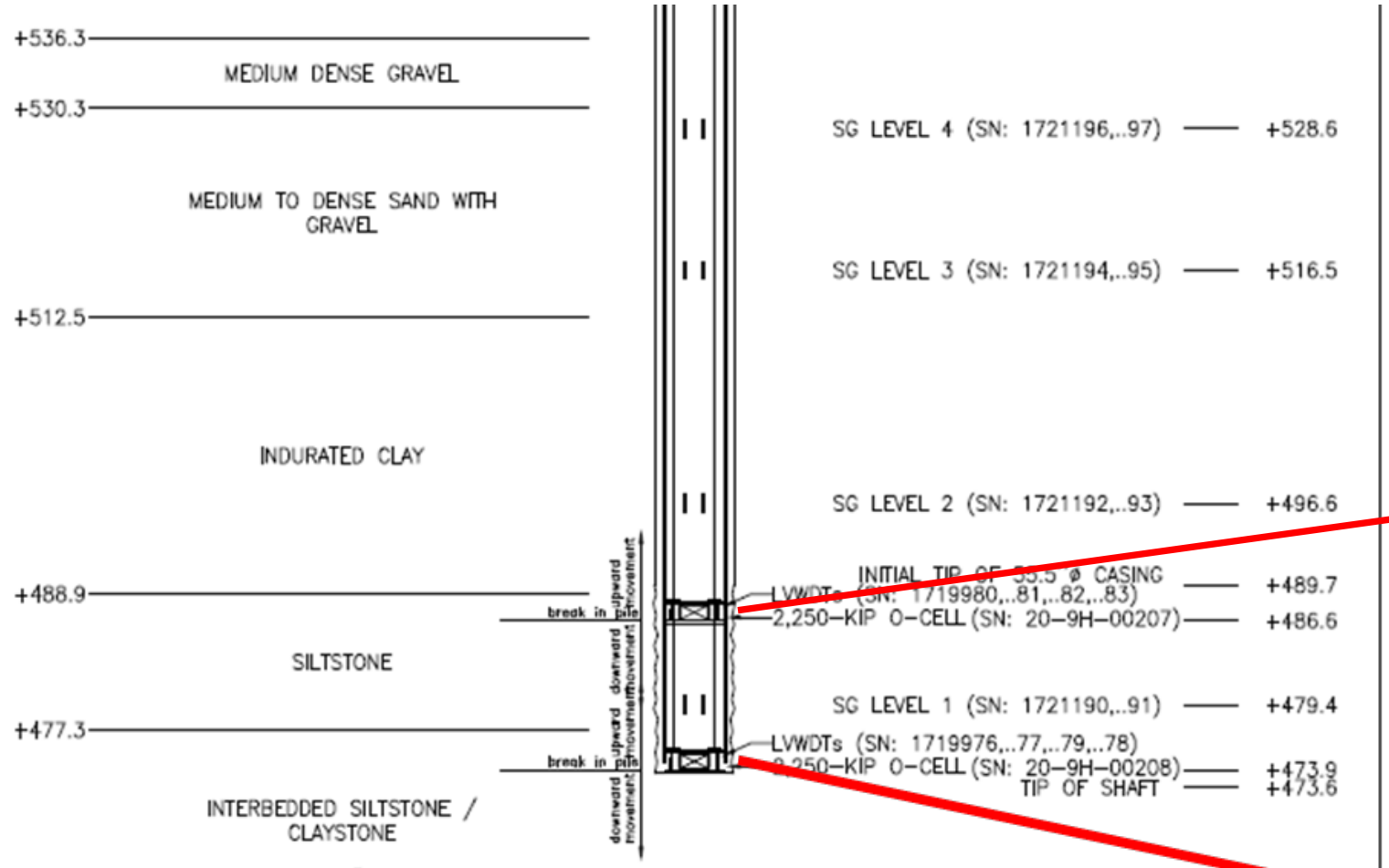






1,200-kip Load Test on 9.875-inch Micropile

Pre-Construction Testing for Design – Osterberg Tests



NOTE: SOIL BASED ON BORING # B-101-17 AND OBSERVATION DURING DRILLING

LOADTEST
www.LOADTEST.com
2631-D NW 41st St.
Gainesville, FL 32606
Phone: 800-368-1138
FAX: 352-378-3934

AS-BUILT SCHEMATIC SECTION OF TEST SHAFT TS-1
RACINE HYDROELECTRIC PROJECT – RACINE, OHIO

DWN BY: DJJ	DATE: 24 Jul 2017	CHECKED BY: WGR	LT-1775-1
REVISED BY: DJJ	DATE: 11 Aug 2017	SCALE: NTS	FIGURE A

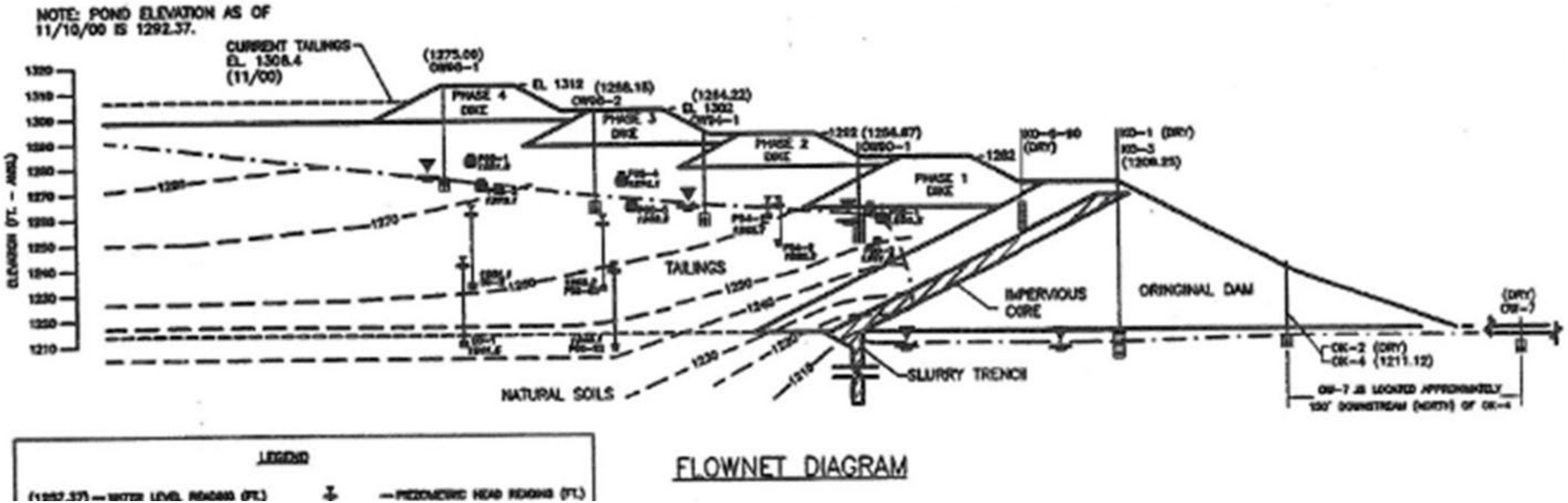


Peck's Observation Method (OM)

- Beginning in early 1960's, Dr. Ralph Peck, P.E., S.E. of the University of IL.
- Two fundamental approaches to reduce risk are:
  1. ***"Ab Inito"*** From the Beginning – OM is adopted at the beginning of the project
  2. ***"Best Way Out"*** OM is adopted after the project has been started and an unexpected event occurs
- Keys for OM success are:
  - Base the design on the most likely conditions and not the worst. So not unnecessarily conservative.
  - Monitor the structure during construction to compare to predicted performance.
  - Adjust the design or construction procedures as needed to meet performance criteria.

Observational Method – Piezometric Water Levels

- Observation Wells
- Vibrating Wire Piezometers – Nested at depths for downward gradients or artesian conditions

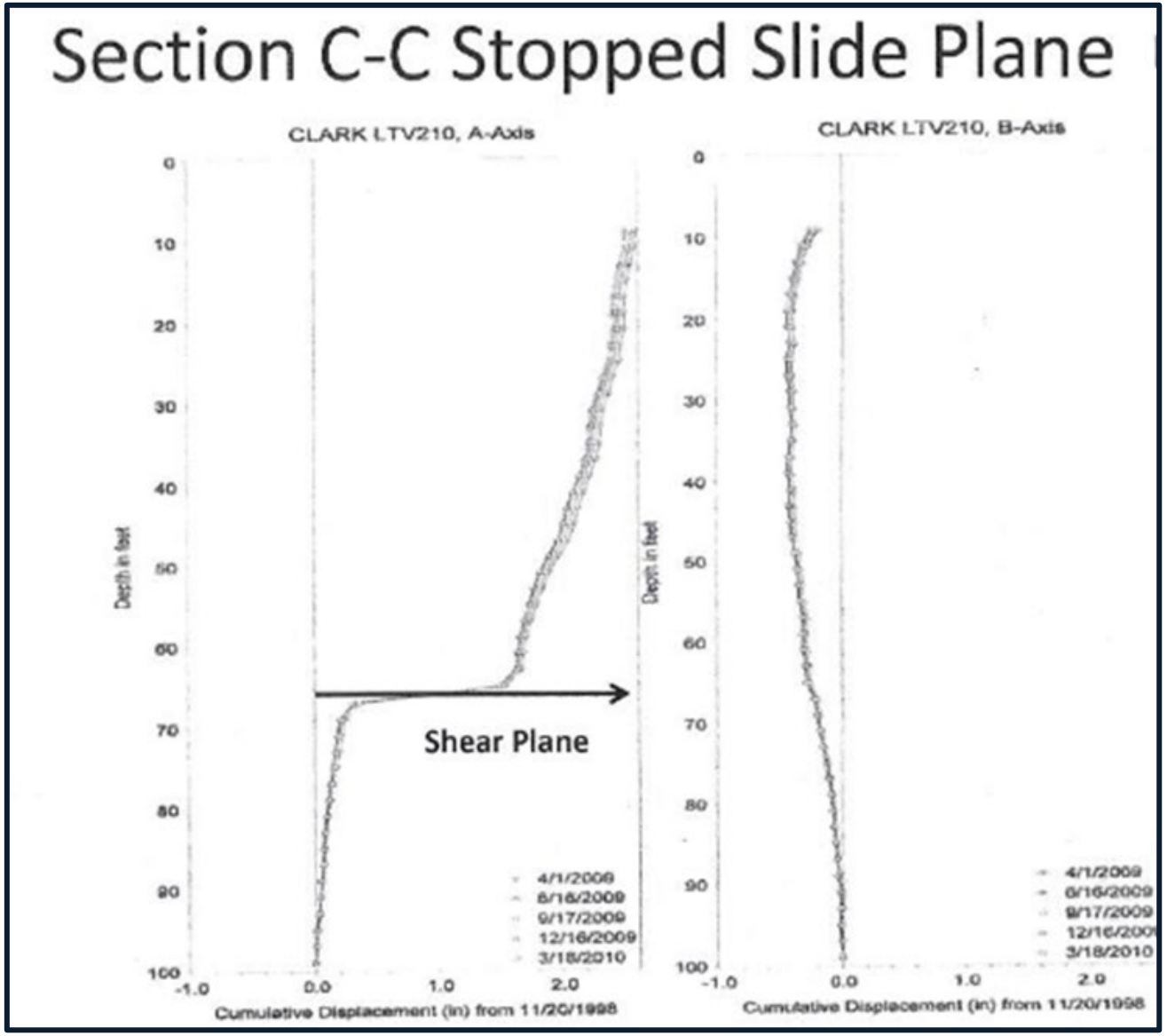


FLOWNET DIAGRAM

Walton et. al., 2002 USSD

Observational Method– Deformations

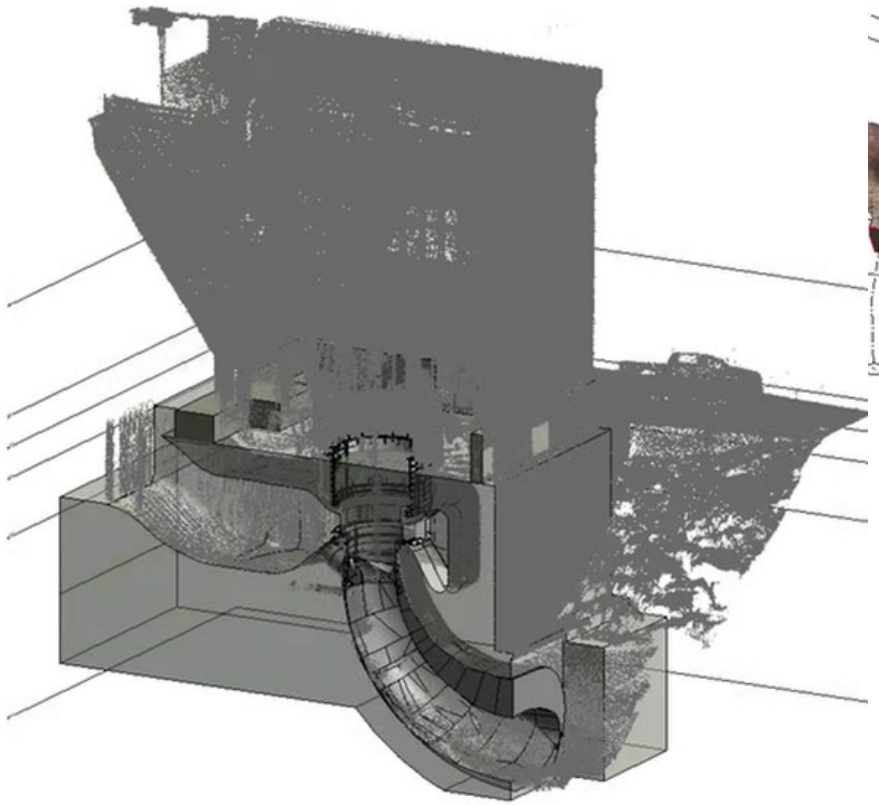
- Survey Points or Prisms for N, E, and El. Measurements
- Inclinometers for Lateral Movements at Depth
- TDR cables for Lateral and Vertical Movements
- SAA cables for Lateral and Vertical Movements
- Extensometers for settlement
- Settlement Plates

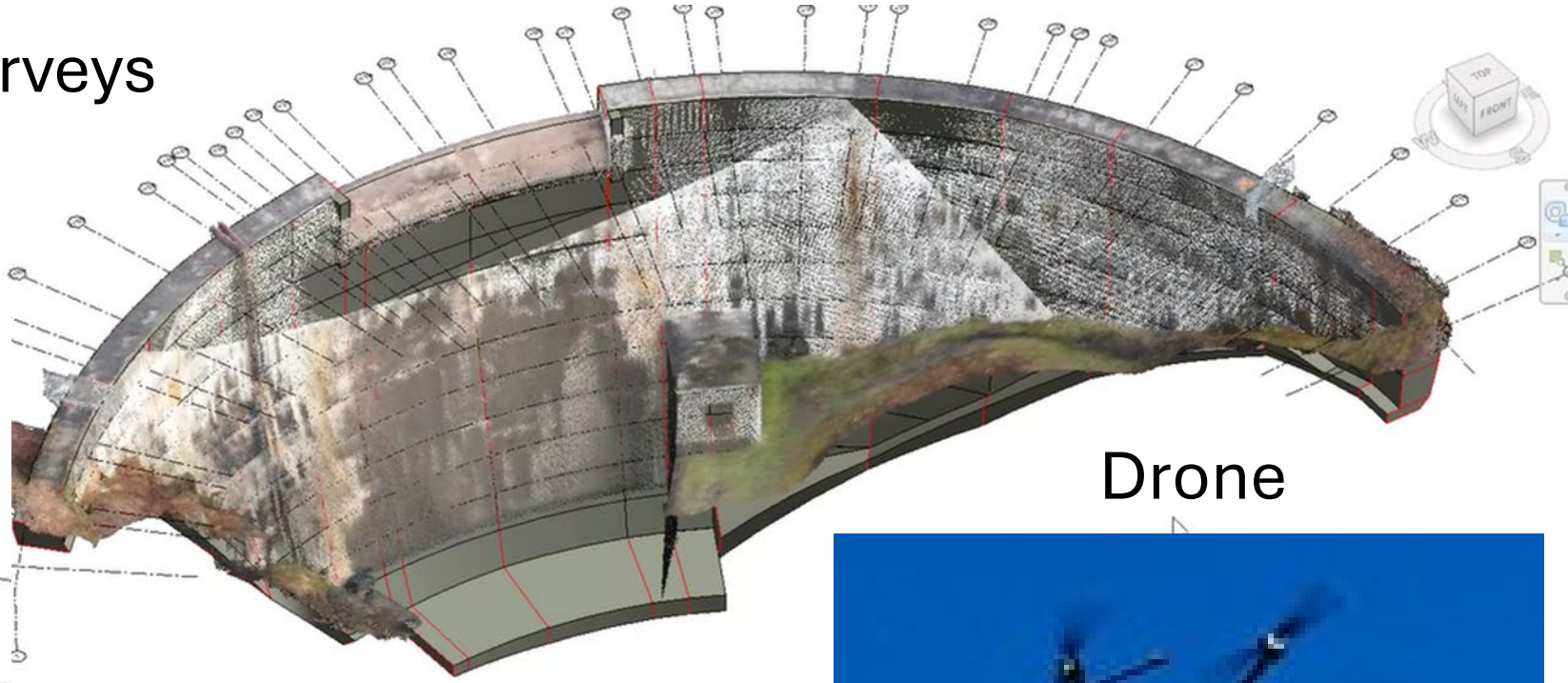


Walton & Baker (2013)

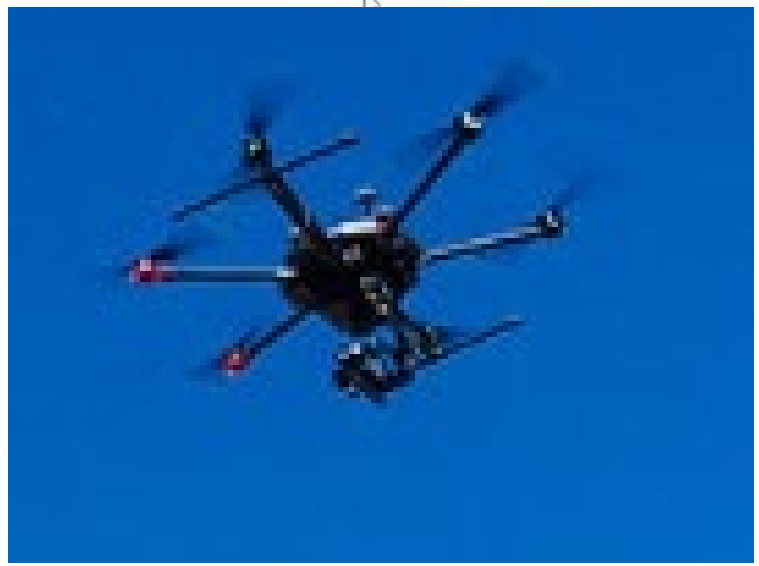
Observational Method – Cool Tools

- LiDAR Drone and ROV Surveys
- Satellite Imagery





Drone



Drone, Backpack Camera for Interior and ROV for Underwater

Walton 2025 Projects

Chicago Caisson Experience

We’re Standing on the Shoulders of Our Mentors

- **Cambridge, MA Geotechnical Legends**
  - Dr. Karl Terzaghi – Theoretical Soil Mechanics and Theory of 1-D Consolidation
  - Dr. Arthur Casagrande – Soil Liquefaction and Embankment Seepage
  - Dr. T. William Lambe – Geotechnical Educator and Risk Awareness
- **Chicago, IL Geotechnical Legends**
  - Dr. Ralph Peck (U of IL Bulletins) CTA Tunnel expert. He verified Terzaghi’s 1-D Consolidation Theory by back analyzing settlements 4 old Chicago Buildings
  - Clyde N. Baker, Jr. of STS and GEI - Tall Building Foundations and Mentor
  - Bob Lukas STS and GEC – Advanced the Pressuremeter and Deep Dynamic Compaction

Foundation Monitoring in Chicago and North America

- **Settlement Predictions Calculated using:**
  - Numerical methods using real soil properties (modulus and strength): Believable, avoids over-conservatism
  - Empirical methods using just index tests (N-value or Cone Tip Resistance): Overly conservative:
  - Quality in quality out. Spend the money for high quality testing and high-end analysis.
- **Performance Monitoring Challenge:**
  - Convince others that designs are acceptable based on performance of previous projects.
  - Convince owners and contractors to pay for performance monitoring of new projects.

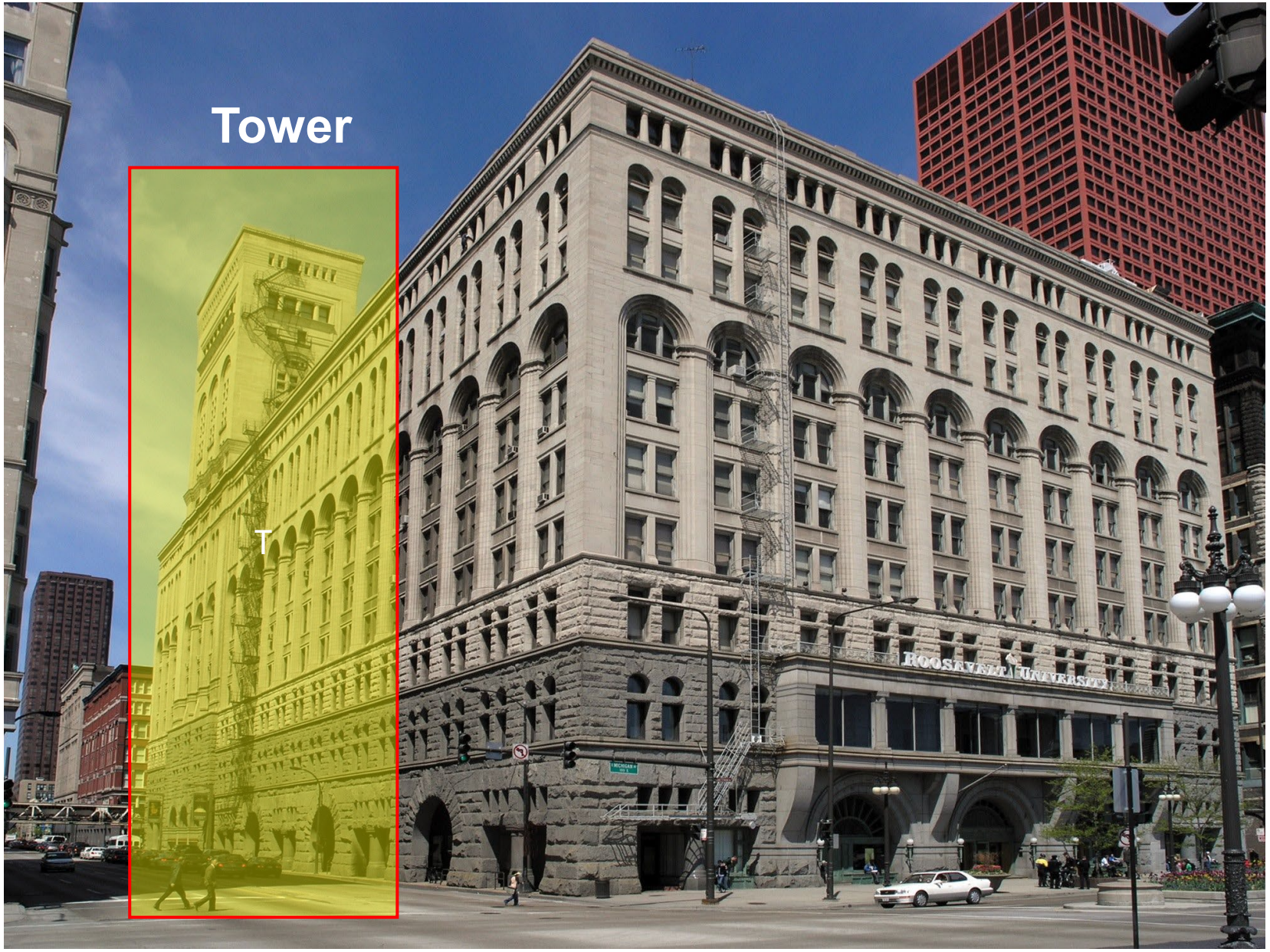
Chicago Building Settlement - 1883 to 1894 on Shallow Spread Footings

Shallow Spread Footing Building Settlements in Chicago (Peck, 1948)

Building	Date	Bearing Pressure	Settlements	Notes
Polk Street Station	1883	3,100 psf	8 inch within tower area and 1 inch for 2 story portion	170-foot tower & 2 story annex
Auditorium Theatre	**1887**	**4,100 to 4,500 psf**	**4 to 17.5 inches after 50 years**	**19 story tower, 10 story building**
Old Board of Trade	1889	5,680 to 8,800 psf	5 to 10 inches	Exterior Walls
Monadnock Block	1891	3,300 to 3,750 psf	21 inches (N side)	16 story Masonry
Old Colony Building	1894	2,500 psf	5 inches	17 story steel frame

1887 Chicago Auditorium Theater

Univ. IL Bulletin 429, Peck,1948



Tower settlement:
17 inches over 50 years

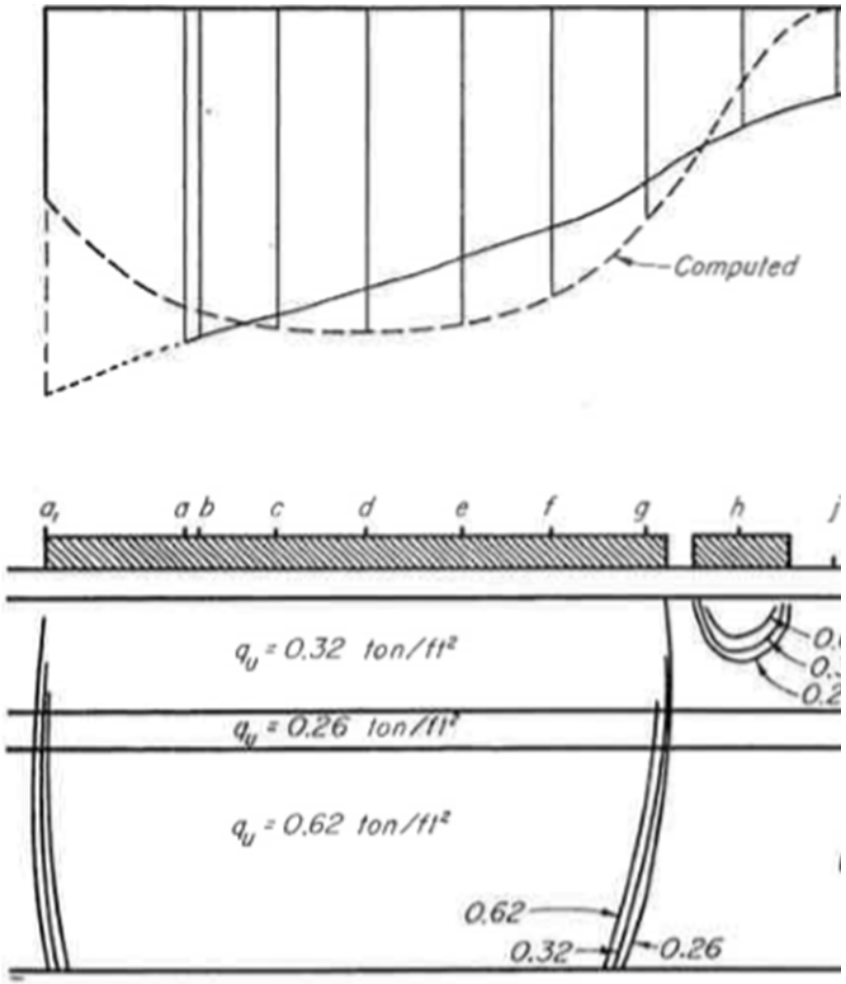


Fig. 10. Comparison for Chicago Auditorium
Pressure in Subsoil and Unco

Advances by Small Steps to Support Taller and Heavier Buildings in Chicago

Used deep foundations to Chicago hardpan to reduce building settlements.

Increased the allowable bearing on deep Chicago hardpan from 6 tsf to 30 tsf since 1920:

- D'Esposito Load Test (1920) at Union Station: 6 tsf
- Undisturbed tube samples and drained triaxial tests: > 9 to 10 tsf
- In-situ pressuremeter testing (since 1970): 10 to 30 tsf
- Settlement observations of constructed buildings (since 1920)

Quoting my mentor and dear friend Clyde Baker:

- *“Take small steps”*
- *“Assume and then Verify”*
- *“Enjoy your work”*



Load Testing to Justify Higher Bearing on Rock

- Static Pile Load Tests with Tension Piles and Kentledge
- Osterberg O-Cell Load Tests
- Dynamic Apple® - Pile Dynamic Analysis (PDA-CAPWAP)
- Statnamic® Testing

1 North Wacker, 1999



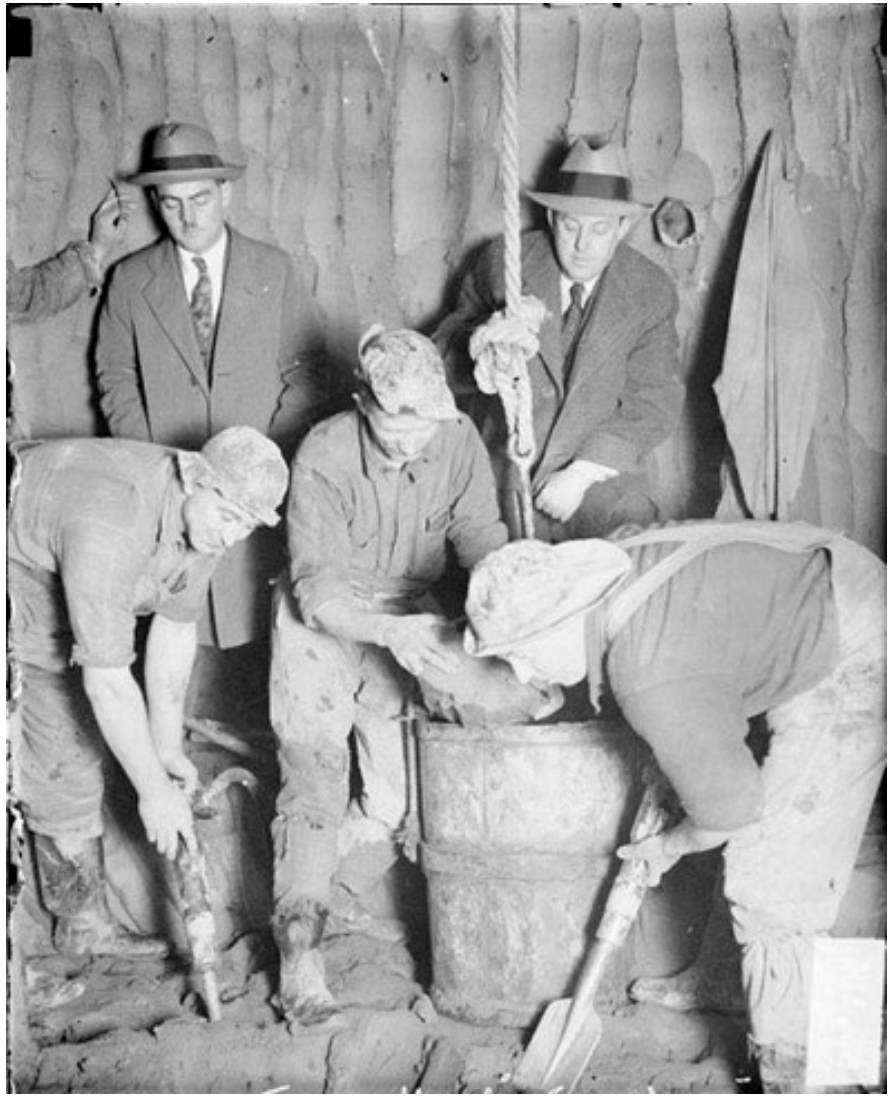
McCormick Place South, 2000s



1894 – General Sooy Smith

- First hand-dug caisson founded on deeper clay soils





1920 - Union Station in Chicago



Union Station Caisson Load Test - 1920

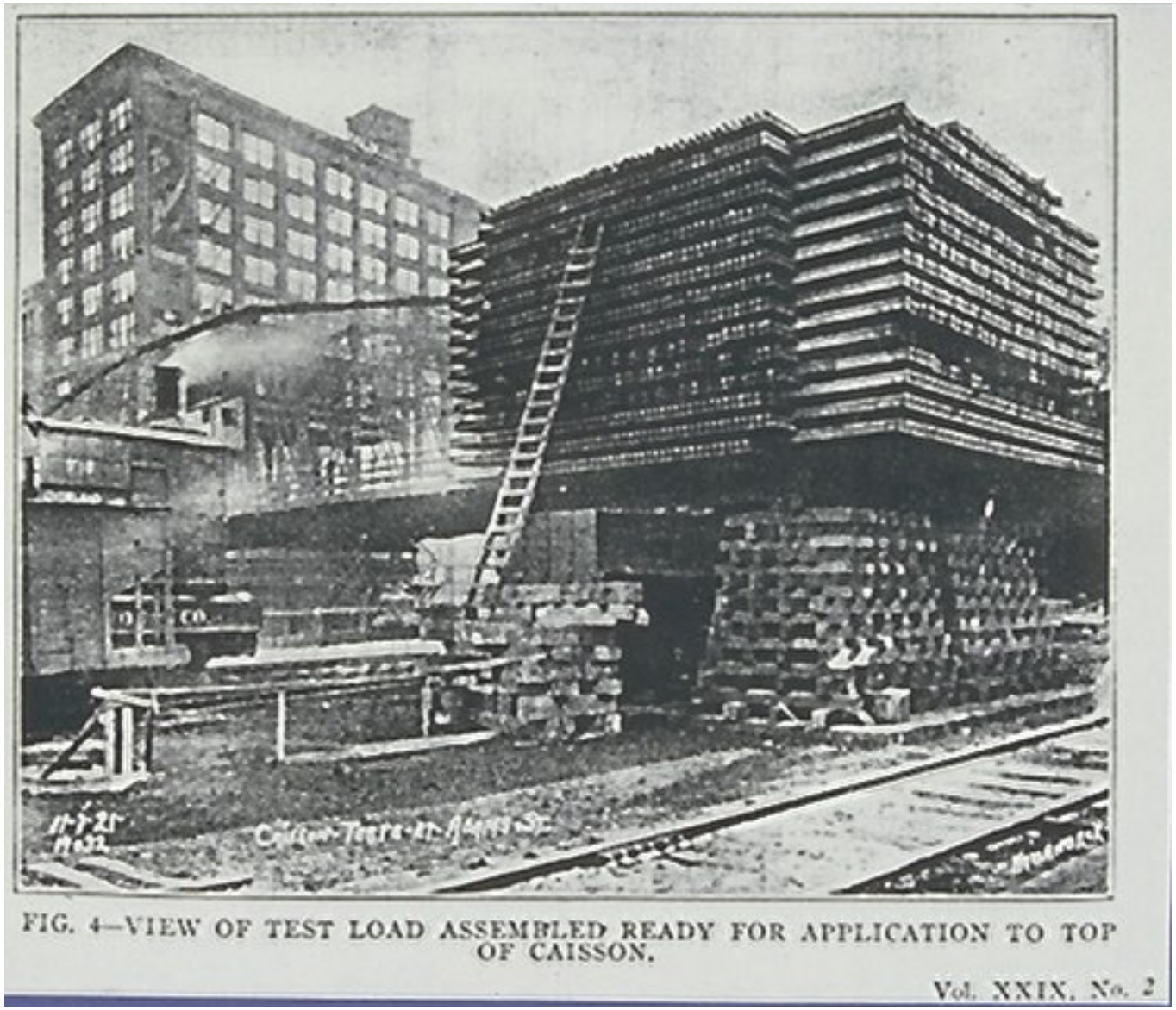
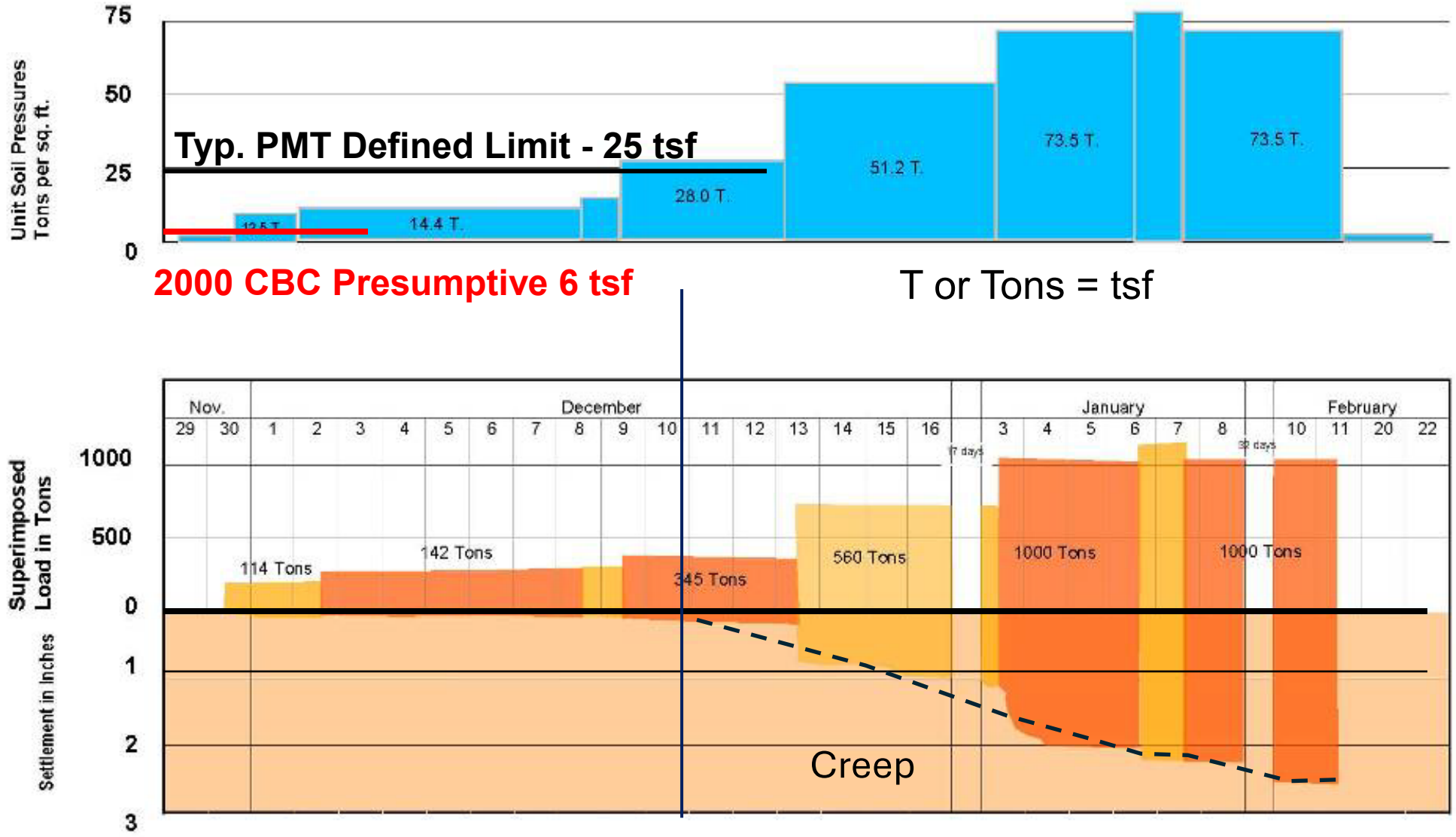


FIG. 4—VIEW OF TEST LOAD ASSEMBLED READY FOR APPLICATION TO TOP OF CAISSON.

Vol. XXIX, No. 2

Western Society of Engineers

1920 Union Station Compression Load Test on Hardpan



Increased Bearing Pressure on Hardpan, Chicago, 1950 - 1983

Using pressuremeter and drained triaxial tests, design bearing of belled caissons increased from 6 tsf to up to 25 tsf on hardpan clay till.

High Rise Foundations Supported by Belled Caissons on Hardpan (CCHRB, 1998)

Building	Year	Design Pressure	Predicted Settlement	Measured Settlement
CHA - Robert Taylor Homes (16-story)	Circa 1950	7.5 to 10 tsf	NA	0.38 to 1 inch
Water Tower Place (75-story)	1976	12.5 tsf	2.0 inches	1.69 to 2.19 Inches
33 W. Monroe (27-story)	1978	4.5 to 7.5 tsf (no hardpan)	1.25 inches	0.63 inches
AT&T Tower (65-story)	1983	22.5 to 25 tsf	0.95 to 1.0 inches	0.55 inch at 80% Dead Load

Early Machine Dug Caissons

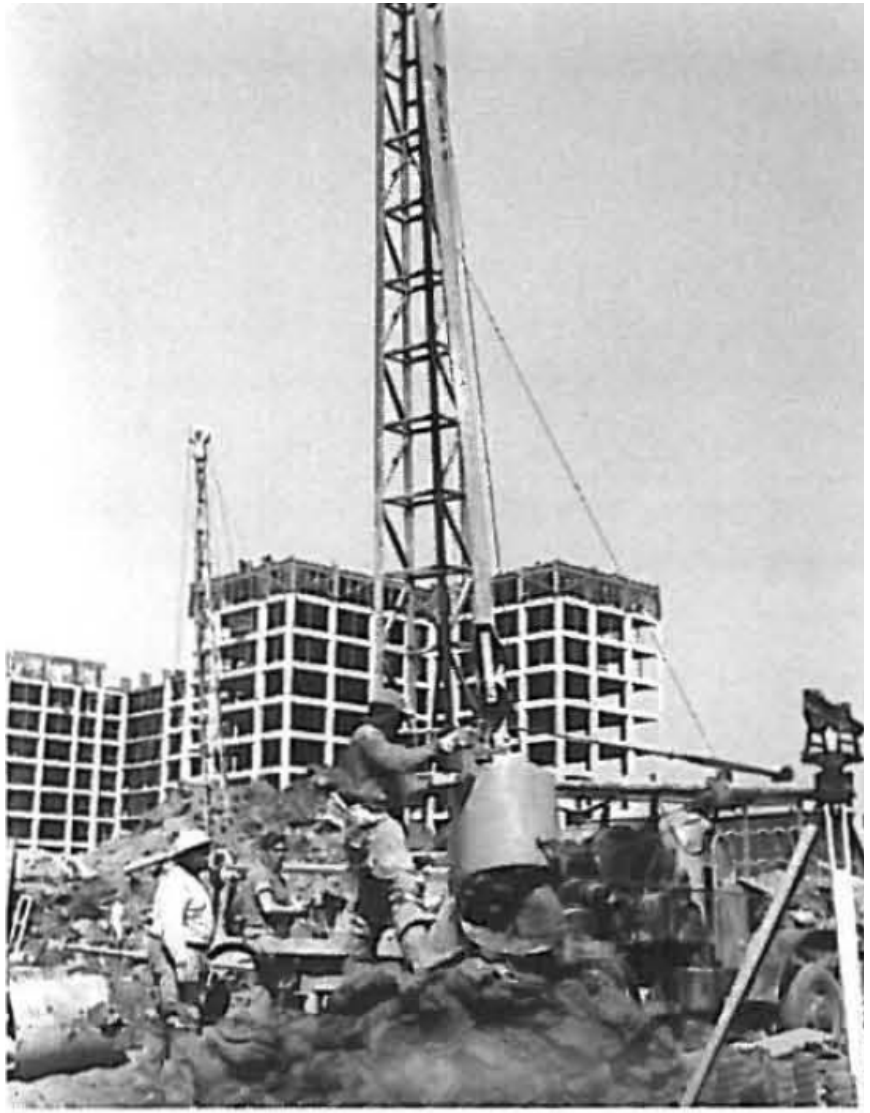


Fig. 15. 1950's era Bucket Rig.



Fig. 17. Early Crane Attachment with First 7 ft. belling bucket capable of cutting a maximum 21'-0" diameter bell. (Circa 1963)

From Bob Schock, Retired President of Case Foundation Company

Largest Bell Caisson - Water Tower Place



Fig.20 Water Tower Place - large "Apex" belling bucket (1972)

9 ft Shaft and 27 ft bell diameter



Fig. 21. Water Tower Place - hand cleaning 27 ft. bell. (1972)

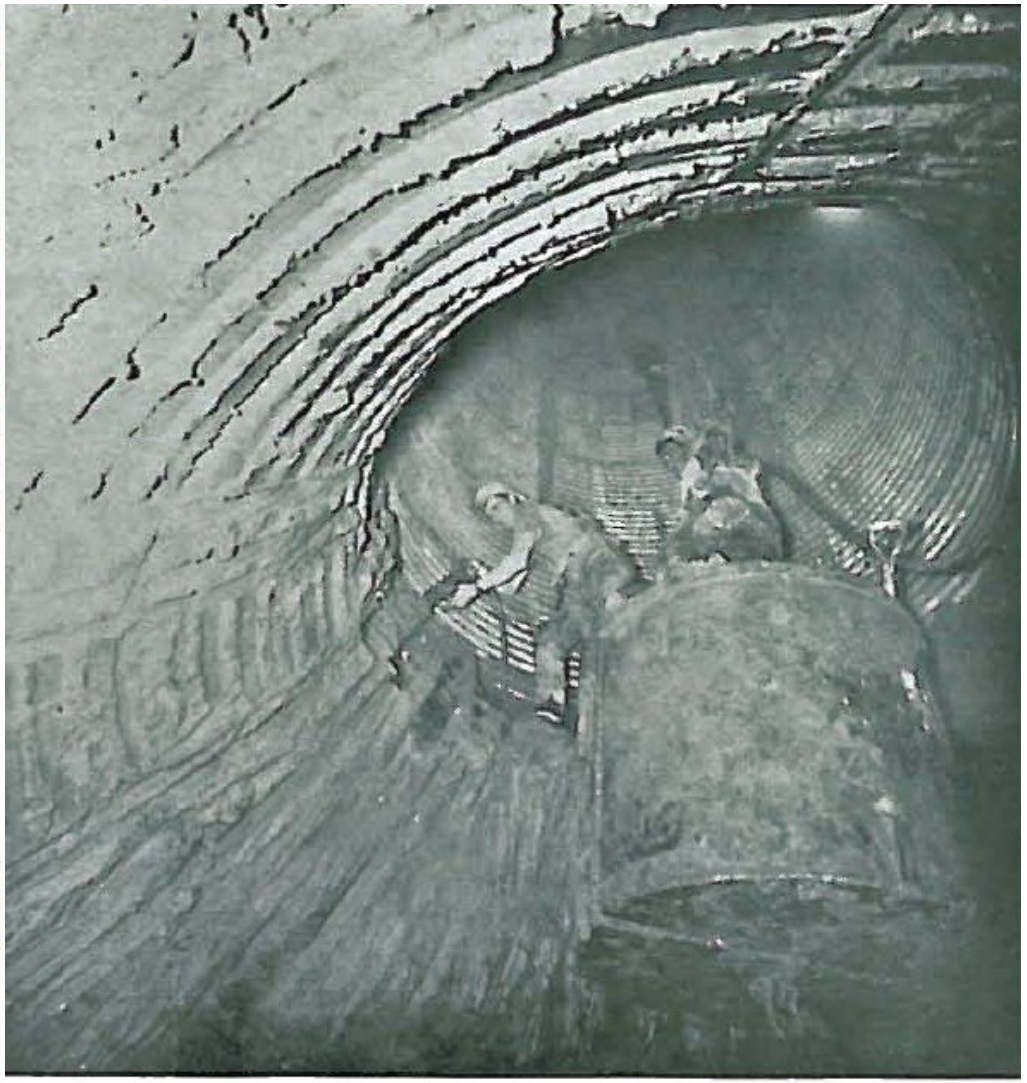


Fig. 22. Water Tower Place - hand trimming top of 27 ft. diameter bell. (1972)

From Bob Schock, Past President of Case Foundation Company

Recent High-Bearing Belled Caissons on Hardpan

- Use pressuremeter testing and analyses to predict settlement.

Building	Height	Design Bearing Pressure	Predicted Settlement	Measured Settlement
Pinnacle Tower (2004)	52-story	22.5 tsf	0.75 inches	0.38 to 0.75 inches
111 South Wacker Drive (2005)	51-story	22.5 tsf	0.5 to 1.0 inches	0.5 to 0.88 inches
300 North LaSalle (2008)	57-story	20 tsf (25-foot diameter bells)	0.75 to 1.2 (core) inches	1.2 inches at core
444 West Lake St. – Riverpoint (2017)	52-Story	25 tsf (25-foot diameter bells)	0.5 to 0.75 inches	36 stories 0.48 to 0.72 inches (70% of DL)

Case Histories
Pressuremeter Sites

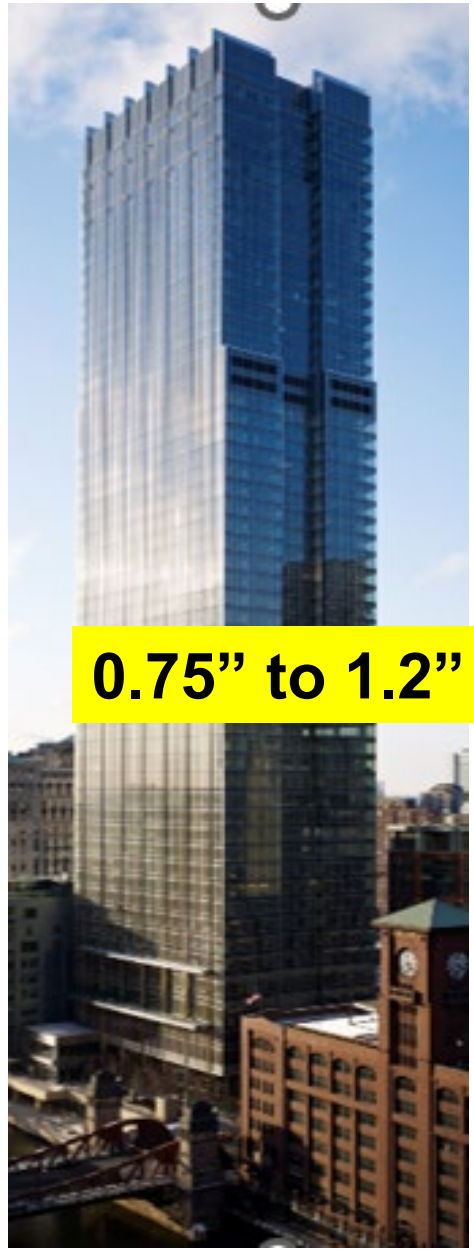
Pinnacle Tower



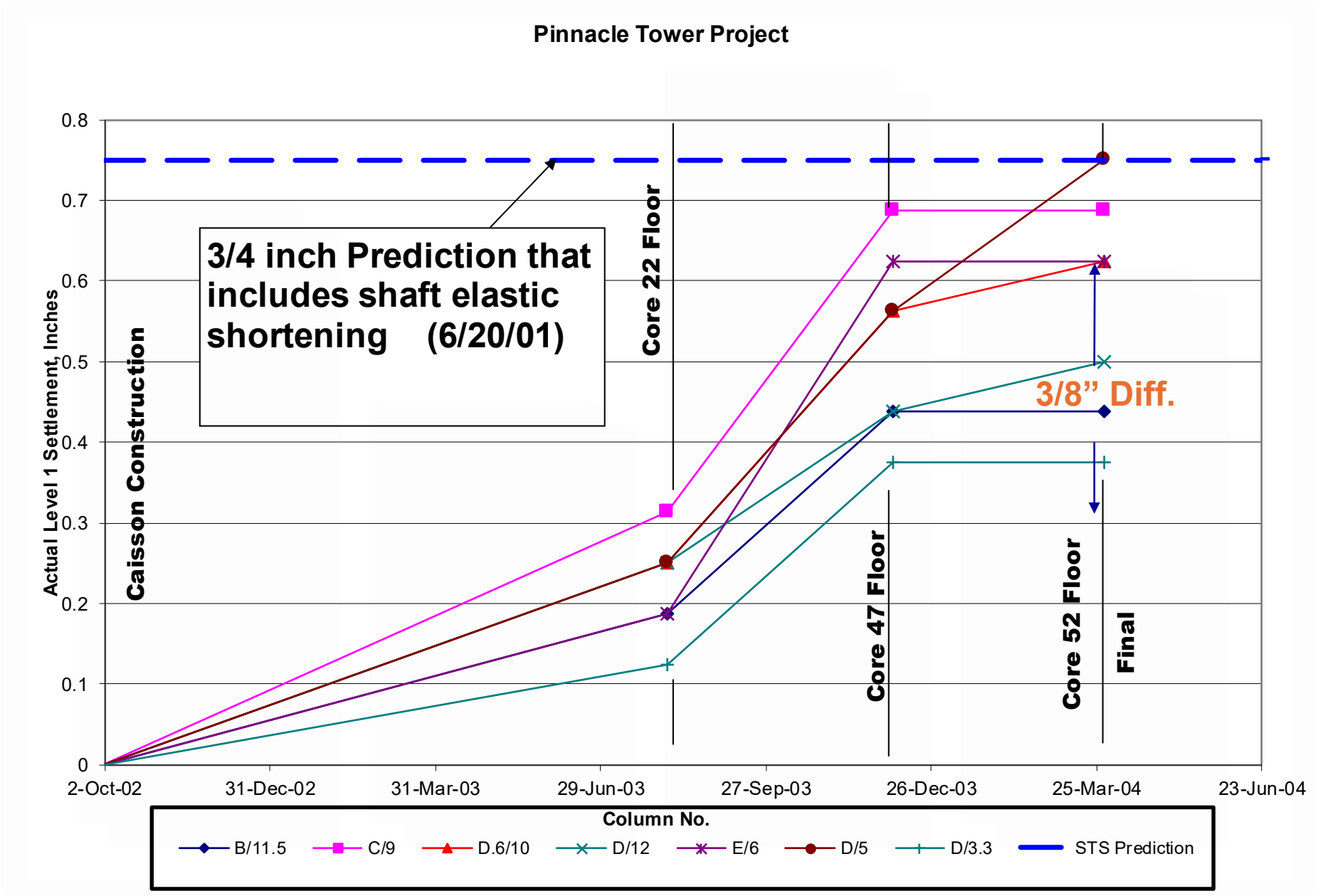
111 S. Wacker



300 N. LaSalle



Pinnacle Tower Settlement Data – 7 DMPs

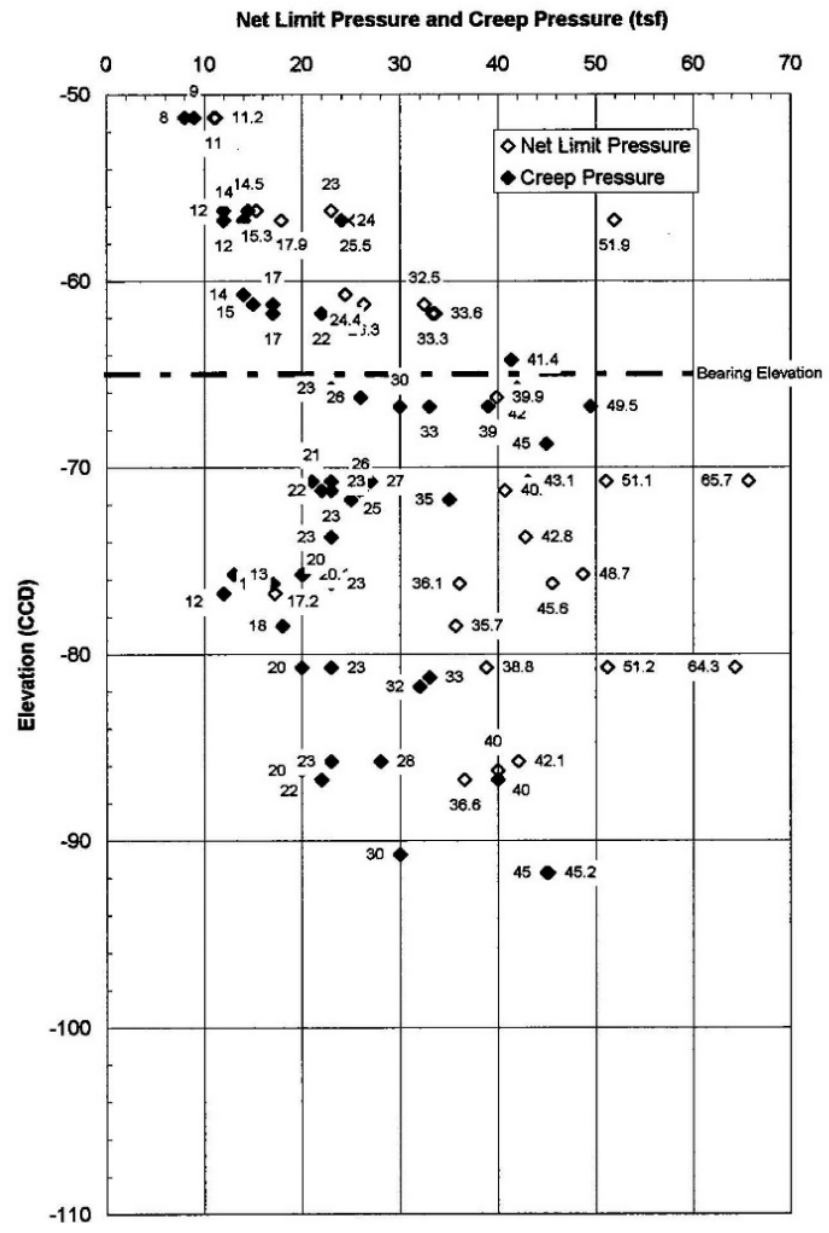


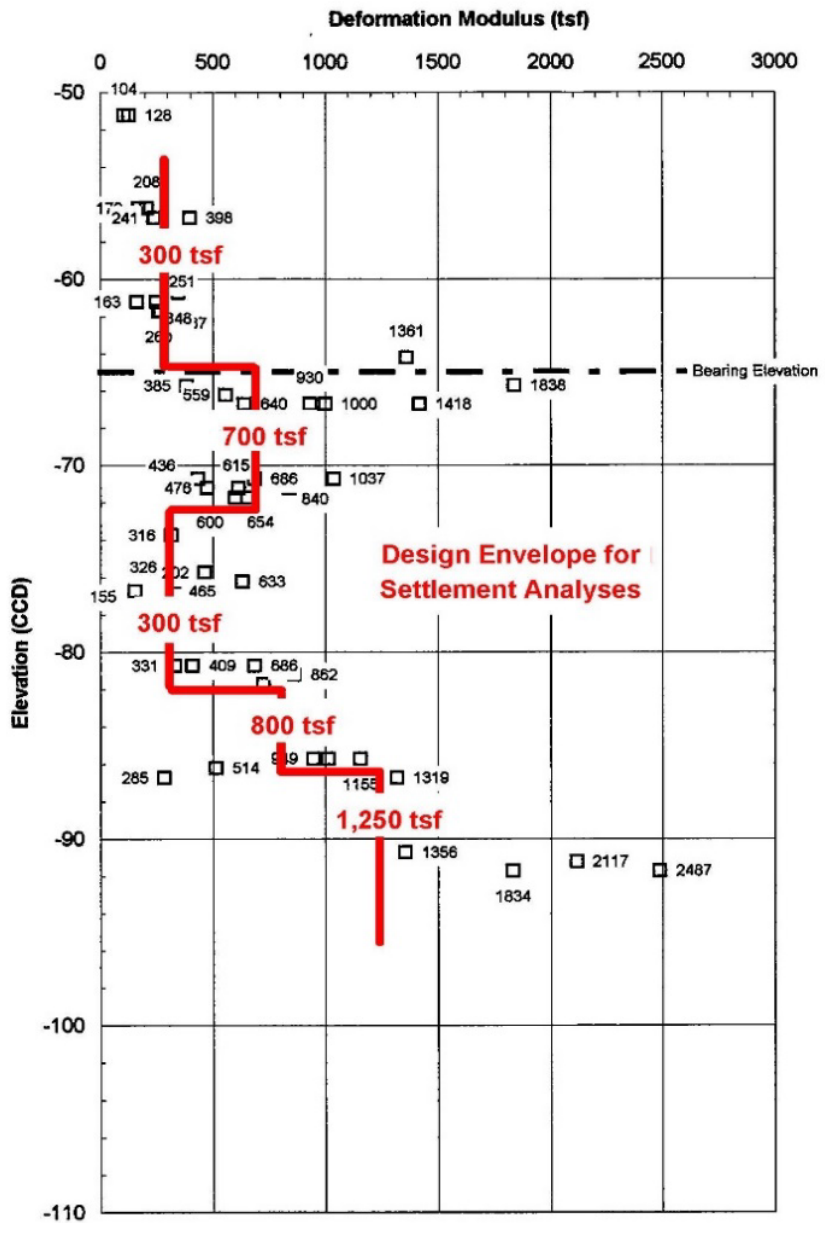
111 South Wacker Tower

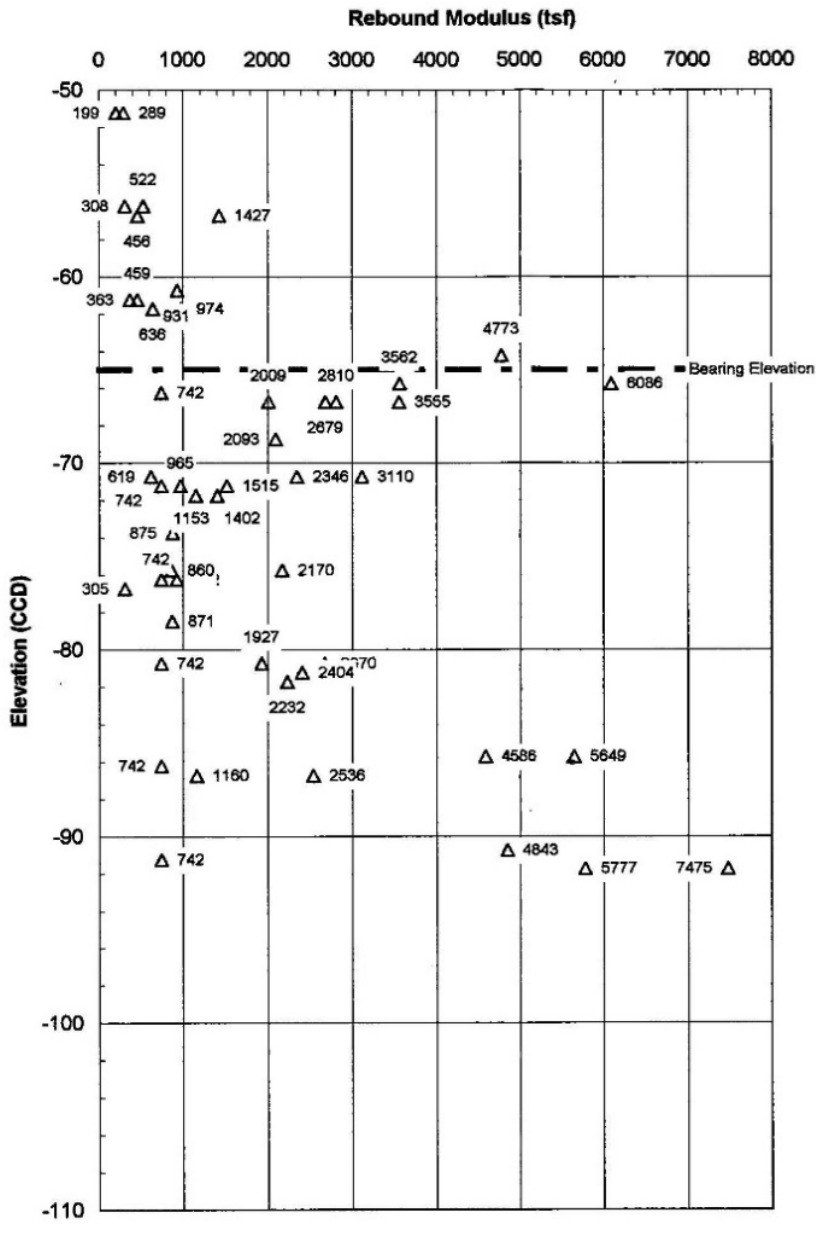
- New 51 story tower, reinforced concrete core, steel exterior frame.
- 12 new bells and 22 reused USG bells to support 100 ft x 100 ft by 10-ft-thick core mat.
- 12 new 16 ft Bell Caissons - 22.5 tsf at El - 65 CCD
- 24 reused Bell Caissons – 8 tsf increased to 12 tsf at El. -55 CCD using PMT (*Clyde Baker was the USG's foundation engineer in 1961*)
- 14 new 200-ton rock bearing micropiles to replace one 20'Φ bell where the bottom blew-in with lots of water
- We predicted mat on core caissons would settle 0.5 inch and external caissons 0.75 inch. Actual was slightly ranging from 0.5 to 1.0 inches.
- All allowable bearing pressures based on PMT testing.



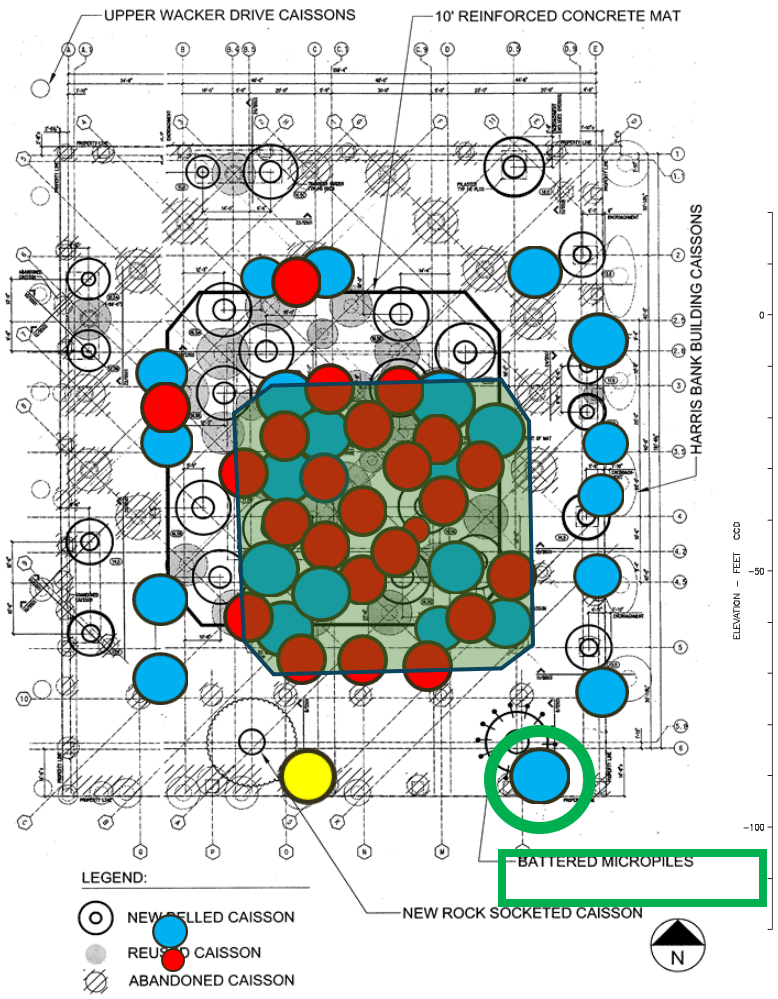
Pressuremeter Test Results

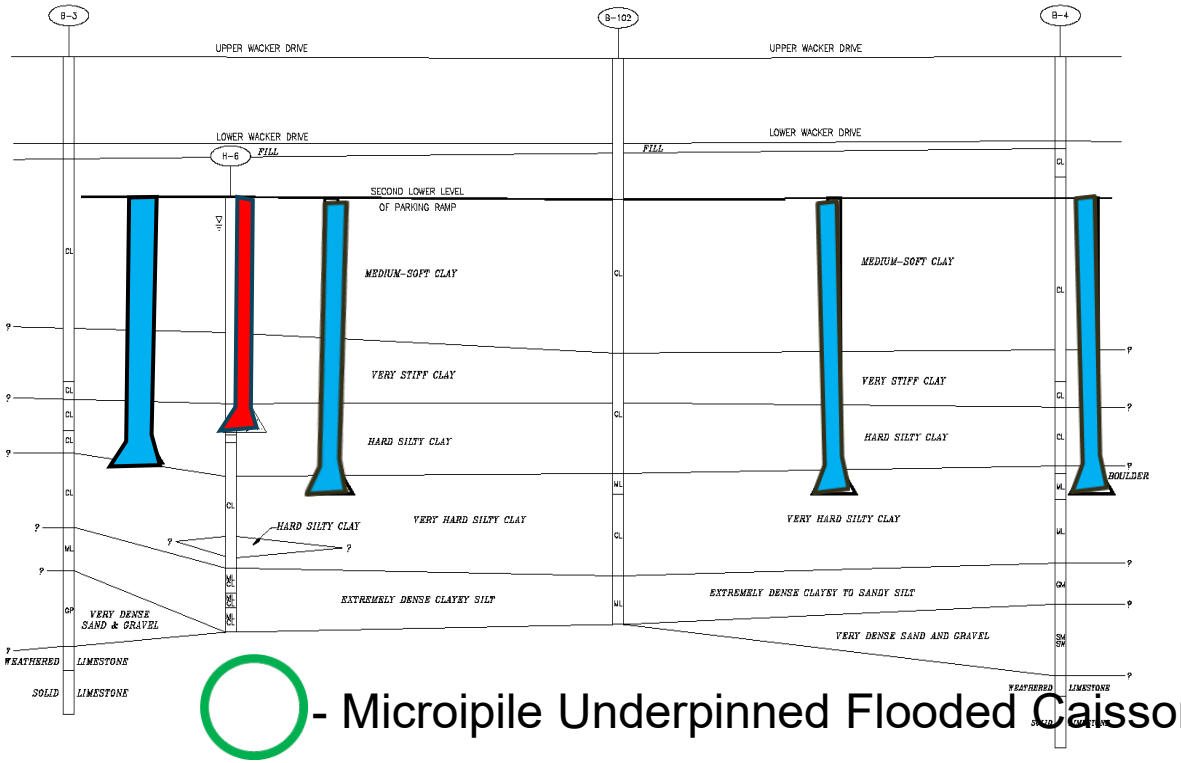






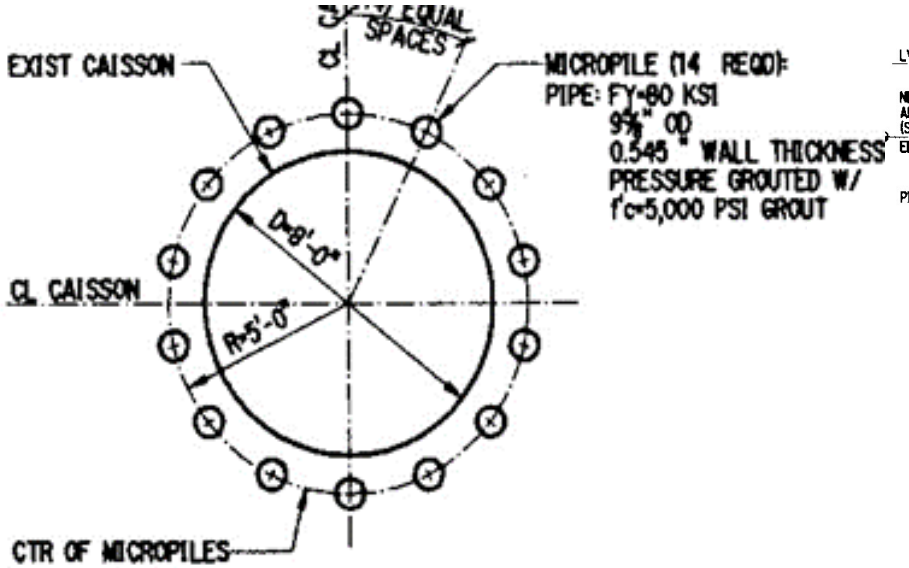
New and Reused Bell Caissons and Soil Profile

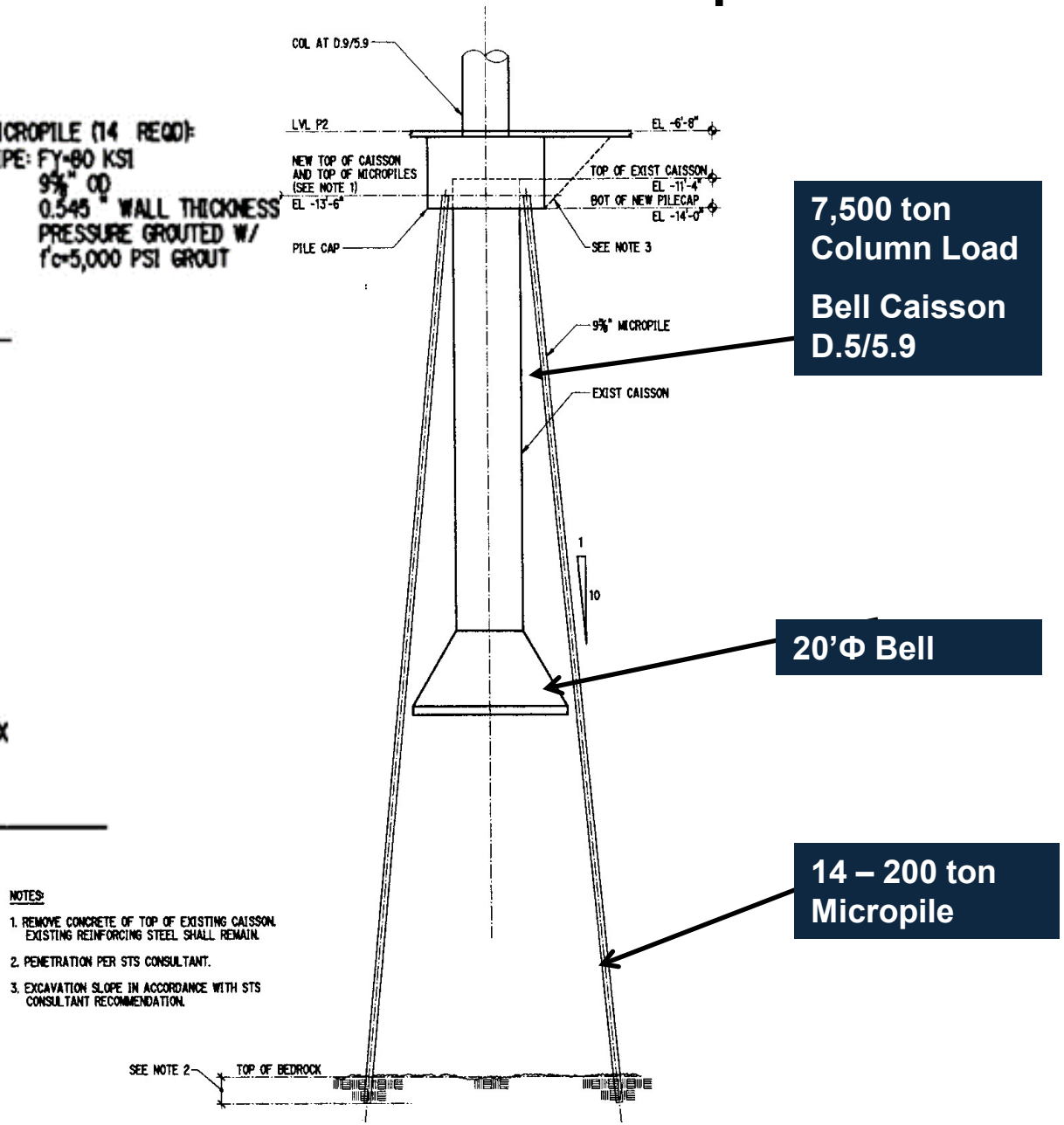




- Microipile Underpinned Flooded Caisson
- Rock Socket Caisson
- Core Mat Limits

Micropile Support of a Blow-in Hardpan Caisson





7,500 ton Column Load

Bell Caisson D.5/5.9

20'Φ Bell

14 – 200 ton Micropile

300 N. LaSalle Tower



300 N. LaSalle

- 57-story office building
- Bearing pressure = 20 tsf (25-foot diameter bells)
- First Bells constructed under Polymer Slurry (3 days to construct)
- Predicted Settlement was 0.75 to 1.2 (core) inches

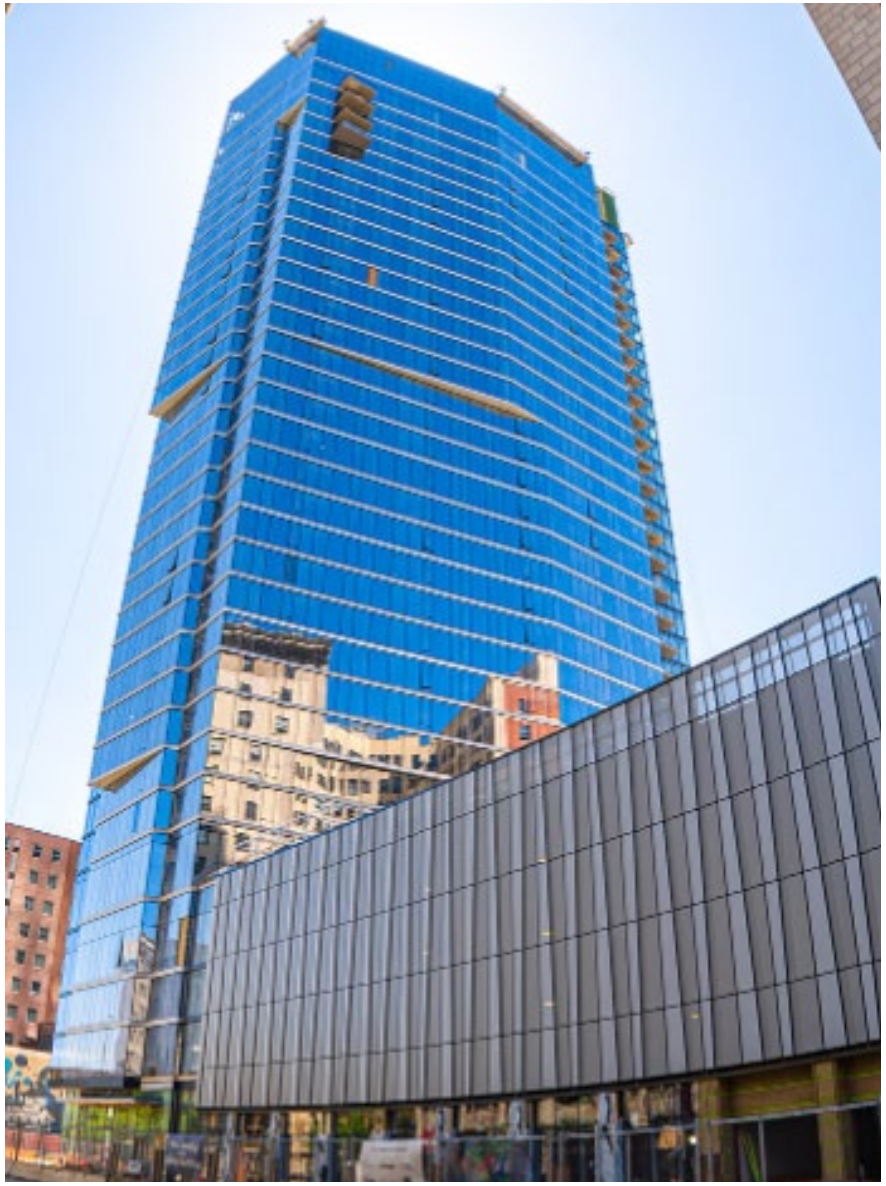
Date	Construction Completed	Measured Settlement at Core
March 9, 2007	Mat Poured	0 inches
May 14, 2008	43-stories at core	0.75 inches
August 18, 2008	57-stories at core	1.2 inches

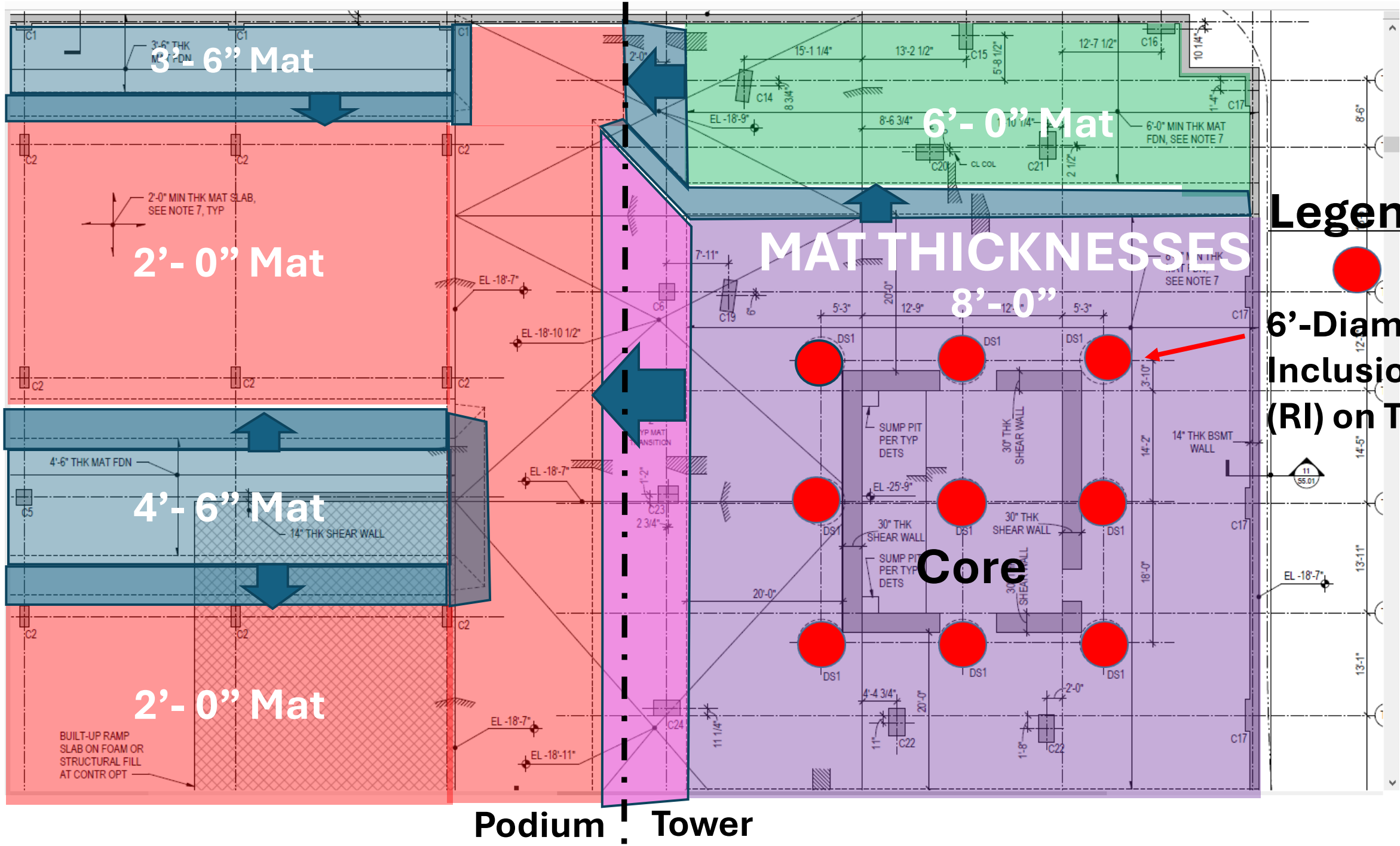
Playhouse Square - Cleveland, OH

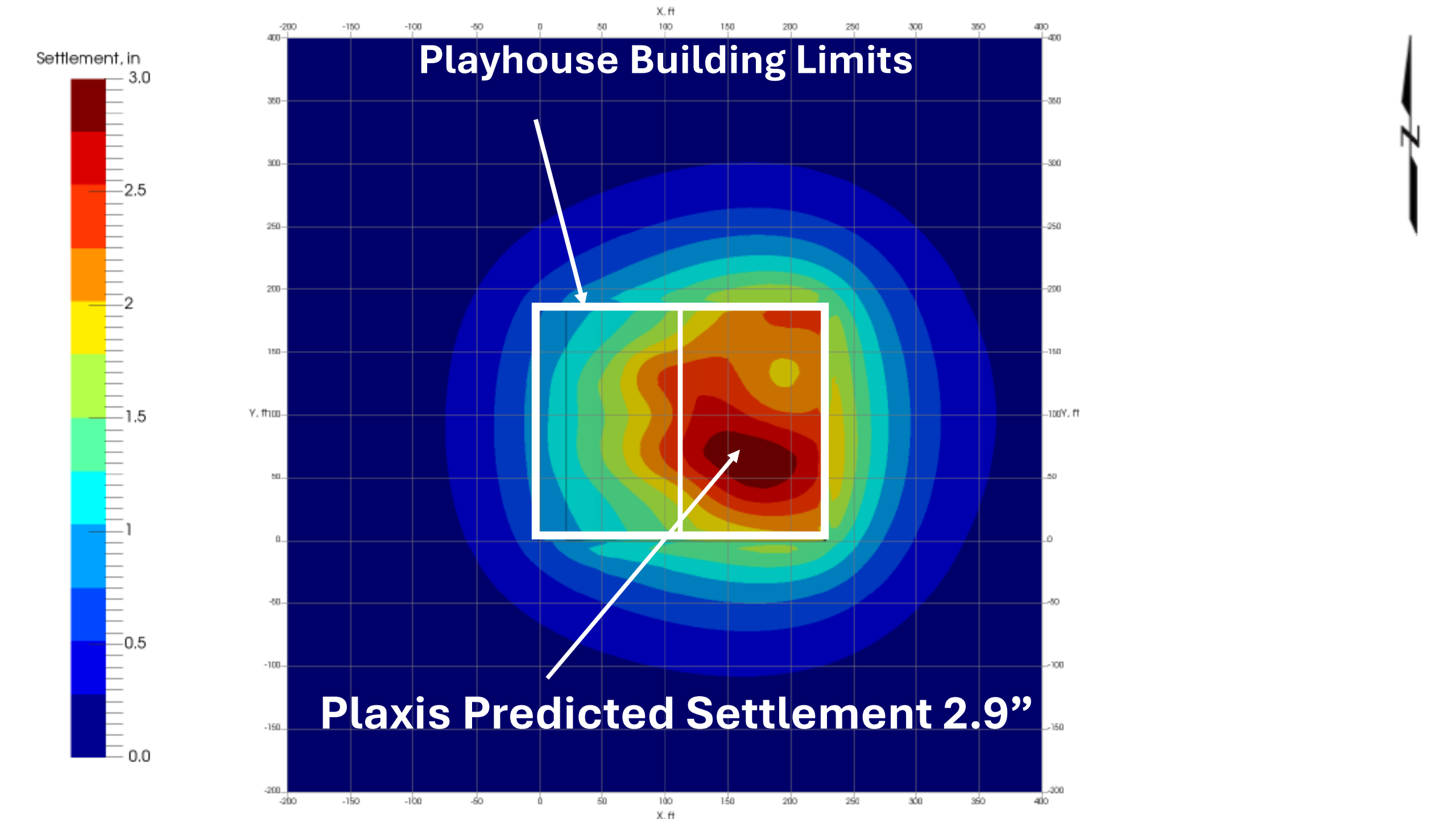
- Mat on dense sand. Core mat on dense sand with nine Rigid Inclusions as settlement reducers.
- Inclusions go into till and not to deep shale rock.
- **Use of inclusions NOT extending to shale rock made possible by pressuremeter data and experience with previous buildings in Chicago that had been monitored.**
- Settlements were less than predicted.

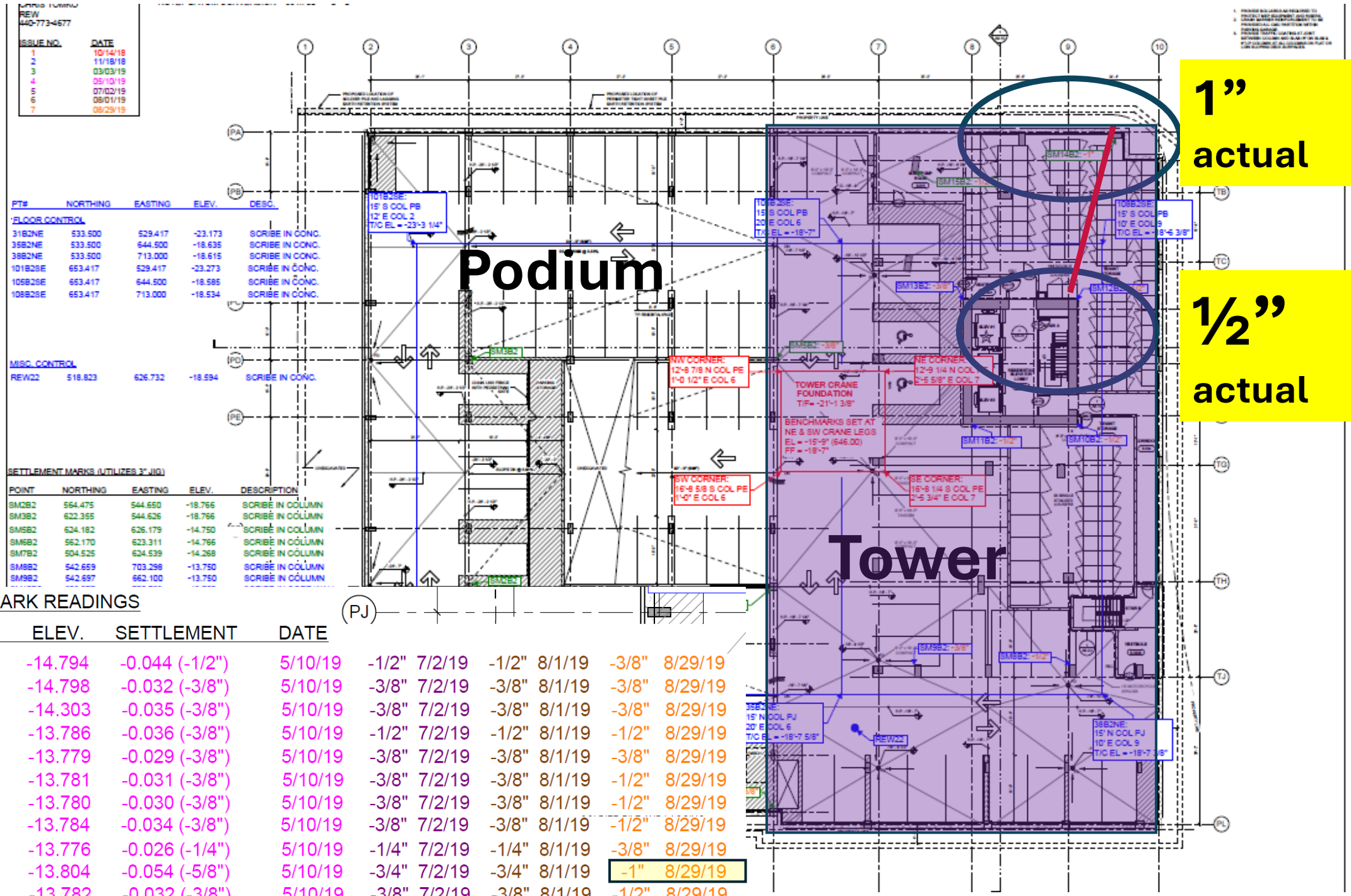
Playhouse Square, Cleveland, OH






3’- 6” Mat
6’- 0” Mat
2’- 0” Mat
MAT THICKNESSES
8’-0”
4’- 6” Mat
2’- 0” Mat
Core
Podium
Tower
Legend:
6’-Diam.Rigid Inclusions (RI) on Till


Settlement, in
3.0
2.5
2
1.5
1
0.5
0.0
X, ft
Y, ft
Playhouse Building Limits
Plaxis Predicted Settlement 2.9”
N



SETTLEMENT MARK READINGS

POINT	ELEV.	SETTLEMENT	DATE						
SM5B2	-14.794	-0.044 (-1/2")	5/10/19	-1/2"	7/2/19	-1/2"	8/1/19	-3/8"	8/29/19
SM6B2	-14.798	-0.032 (-3/8")	5/10/19	-3/8"	7/2/19	-3/8"	8/1/19	-3/8"	8/29/19
SM7B2	-14.303	-0.035 (-3/8")	5/10/19	-3/8"	7/2/19	-3/8"	8/1/19	-3/8"	8/29/19
SM8B2	-13.786	-0.036 (-3/8")	5/10/19	-1/2"	7/2/19	-1/2"	8/1/19	-1/2"	8/29/19
SM9B2	-13.779	-0.029 (-3/8")	5/10/19	-3/8"	7/2/19	-3/8"	8/1/19	-3/8"	8/29/19
SM10B2	-13.781	-0.031 (-3/8")	5/10/19	-3/8"	7/2/19	-3/8"	8/1/19	-1/2"	8/29/19
SM11B2	-13.780	-0.030 (-3/8")	5/10/19	-3/8"	7/2/19	-3/8"	8/1/19	-1/2"	8/29/19
SM12B2	-13.784	-0.034 (-3/8")	5/10/19	-3/8"	7/2/19	-3/8"	8/1/19	-1/2"	8/29/19
SM13B2	-13.776	-0.026 (-1/4")	5/10/19	-1/4"	7/2/19	-1/4"	8/1/19	-3/8"	8/29/19
SM14B2	-13.804	-0.054 (-5/8")	5/10/19	-3/4"	7/2/19	-3/4"	8/1/19	-1"	8/29/19
SM15B2	-13.782	-0.032 (-3/8")	5/10/19	-3/8"	7/2/19	-3/8"	8/1/19	-1/2"	8/29/19

Case History - Earth Retention System

Deep Space Auditorium at Epic Systems,
Verona, Wisconsin

The Challenge – Get it Done Quick

Epic in Wisconsin needed an underground auditorium for client training and meetings

<u>Details</u>:

- 600,000 square feet
- 11,400 seats
- Six stories underground
- 90 ft below grade
- Green roof





<u>Location</u>:

- No street parking
- Mine 2M CY of Soil and Rock
- Between the CEO’s office and an idyllic countryside view

Protect Nearby Buildings and Infrastructure

To obtain builders risk insurance, needed to limit settlement of existing buildings and lateral movement of earth retention system.

Permanent excavation support wall, independent of new auditorium.

Design of excavation support went from:

- Massive reinforced concrete counterfort wall (on micropiles) due to karstic limestone with caves

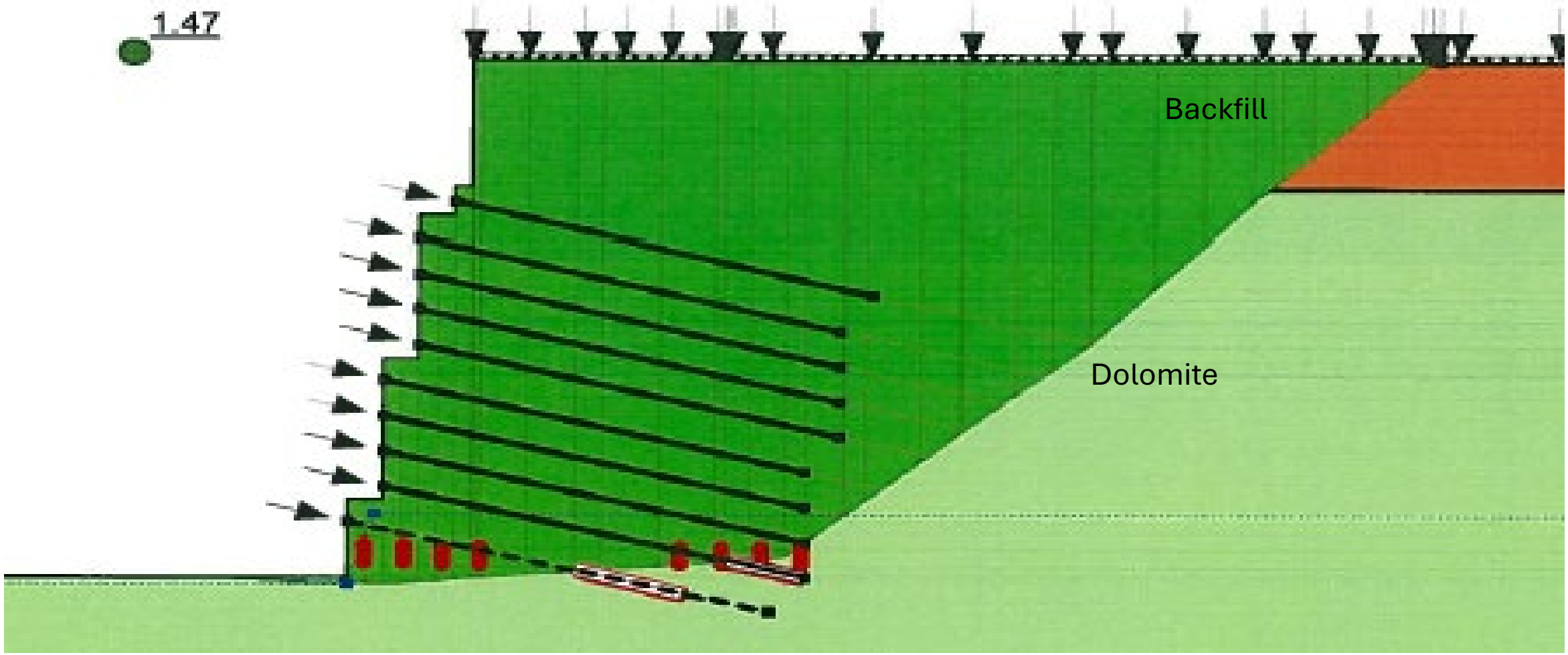
To:

- A flexible active and passive soil and rock nail system with shotcrete and reinforced concrete fascia wall
- As work progressed, geology required use of deeper and steeper active rock nails instead of all passive rock nails
- Provide a watertight wall with drainage

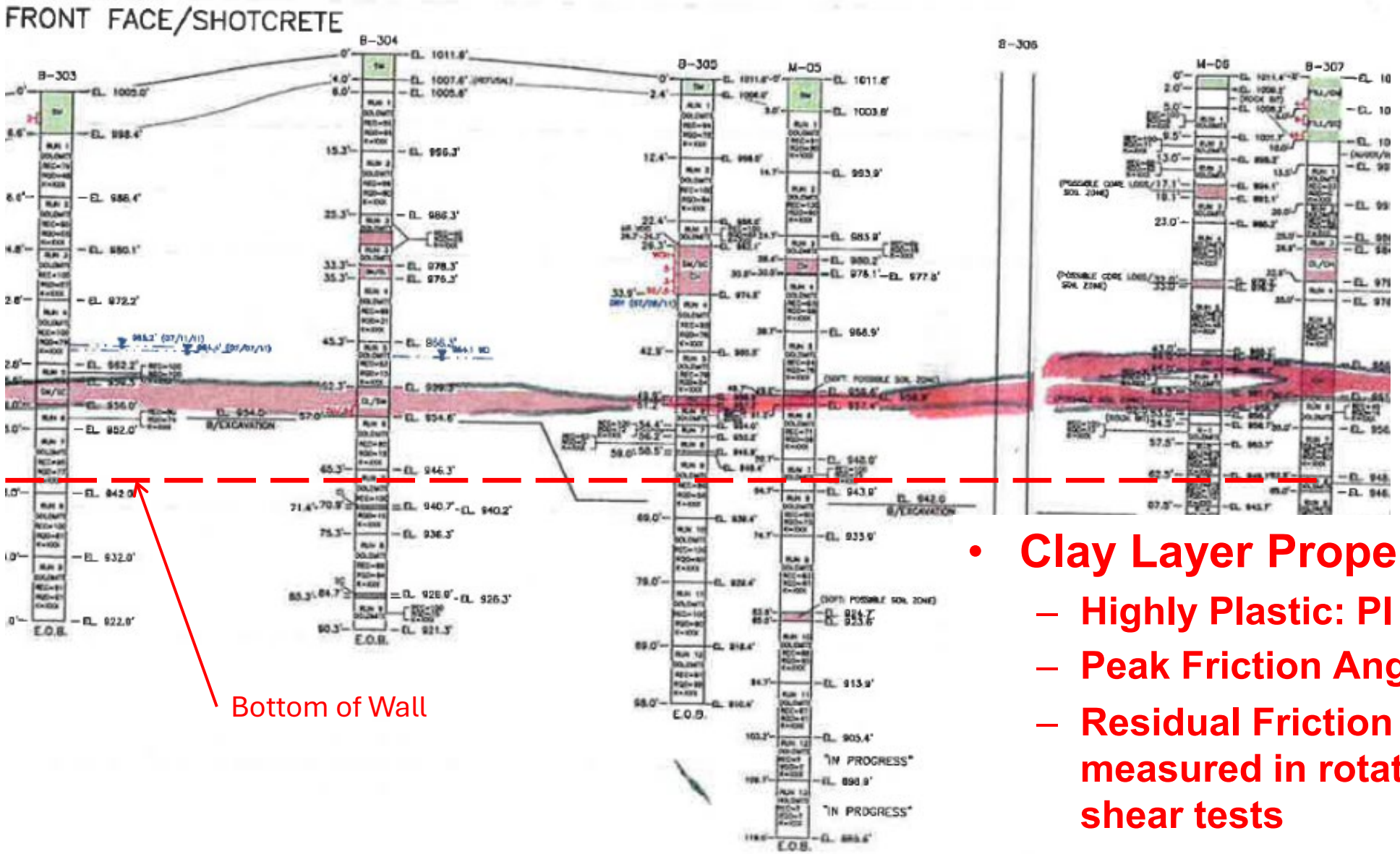
Global Stability 1- original design

Sliding stability results with no weak layer

FS = 1.5 - OK



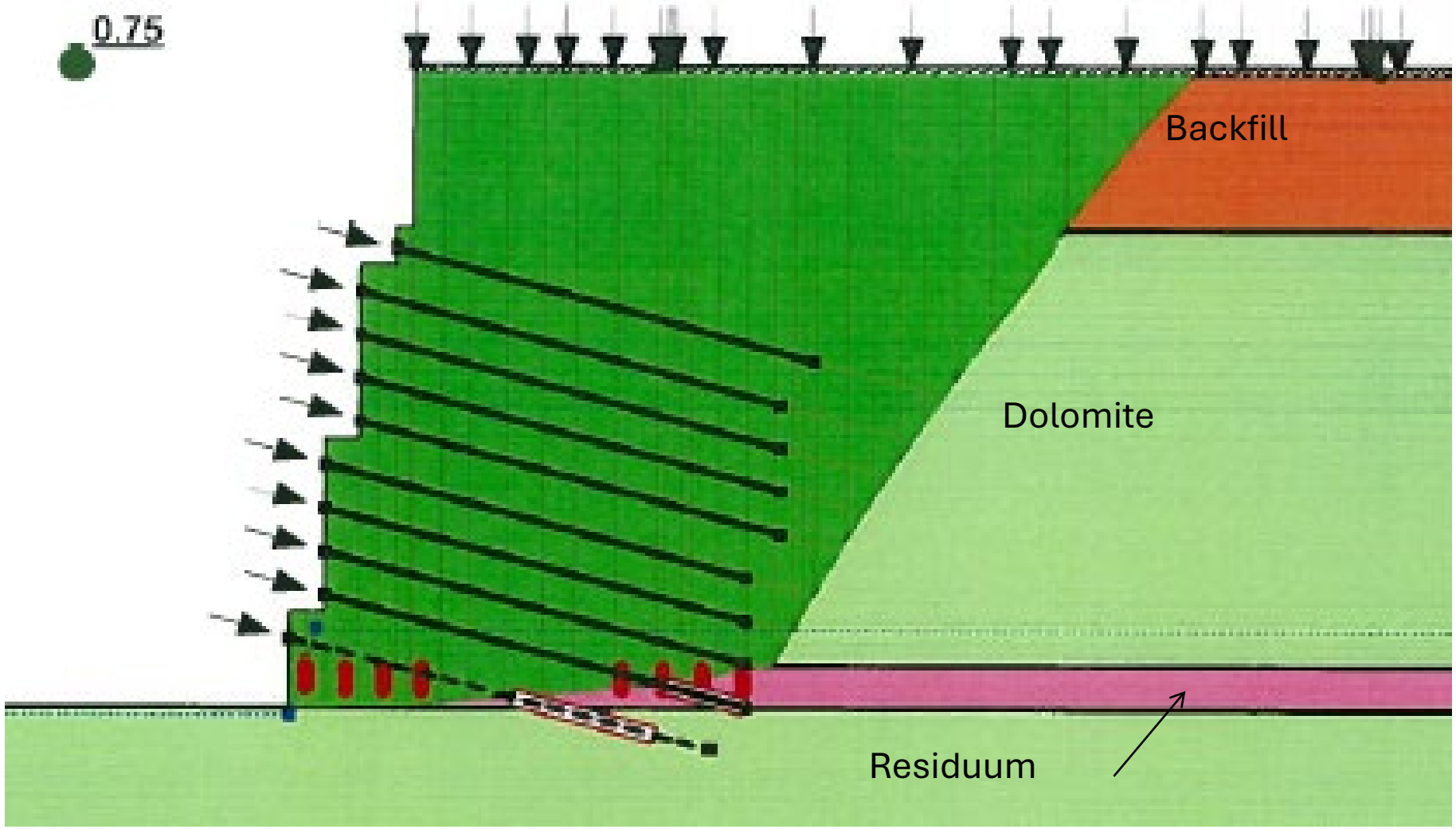
Global Stability 2- A layer of residual fat clay was found near the bottom of excavation



- **Clay Layer Properties**
  - **Highly Plastic: PI = 80, LL = 110**
  - **Peak Friction Angle = 20°**
  - **Residual Friction Angle of 5°-7° measured in rotational (ring) shear tests**

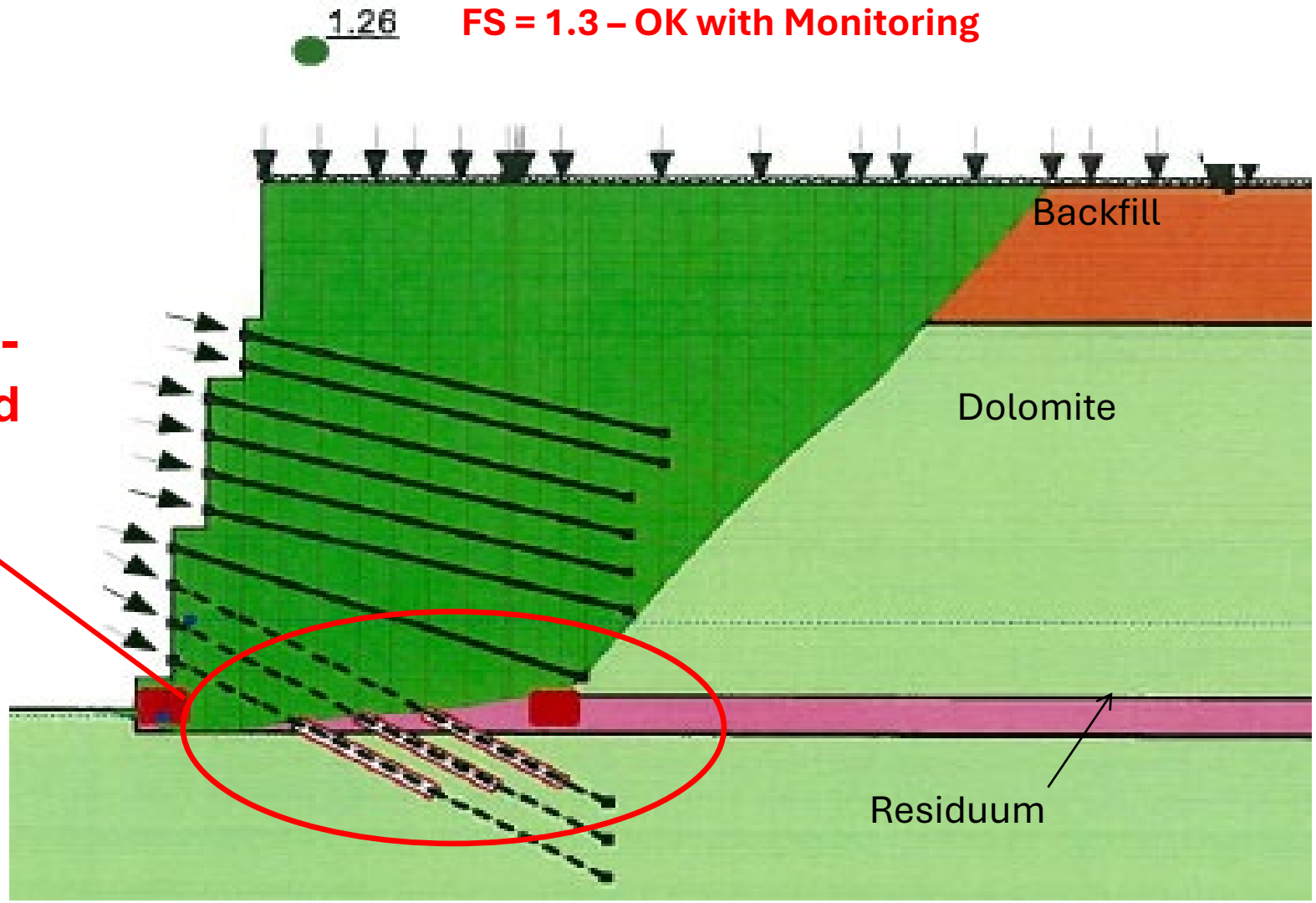
Global Stability 3 - Re-analyzing global stability with the fat clay layer showed original design not stable

FS = 0.8 - NG



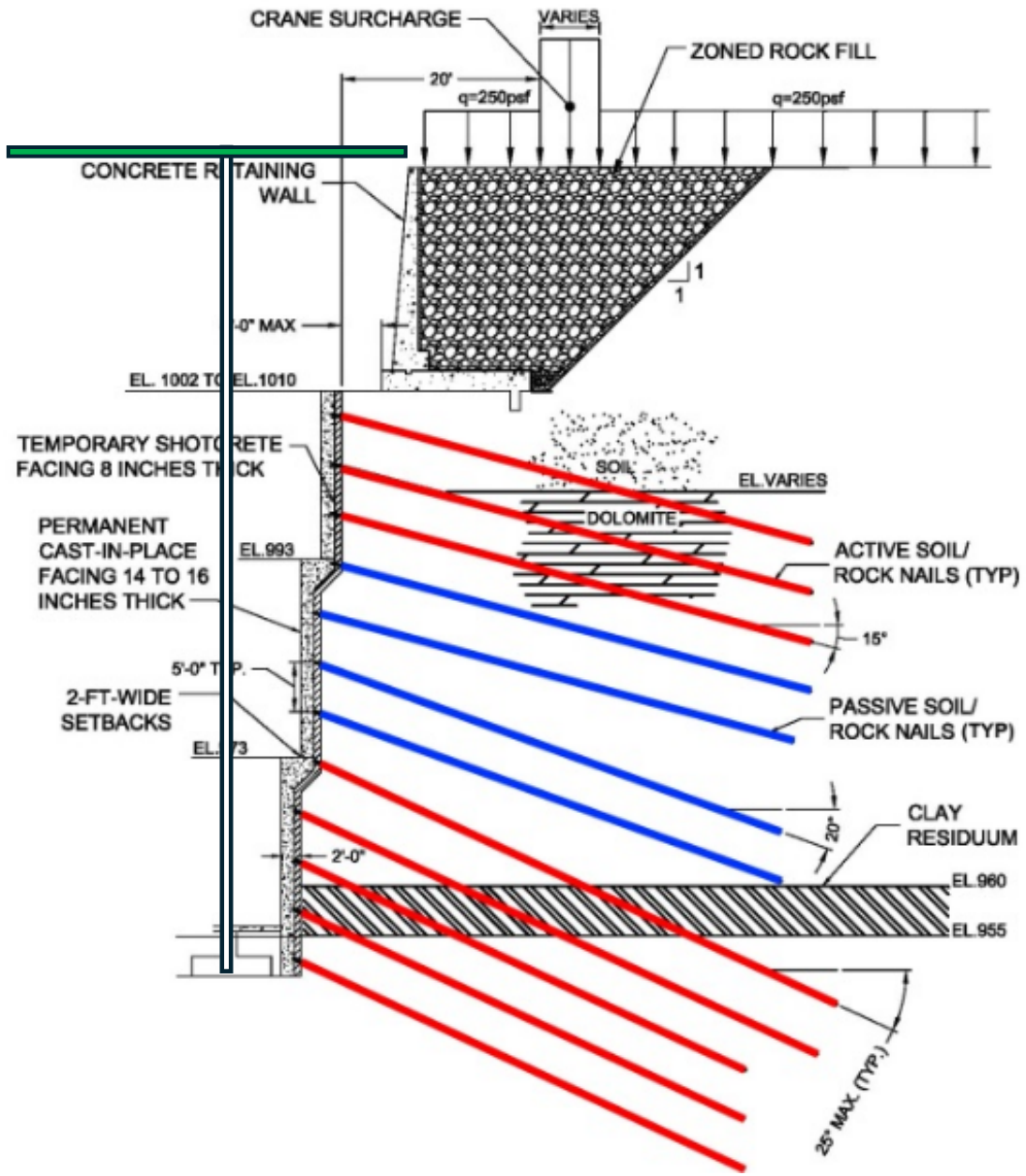
Global Stability 4 - Redesigned nails at bottom of excavation to be steeper and deeper, and pre-stressed with lockoff instead of passive

FS = 1.3 – OK with Monitoring



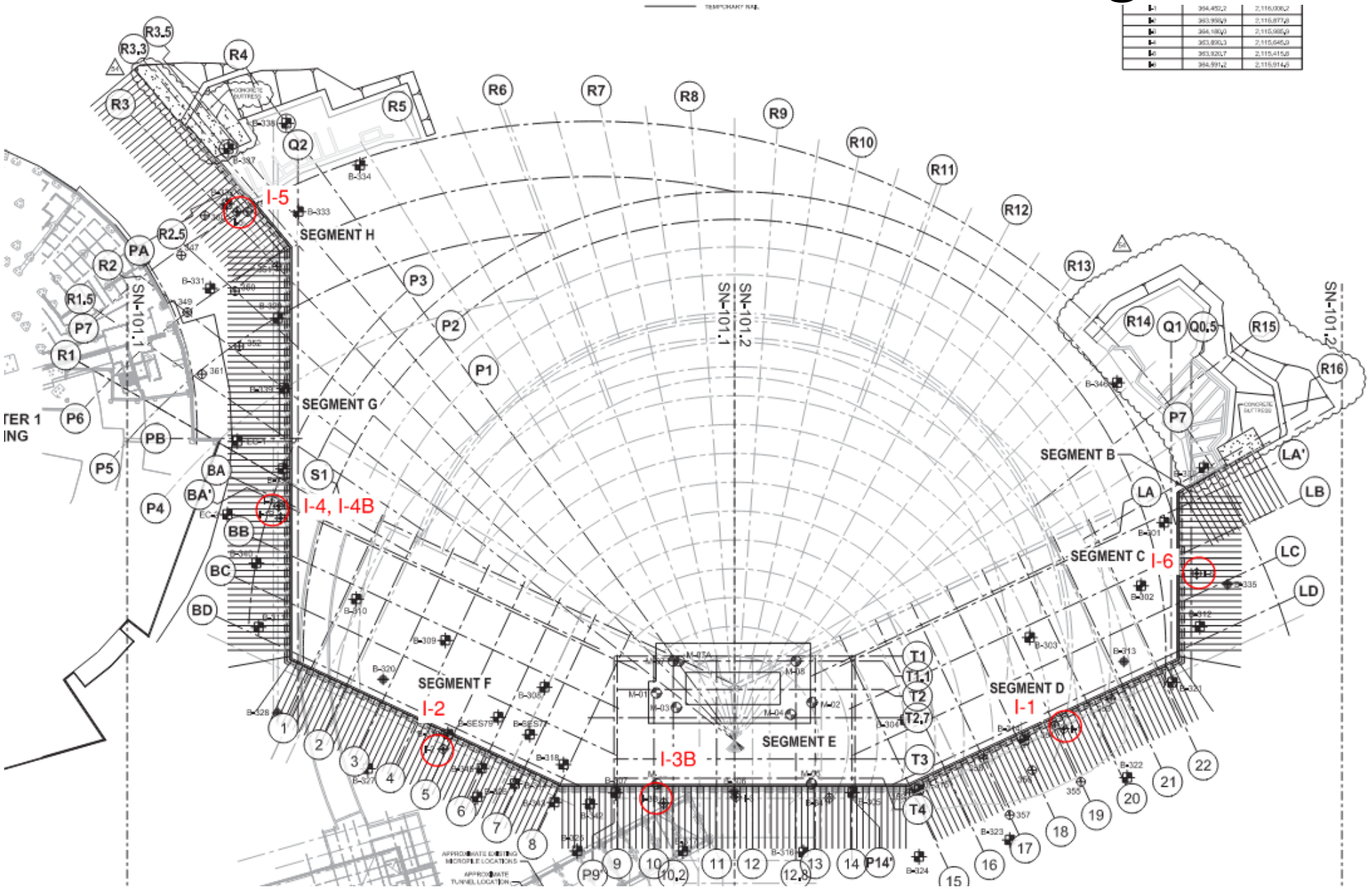
Active rock nail - angle steepened

Design Section – Building Independent of Wall



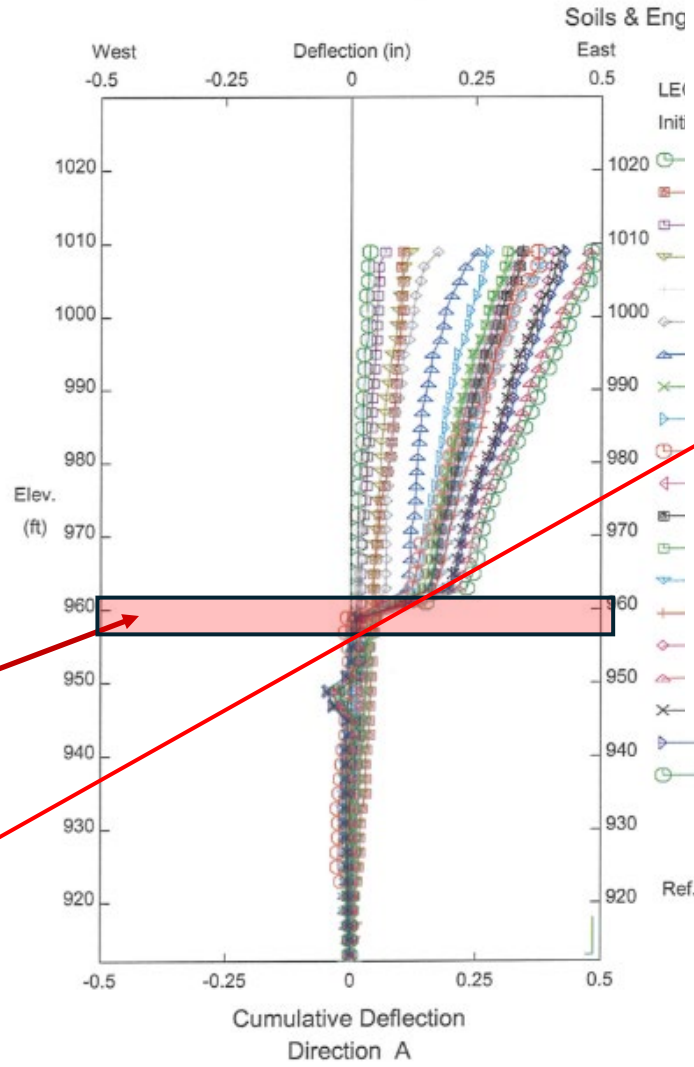
- Up to 12 rows of nails spaced on grid at 5 feet on-center
- Design loads up to 112 kips
- #11 or #14 Grade 75 thread bars
- Bond stress increased with depth from 5 psi to 25 psi
- Combination of Passive and Active (post-tensioned) soil and rock nails
- Class I Corrosion Protection
- **Blue is passive anchors – no load locked in**
- **Red is active anchors – preloaded and locked off**

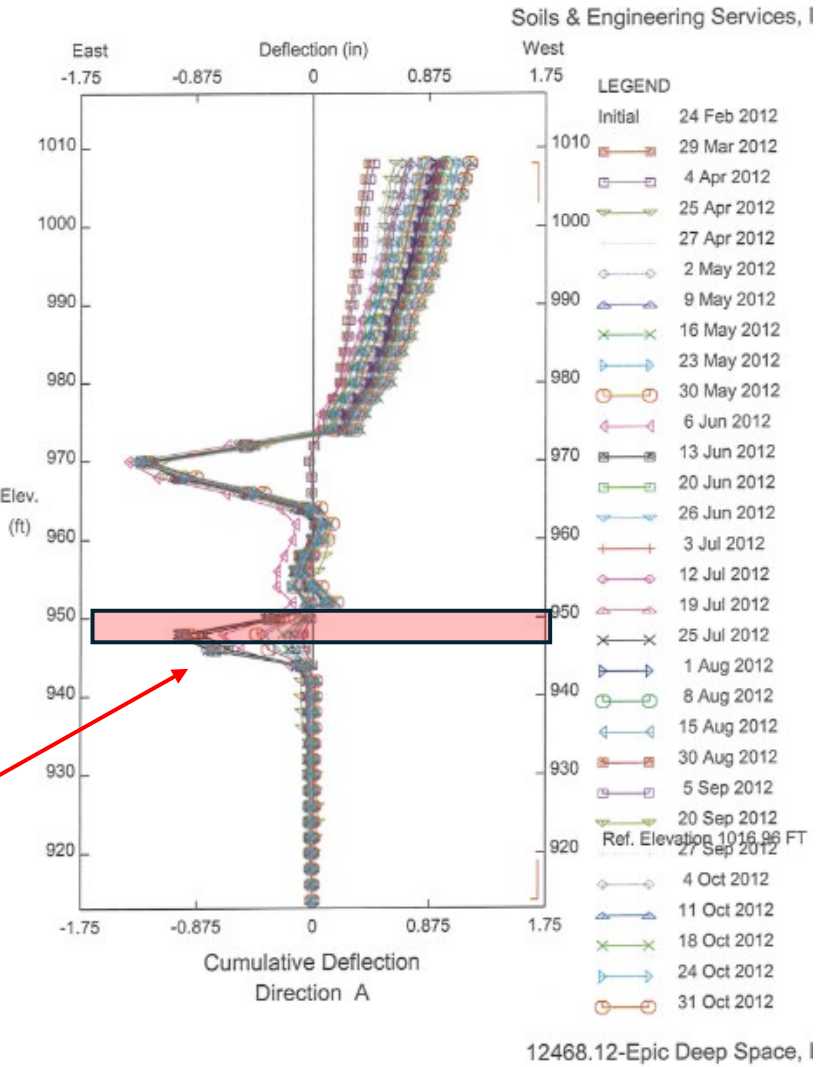
Inclinometer & Piezometer Monitoring Locations



Wall System Performance

- Spec required <2” movement
- Cumulative top of wall movements generally <1.3” next to CEO office
- Movements stabilized at end of construction
- Movement was concentrated near fat red clay layer
- Active anchors compressed the red clay





November 2011

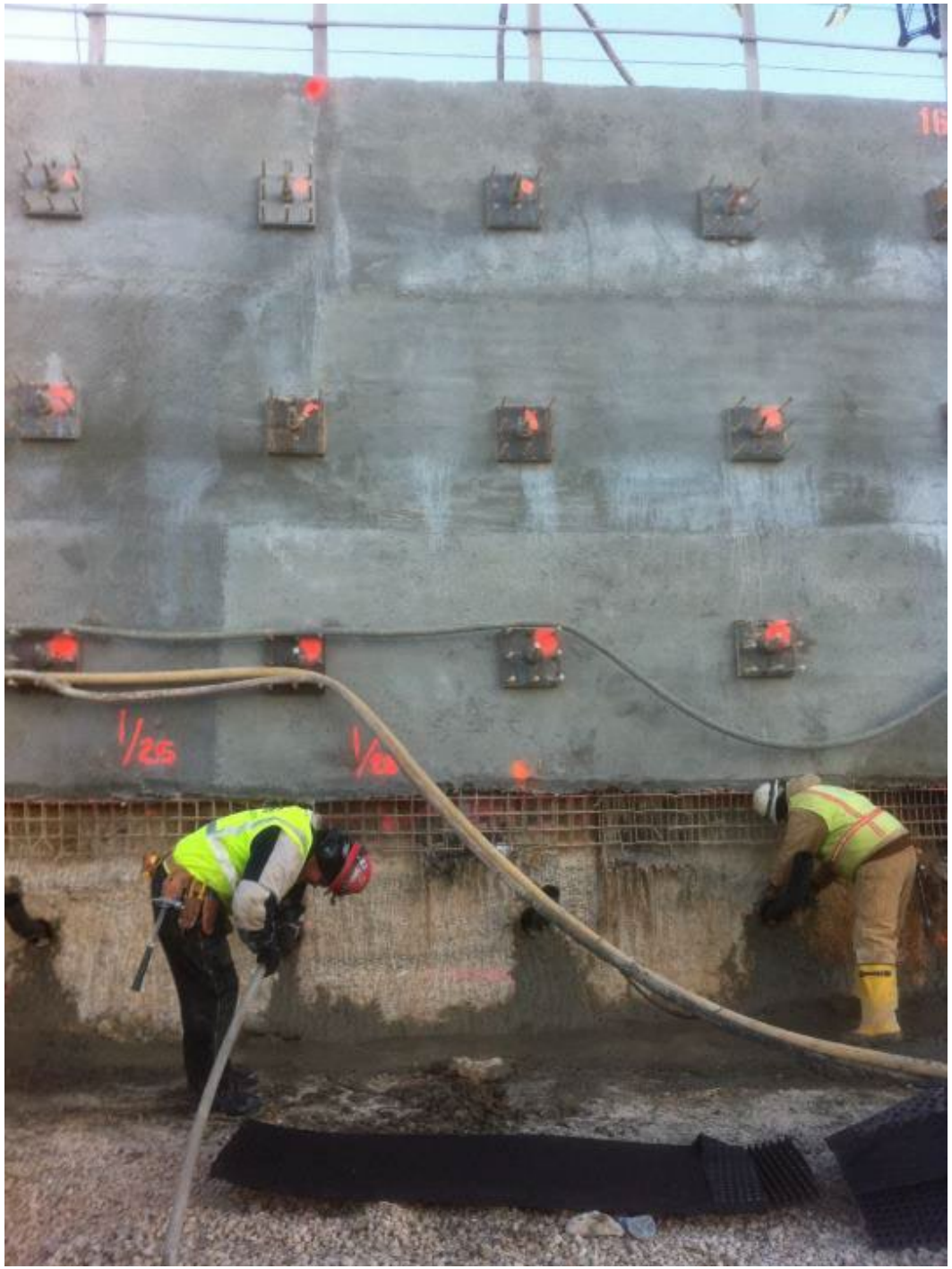


February 2012



Order of Construction

- Cleaned rock face
- Placed Miradrain strips
- Rock nails
- Welded wire mesh
- Shotcrete



May 2012
Mary Nodine
ASCE's Geo-Poet

Red Fat Clay Layer



June 2012



July 2012



Final Wall Treatment

- Shotcrete
- PrePrufe
- Reinforcing steel grid
- Moving slip form for 24-inch-thick concrete facing

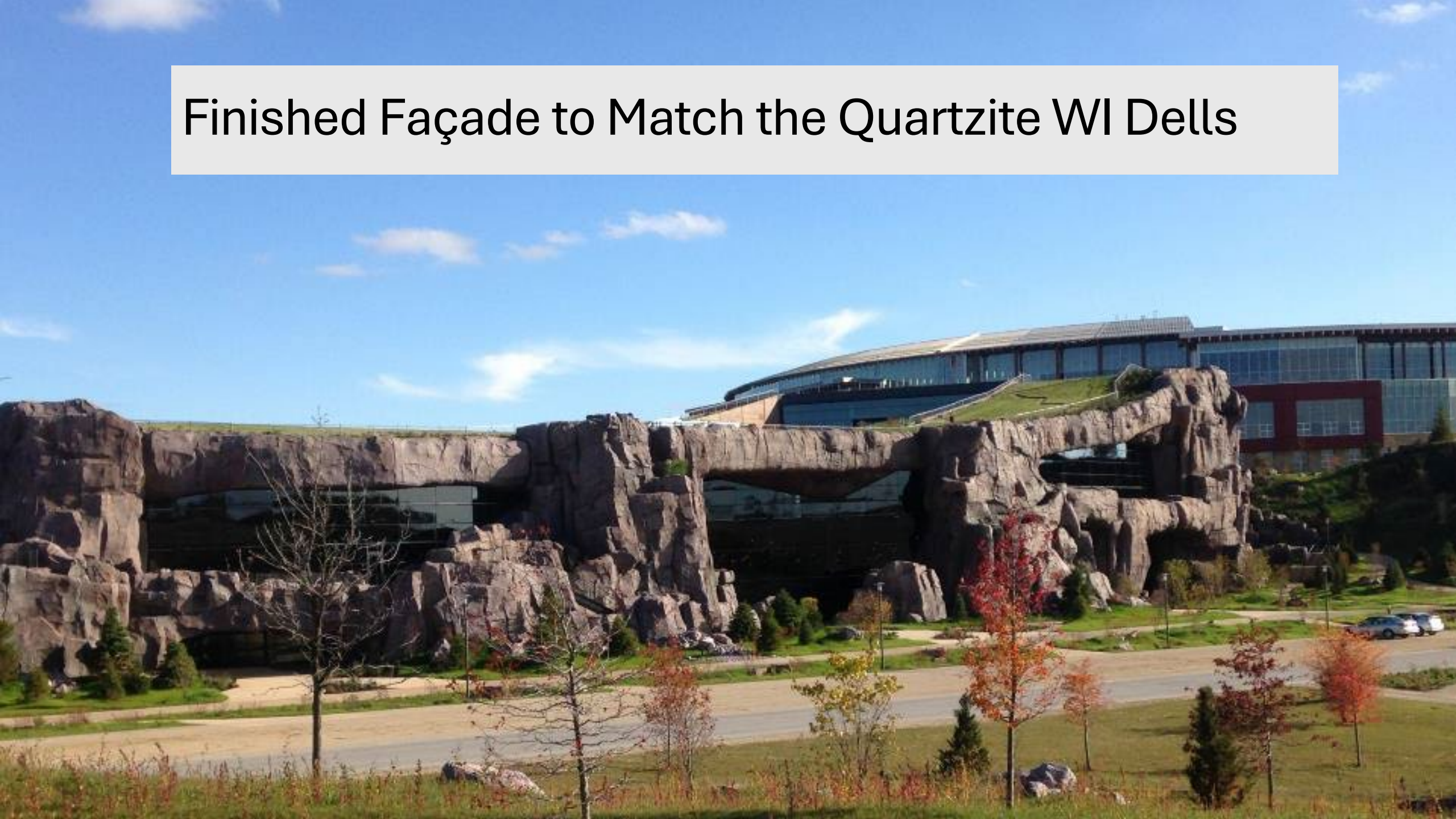
Note rock benches every 20 vertical feet.




October 2012


Winter 2013

Finished Façade to Match the Quartzite WI Dells



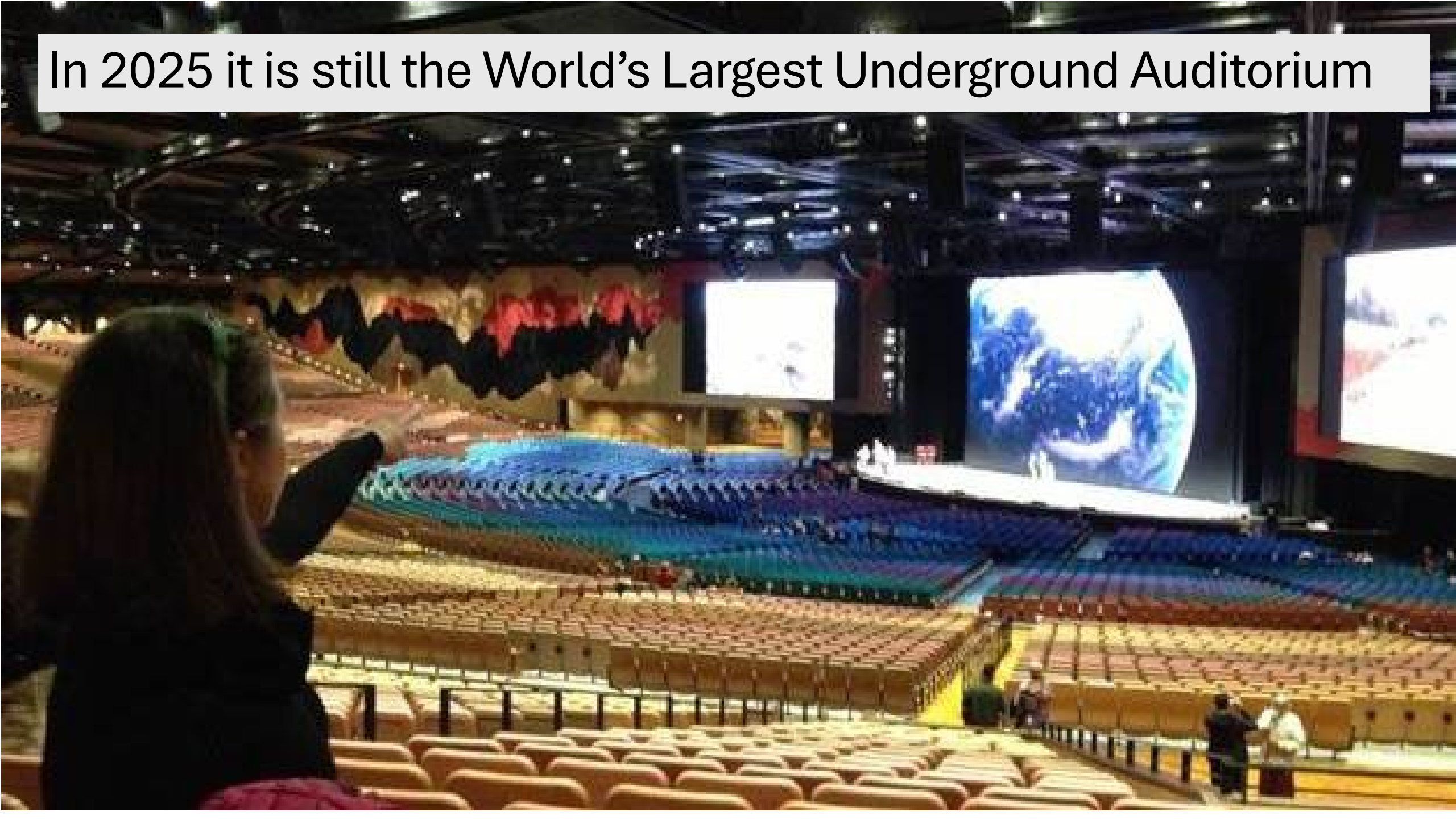
Finished Landscape 9-Acre Roof



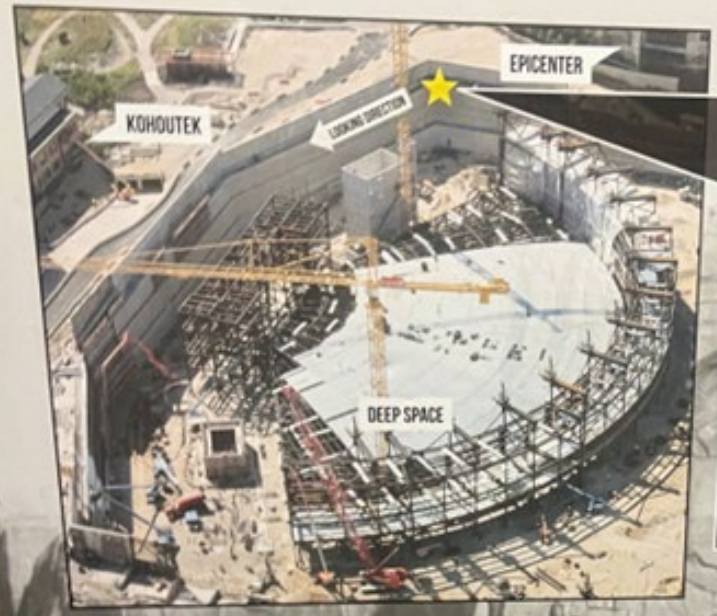
Wall Statistics:

- **80 ft deep**
- **1,600 feet long**
- **77,000 square ft**
- **2,819 soil & rock nails**
- **Doesn't leak**

In 2025 it is still the World’s Largest Underground Auditorium



DEEP SPACE
PERMANENT EARTH RETENTION WALL



YOU ARE HERE

As you look through the glass, you will see the permanent earth retention wall that surrounds Deep Space. The function of the wall is to resist the loads that Kohoutek, the parking ramp Yoda, and Epicenter impose on Deep Space. This allows Deep Space to be a free standing structure which minimizes the structural member sizes necessary and ultimately reduced construction costs.



Over 477,000 cubic yards of rock needed to be removed from the site to create the hole for Deep Space, which is enough to fill 146 Olympic sized pools. Blasts were held 3 times daily for 14 months.



Workers drill one of 2,800 holes for soil nails into bedrock. Each hole is 6" in diameter by 50' long and gets filled with grout and a threaded rod, also called a nail. If you lined up all of the nails end to end, they would be over 26 miles long.



A 16 inch thick wall is placed over the soil nails. It is almost 1,400 feet long, 60 feet tall, and has an area of 72,000 sqft. That means the wall is over 1/4 mile long and has an area equivalent to 1.5 football fields.

The image to the right shows a section through the center of the stage in Deep Space, roadway, Kohoutek, and the permanent earth retention wall (highlighted in pink). The "Building Load" arrow represents the lateral loads that are being put on the retaining wall by Kohoutek and the roadway. The thousands of soil nails counteract this force and allow Deep Space to be a free standing structure.

You are currently about 20 feet below grade between Deep Space and Yoda looking Northeast at the permanent retention wall towards Kohoutek.



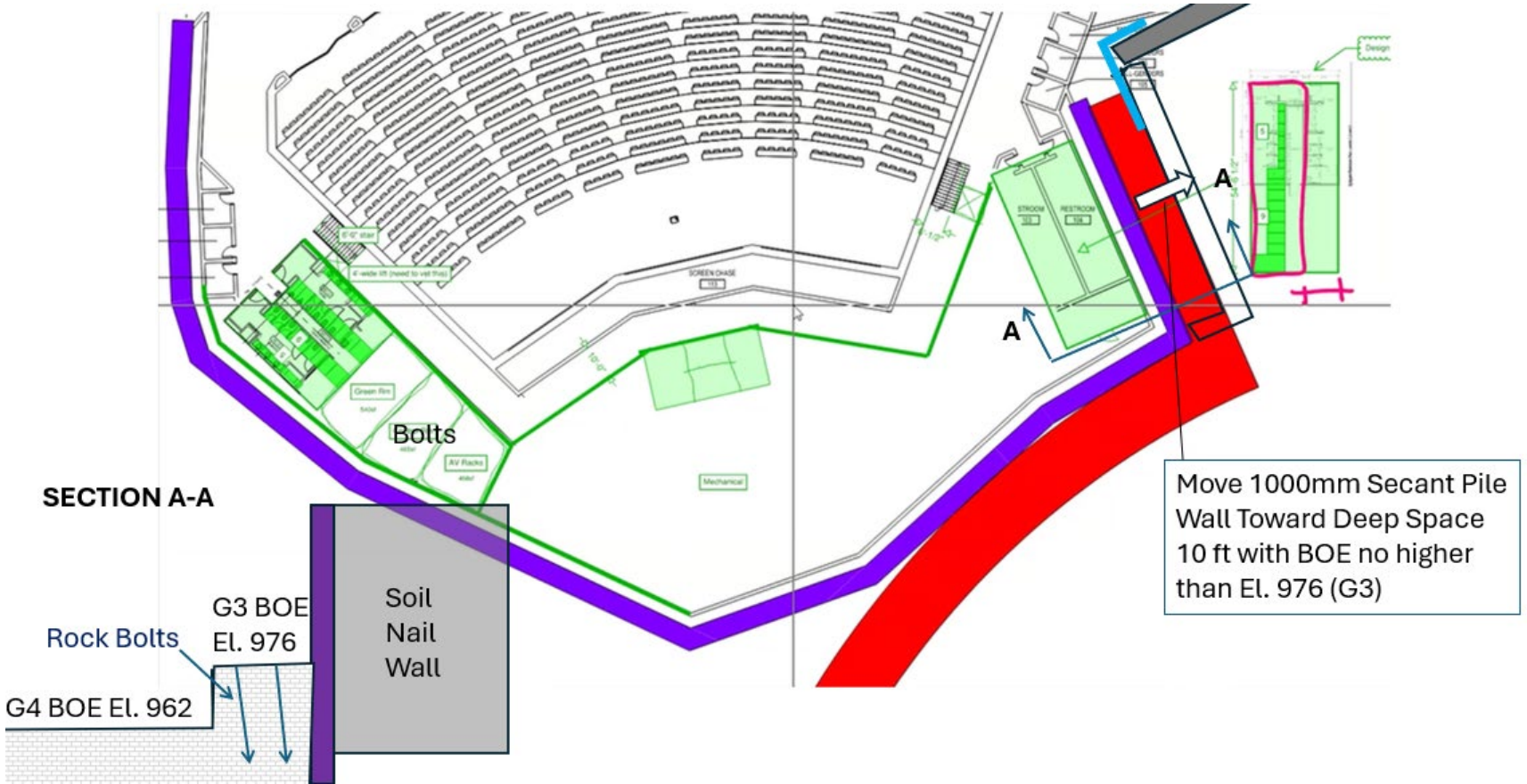
A Window to What is Hidden
or
You Can See our Work

2025 Deep Space Expansion



Current (2025) Challenge - Expansion

- 750 Person Underground Auditorium for Client CTOs and CFOs
- 47 ft Cantilevered 1000mm Tangent & 1500mm Secant Pile and Propped Walls to support Pluto and Deep Space less than 20 ft away
- Wall Shall Not Load Auditorium structure



Deep Space Expansion Progress – Fall 2025





Thank you very much – This has been an Honor

Questions?

Lyrik - Air Rights Parcel 12

Building Over the Massachusetts Turnpike
Foundation Design and Construction Challenges

Deep Foundation Institute (DFI)
50th Annual Conference - October 2025

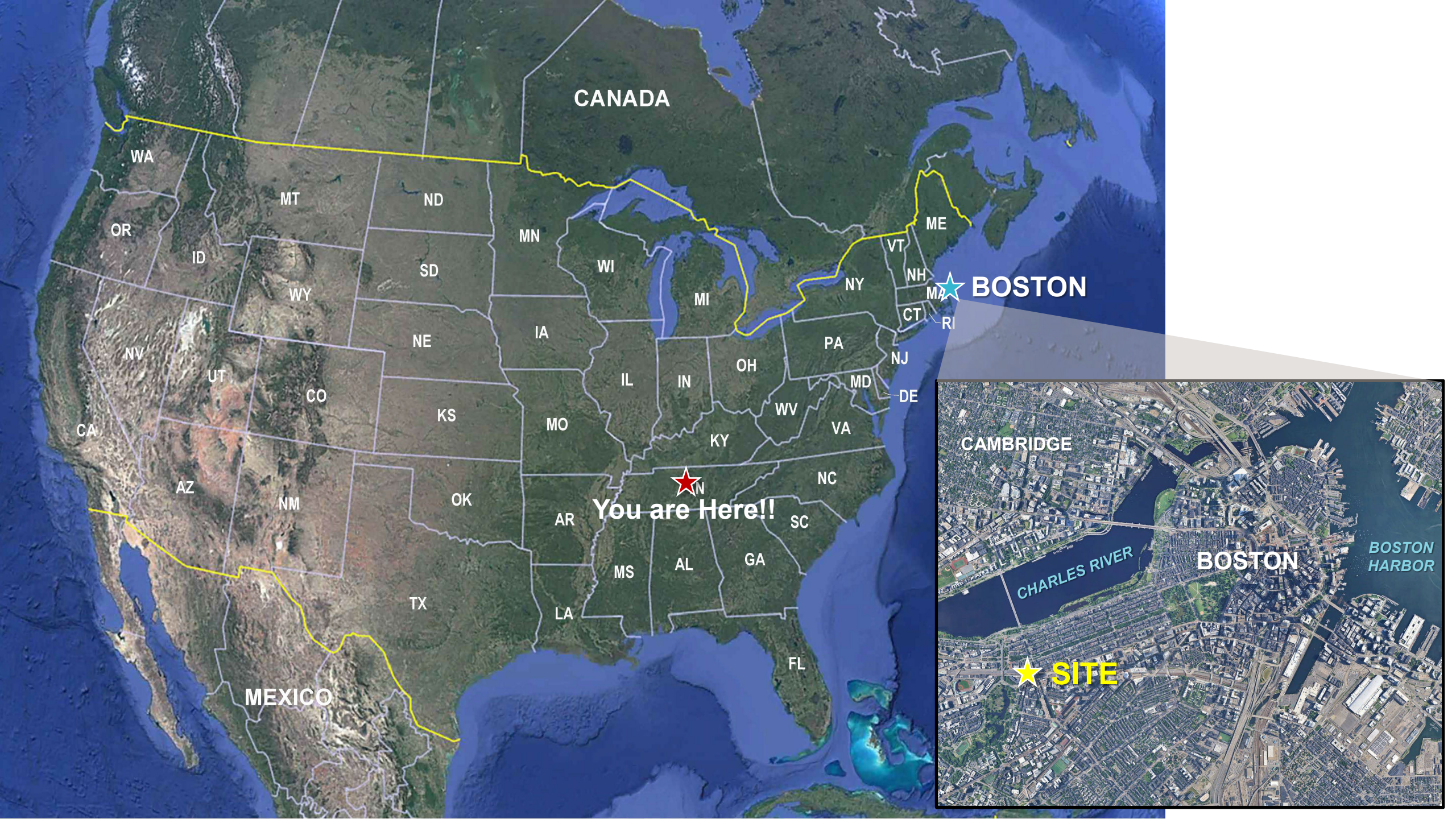
The opinions expressed in this presentation and on the following slides are solely those of the presenter(s) and not necessarily those of DFI. DFI does not guarantee the accuracy or reliability of the information nor endorse the recommendations offered by the presenter(s)

Collaboration was key to project success

- **Samuels & Associates:** Designated Developer
- **Massachusetts Department of Transportation** (MassDOT): Owner
- **Massachusetts Bay Transportation Authority** (MBTA): Owner
- **City of Boston:** location
- **Elkus Manfredi Architects**: Project architect
- **McNamara/Salvia Structural Engineers**: Structural engineer
- **Vanasse Hangen Brustlin (VHB)**: Civil and highway engineer, permits and planning
- **Howard Stein Hudson (HSH)**: Transportation planning and traffic management
- **WSP**: Mechanical, electrical, plumbing engineer
- **Suffolk**: Construction Manager
- **Trevi ICOS Corporation**: Foundation sub-contractor
- **Keller North America**: Foundation sub-contractor
- **J DeRenzo Company**: Site work contractor
- **GeoInstruments**: Instrumentation specialist
- **Feldman Geospatial**: Surveyor

Lyrik – Air Rights Parcel 12 is the first Air Rights project completed in Boston's Back Bay in over 40 years



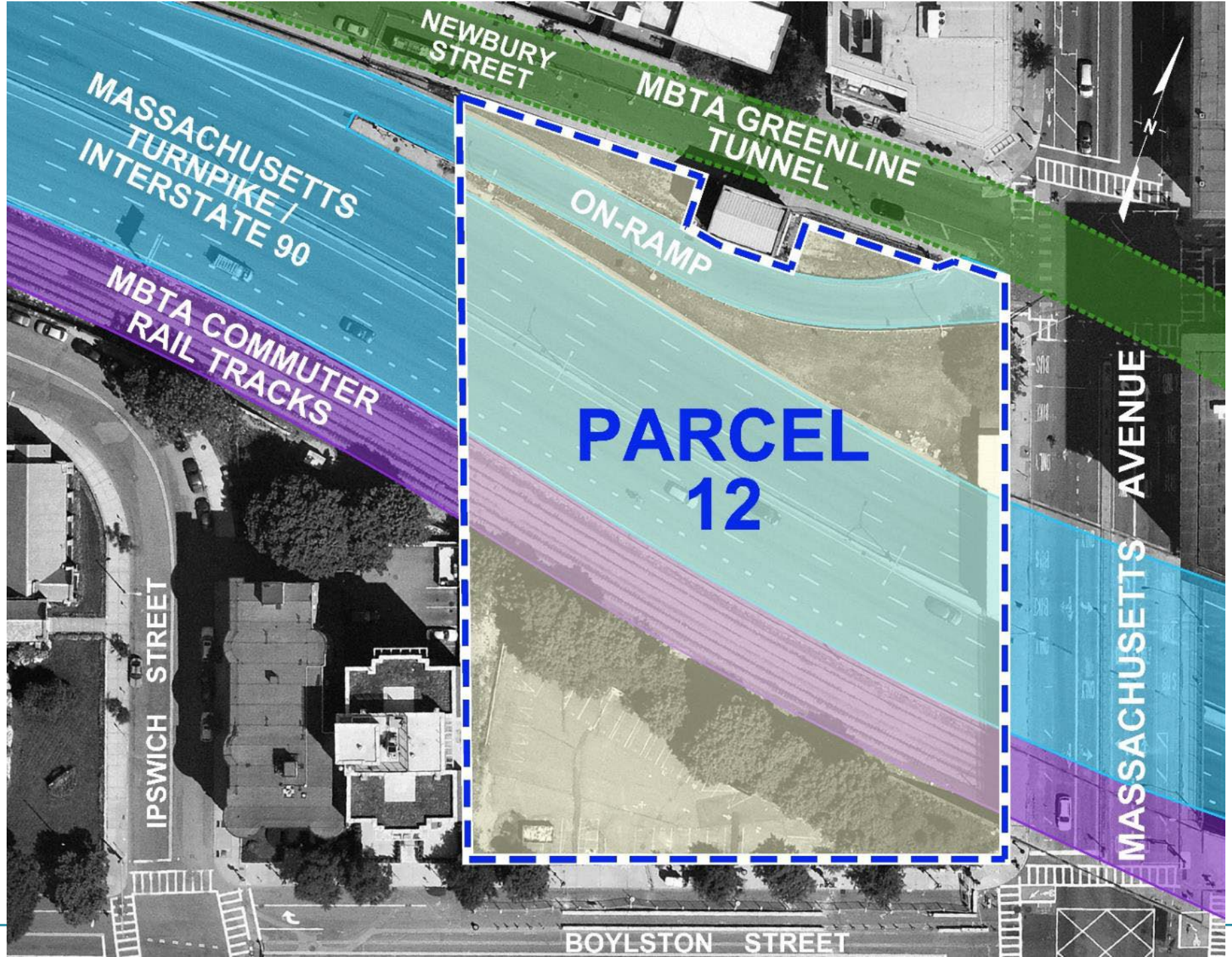

CANADA
WA
OR
CA
ID
NV
MT
WY
UT
AZ
CO
NM
ND
SD
NE
KS
OK
TX
MN
IA
MO
AR
LA
WI
IL
MS
MI
IN
KY
TN
AL
OH
WV
GA
FL
SC
NC
VA
PA
NY
MD
DE
NJ
CT
RI
MA
VT
NH
ME
BOSTON
You are Here!!
MEXICO
CAMBRIDGE
CHARLES RIVER
BOSTON
BOSTON HARBOR
SITE

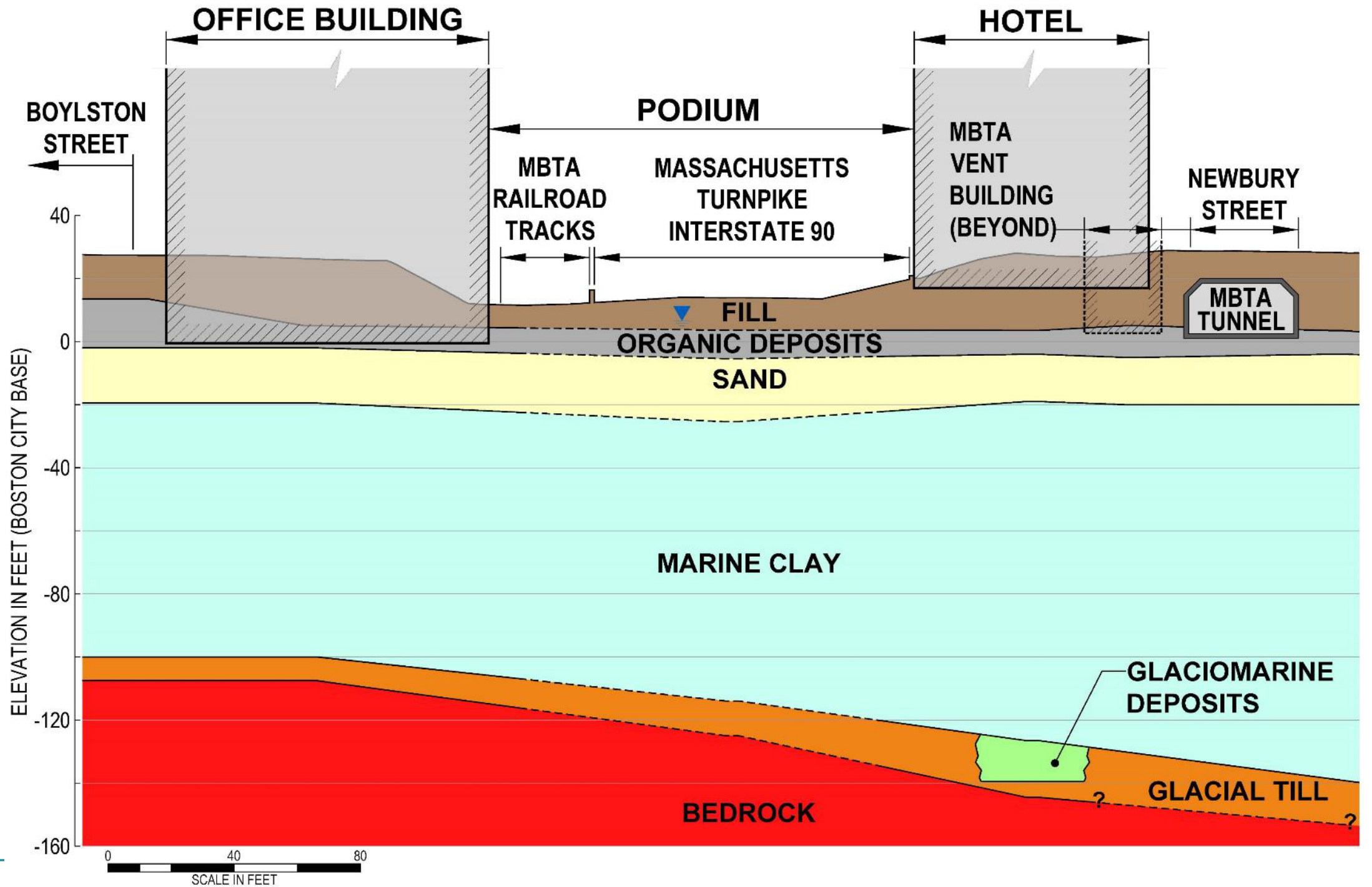

Harvard
Tufts
MIT
SITE
Haley & Aldrich
Office
CAMBRIDGE
CHARLES RIVER
MASSACHUSETTS AVENUE
MASS PIKE (I-90)
BOSTON

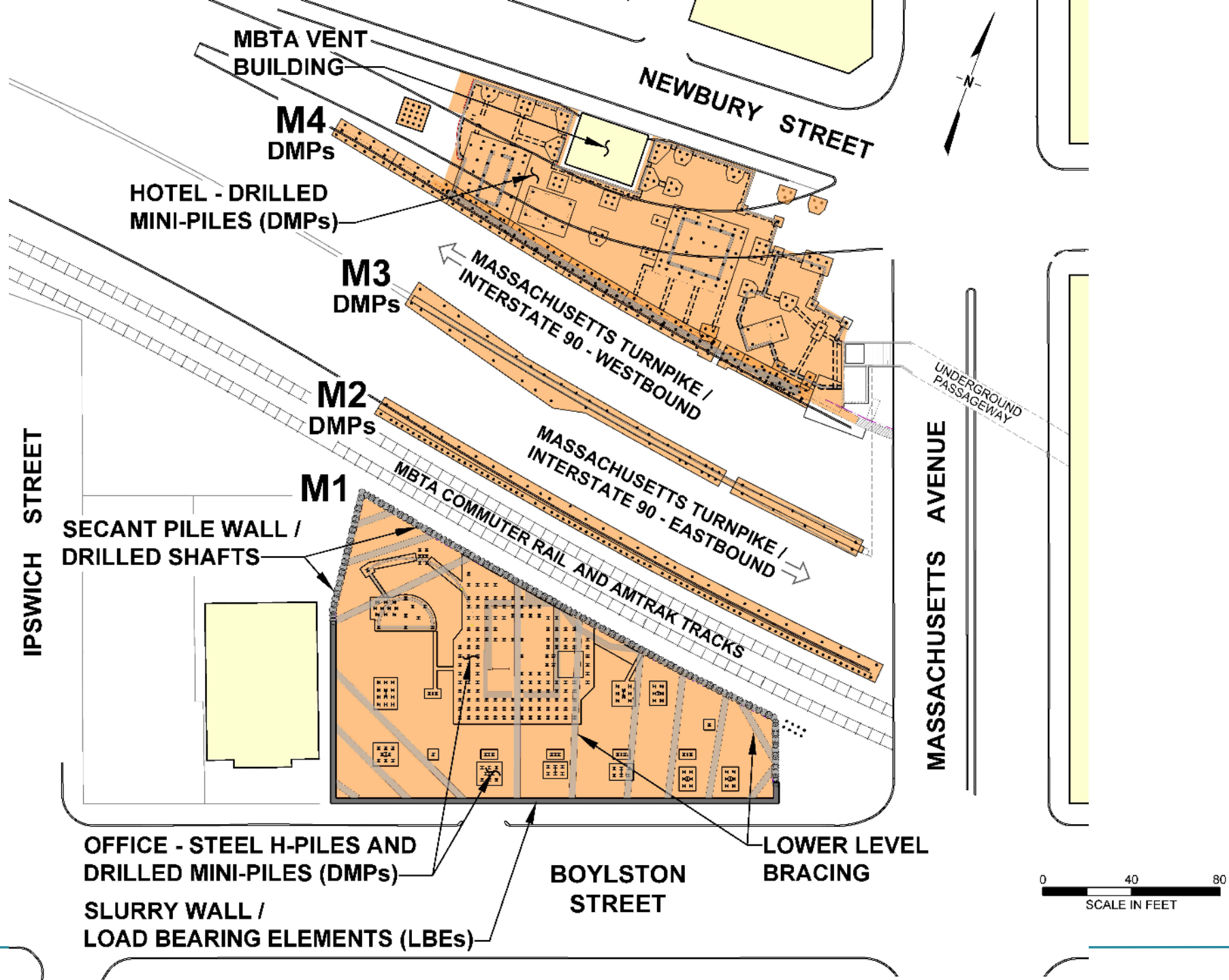
MassDOT envisioned Air Rights parcels to fill a gaping hole created by I-90 and reconnect neighborhoods

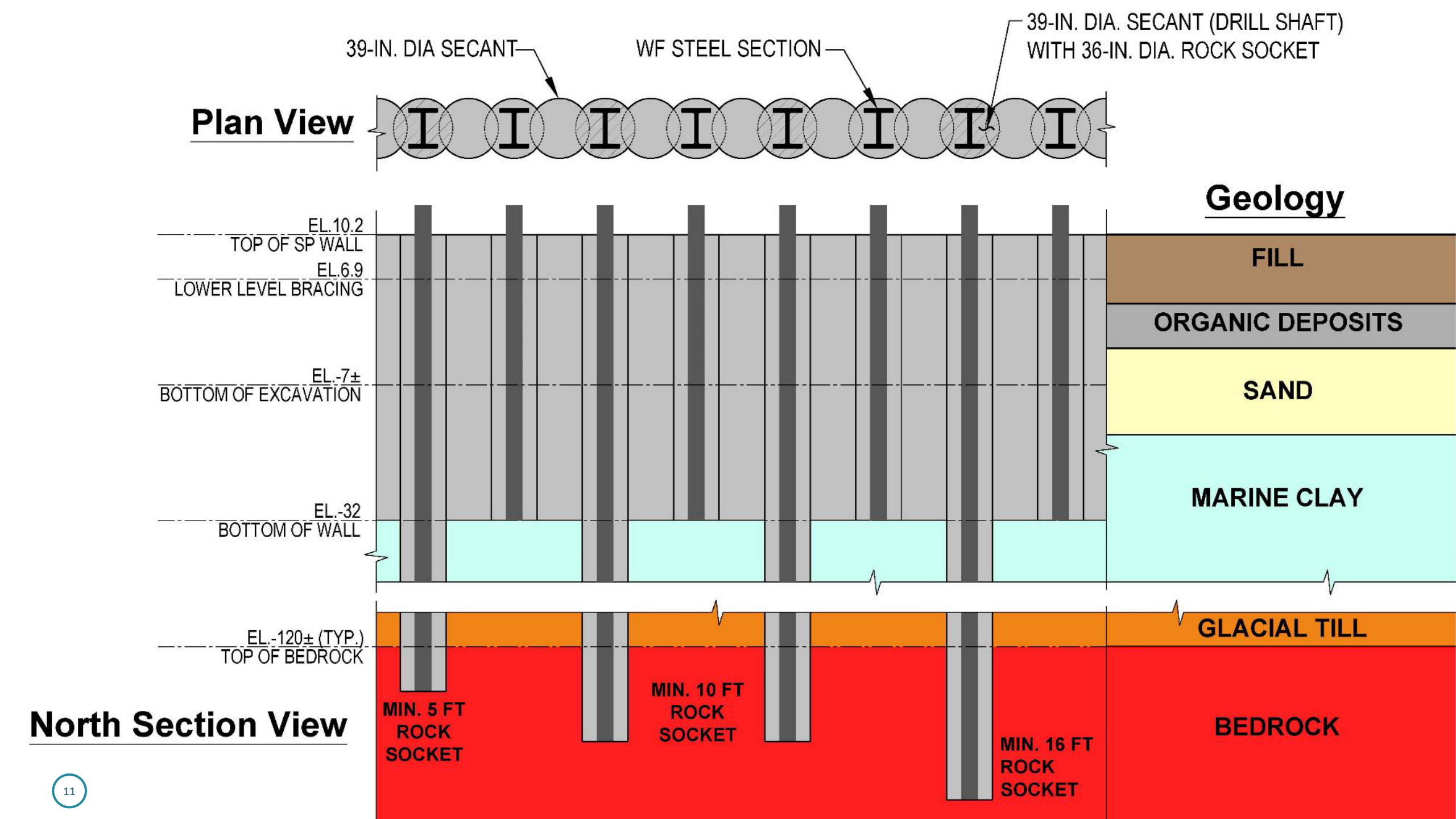



PARCEL 12
PARCEL 13
MASSACHUSETTS AVENUE
NEWBURY STREET
BOYLSTON STREET


NEWBURY STREET
MBTA GREENLINE TUNNEL
MASSACHUSETTS TURNPIKE / INTERSTATE 90
ON-RAMP
MBTA COMMUTER RAIL TRACKS
PARCEL 12
IPSWICH STREET
MASSACHUSETTS AVENUE
BOYLSTON STREET
N


OFFICE BUILDING
HOTEL
BOYLSTON STREET
PODIUM
MBTA RAILROAD TRACKS
MASSACHUSETTS TURNPIKE INTERSTATE 90
MBTA VENT BUILDING (BEYOND)
NEWBURY STREET
MBTA TUNNEL
FILL
ORGANIC DEPOSITS
SAND
MARINE CLAY
GLACIOMARINE DEPOSITS
GLACIAL TILL
BEDROCK
ELEVATION IN FEET (BOSTON CITY BASE)
40
0
-40
-80
-120
-160
0
40
80
SCALE IN FEET


MBTA VENT BUILDING
NEWBURY STREET
M4
DMPs
HOTEL - DRILLED MINI-PILES (DMPs)
M3
DMPs
MASSACHUSETTS TURNPIKE / INTERSTATE 90 - WESTBOUND
M2
DMPs
MASSACHUSETTS TURNPIKE / INTERSTATE 90 - EASTBOUND
UNDERGROUND PASSAGEWAY
IPSWICH STREET
M1
SECANT PILE WALL / DRILLED SHAFTS
MBTA COMMUTER RAIL AND AMTRAK TRACKS
MASSACHUSETTS AVENUE
OFFICE - STEEL H-PILES AND DRILLED MINI-PILES (DMPs)
SLURRY WALL / LOAD BEARING ELEMENTS (LBEs)
BOYLSTON STREET
LOWER LEVEL BRACING
N
0
40
80
SCALE IN FEET


39-IN. DIA SECANT
WF STEEL SECTION
39-IN. DIA. SECANT (DRILL SHAFT)
WITH 36-IN. DIA. ROCK SOCKET
Plan View
Geology
EL.10.2
TOP OF SP WALL
EL.6.9
LOWER LEVEL BRACING
EL.-7±
BOTTOM OF EXCAVATION
EL.-32
BOTTOM OF WALL
EL.-120± (TYP.)
TOP OF BEDROCK
FILL
ORGANIC DEPOSITS
SAND
MARINE CLAY
GLACIAL TILL
BEDROCK
MIN. 5 FT ROCK SOCKET
MIN. 10 FT ROCK SOCKET
MIN. 16 FT ROCK SOCKET
North Section View

Conditions at Start of Work



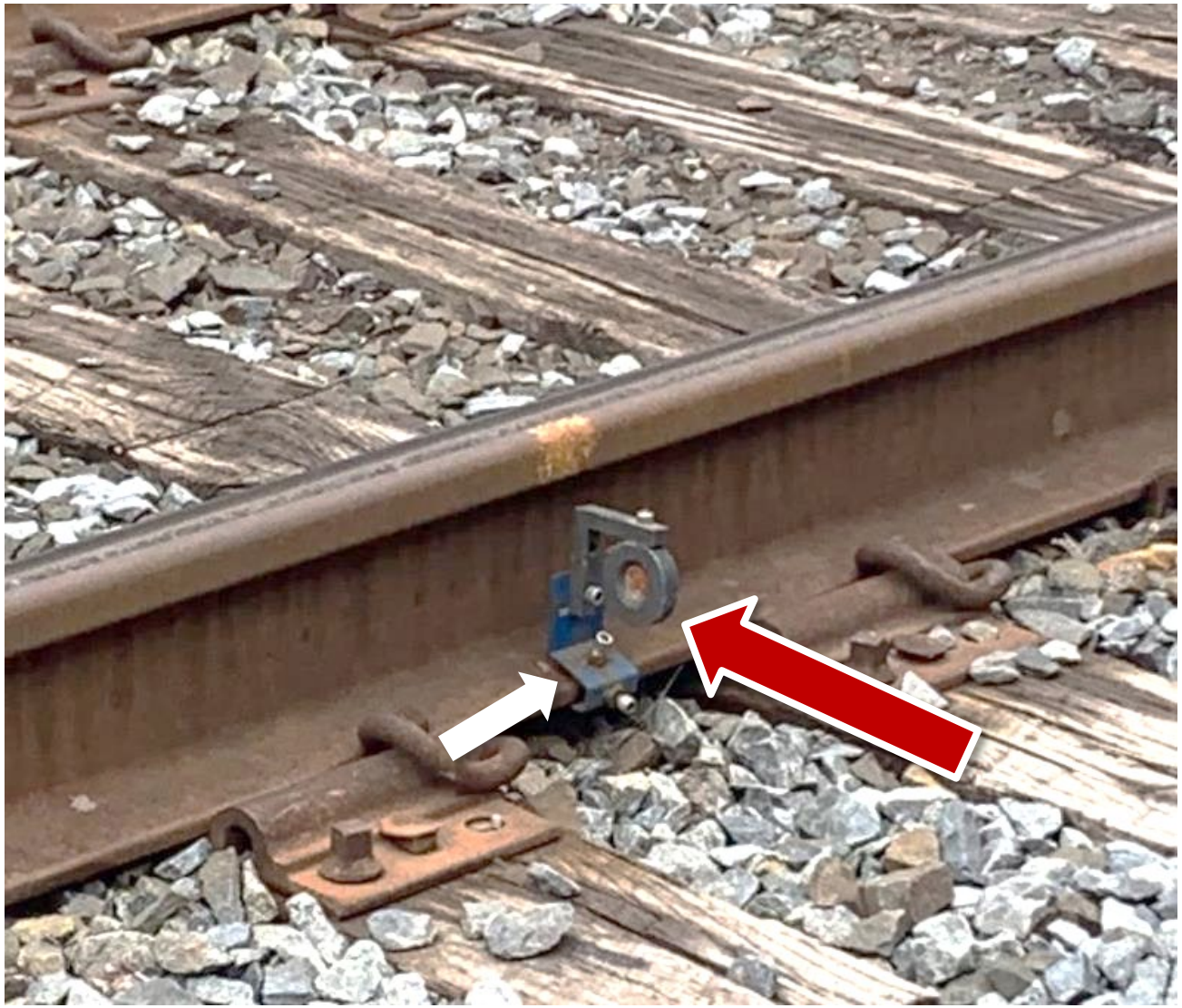

LIEBHERR
HS 885
LB 36
410


J. DERENZO CO.





Automated and remote monitoring program on tracks

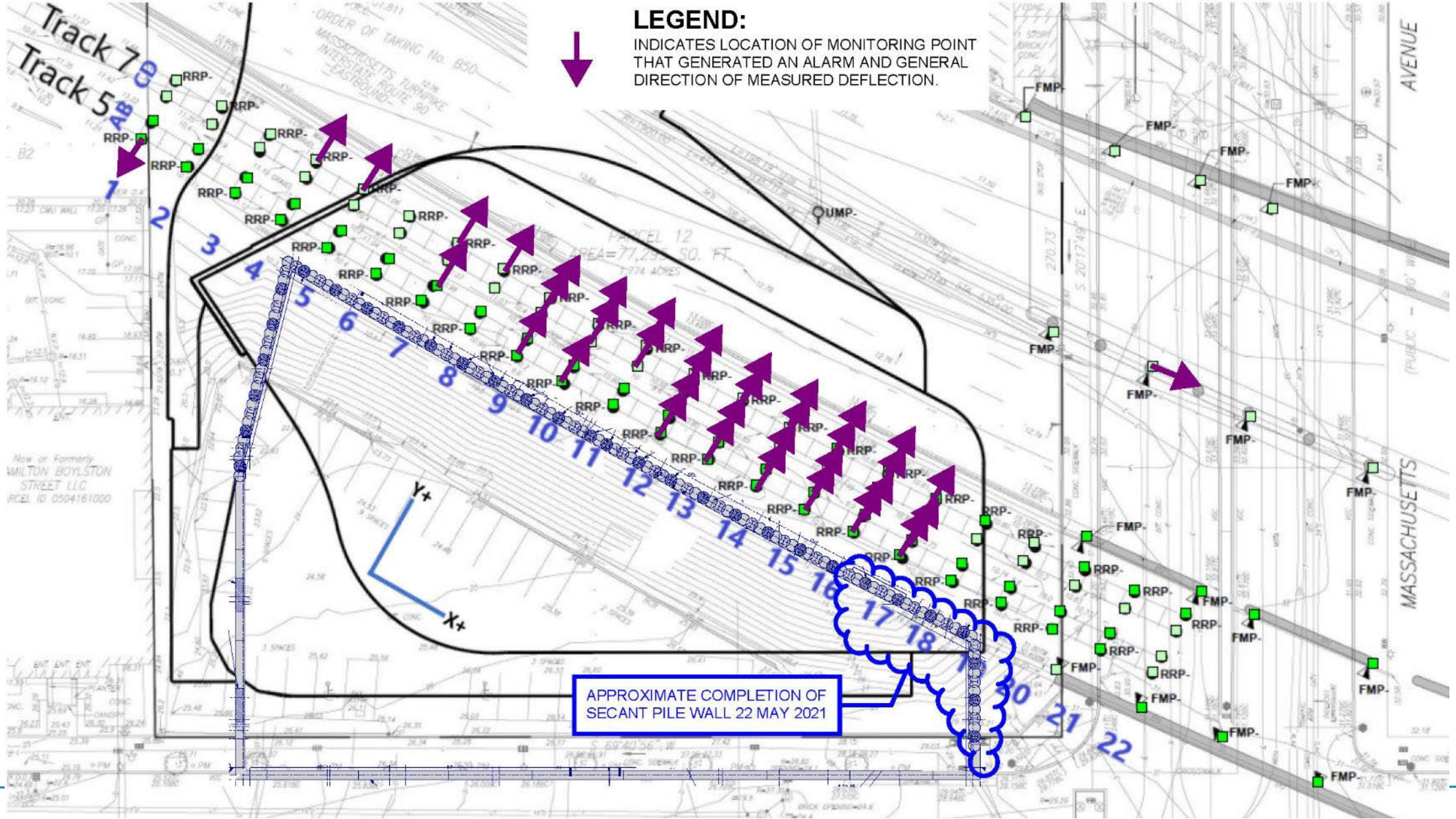


Rail Target

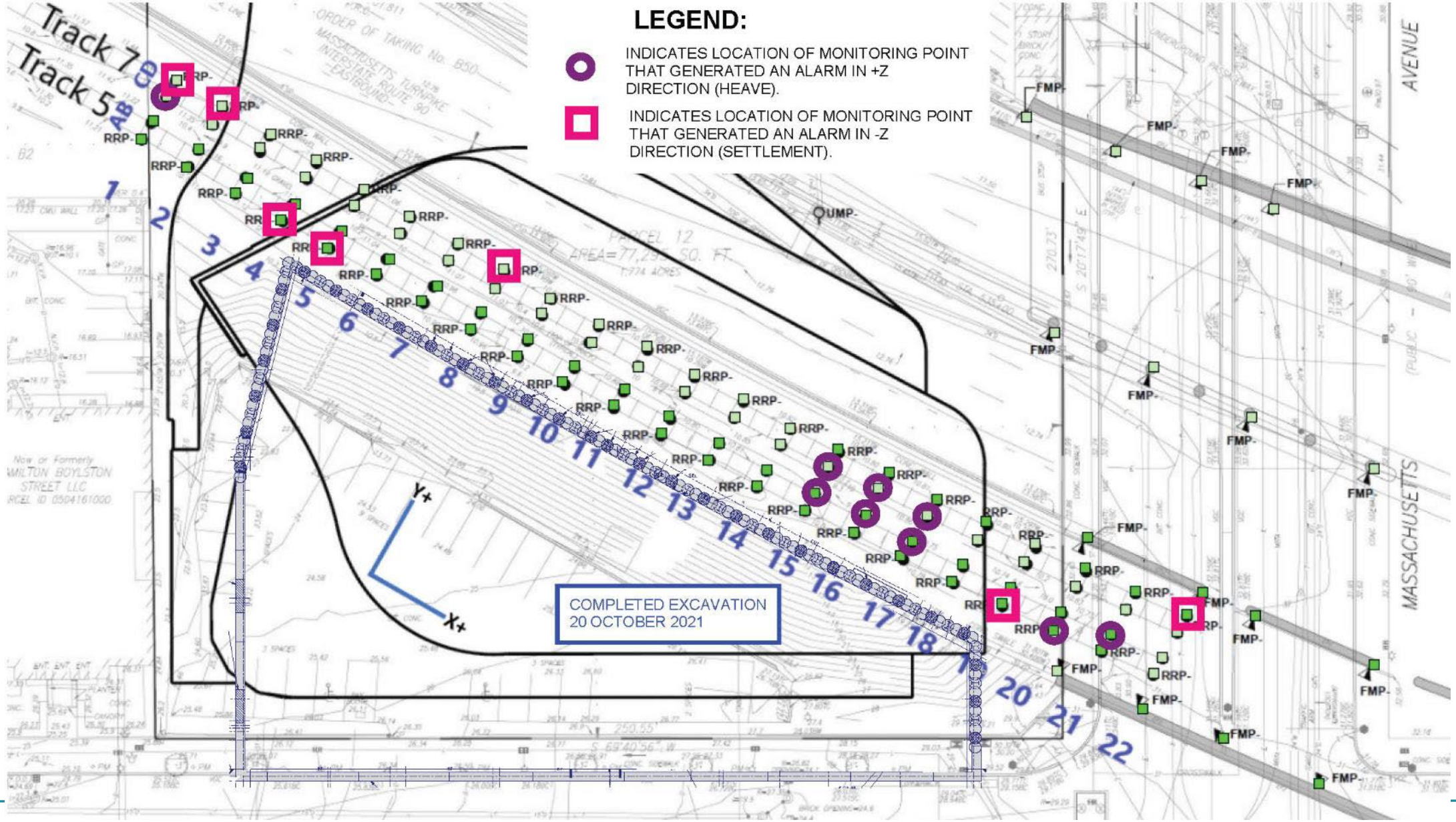


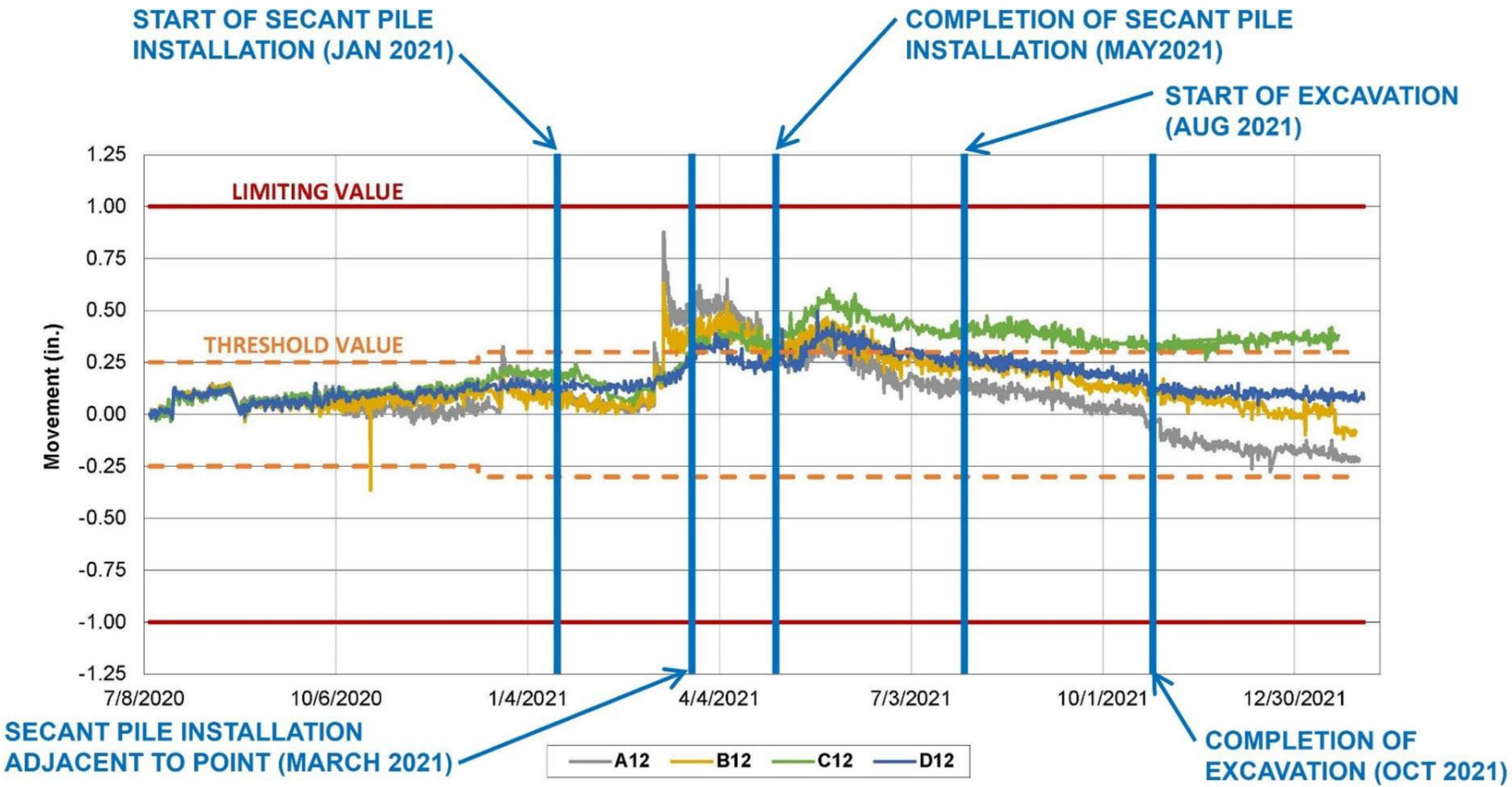
Automated Total Station (AMTS)

May 2021 – SPW Completed – Lateral Movement

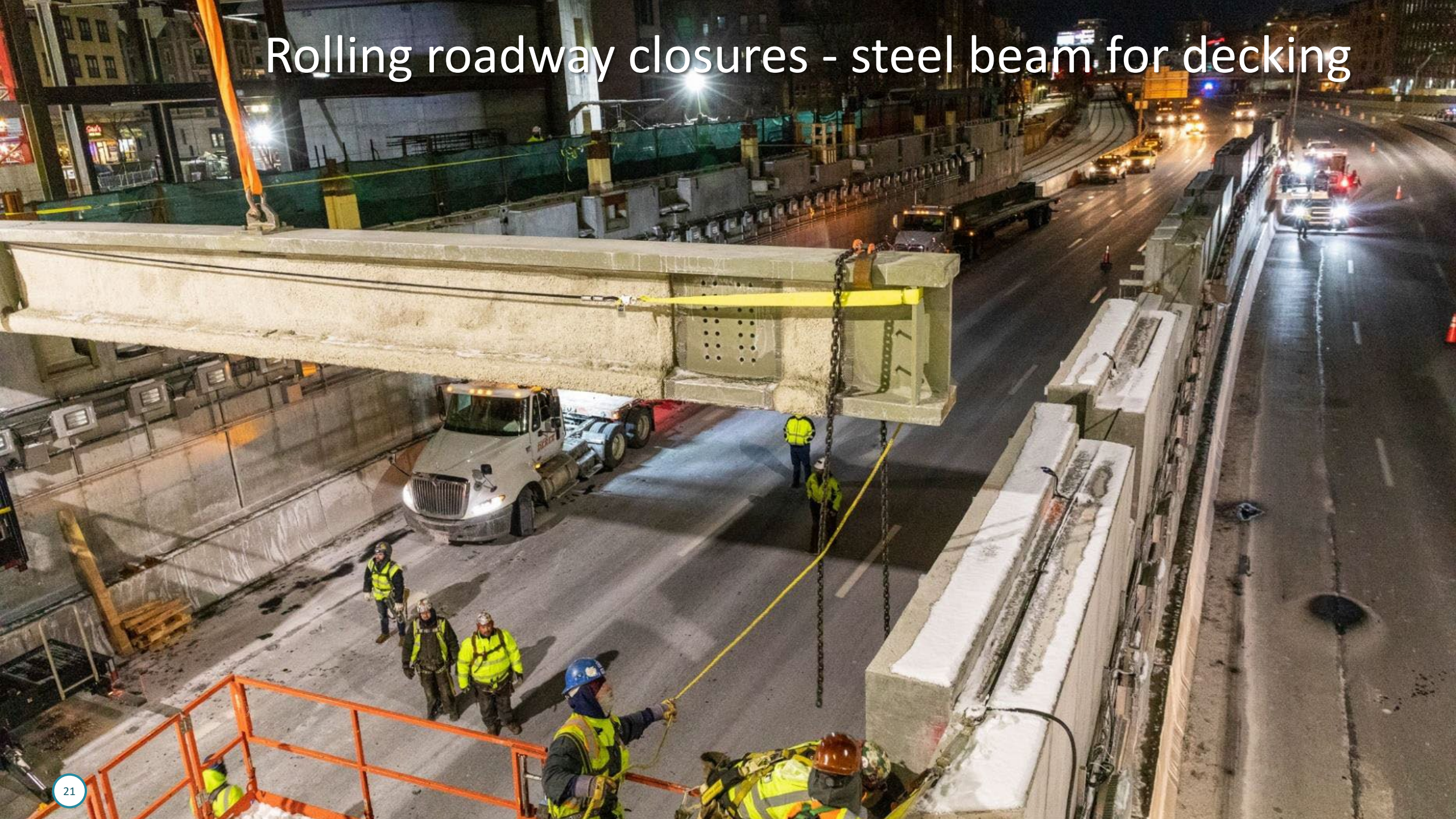


20 October 2021 – Excavation – Vertical Movement




START OF SECANT PILE INSTALLATION (JAN 2021)
COMPLETION OF SECANT PILE INSTALLATION (MAY2021)
START OF EXCAVATION (AUG 2021)
LIMITING VALUE
THRESHOLD VALUE
Movement (in.)
1.25
1.00
0.75
0.50
0.25
0.00
-0.25
-0.50
-0.75
-1.00
-1.25
7/8/2020
10/6/2020
1/4/2021
4/4/2021
7/3/2021
10/1/2021
12/30/2021
A12
B12
C12
D12
SECANT PILE INSTALLATION ADJACENT TO POINT (MARCH 2021)
COMPLETION OF EXCAVATION (OCT 2021)

Rolling roadway closures - steel beam for decking





Public benefits



Encouraging STEM education



Encouraging Careers in AEC Industry





Questions?..



MGorczyca@HaleyAldrich.com

HALEY ALDRICH

Investigation of Vibratory Hammer Installation for Deeply Embedded Ring Anchor System in Sand

Song Qin

Texas A&M University

The opinions expressed in this presentation and on the following slides are solely those of the presenter(s) and not necessarily those of DFI. DFI does not guarantee the accuracy or reliability of the information nor endorse the recommendations offered by the presenter(s).

Introduction

Suction Caisson

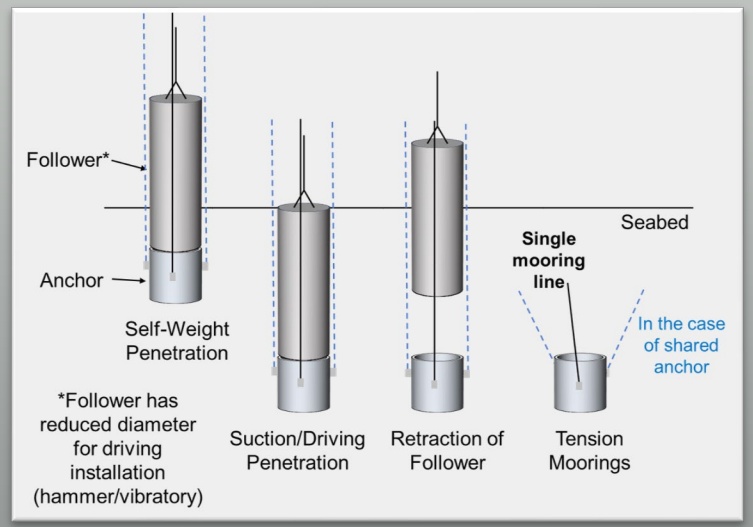
- D: 5m × L: 25m
- Complex Manufacturing
- High Cost
- Not suitable for clean sand or stratified soils



VS

Deeply Embedded Ring Anchor (DERA)

- D: 2.5m × L: 5m (1/4th size)
- Simple Manufacturing
- Lower Cost (Cut CAPEX by up to 75%)
- Suitable for various soil profile (Suction installation in clay; Vibratory installation in sand)



Introduction

Vibratory Hammer Installation

- Reduced noise emission
- Lower cyclic stress
- Rapid installation
- Effective in heterogeneous soil profiles
- Ability to extract and reposition the anchor

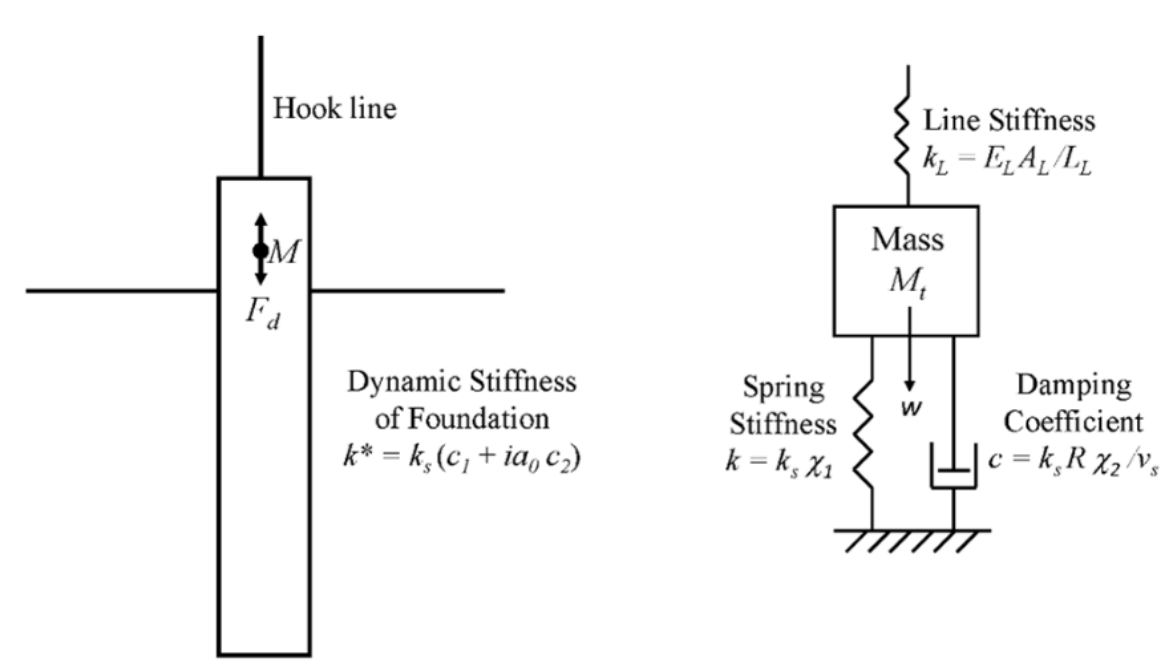
Hypothesis:

- Follower and DERA has same diameter and wall thickness
- Follower-DERA system behaves like pile
- Penetration in sand governed by whether cyclic shear strain exceeds a threshold



Source: Offshore-mag.com

Vibratory Installation of DERA



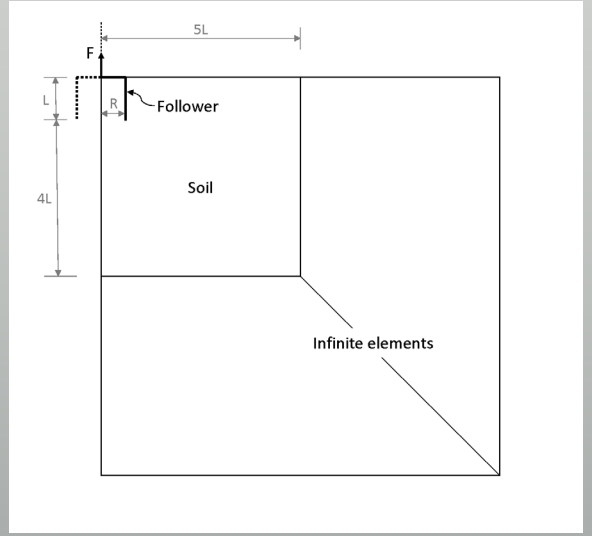
Soil-structure interaction model (Aubeny, 2023)

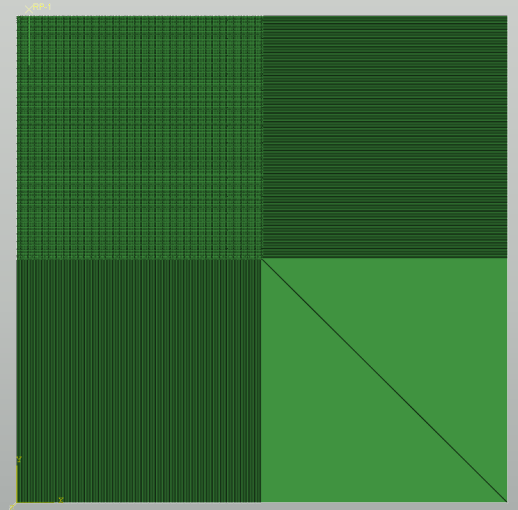
- Inertial force: $F_d = m_e e \Omega^2 sin\Omega t$ (Jonker,1987)
- k_s: static stiffness
- c: the effect of radiation damping
- R: the foundation radius
- V_s: the wave propagation velocity
- χ1 and χ2: the effects of frequency-dependent dynamics stiffness and radiation damping

Finite-element Model

- Axisymmetric continuum elements (CAX4R) for soil mass
- Axisymmetric shell elements (SAX1) for the DERA and follower
- Bottom and side boundary comprised infinite elements (CINAX4)
- D = 4 m, L/D =0, 2, 5, 10
- Evaluate k_s, $\chi 1$ and $\chi 2$

	Soil Properties
Density ρ	2,200 kg/m3
Young's Modulus E	228113.6 kPa
Poisson's Ratio μ	0.3
structural damping factor S	0.1

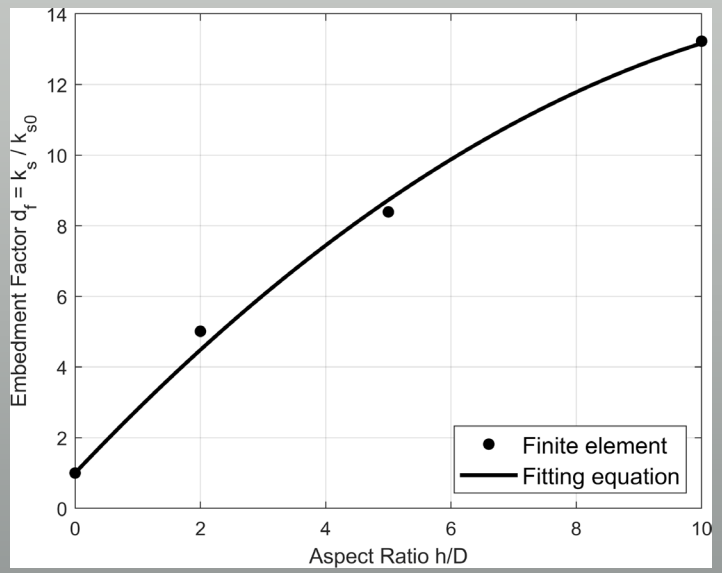




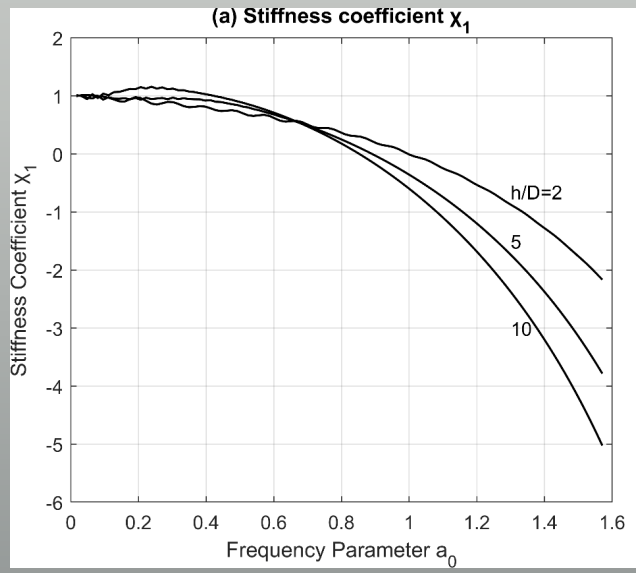
FE model using ABAQUS, 2021

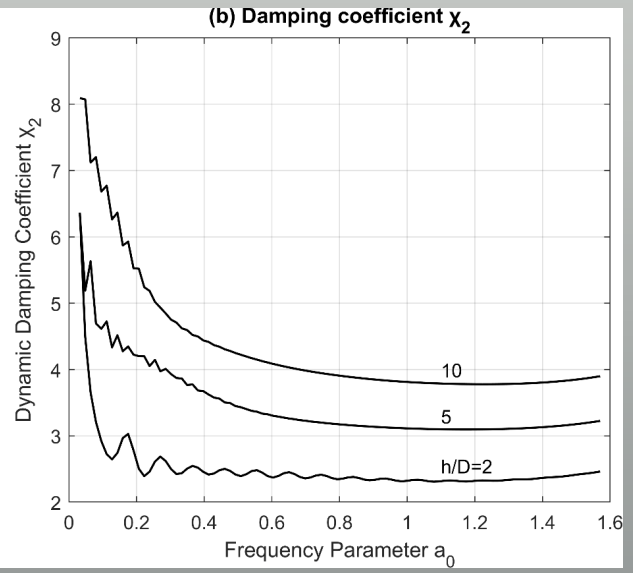
Finite-element Model

- Based on static finite element estimates of stiffness
- $k_s = d_f k_{s0} = \frac{d_f s_f G R}{1-\mu}$
- d_f = embedment factor = -0.066 $(L/D)^2$ + 1.874 L/D +1

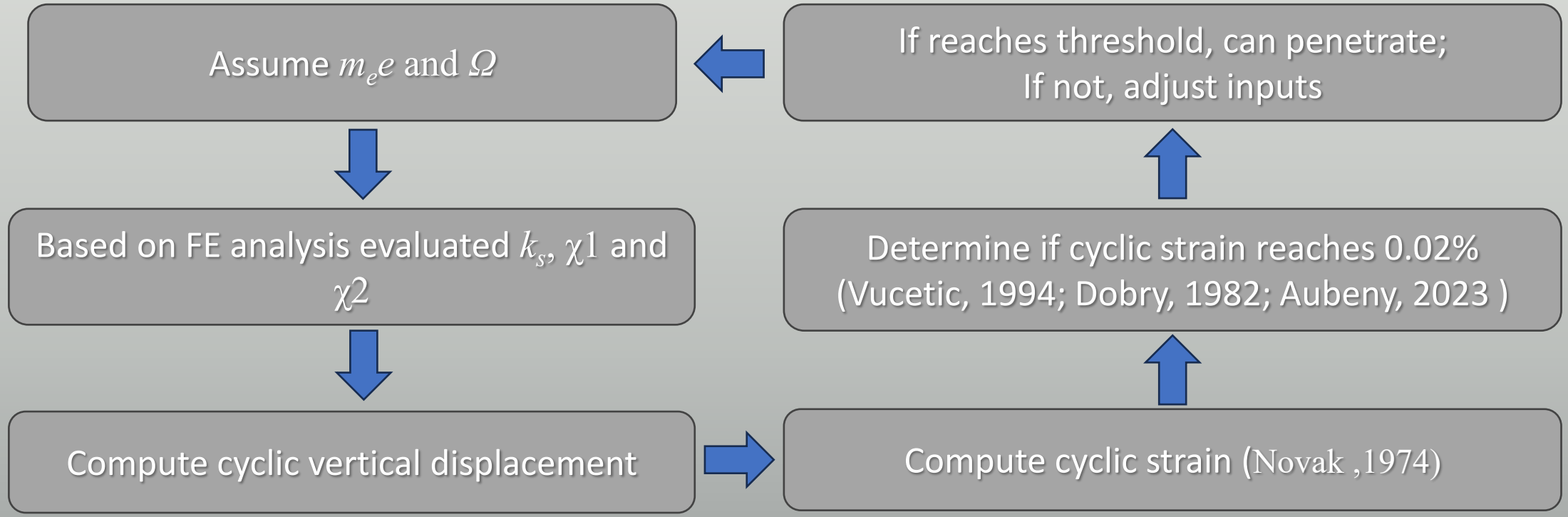


- The real and imaginary components of the follower displacement, a and b, can be calculated using FE steady state analysis.
- With the known a and b, the equivalent stiffness k and damping c are computed.
- $\chi_1 = 1 + c_1 a_0 + c_2 a_0^2$ $\chi_2 = 1.7006 + 1.144 \ln\left(\frac{L}{D}\right)$





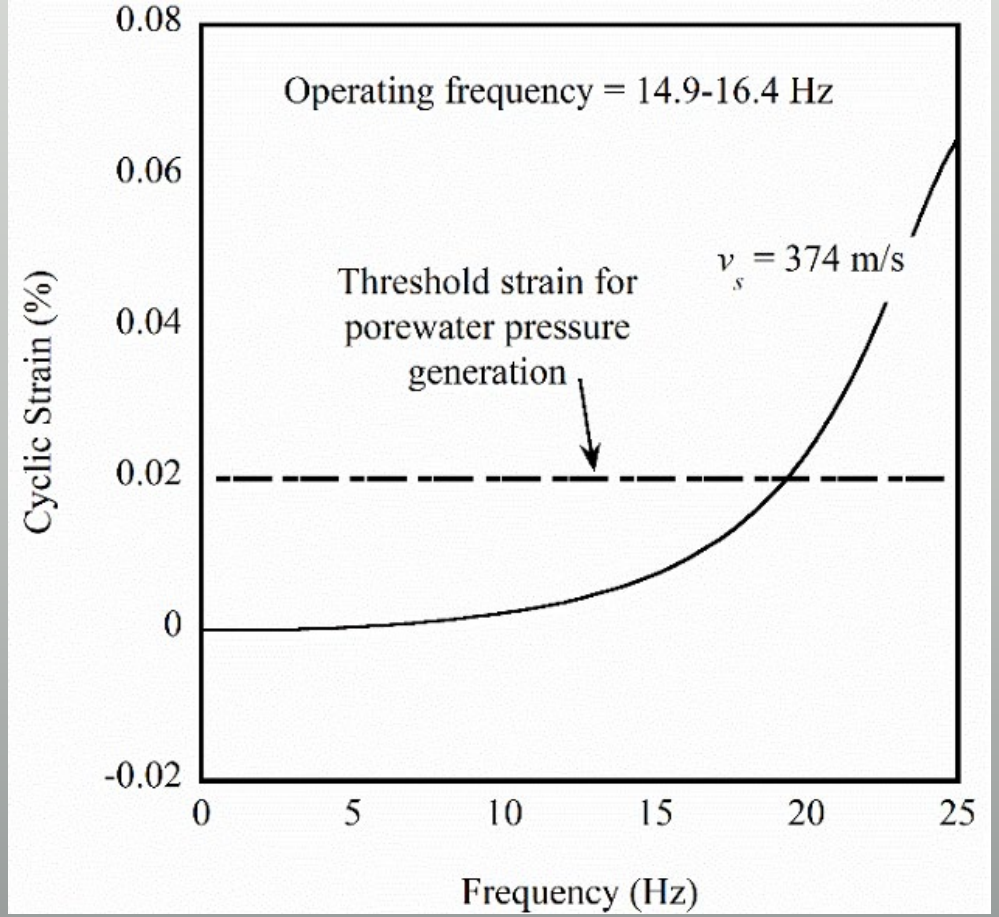
Vibratory Installation Model



$$DLF = \frac{1}{\sqrt{\left[1-\left(\frac{\Omega}{\omega_n}\right)^2\right]^2 + 4\beta_v{}^2\left(\frac{\Omega}{\omega_n}\right)^2}}$$

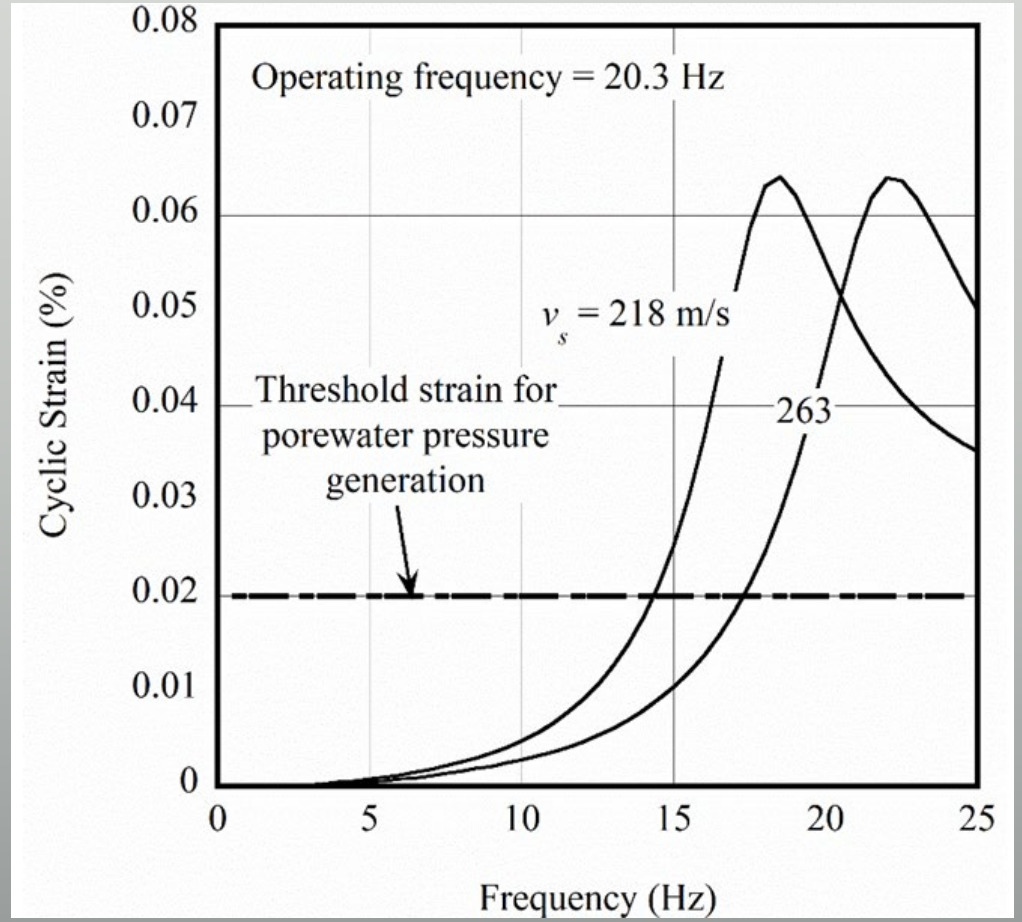
Model Validation

- Vibratory pile installation tests conducted in Rotterdam Harbor (Van Dorp et al., 2019).
- According to the model, 19 Hz would be required to initiate porewater pressure generation.
- Actual operating frequency:14.9 - 16.4 Hz.
- The discrepancy maybe due to the uncertainty in soil property inputs and assumed cyclic strain (0.02%).



Model Validation

- Jonker (1988) presents a case study involving the vibratory installation of four large dolphins near Copenhagen, Denmark.
- Operating frequency: 20 Hz.
- Model estimated required frequency: 14 - 17 Hz.
- The discrepancy maybe due to the uncertainties in inputs, or operator applied a frequency higher than necessary.



Application of Model to DERA System

- Load target ~15 MN;
- Three soil types
- DERA D ~2.5–2.9 m;
- tip depth ~15D → ~37.5–43.5 m;
- follower length accordingly

Case	Soil type	Relative density (%)	Shear wave velocity (m/s)	DERA diameter (m)	Anchor tip embedment depth (m)	Follower length (m)
1	Very dense sand	95	350	2.5	37.5	33.75
2	Dense sand	80	300	2.7	40.5	36.45
3	Medium dense sand	65	250	2.9	43.5	39.15

DERA Installation

Case	DERA diameter (m)	Anchor tip embedment depth (m)	Required hammer size (tonne-m)		
			16 Hz	20 Hz	23 Hz
1	2.5	37.5	0.533	0.293	0.192
2	2.7	40.5	0.394	0.182	0.092
3	2.9	43.5	0.213	0.060	0.036

APE model	Type	Eccentric mass (tonne-m)	Maximum frequency (Hz)
300	Standard	0.06625	23.3
400	Standard	0.13329	22.5
600	Standard	0.19817	22.5
400	Tandem	0.26499	22.5
600	Tandem	0.39633	22.5

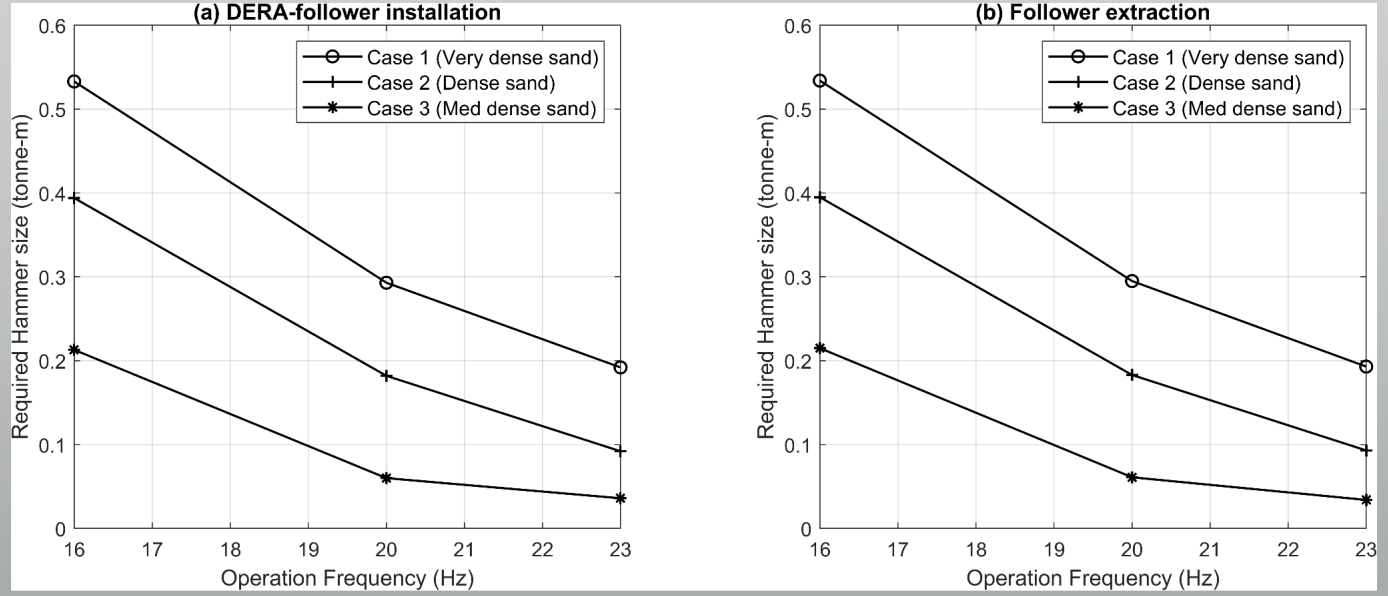
Follower Extraction

- Higher soil density and lower operating frequencies are associated with increased hammer size requirements.
- The hammer size needed for follower extraction is comparable to that required for DERA installation .

Case	Follower diameter (m)	Follower length (m)	Required hammer size (tonne-m)		
			16 Hz	20 Hz	23 Hz
1	2.5	33.75	0.534	0.295	0.193
2	2.7	36.45	0.395	0.183	0.093
3	2.9	39.15	0.215	0.061	0.034

Discussion

- The required hammer size increases with sand density.
- Lower operating frequencies necessitate larger hammer sizes to achieve sufficient cyclic strain.
- DERA–follower installation (Figure a) and follower extraction (Figure b) reveals a consistent trend.
- However, the hammer sizes required for follower extraction are slightly greater than those for installation.
- The findings confirm the technical feasibility of installing and extracting the DERA–follower system in sandy soils using vibratory hammers.



Conclusion

- The vibratory installation model effectively predicted the hammer size needed to generate porewater pressure in sands. The results confirm that the DERA can be driven to its target depth using currently available vibratory hammers.
- For operating frequencies of 20 Hz or greater, the required hammer eccentric moment for DERA installation does not exceed 0.3 tonne-m. This falls within the performance capabilities of commercially available vibratory hammers.
- Although follower extraction requires slightly higher hammer sizes compared to installation, it remains feasible within the range of standard equipment.
- Further research should investigate other soil types, such as clays and interbedded deposits. Additionally, the effects of vibratory driving-induced stress levels on the long-term fatigue performance of the DERA system warrant further examination.

Acknowledgements and Reference

The authors would also like to acknowledge the support from the National Science Foundation, award numbers CMMI-1936901 and TI-2214009, and the Department of Energy, award number DE-SC0040602.

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Thank you!

Q&A

PLENARY SESSION 2

The Golden Age



Dan Brown

Consultant

2017-2018

50 Years of Drilled Shafts

Dan Brown, P.E.

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Construction Equipment & Methods

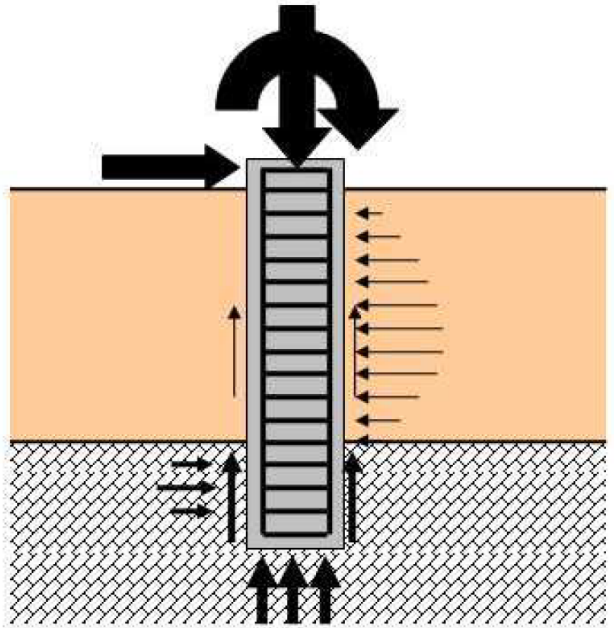












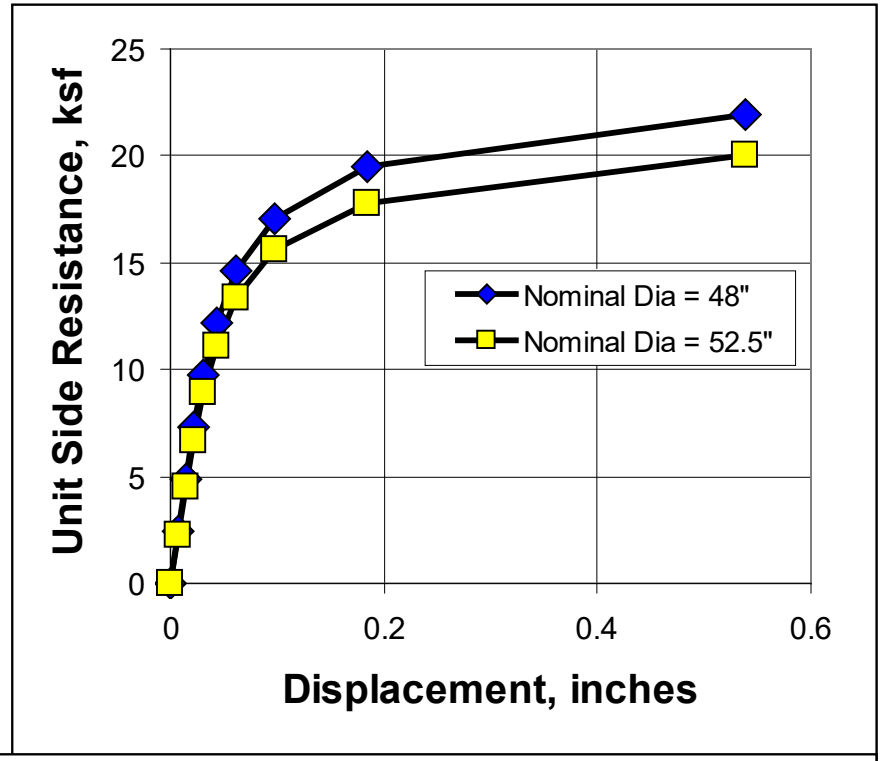
Design:
LRFD,
Seismic,
Lateral Loading

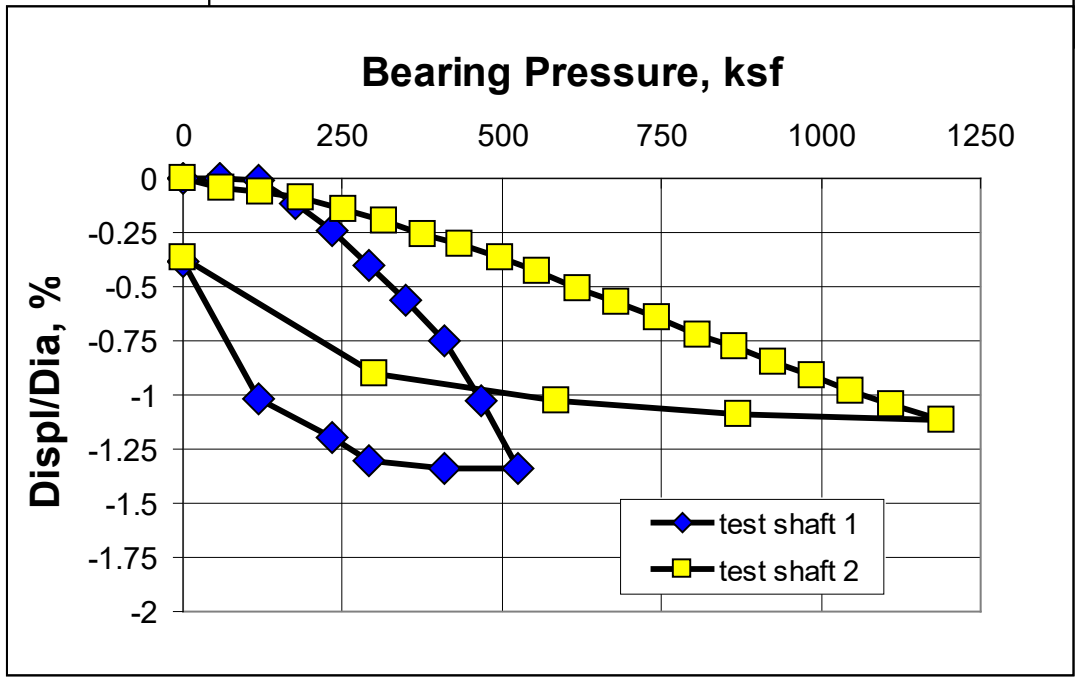




Load Testing



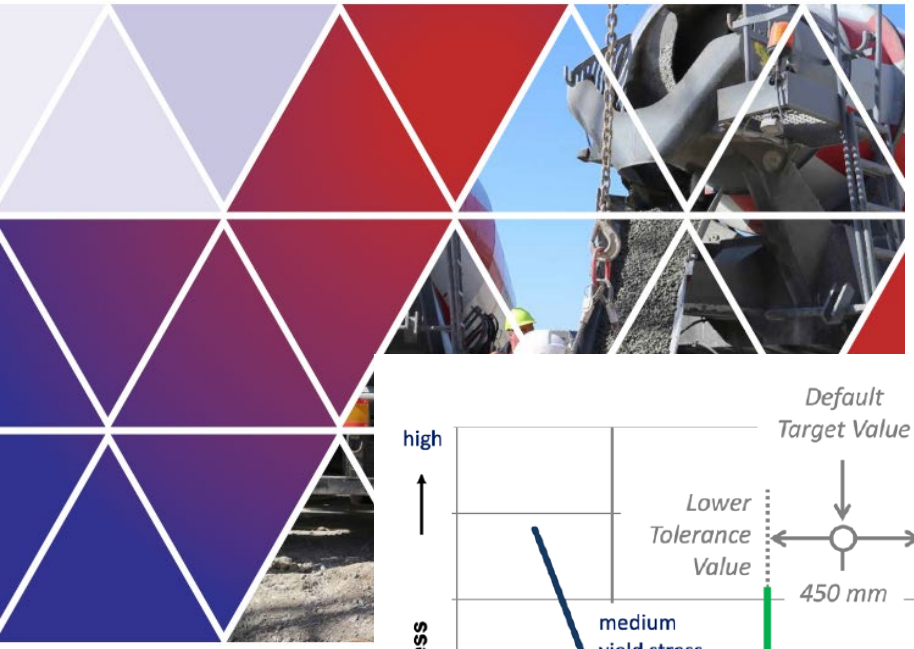




Quality Control / Assurance

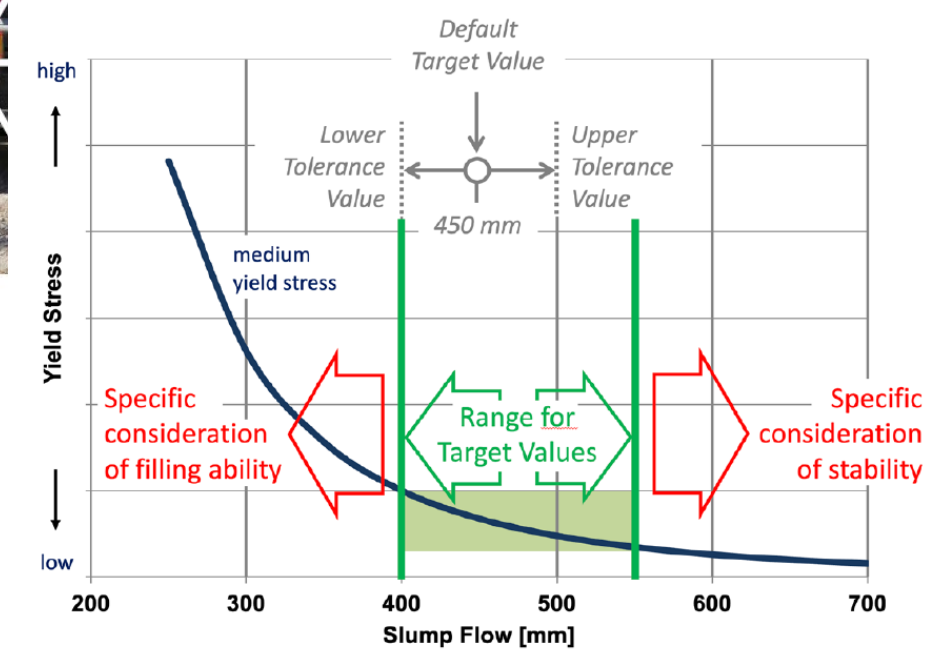
Guide to
Tremie Concrete
for Deep Foundations

By the joint **EFFC/DFI** Concrete Task Group













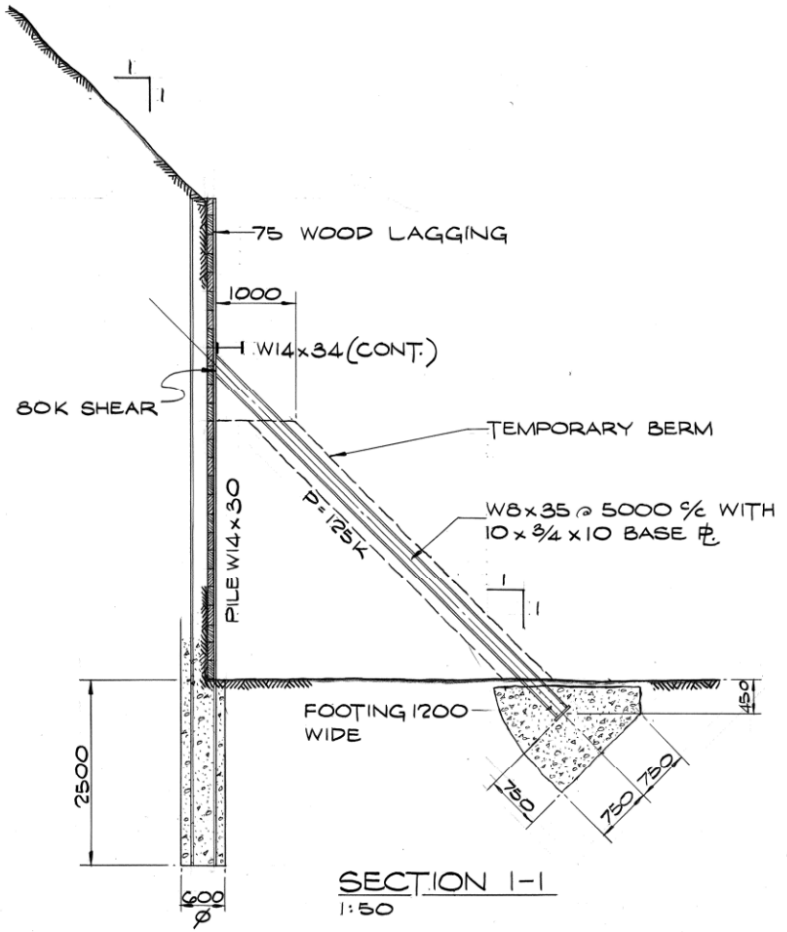
Matthew Janes

Isherwood Geostructural Engineers

2019-2020

ISHERWOOD
GEOSTRUCTURAL ENGINEERS

Blueprints, Rakers, Caldwell drills, Vibratory hammers & Telescoping casings



FAX

CAD, Tie backs

Fast-Tracking

Inclinometers,
Precision survey,

Observational
Method

Cell phones

Shotcrete, Secant pile walls, Diaphragm walls

Hollow Core Bars, Post grouted anchors, High early grouts

Finite Element Analysis

CPT, Strain gauges

Advanced Instrumentation, Composite Design

ISHERWOOD
GEOSTRUCTURAL ENGINEERS

Blueprints to BIM

Mainframe to Laptop

Pagers to Cell Phones

1 – 3% H Movement to 0.1 – 0.3% H

Observational Method – Away from Observational Method

Future

Composites and Weak Concrete Composite Design

AI Design

Real time BIM / Survey / Verification



John Wolosick

Keller

2015-2016

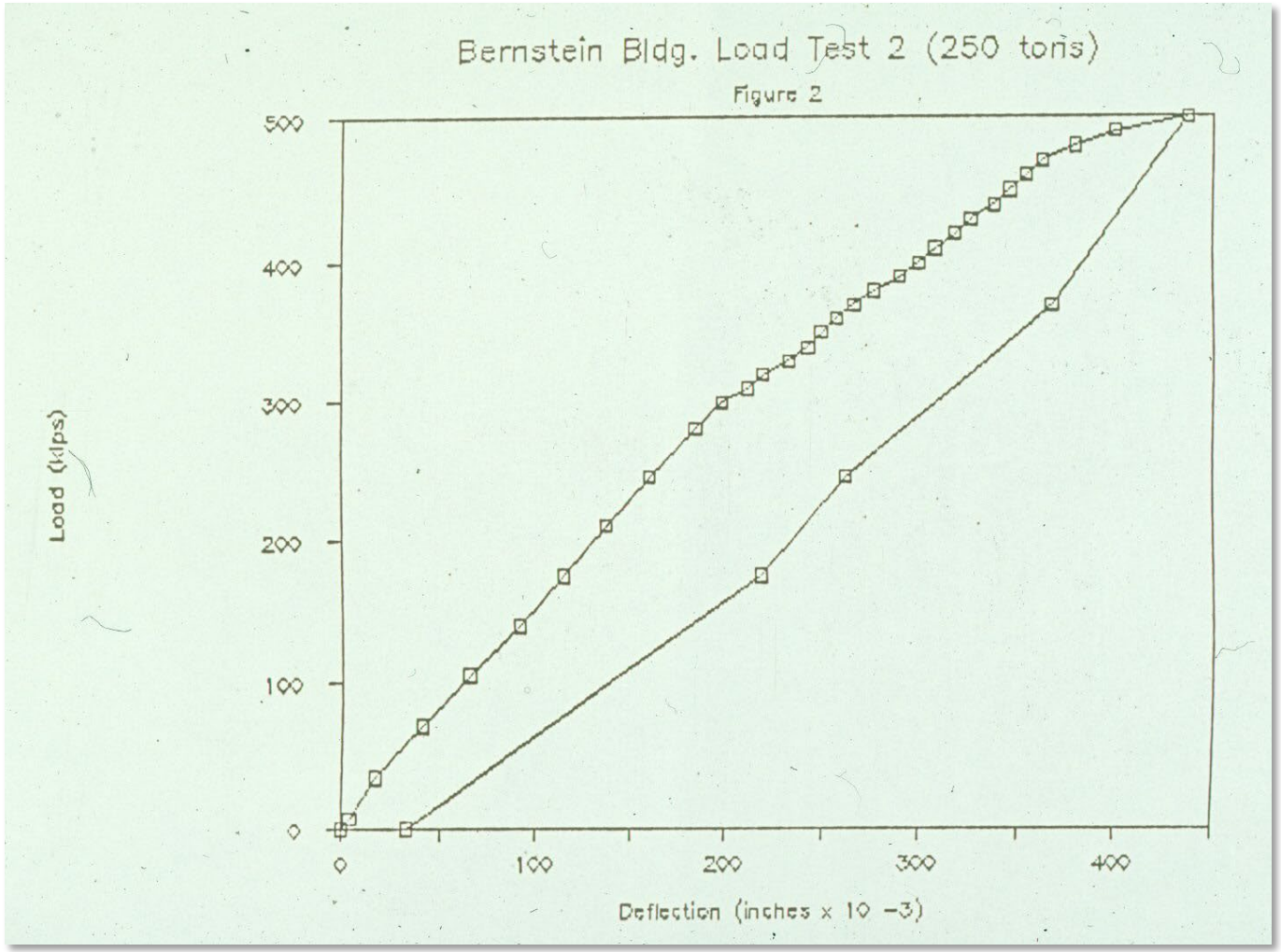
keller.com

A Few Thoughts about Micropile Design and Construction Progress and Some Regression since 1989…..

John R Wolosick, PE, SE, BC.GE, F.ASCE

Keller Atlanta Office

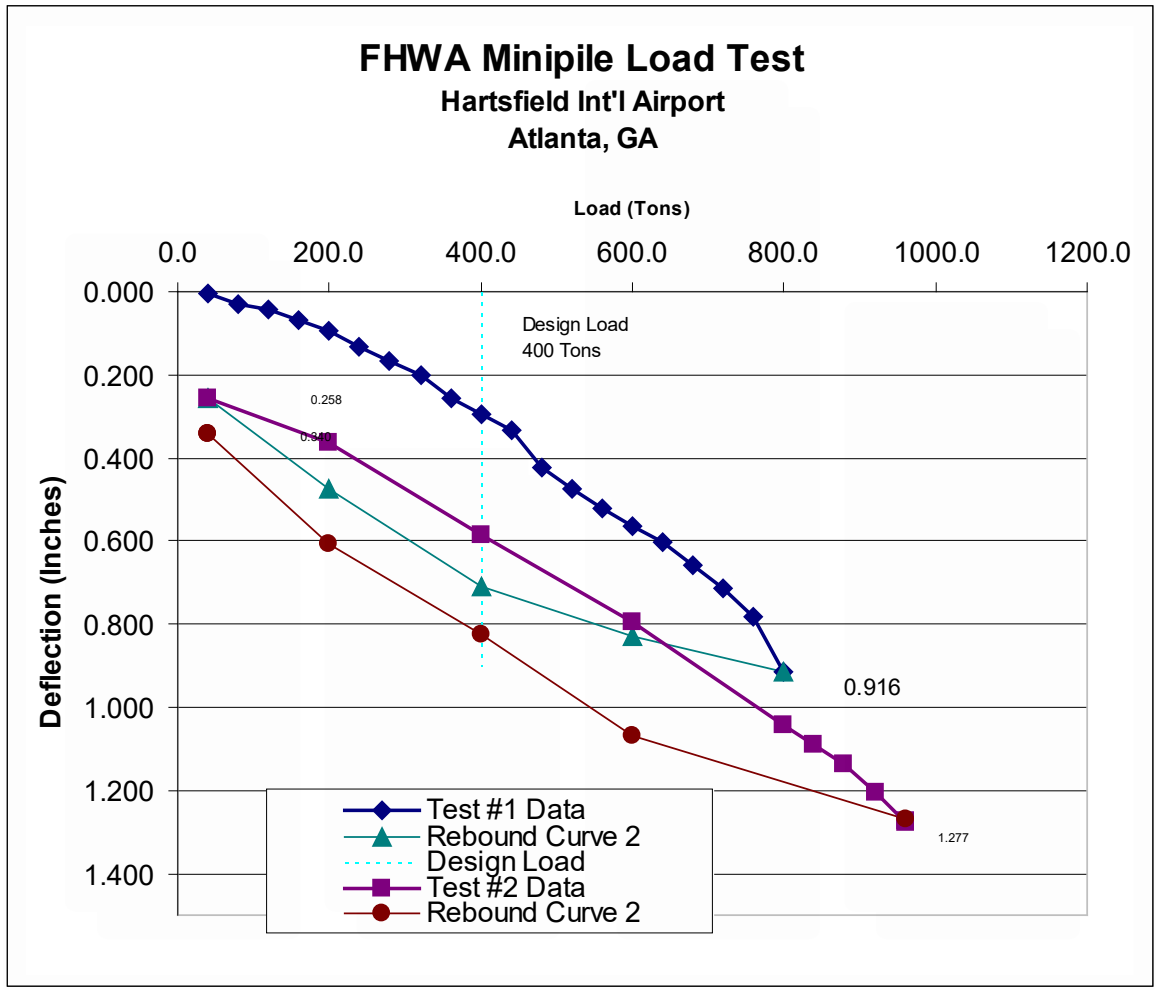
Bernstein Building, Baltimore – 1990. 500 kips Test Load in Dense Sand



Initial Design Load was 140 kips. We could not fail it, so we upped the load until unsafe….



Atlanta Airport – 2002. 970 Tons Test Load in Granitic Bedrock Rock Socket



I submitted 800 kip design load. DOT balked.
To prove a point, we tested it to 970 TONS.



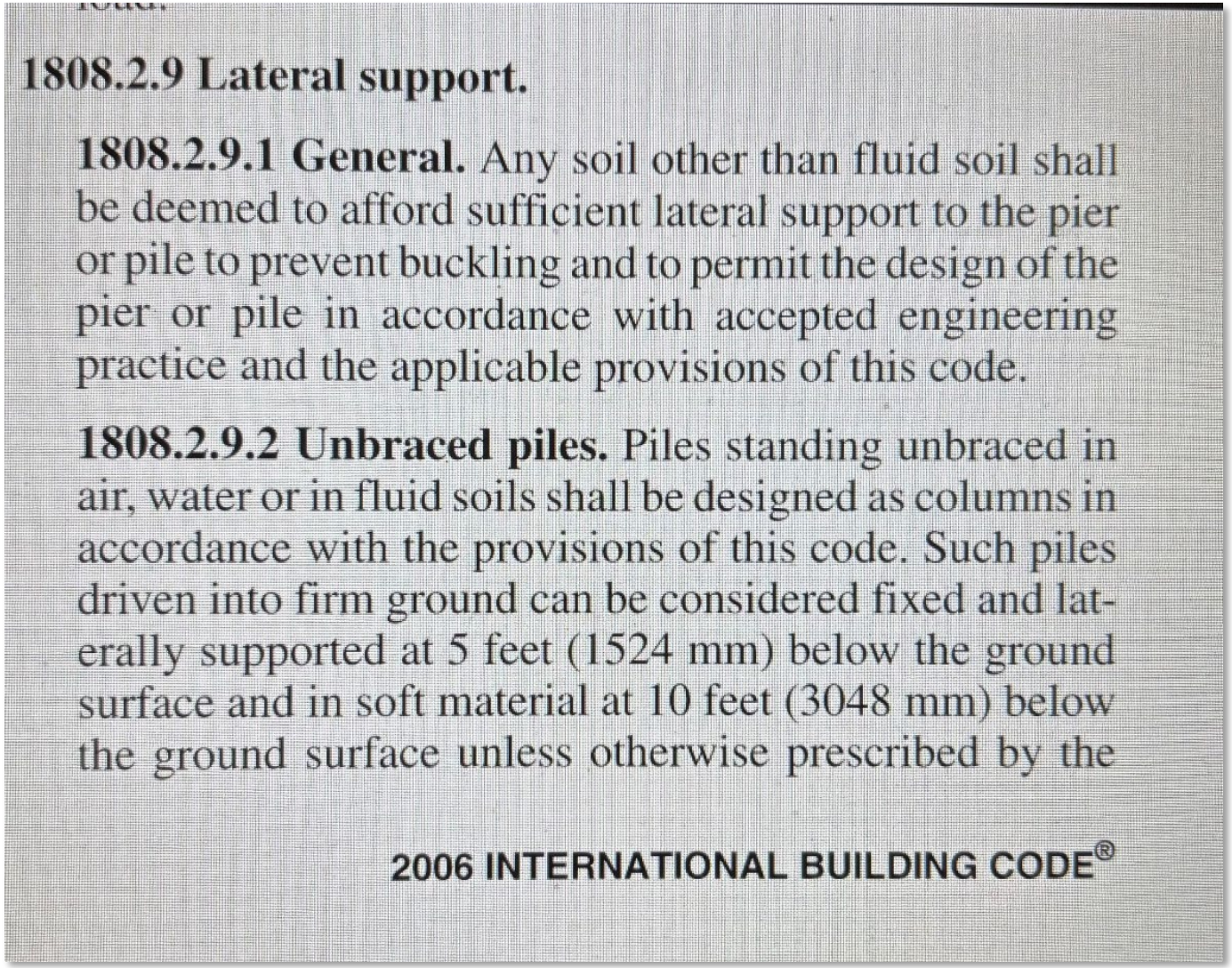
At the recent International Society for Micropiles meeting in Valencia Spain, many European colleagues presented discussions about buckling of micropiles. I apologize, but I find this laughable.

There has never been a deep foundation that has buckled in service. EVER.

I have designed and tested 7 inch micropiles, installed through 150 feet of N = WOH Bay Mud, founded in 35 feet of dense sands to 500 kips with no problem.

These types of discussions are a waste of time. And US building code (IBC) says you don't have to worry about it, as long as you don't have liquefaction.

This has been Codified for 20 years……



1808.2.9 Lateral support.

1808.2.9.1 General. Any soil other than fluid soil shall be deemed to afford sufficient lateral support to the pier or pile to prevent buckling and to permit the design of the pier or pile in accordance with accepted engineering practice and the applicable provisions of this code.

1808.2.9.2 Unbraced piles. Piles standing unbraced in air, water or in fluid soils shall be designed as columns in accordance with the provisions of this code. Such piles driven into firm ground can be considered fixed and laterally supported at 5 feet (1524 mm) below the ground surface and in soft material at 10 feet (3048 mm) below the ground surface unless otherwise prescribed by the

2006 INTERNATIONAL BUILDING CODE®

Worry about things like caves, not soft soils!

PS: We do caves too!

National Corvette Museum

Bowling Green, Kentucky



KELLER

keller.com

Thank you!

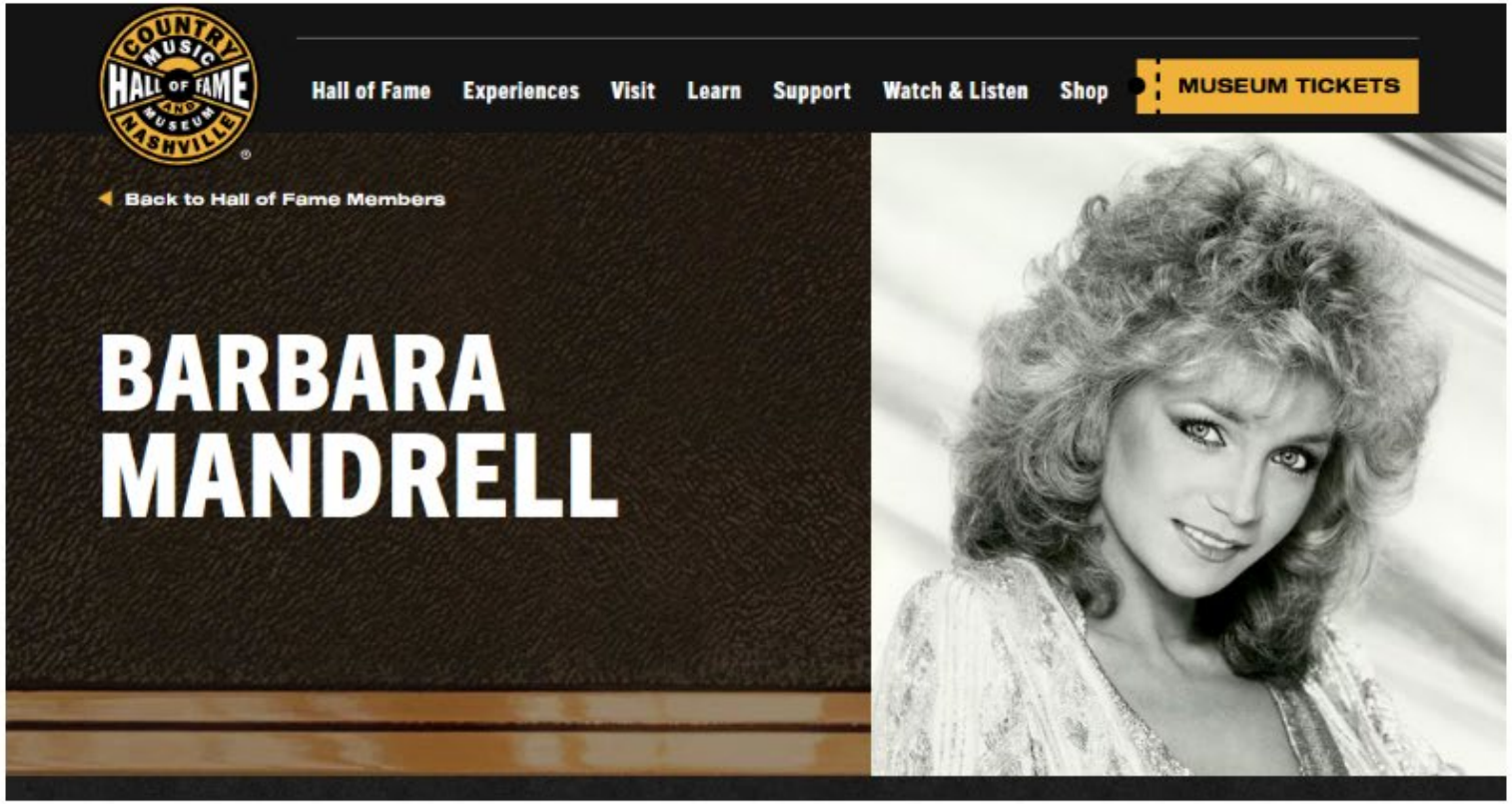




Rudolph Frizzi

LANGAN Engineering and Environmental Services

2009-2010





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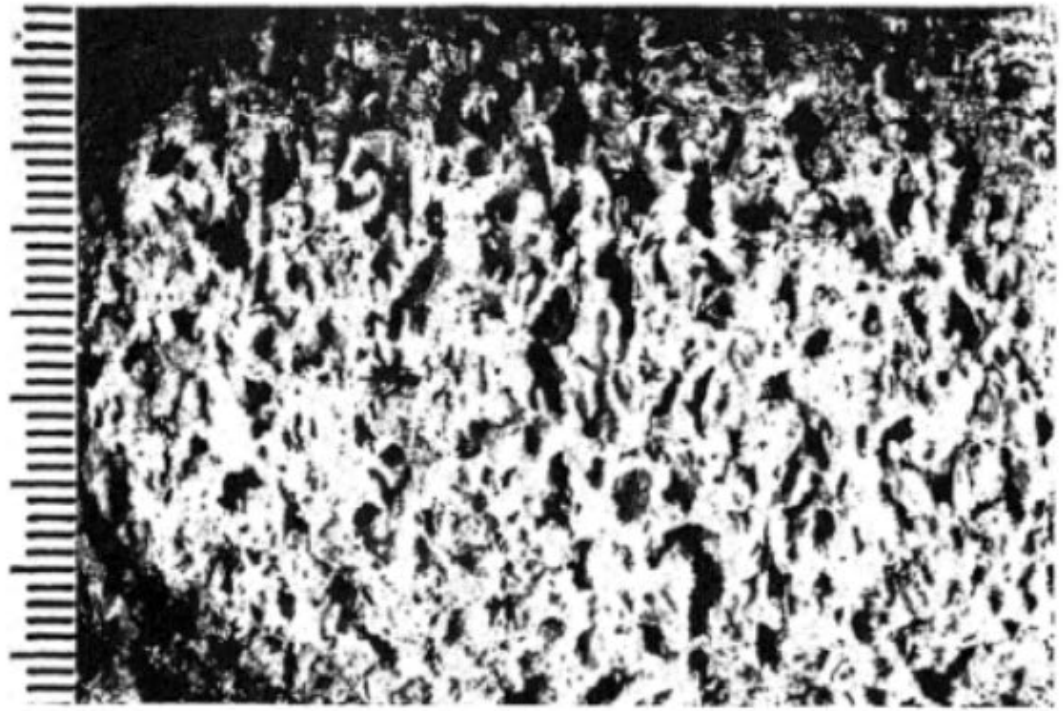
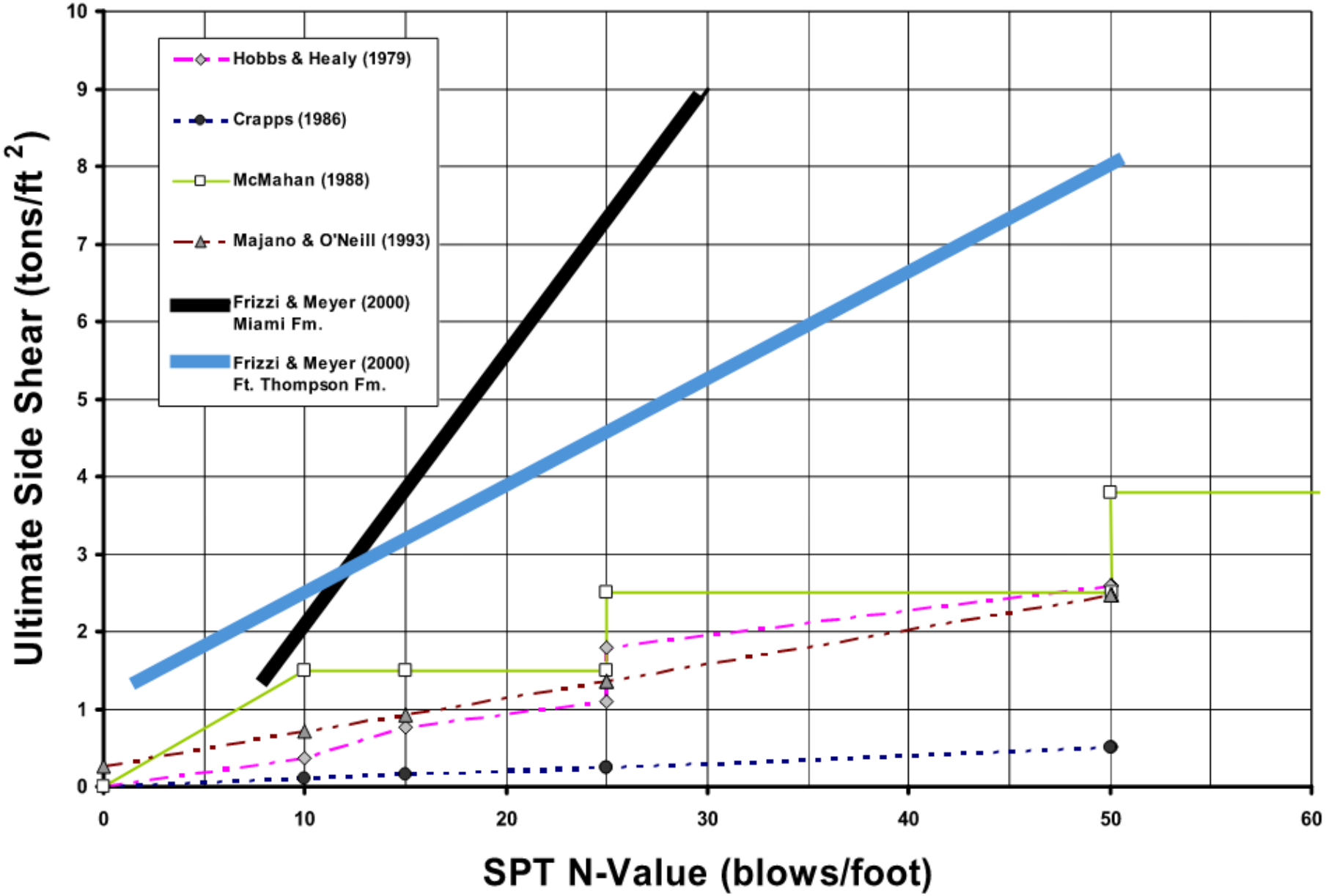
(a)

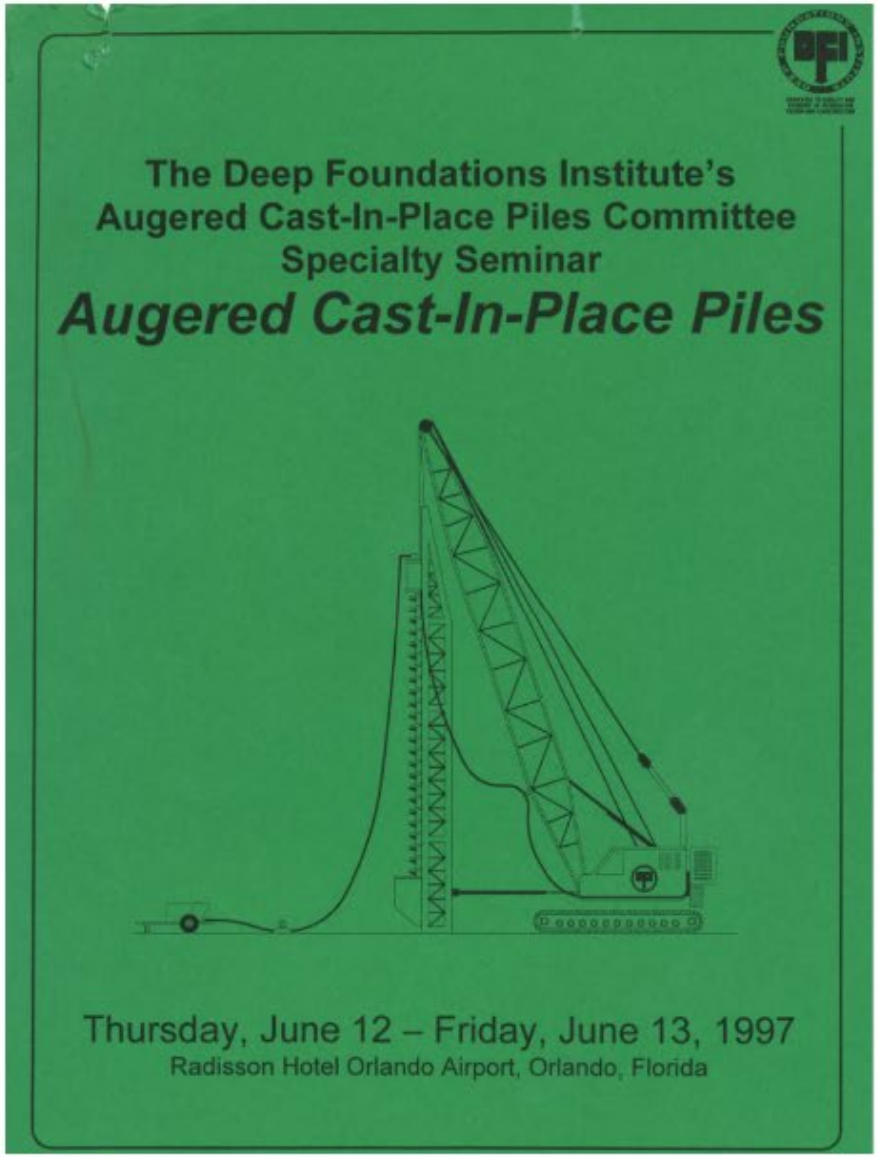
Ref: Sowers (1996)



(b)

FIG. 2.5. Solution-enlarged Primary Porosity in Limestones From Miami, FL: a. Porosity 75% Approximately (popcorn rock); b. Porosity 30% Approximately (1-mm Scale Marks)


Hobbs & Healy (1979)
Crapps (1986)
McMahan (1988)
Majano & O'Neill (1993)
Frizzi & Meyer (2000) Miami Fm.
Frizzi & Meyer (2000) Ft. Thompson Fm.
Ultimate Side Shear (tons/ft 2)
SPT N-Value (blows/foot)
0
1
2
3
4
5
6
7
8
9
10
0
10
20
30
40
50
60



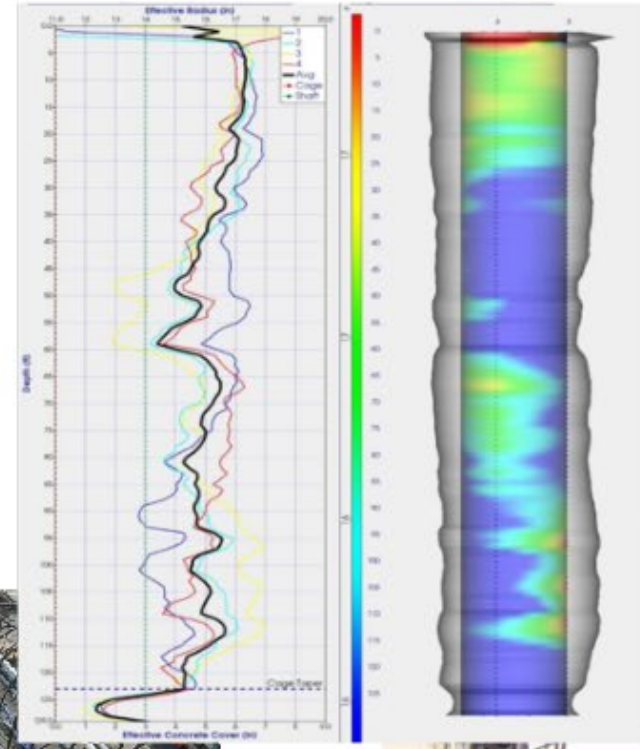


16 *Moderator, chair, speakers, and DFI's operations manager strike a satisfied pose at the conclusion of the Orlando seminar*






DAVIS
Depth:
0.19ft
REC











Michael Wysockey

Thatcher Engineering Corporation

2021-2022

PILE DRIVING THEN AND NOW

Michael H. Wysockey PE, PhD

Thatcher Foundations

October 21, 2025

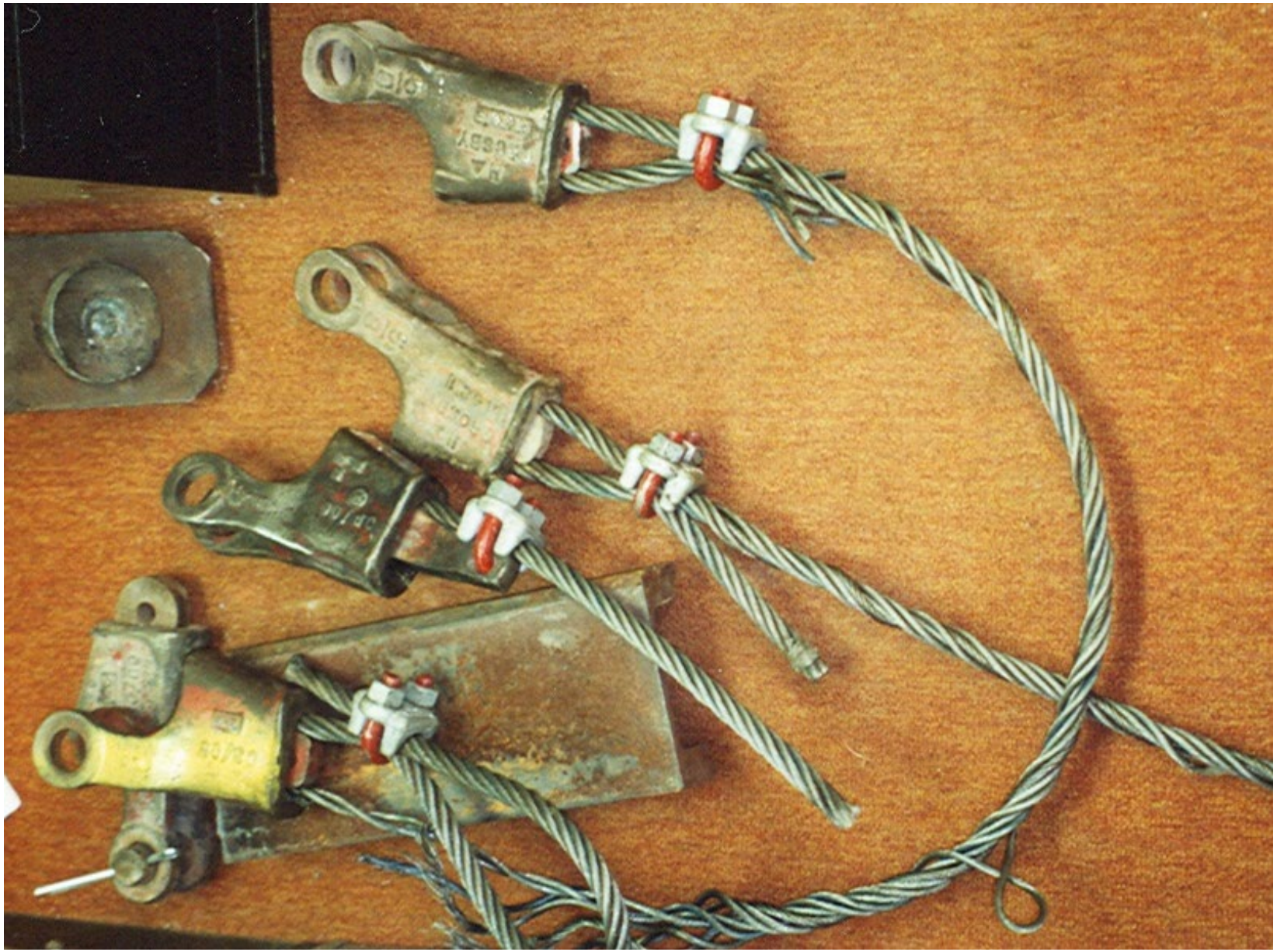
THATCHER


THATCHER

Harry Thatcher and Tom Wysockey



SAFETY



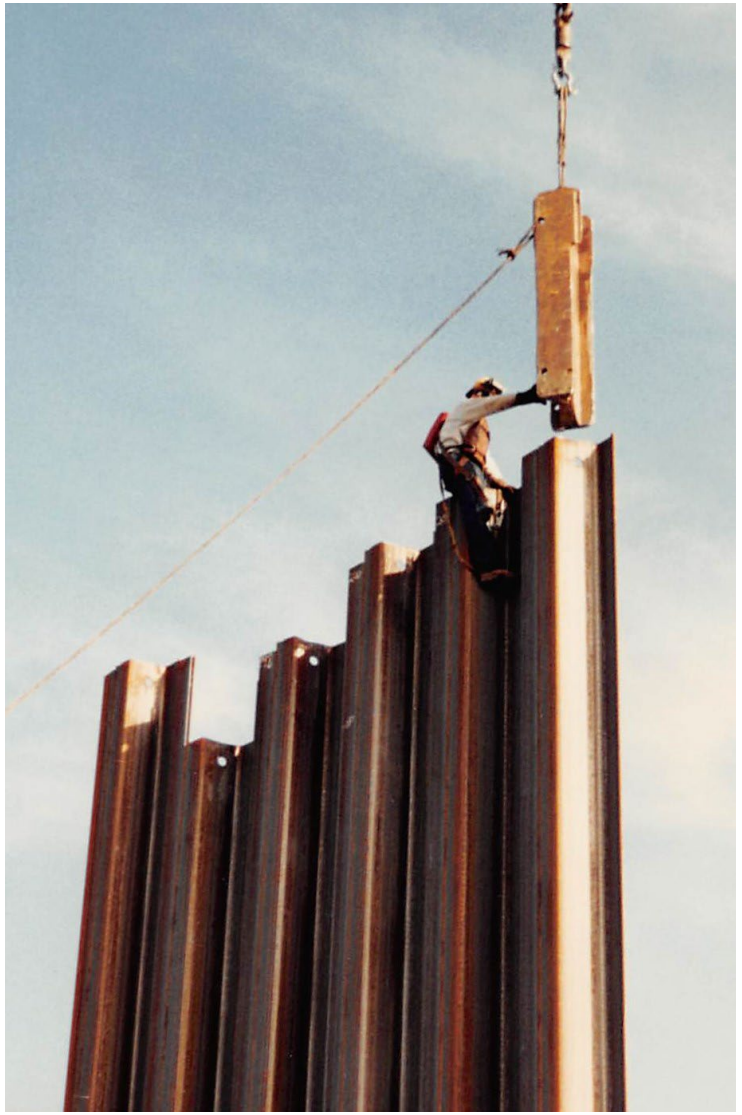
Manitowoc Brochure




SON CO.
ROBT R ANDERSON CO

Top man

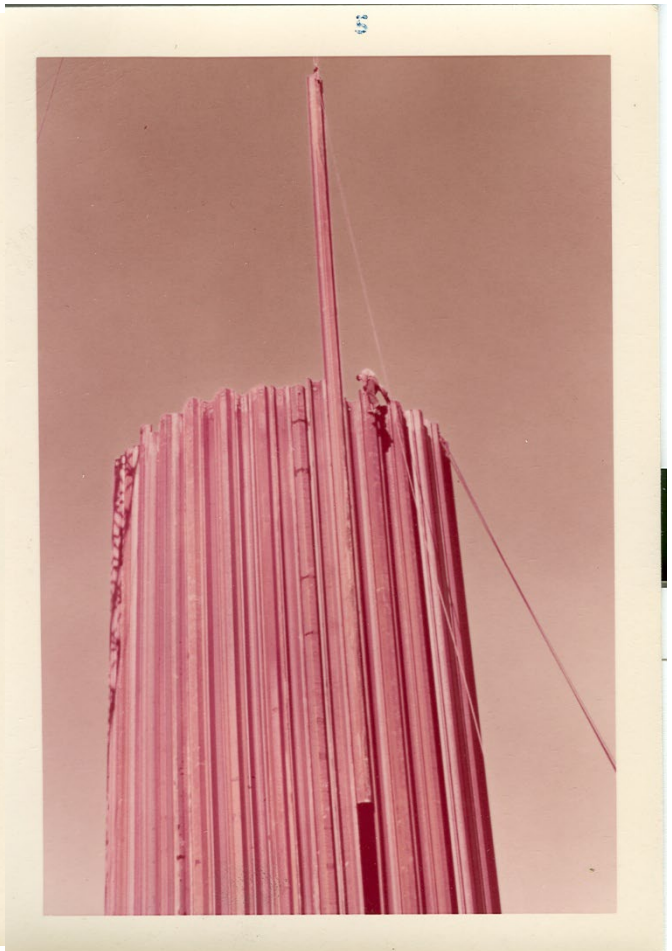




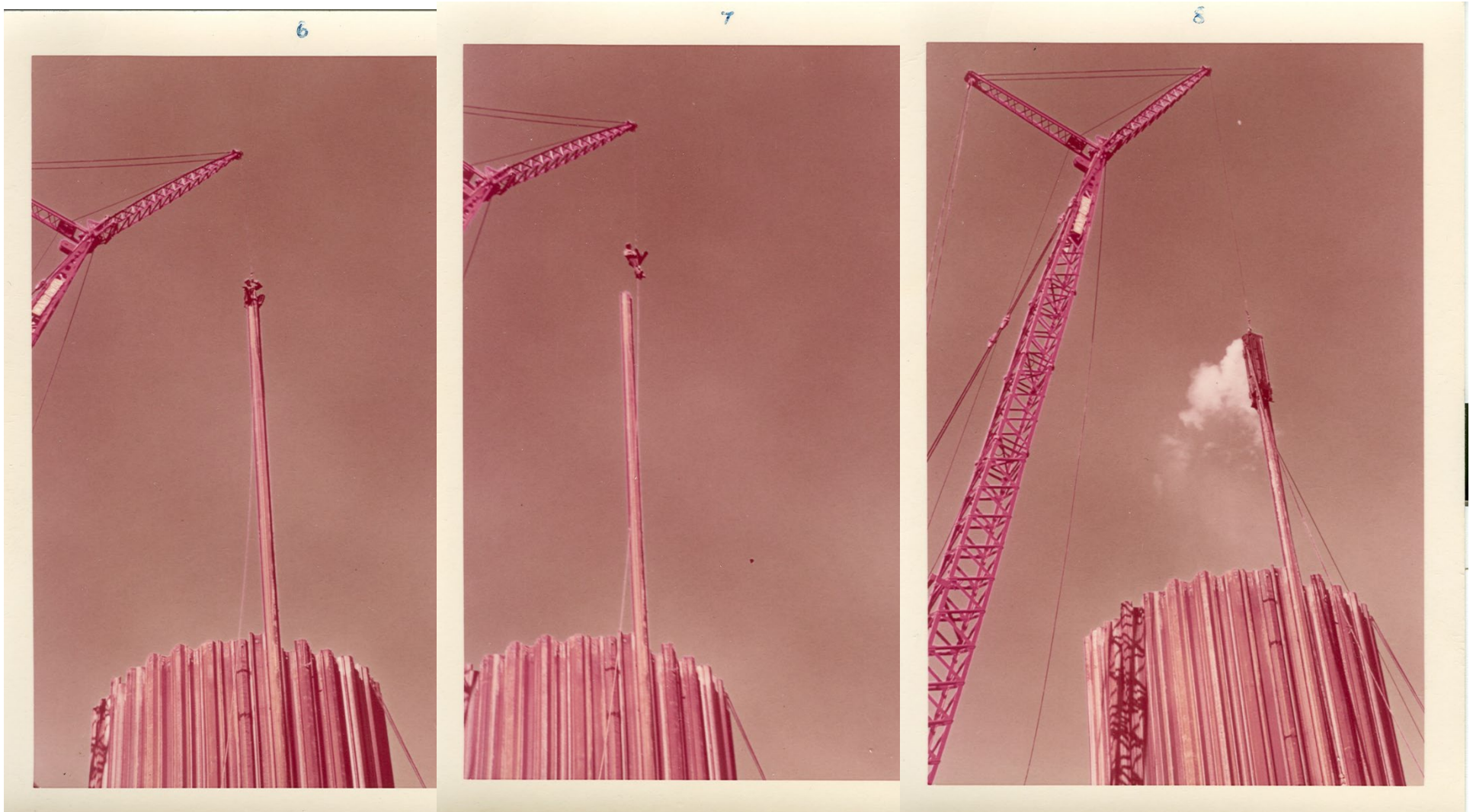
Old Cellular Cofferdam







OSHA still just a town in Wisc.



Steam Hammer



Monkey Stick



Inland Steel (Cleveland Cliffs)



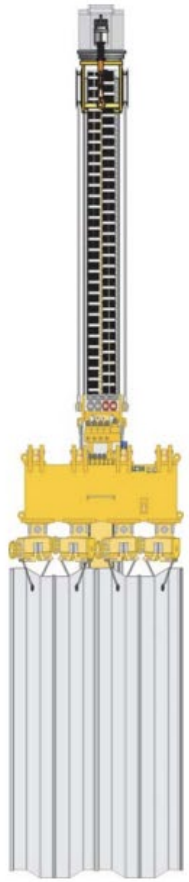


East Chicago, IN
2.5 miles of cell sheeting @ 60’ long made up of about 8 miles of interconnected walls

Menck Underwater Hammer



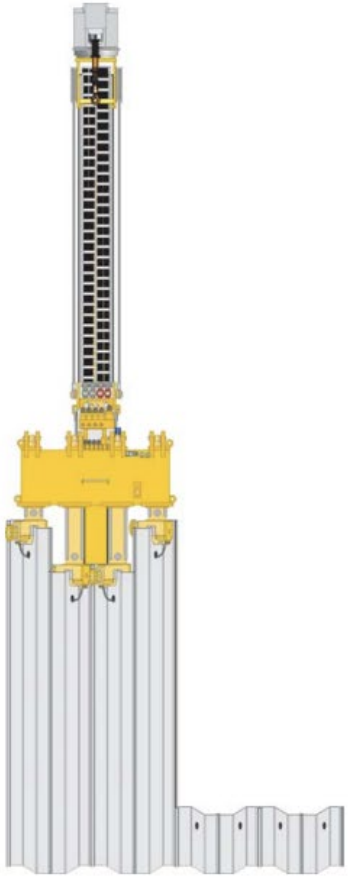




Pitching sheet piles panel with the pitching chain.

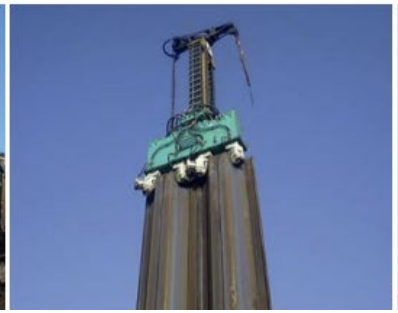


The sheet piles are clamped into the clamp assemblies and aligned. Lifted sheet piles are going to be threaded in the privious pressed sheet piles interlock.



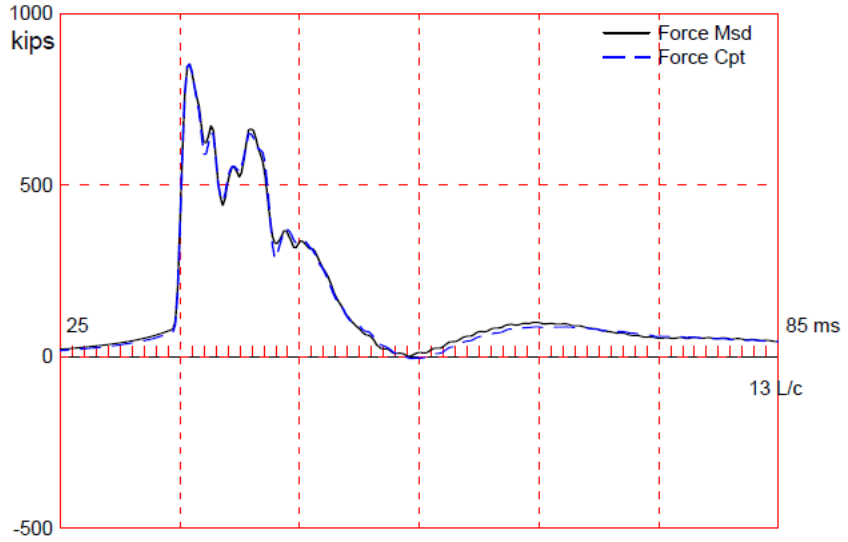
Pressing operation
The piles in the middle are pressed individually first, then followed by the outer piles.

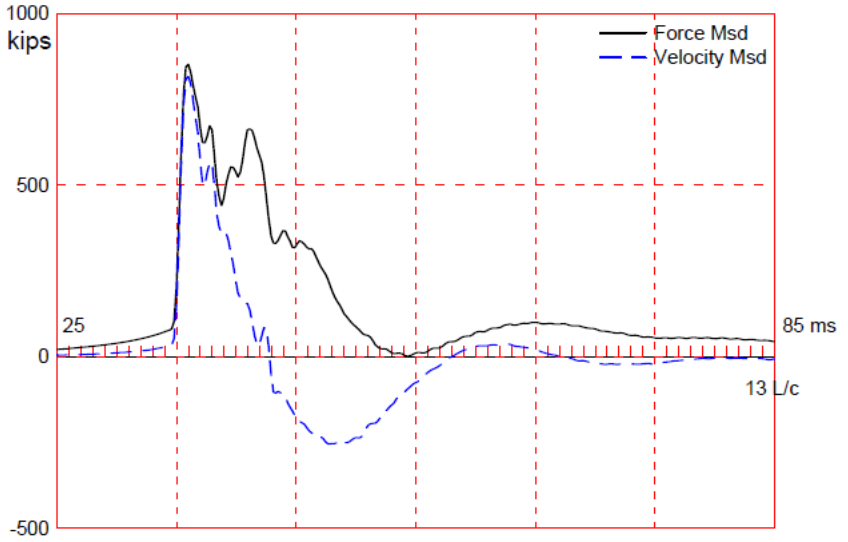


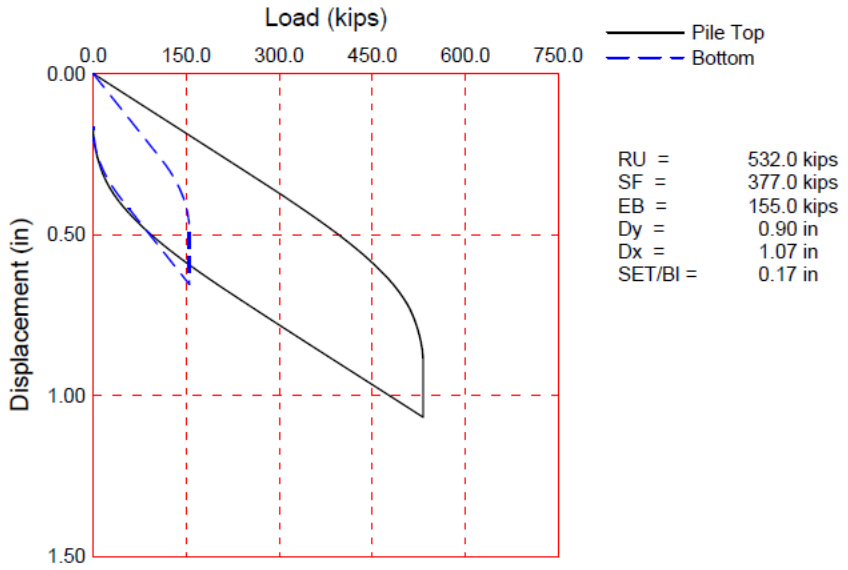




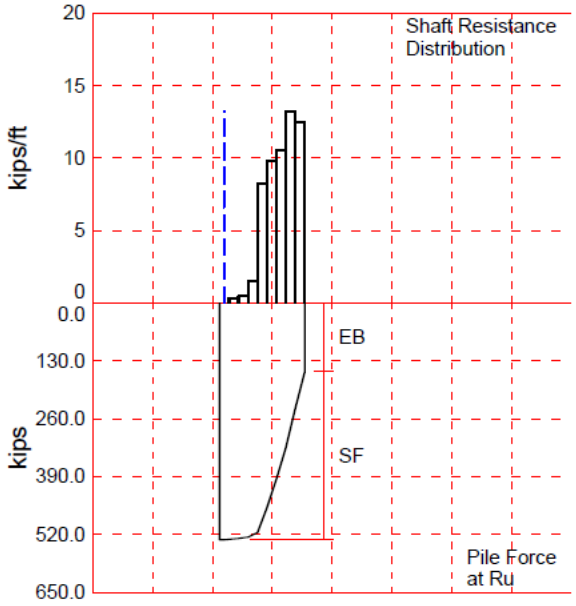








RU =	532.0 kips
SF =	377.0 kips
EB =	155.0 kips
Dy =	0.90 in
Dx =	1.07 in
SET/Bl =	0.17 in



Length b. Sensors	59.8 ft
Embedment	56.0 ft
Top Area	26.1 in²
End Bearing Area	203.2 in²
Top Perimeter	4.75 ft
Top E-Modulus	30000 ksi
Top Spec. Weight	492.0 lb/ft³
Top Wave Spd.	16808 ft/s
Overall W.S.	16808 ft/s
Match Quality	1.91
Top Compr. Stress	32.7 ksi
Max Compr. Stress	36.3 ksi
Max Tension Stress	-1.87 ksi
Avg. Shaft Quake	0.28 in
Toe Quake	0.38 in
Avg. Shaft Smith Dpg.	0.20 s/ft
Toe Smith Damping	0.12 s/ft

CAPWAP(R) 2014-4 Licensed to GRL Engineers, Inc.

Thank you



THATCHER

PARALLEL SESSION A1
Golden State of Practice

Drilled Shafts as Mass Concrete: Recommended Specification Provisions

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DFI 50

October 21, 2025

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Acknowledgements

- **Co-authors: Erik Loehr and Paul Axtell**
- University of Missouri
- FHWA
- WSP
- DBA colleagues
- Many patient clients







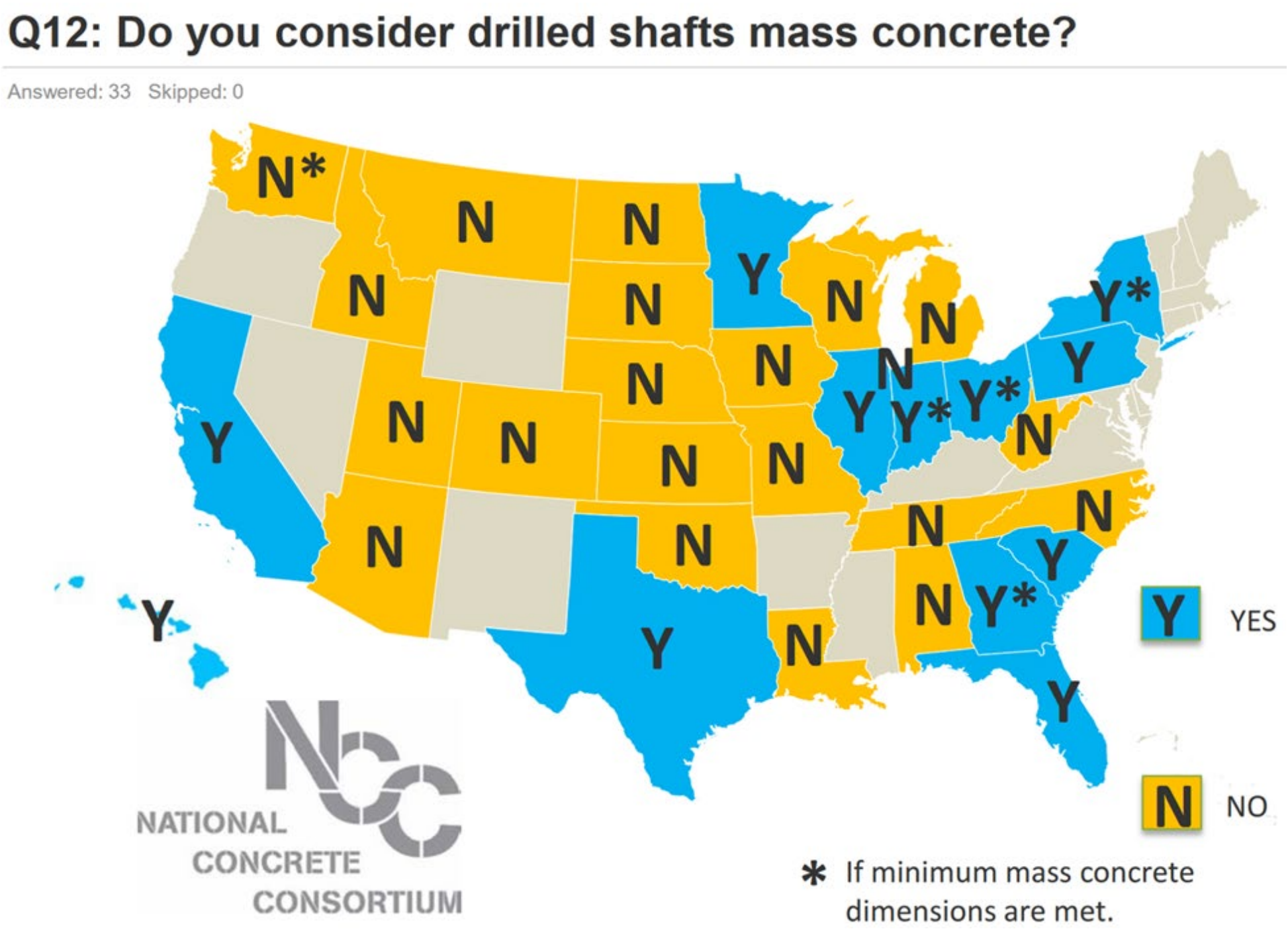


Motivation: The Topic, Generally

- Large diameter shafts are increasingly common
- Large diameter shafts frequently develop high temperatures
- High concrete temperatures are associated with potential durability problems
- Many agencies are imposing mass concrete specifications
- The specifications have significant consequences for constructability and cost

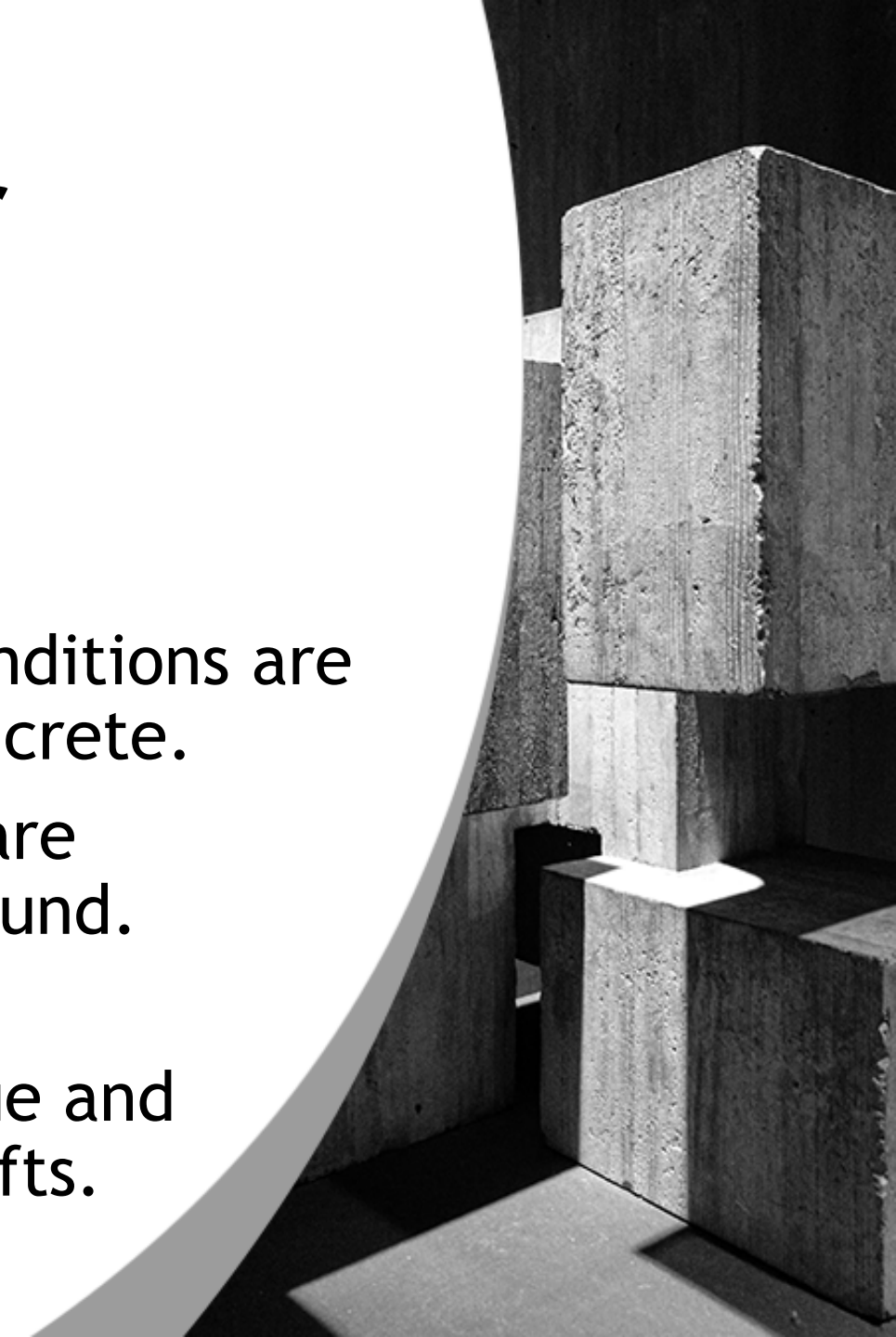


Masten, M. (2018), *State Reports Summary: Types of Reinforcement and Mass Concrete*, Presentation to National Concrete Consortium



Motivation: This Paper

- Mass concrete for drilled shafts is here.
- It’s not going away.
- It’s going to become more common.
- Drilled shaft construction and boundary conditions are very different from above-ground mass concrete.
- Drilled shaft mass concrete specifications are typically not very different from above-ground.
  - i.e., they’re not good.
- Specifications should account for the unique and frequently beneficial aspects of drilled shafts.



What are “thermal issues”?

- Two main concerns with high concrete temperatures:

(1) Delayed Ettringite Formation (DEF) in response to large peak temperatures

(2) Thermal cracking in response to large temperature differentials



Mass Concrete

- Mass concrete specifications address thermal issues. Generally include four components:

(1) Definition of mass concrete, e.g. any element with minimum dimension $\geq$ 4 ft

(2) Maximum allowable T_{max}

(3) Maximum allowable ΔT

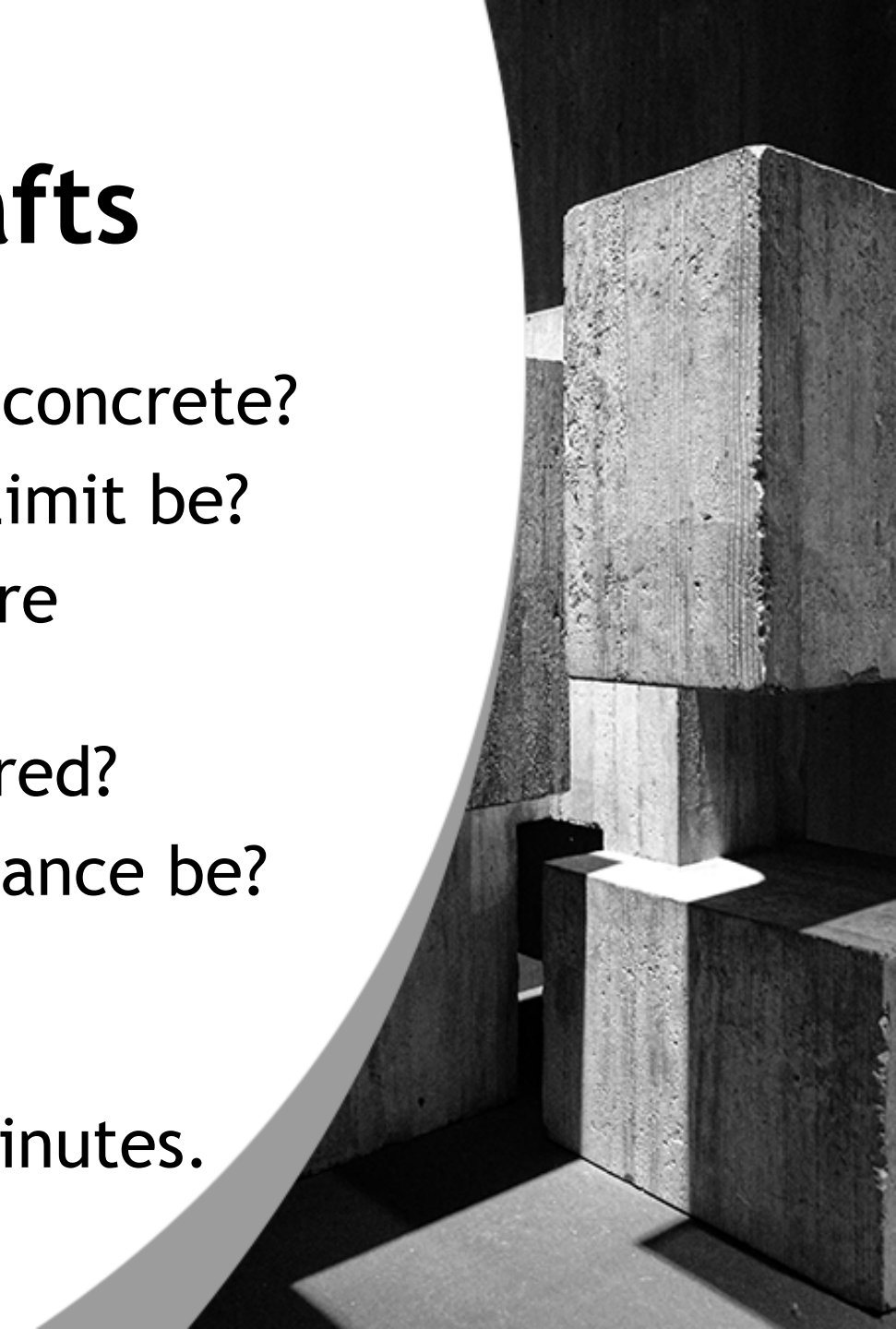
(4) Requirements for Thermal Control Plan and monitoring temperature



Questions for Drilled Shafts

1. Should drilled shafts be considered mass concrete?
2. What should the maximum temperature limit be?
3. What should be the maximum temperature differential be?
4. How long should drilled shafts be monitored?
5. What should the consequences of exceedance be?
6. What about potential strength loss?

- Paper answers these questions with more information than can be presented in 20 minutes.



Basis for Answering Questions

- DBA experience developing Thermal Control Plans (TCPs) on nine projects
- Thermal analysis on two additional projects
- Owners include
  - Two federal agencies
  - Two state transportation agencies
  - Five local or regional transportation agencies, many citing state DOT specs
  - One private developer
- Three owners referenced ACI specifications



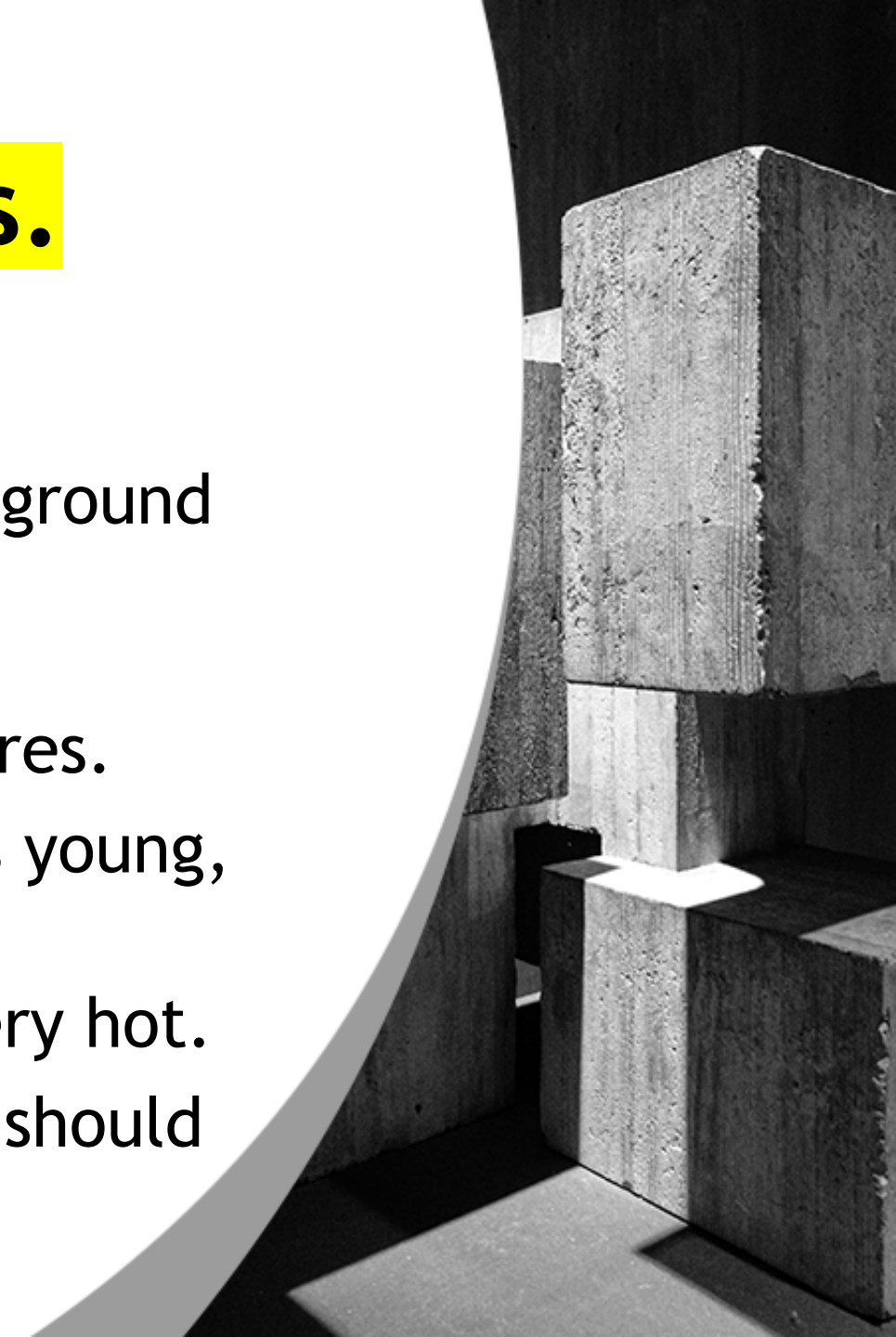
1. Should drilled shafts be considered mass concrete? The case for “NO!”

- No evidence of drilled shaft thermal damage.
- Concrete is heavily reinforced.
- Concrete is confined.
- Consequences would be limited, and likely negligible.
- Cure is worse than the disease.
- No evidence of drilled shaft thermal damage.



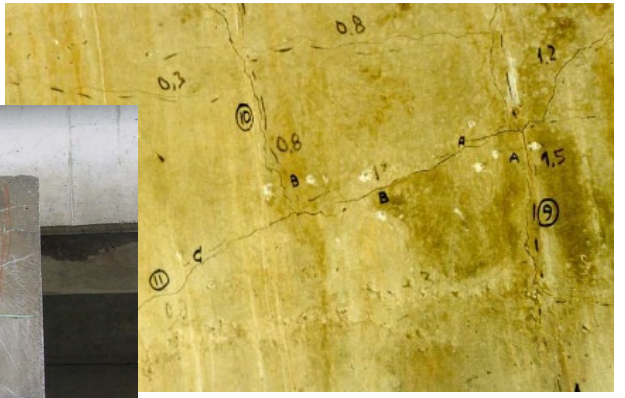
Sorry, the answer is Yes.

- No one has looked for damage.
- Damage has been documented in above-ground concrete elements that are
  - Smaller.
  - Subjected to less extreme temperatures.
- Experience with large-diameter shafts is young, compared to service life.
- Even small diameter elements can be very hot.
- Service life and durability requirements should not be overlooked.



Six bridges with DEF documented in:

Godart, B. and L. Divet (2013), “Lessons learned from structures damaged by delayed ettringite formation and the French prevention strategy,” Fifth International Conference on Forensic Engineering, Institution of Civil Engineers, Apr 2013, France. 12p. hal-00945667







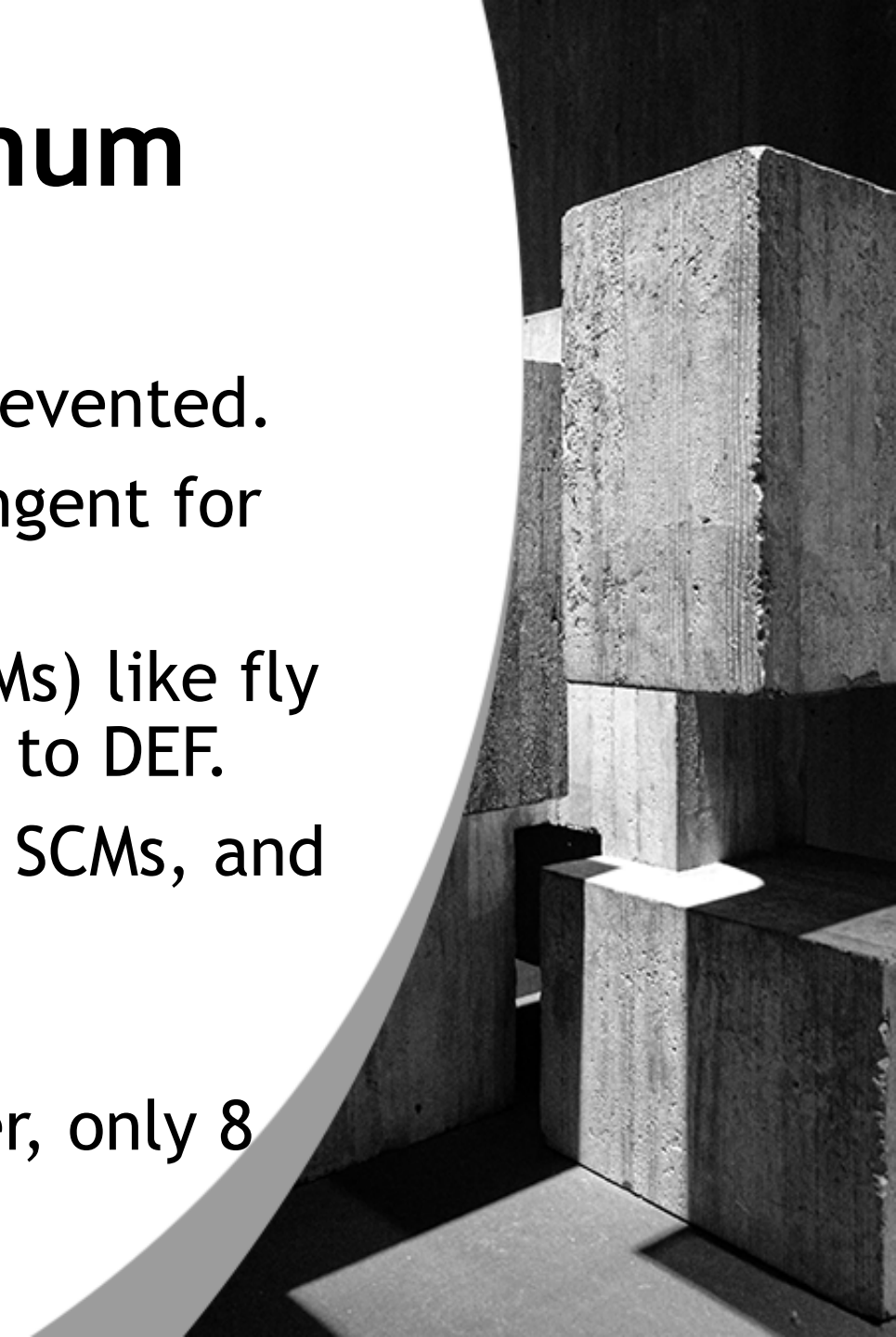






2. What should the maximum temperature limit be?

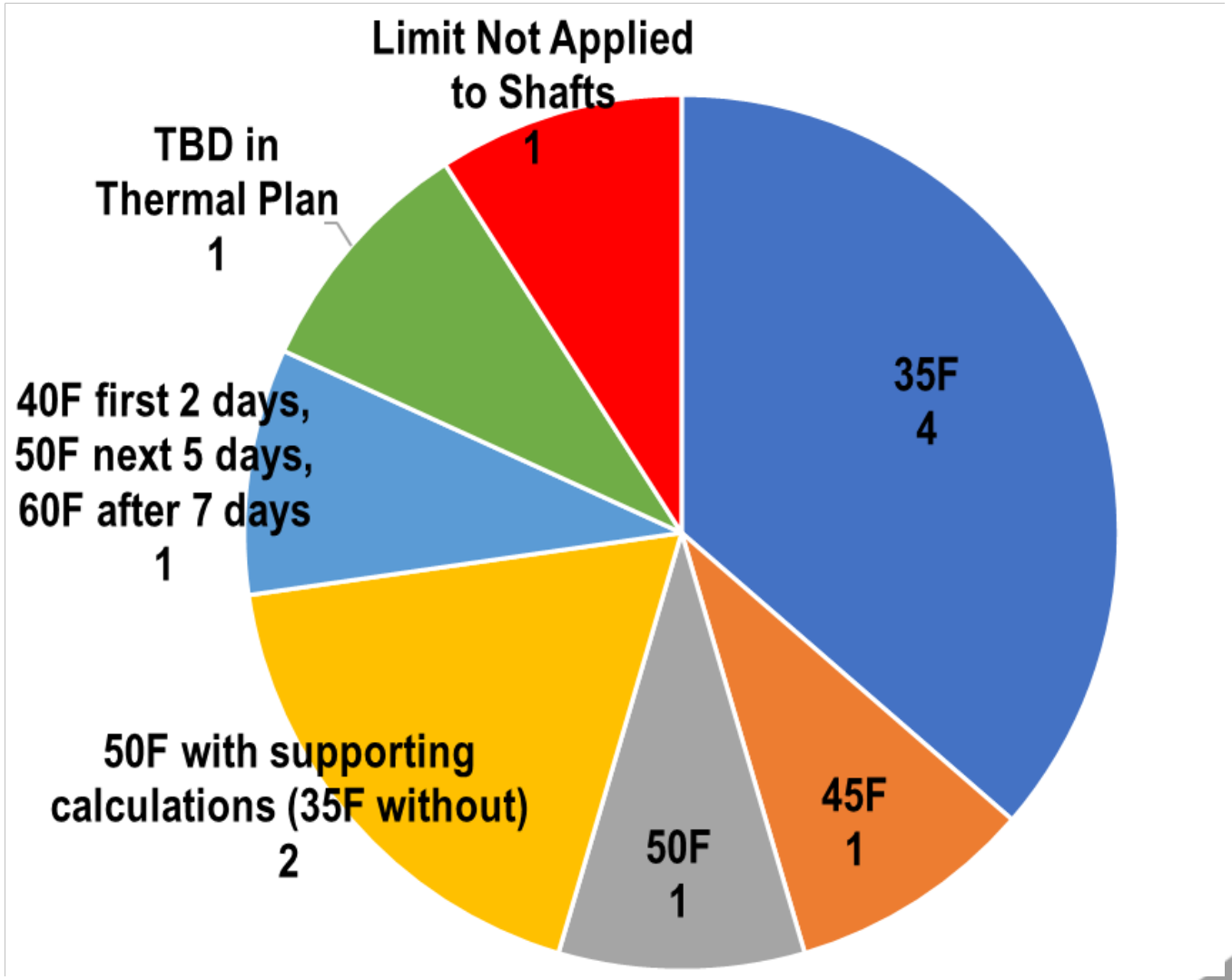
- DEF is real, it's serious, and it should be prevented.
- Service life requirements are typically stringent for large-diameter shafts.
- Supplementary cementitious materials (SCMs) like fly ash and slag increase concrete's resistance to DEF.
- Research has demonstrated the benefits of SCMs, and ACI guidance acknowledges this.
- Use the ACI guidance.
- Of the 11 specifications considered in paper, only 8 allowed the ACI higher limits. ☹



Maximum Concrete Temperature, T	Prevention Required
$T \leq 158\ ^{o}F$	No prevention required.
$158\ ^{o}F < T \leq 185\ ^{o}F$	Use one of following to minimize risk of expansion: 1. Low-alkali Portland cement having moderate or high sulfate resistance (ASTM C150/C150M) 2. Portland cement with 1-day mortar strength ≤ 2850 psi (ASTMC109/C109M) 3. Portland cement in combination with the following supplementary cementitious materials (SCMs) a. ≥ 25 % Class F fly ash (ASTM C618) b. ≥ 35 % Class C fly ash (ASTM C618) c. ≥ 35 % slag cement (ASTM C989/C989M) d. ≥ 5 % silica fume (ASTM C1240) with ≥ 25 % slag cement e. ≥ 5 % silica fume (ASTM C1240) with ≥ 20 % Class F fly ash f. ≥ 10 % metakaolin (ASTM C618) 4. Blended hydraulic cement with SCM content listed in Item 3 (ASTM C595/C595M or ASTM C1157/C1157M).
$T > 185\ ^{o}F$	Not permissible under any circumstances.

From ACI 201.2R *Guide to Durable Concrete*

3. What should the maximum temperature differential be?

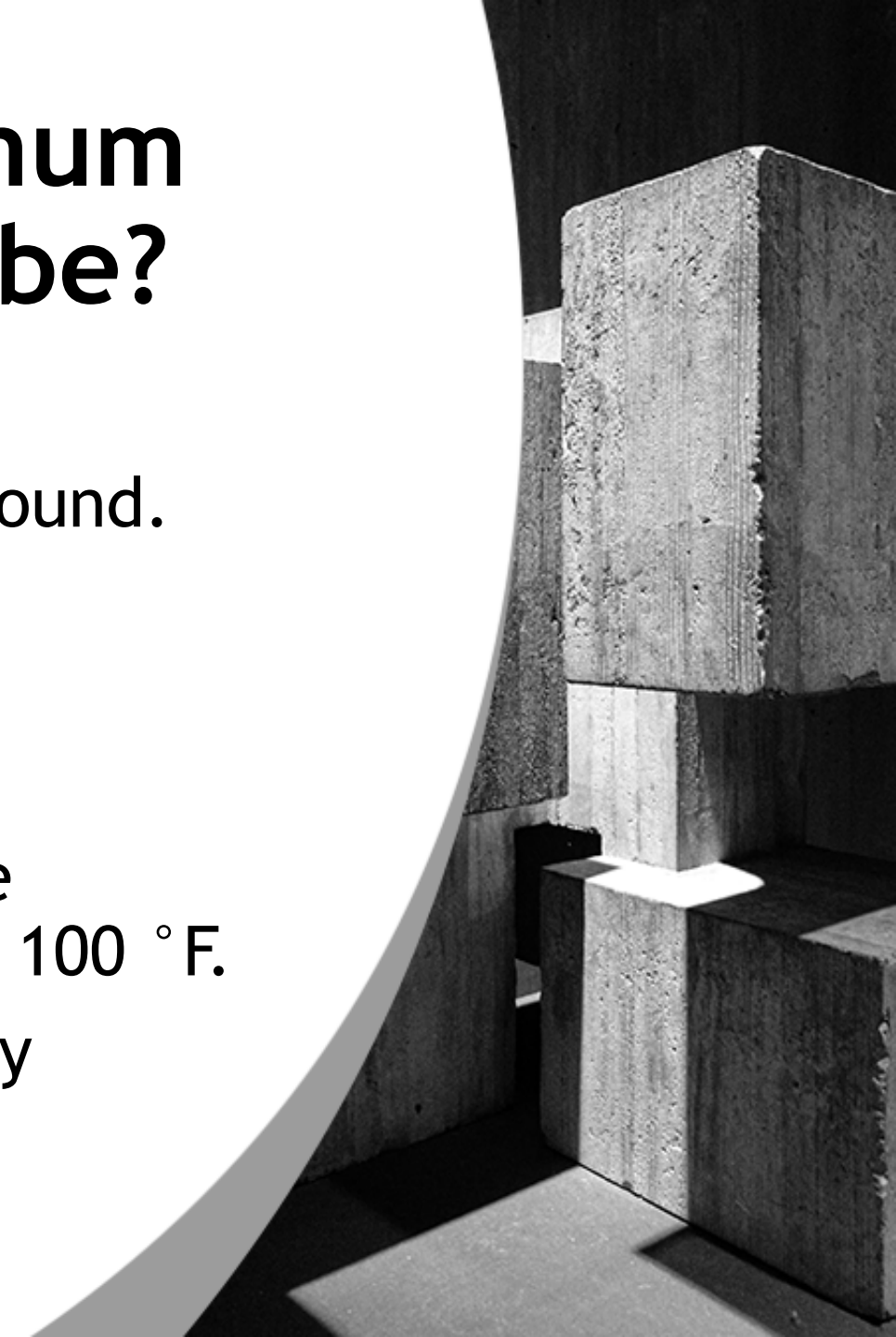




3. What should the maximum temperature differential be?

Background:

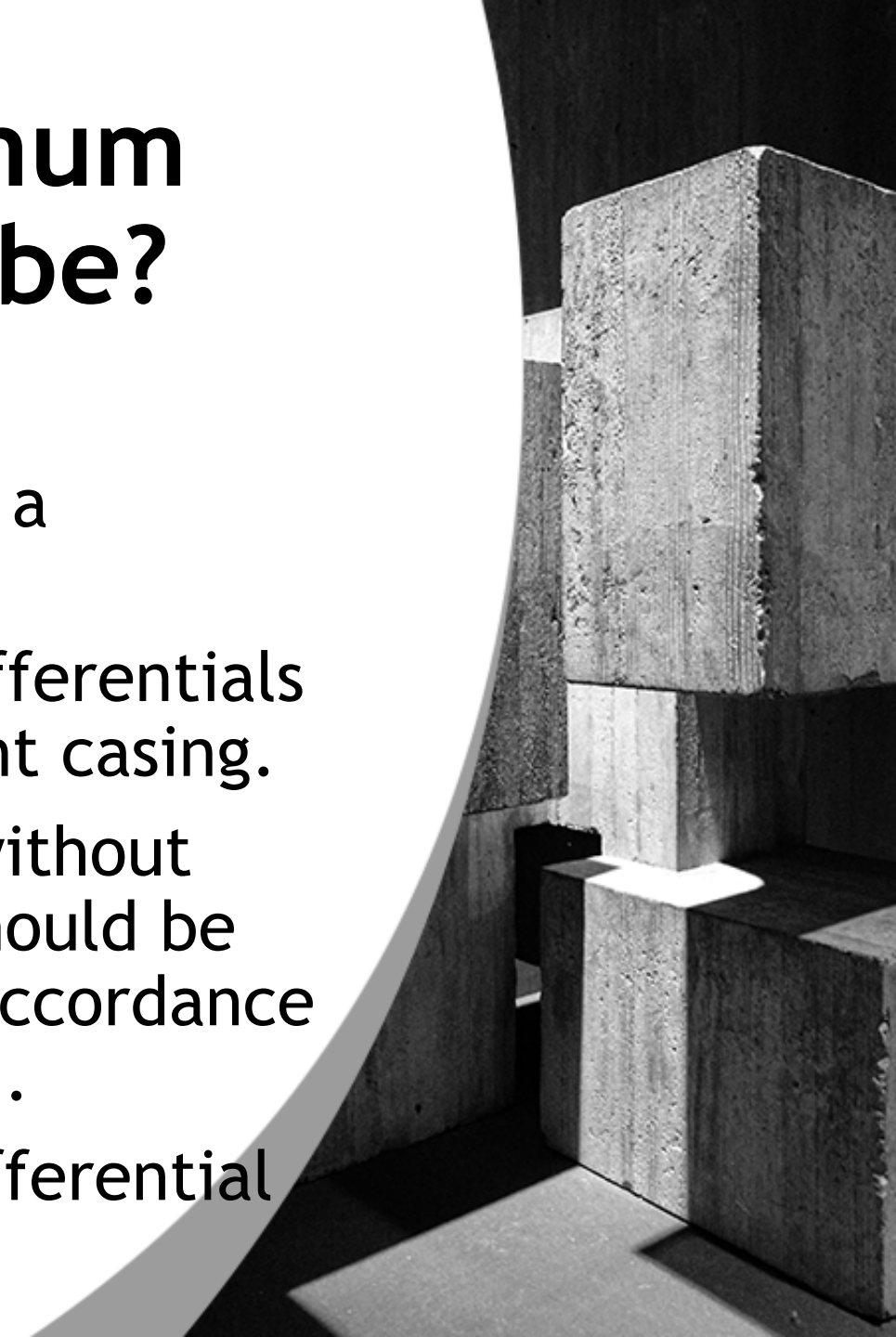
- There is no visual quality concern below ground.
- Durability concerns are…
  - Negligible within a permanent casing
  - Negligible for minor cracking
- If you permit minor cracking, the allowable differential limit is on the order of at least 100 °F.
- Unlike above-ground, drilled shaft boundary conditions are not controllable.
- Therefore…



3. What should the maximum temperature differential be?

Answer:

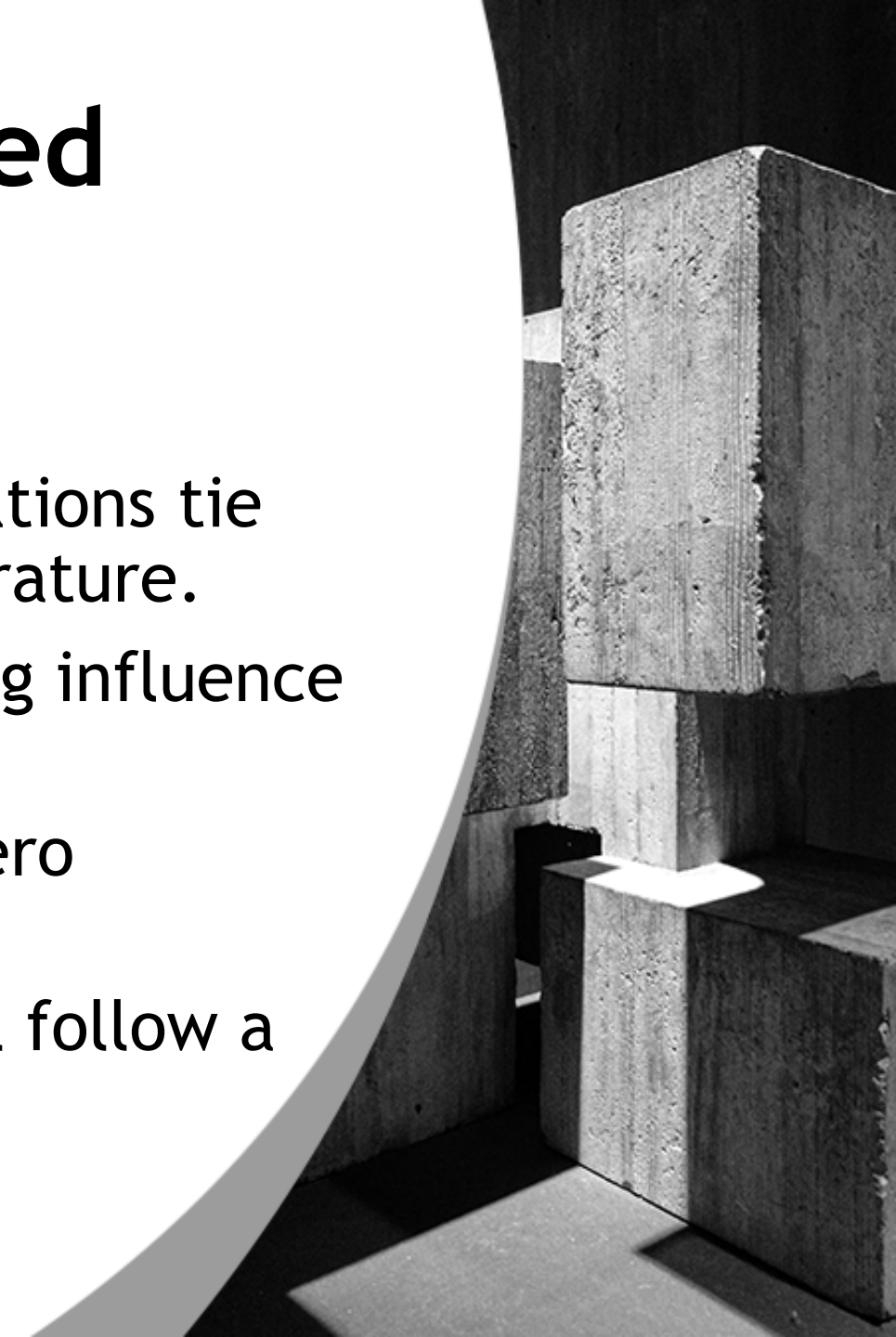
- Differential limits should never apply within a permanent steel casing.
- For most ground conditions, temperature differentials should not apply to shafts without permanent casing.
- For drilled shafts in aggressive ground and without permanent casing, allowable differentials should be established to prevent thermal cracking in accordance with procedures in Boeckmann, et al. (2021).
- More agencies are exempting shafts from differential requirements. ☺

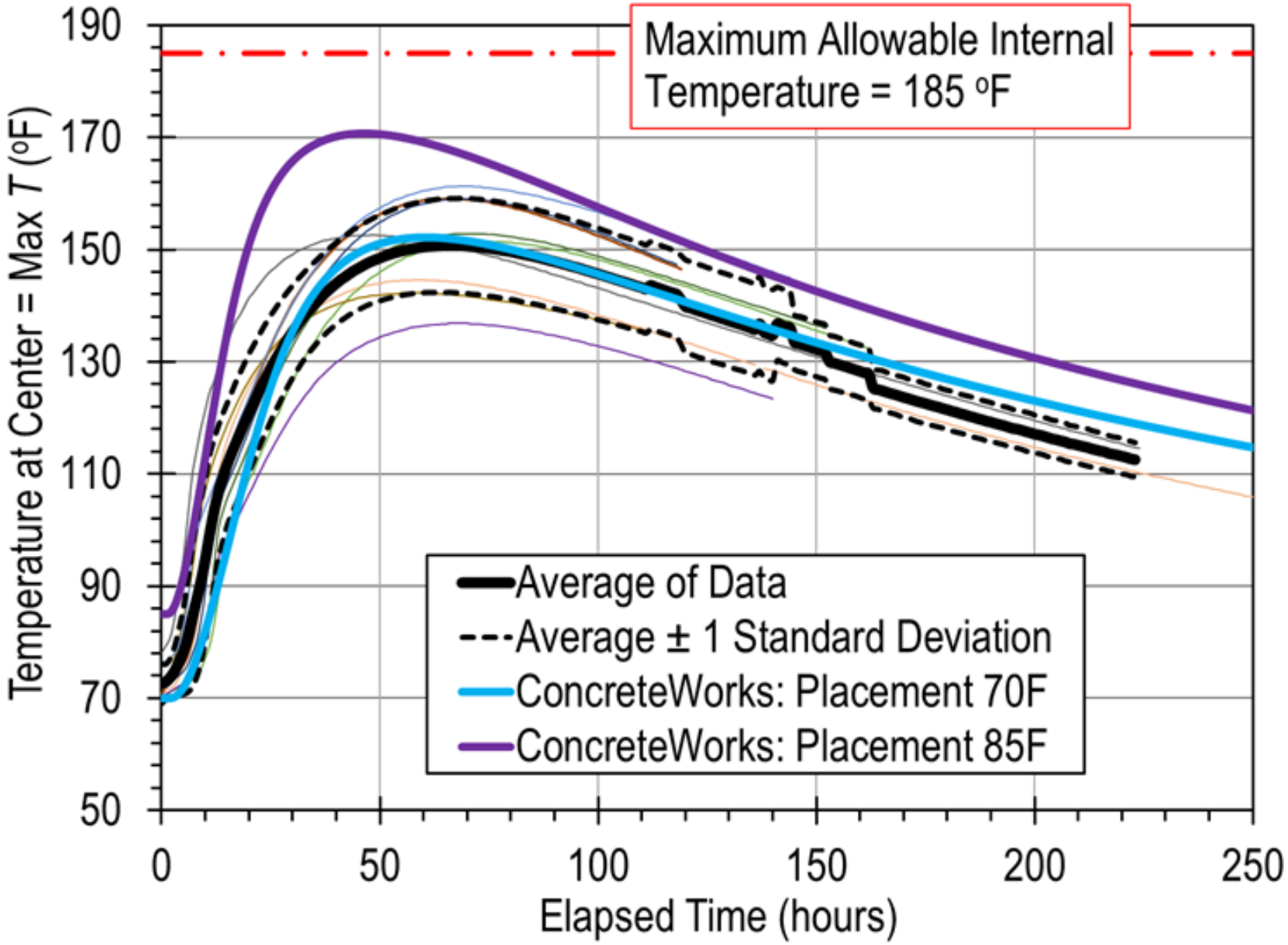


4. How long should drilled shafts be monitored?

Background:

- Most above-ground mass concrete specifications tie the monitoring duration to ambient temperature.
- Above ground, ambient temperature has big influence on differentials.
- Below ground, ambient temperature has zero influence on differentials.
- Below ground, temperature-time curves all follow a similar shape (but the magnitudes vary!)




Maximum Allowable Internal
Temperature = 185 °F
Average of Data
Average ± 1 Standard Deviation
ConcreteWorks: Placement 70F
ConcreteWorks: Placement 85F
Temperature at Center = Max T (°F)
190
170
150
130
110
90
70
50
0
50
100
150
200
250
Elapsed Time (hours)

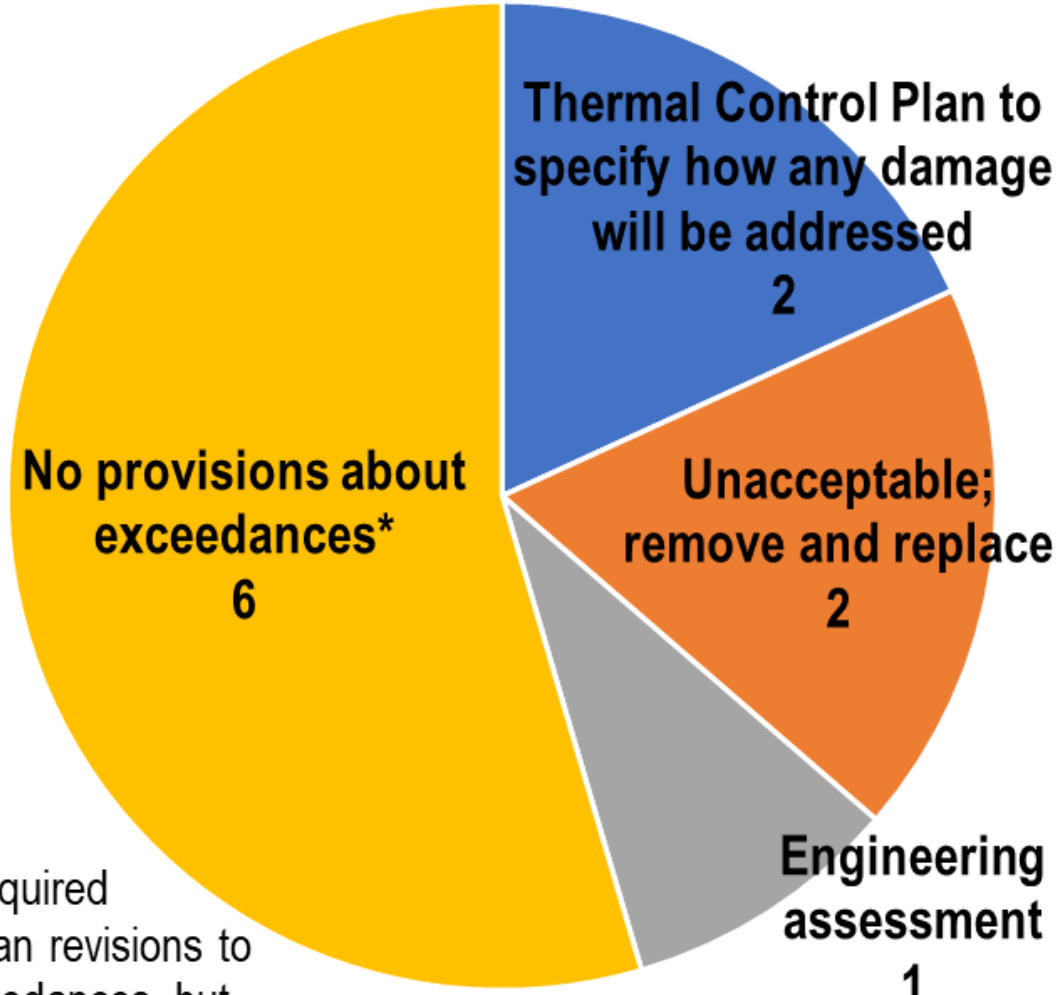
4. How long should drilled shafts be monitored?

Answer:

- Monitor until 24 hours after
  - The temperature in the center of shaft has peaked.
  - The temperature is less than the limit.



5. What should the consequences of exceedance be?

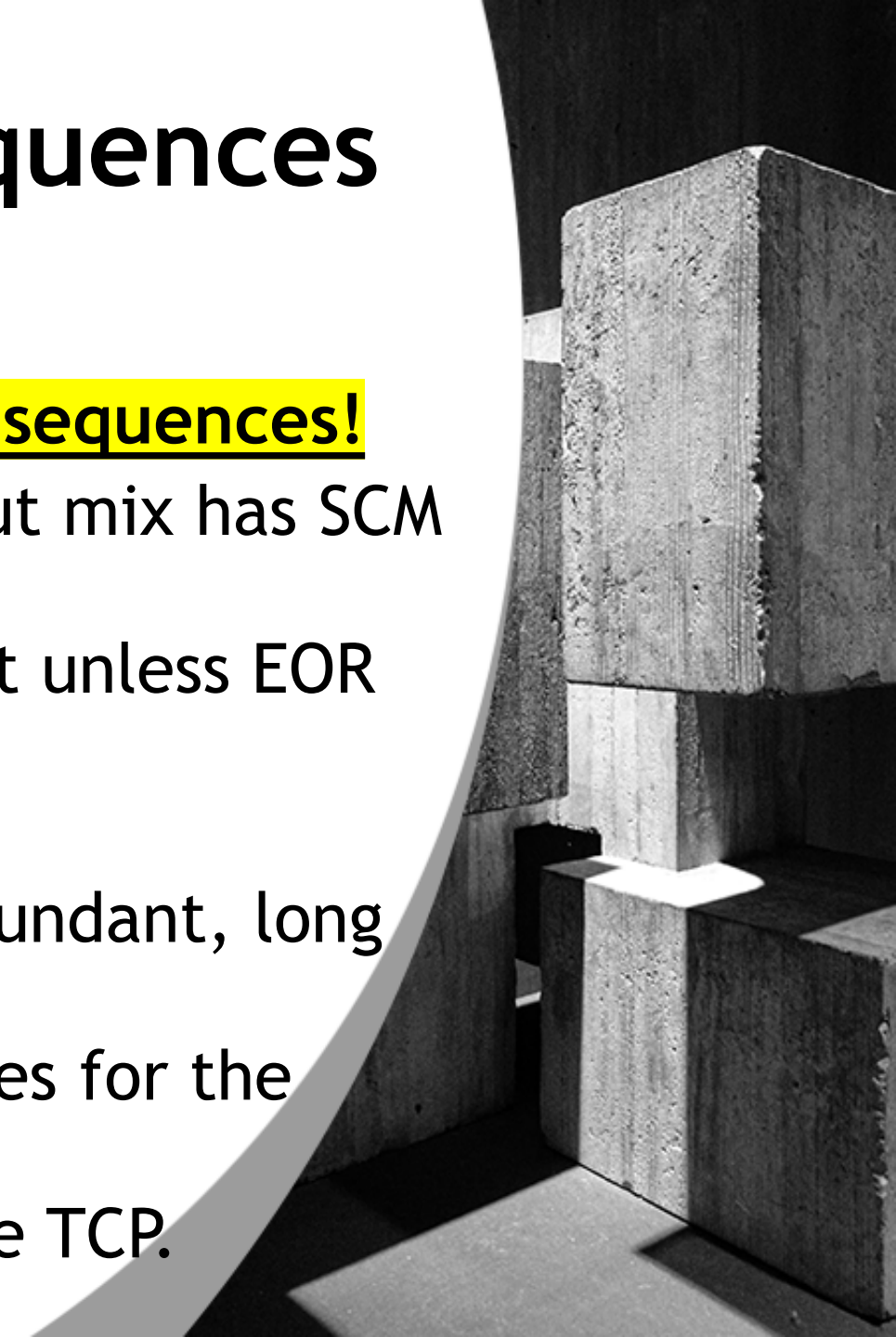


*Three of the six required Thermal Control Plan revisions to prevent future exceedances, but did not address potential damage.



5. What should the consequences of exceedance be?

- **Specifications should be explicit about consequences!**
- If temperature is between 160 and 185 °F but mix has SCM to satisfy ACI 201, accept.
- If temperature exceeds ACI 201 limits, reject unless EOR can justify acceptance.
- Consequences of rejection are huge!
- Consequences of bad concrete for a non-redundant, long service life shaft are huge!
- TCP should be revised to prevent exceedances for the remaining mass concrete shafts.
- If differential applies and is exceeded, revise TCP.

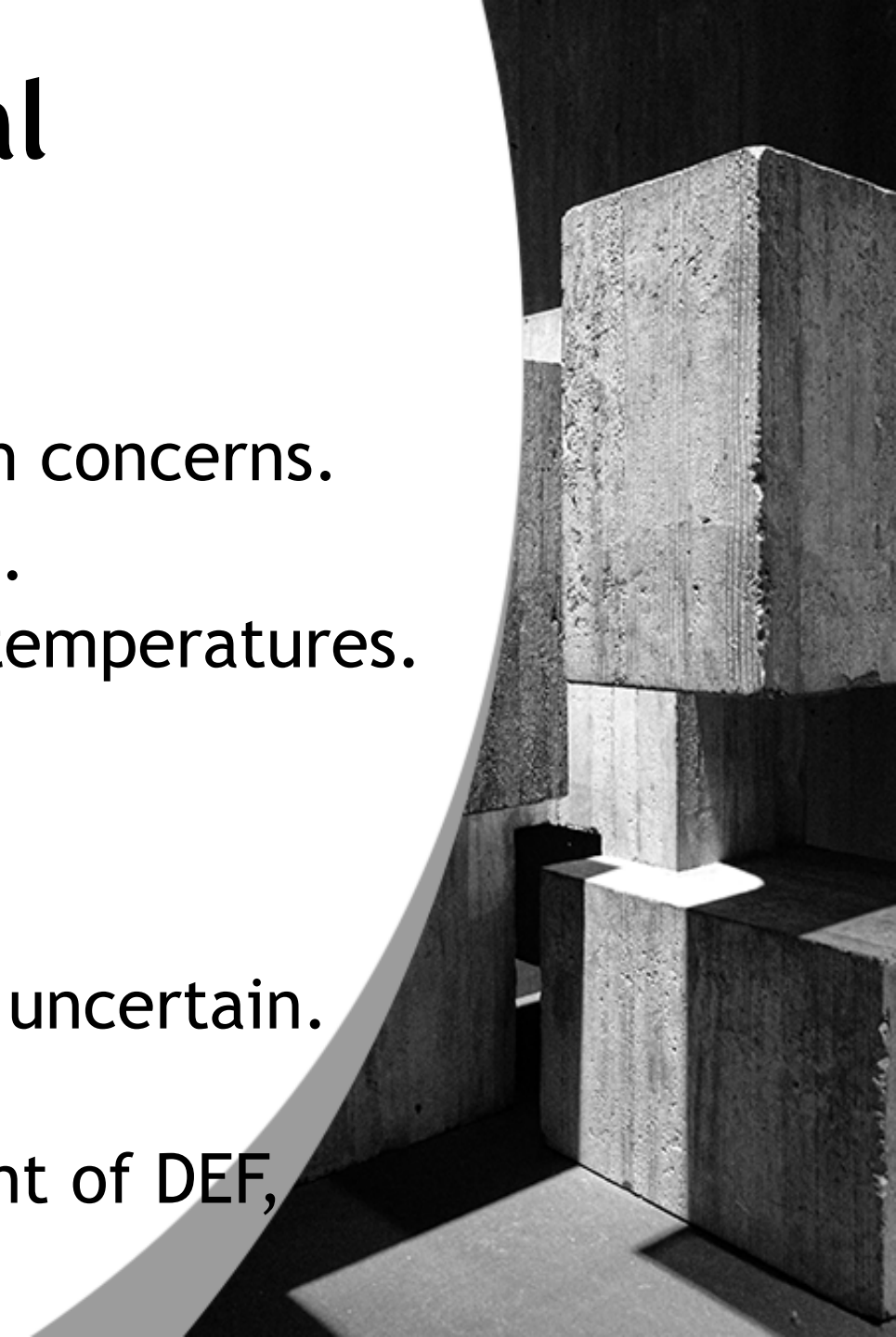


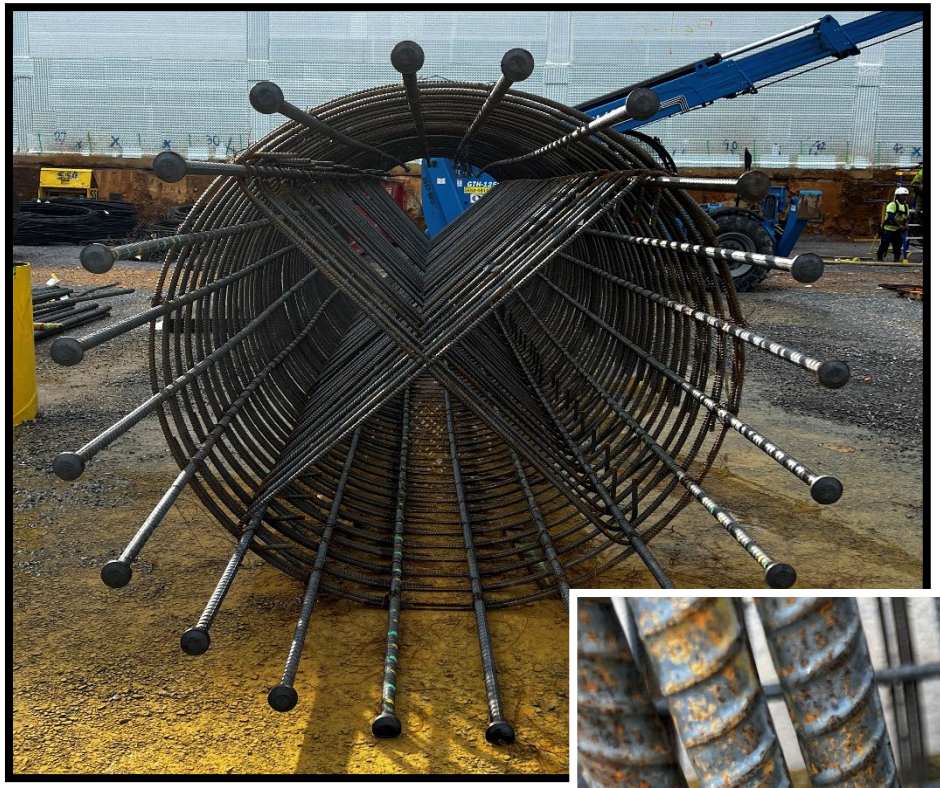
6. What about potential strength loss?

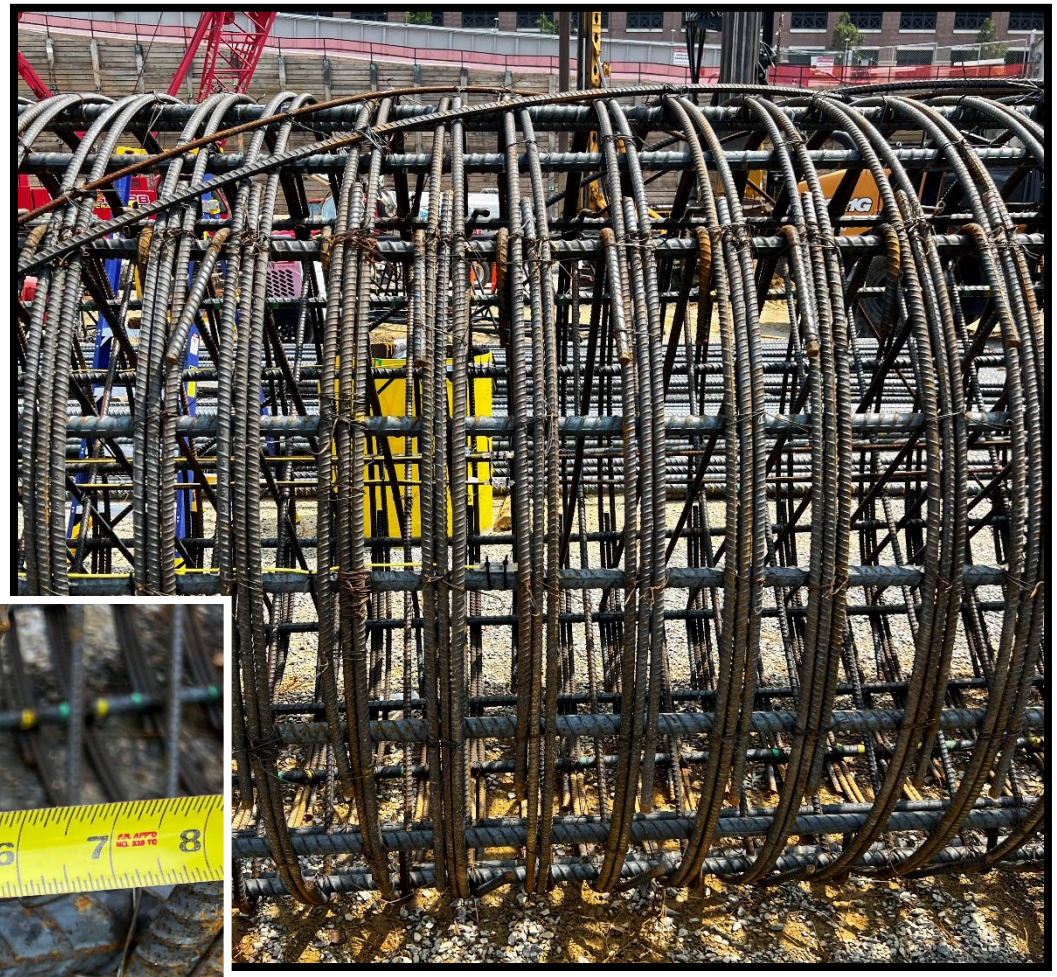
Background:

- Some SEs will not allow 185 °F, citing strength concerns.
- Concrete strength is affected by temperature.
  - At early ages, strength is greater for high temperatures.
  - In long term, strength is reduced.

  → It's a valid concern!
- BUT!
  - The effect is poorly understood and highly uncertain.
  - The effect is less with SCMs.
  - The phenomenon is completely independent of DEF, and does not change at 160 or 180 °F.








STANLEY
2
3
4
5
6
7
8

6. What about potential strength loss?

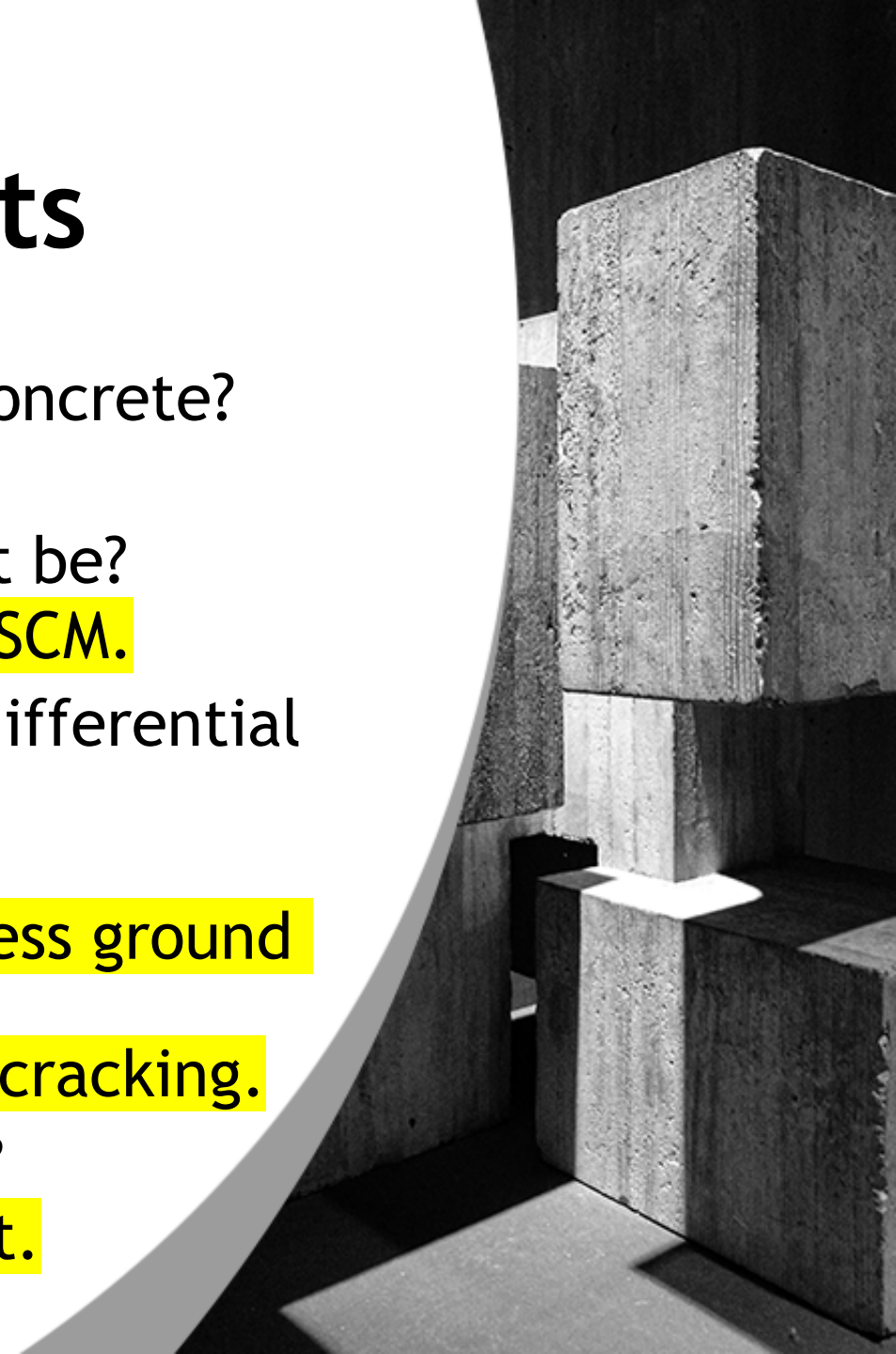
Answer:

- Keep it out of the mass concrete spec!
- At least three methods would be more effective than a blanket 160 °F limit:
  1. Specify strength values that exceed the design value by a margin that accounts for temperature effects.
  2. During trial batching, cure specimens at 185 °F and 73 °F to quantify the temperature effect and adjust the acceptance value accordingly.
  3. Match cure and design with safety margin.



Answers for Drilled Shafts

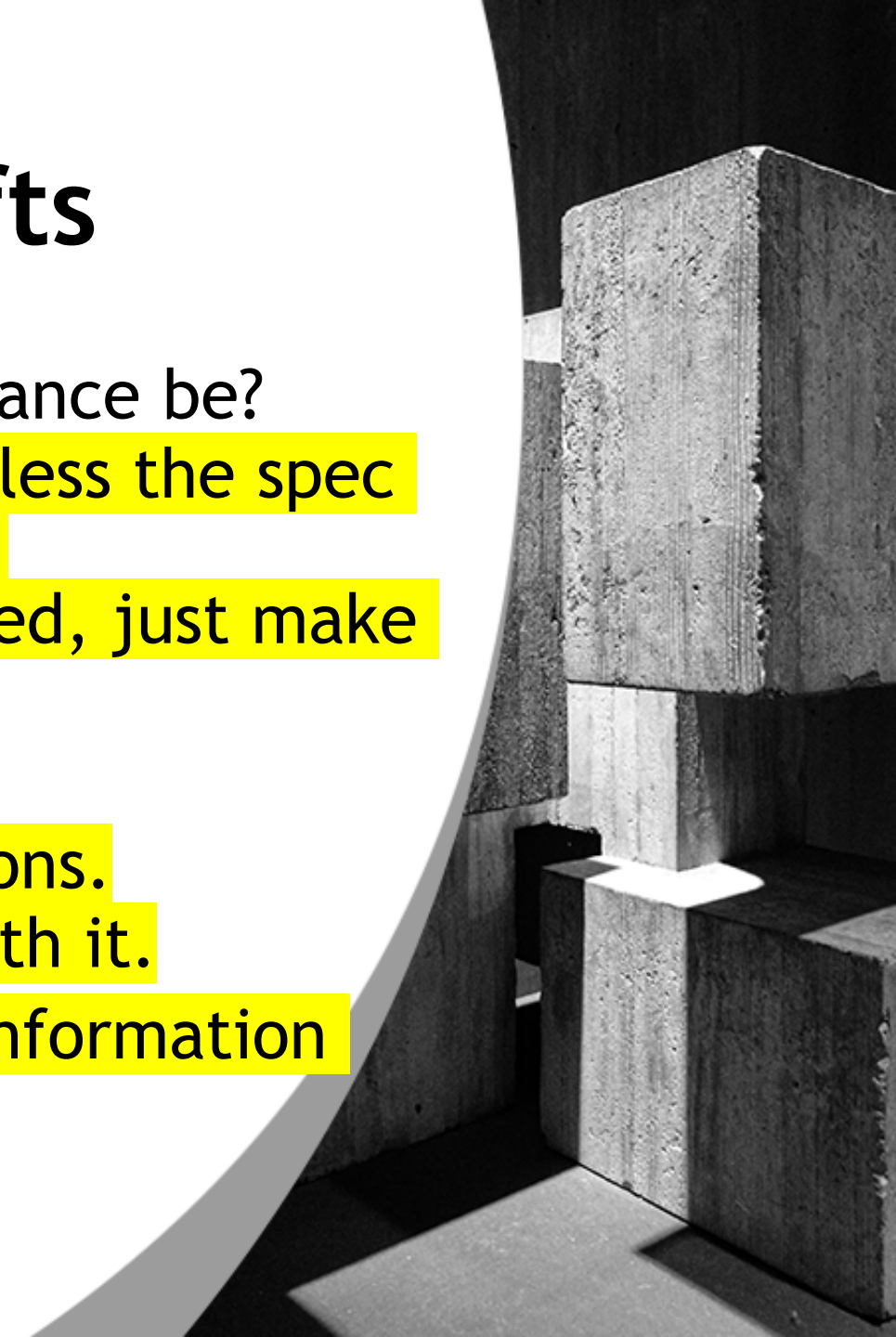
1. Should drilled shafts be exempt from mass concrete?
   - No.
2. What should the maximum temperature limit be?
   - 160 °F without SCM; 185 °F with sufficient SCM.
3. What should be the maximum temperature differential be?
   - Do not apply inside permanent casing.
   - Do not apply without permanent casing unless ground conditions are aggressive.
   - Account for reinforcement and permissible cracking.
4. How long should drilled shafts be monitored?
   - 24 hours after peak and after less than limit.



Answers for Drilled Shafts

5. What should the consequences of exceedance be?
   - Exceeding max temp is unacceptable unless the spec is conservative (e.g., 160 °F with SCMs).
   - If differential limit imposed and exceeded, just make adjustments.
6. What about potential strength loss?
   - Keep it out of mass concrete specifications.
   - There are more rational ways to deal with it.

- Paper answers these questions with more information than can be included in 20 minutes.



Thank you

Questions?

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Drilled Shaft Load Tests in Weakly Cemented Sandstone

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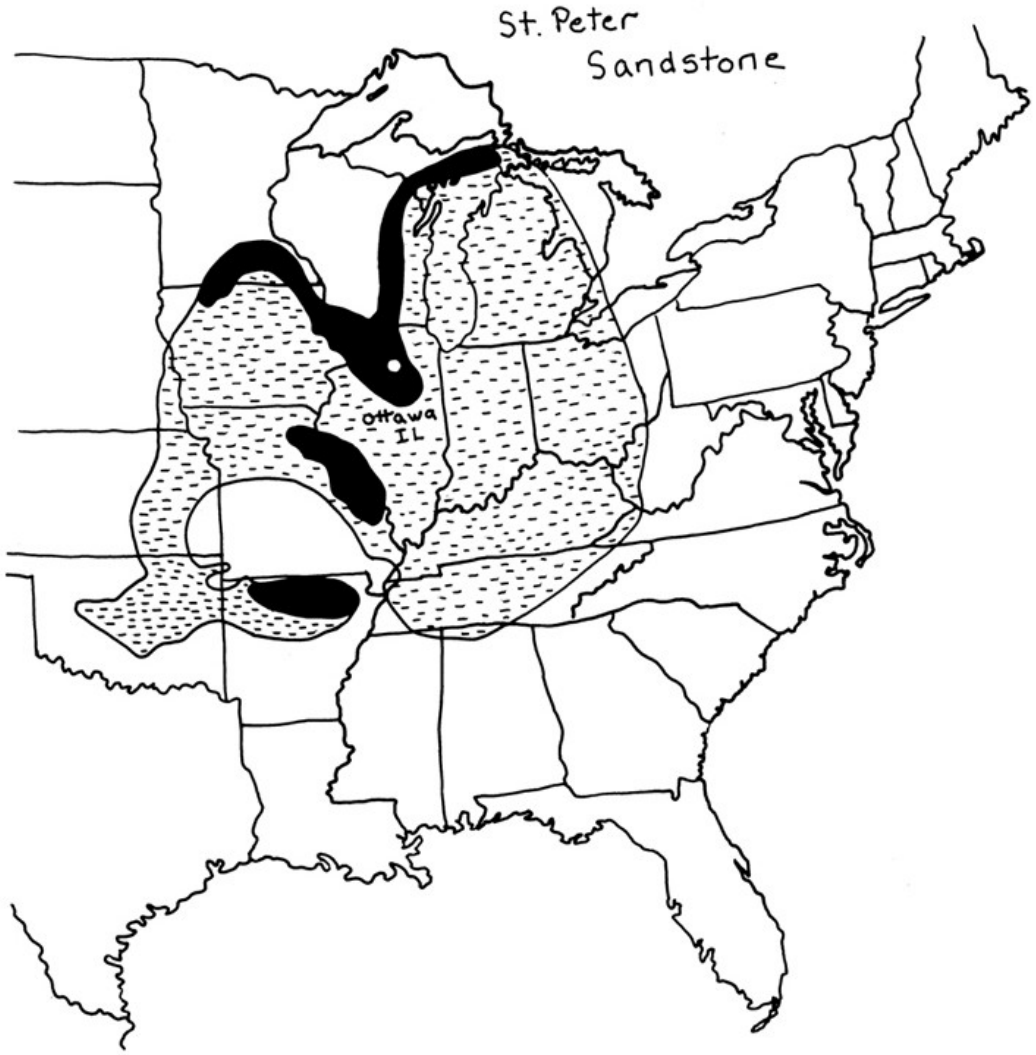
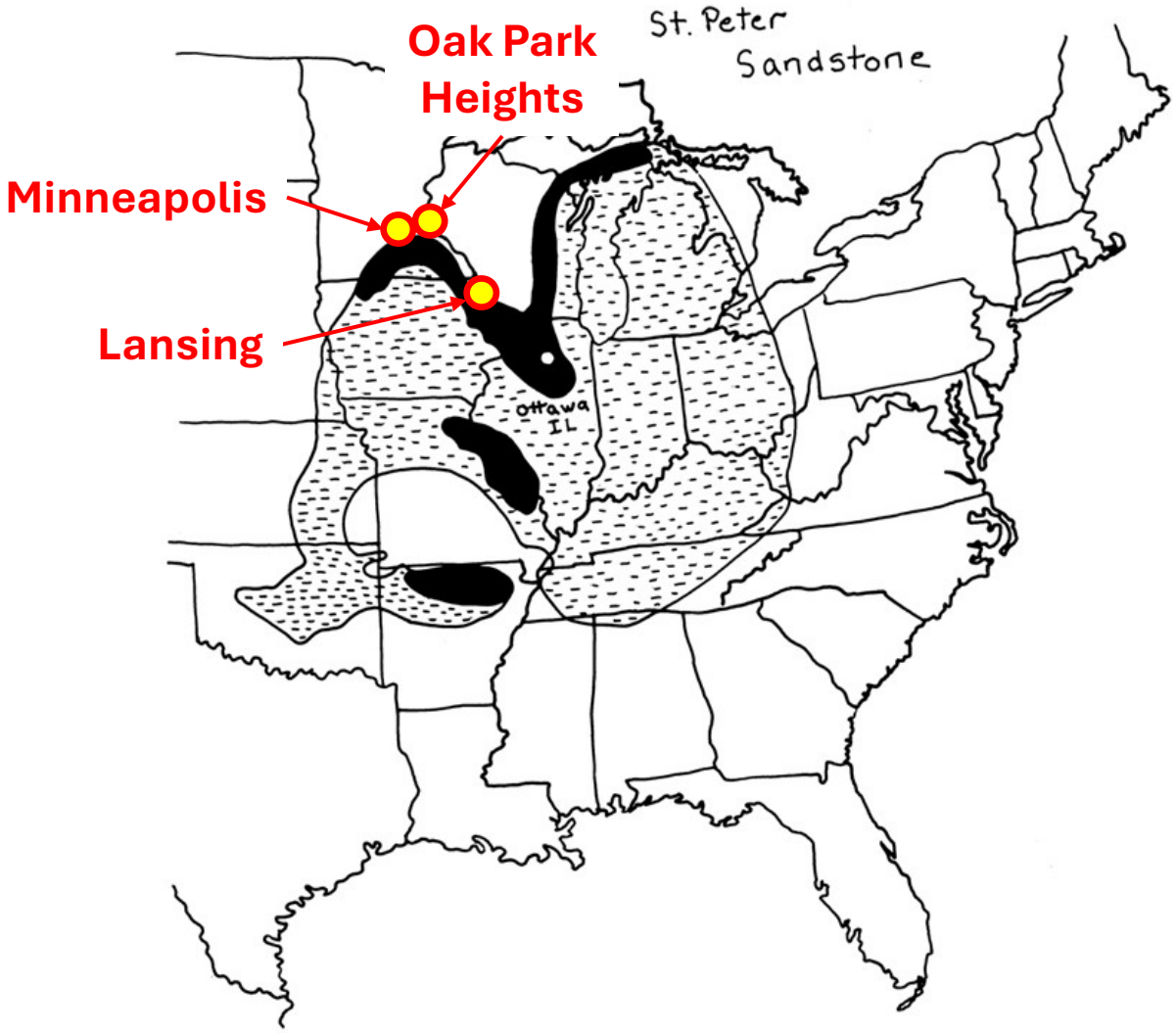
Sandstone Across the Midwestern United States



Load Test Sites



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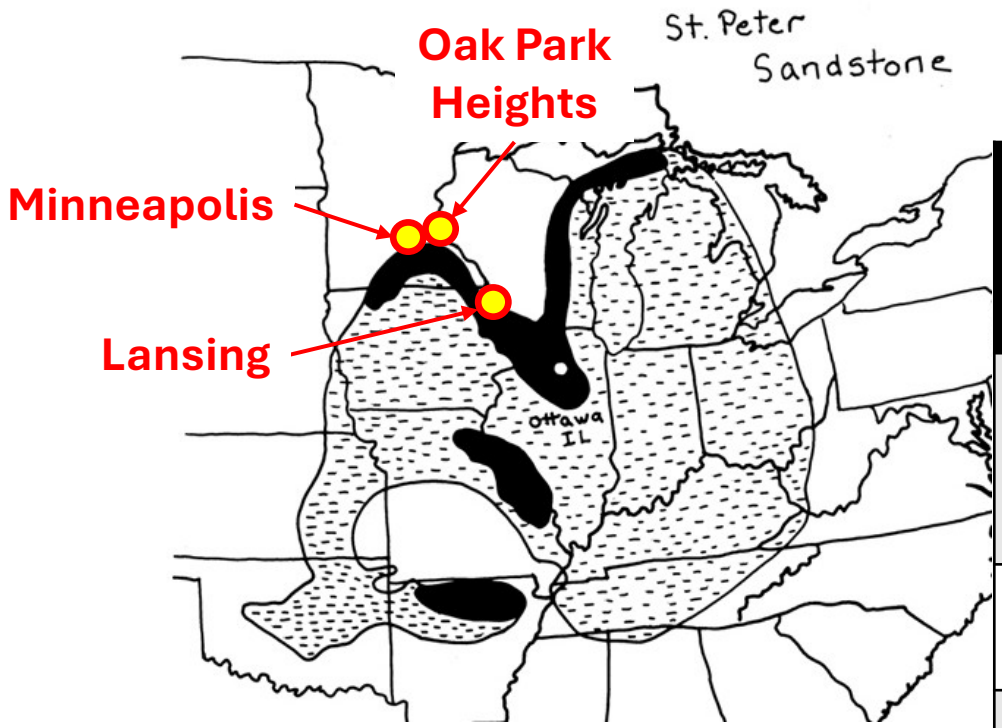
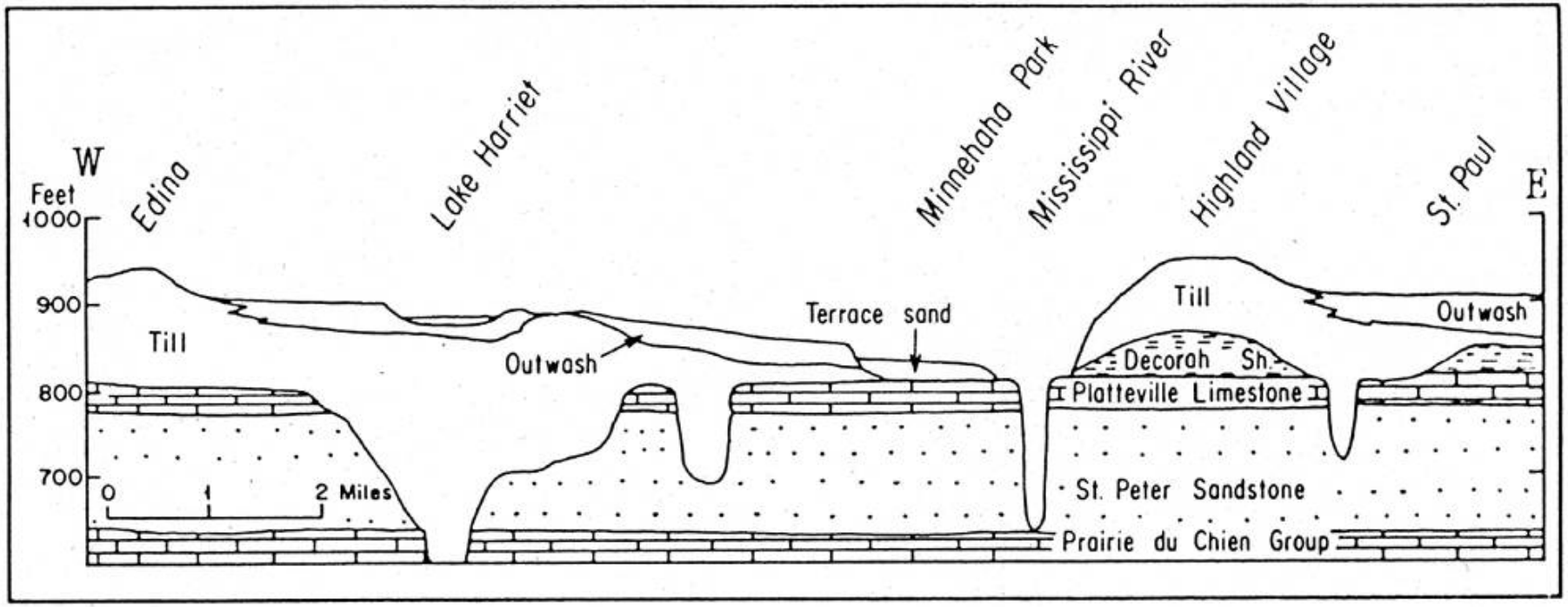


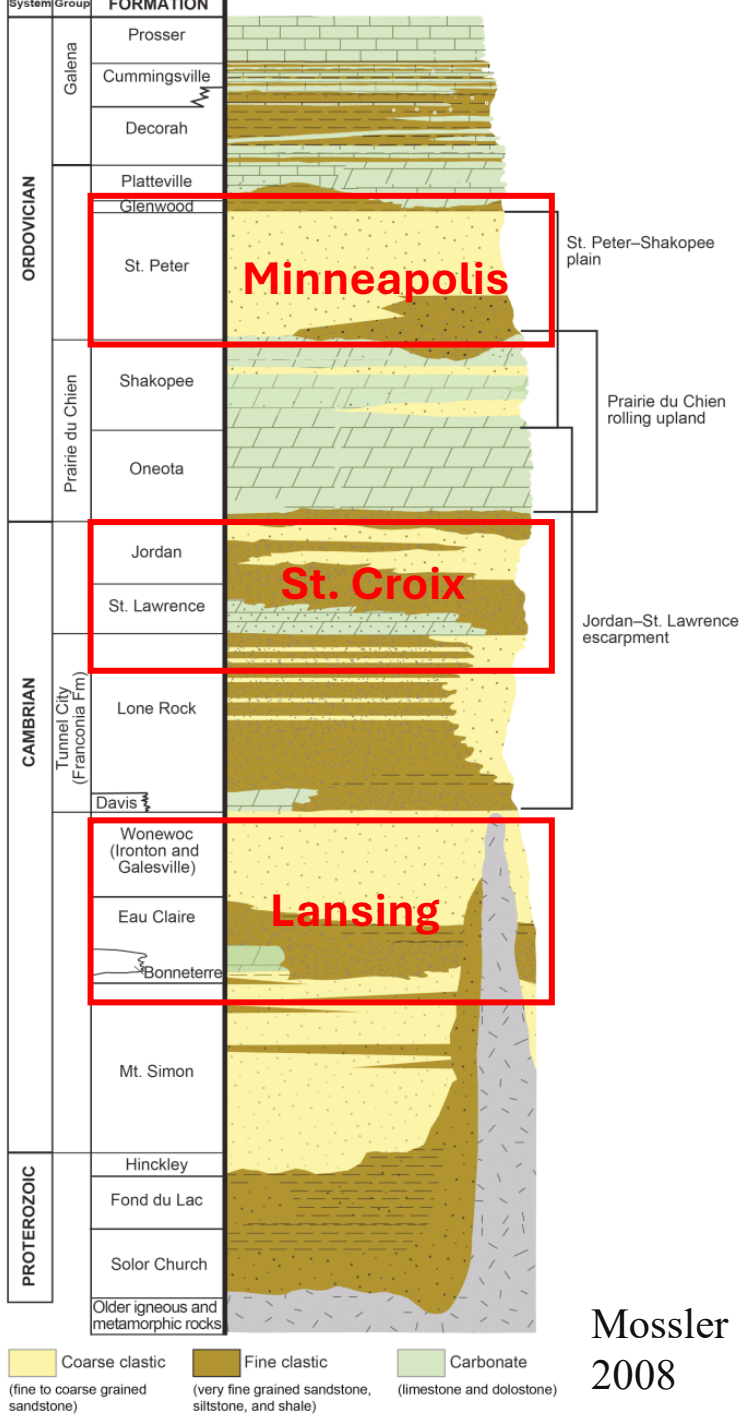
Project	Location	Test Date	Bi-Directional Jacks	Rock Socket Diameter (ft)	Base Load Plate Diameter (ft)	Rock Socket Length (ft)
I-35W St. Anthony Falls Bridge	Minneapolis, MN	November 22, 2007	2 levels of 4 26-inch jacks at the base and 10.0 ft above the base	6.5	5.4	39.9
St. Croix Crossing Bridge	Oak Park Heights, MN	August 27, 2012	3 34-inch jacks 4.8 ft above the base	7.5	-	30.9
IA-9 Black Hawk Bridge	Lansing, IA	February 13, 2024	3 24-inch jacks at the base	8.5	6.0	46.8

Geology

- Paleozoic sedimentary bedrock layers
- Horizontally bedded

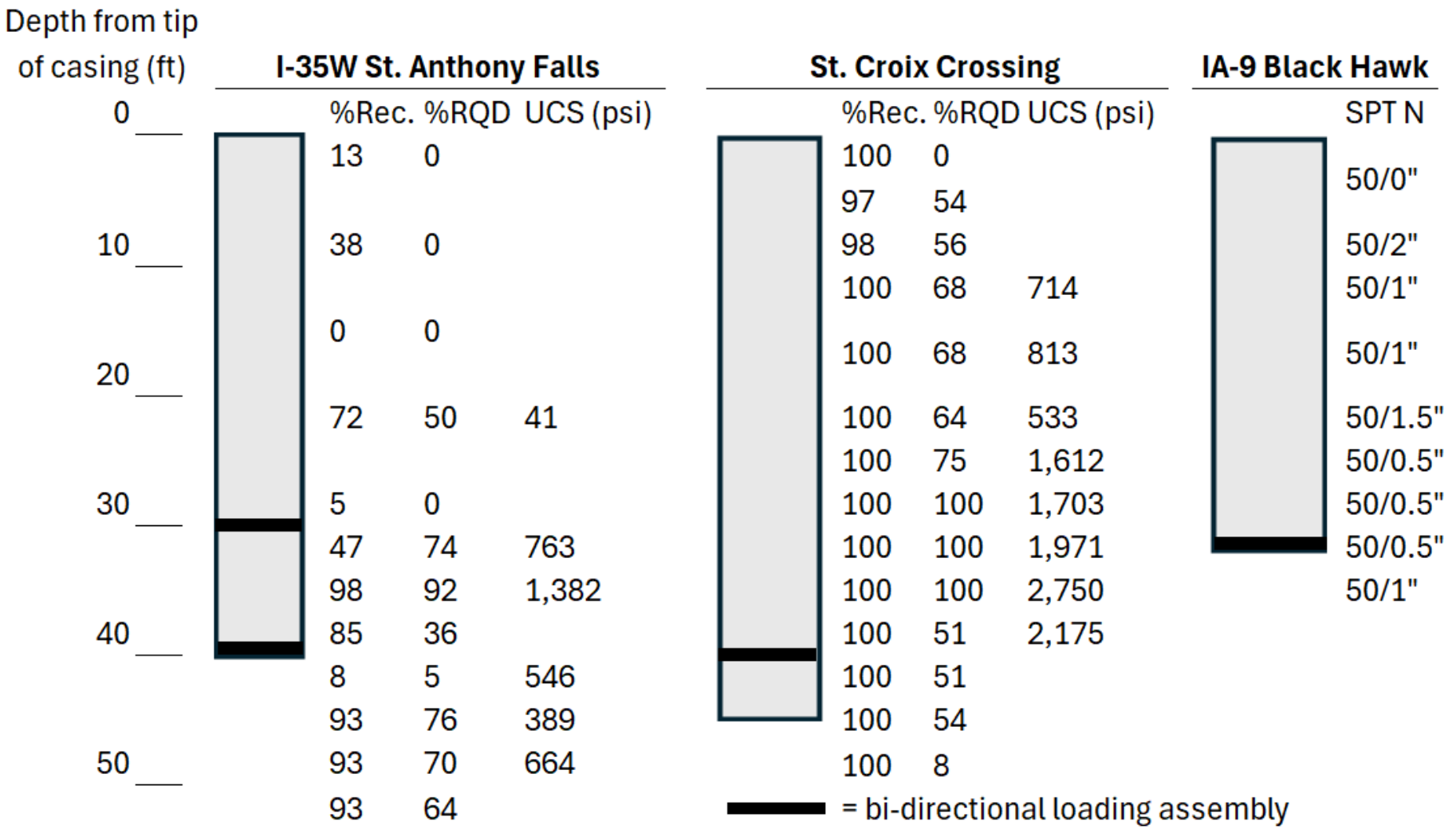


East-West cross section south of Minneapolis (Wright 1972)



Mossler 2008

Rock Socket Conditions



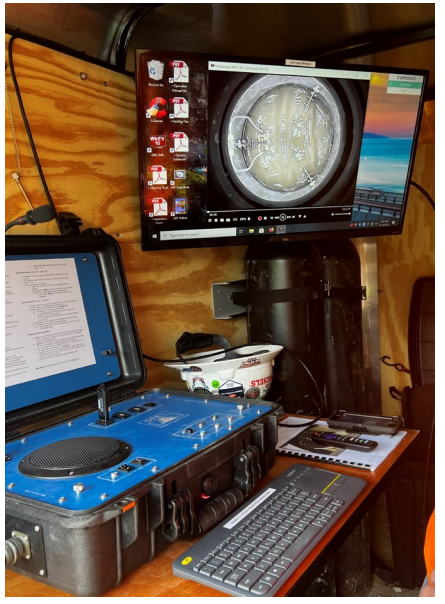
Construction



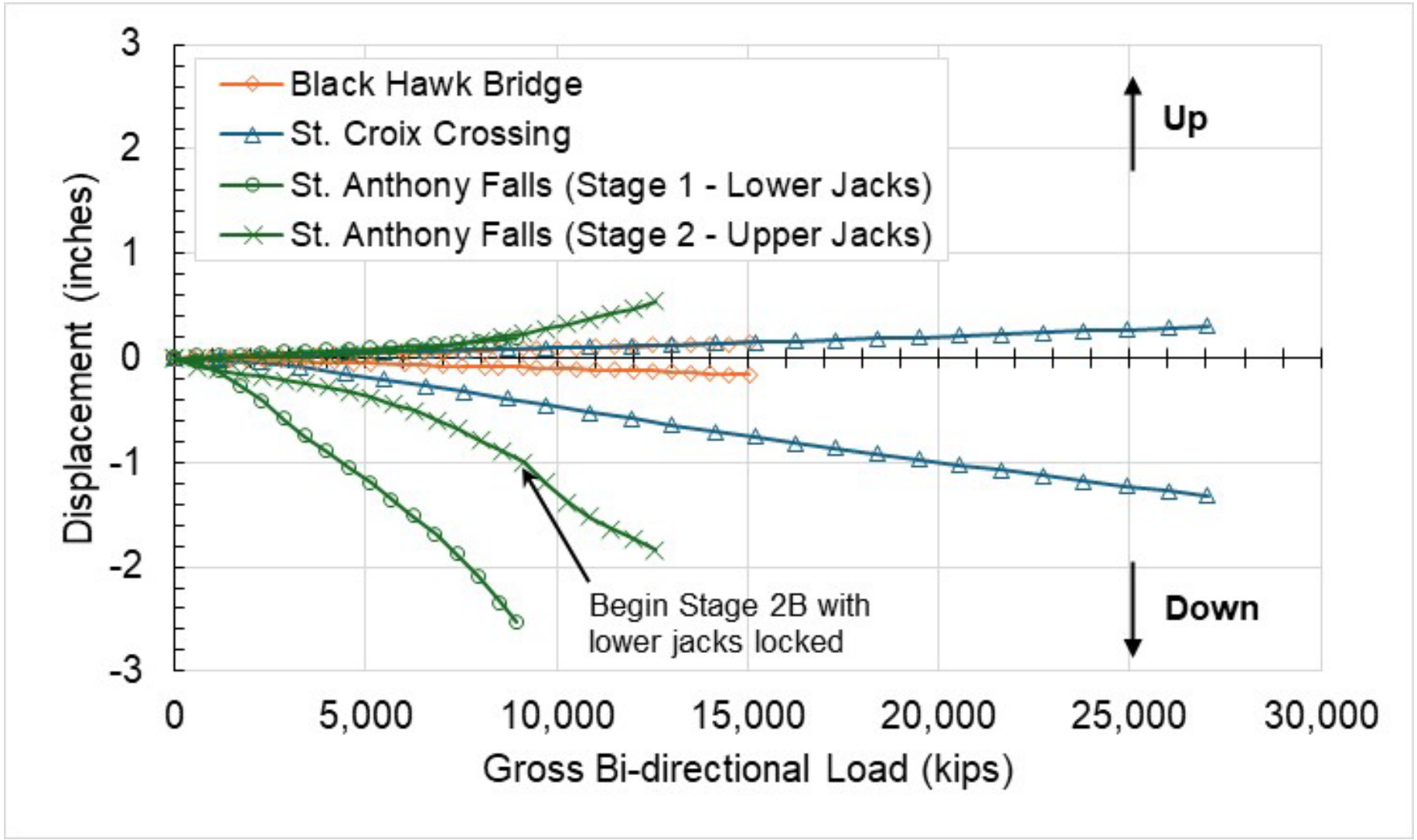




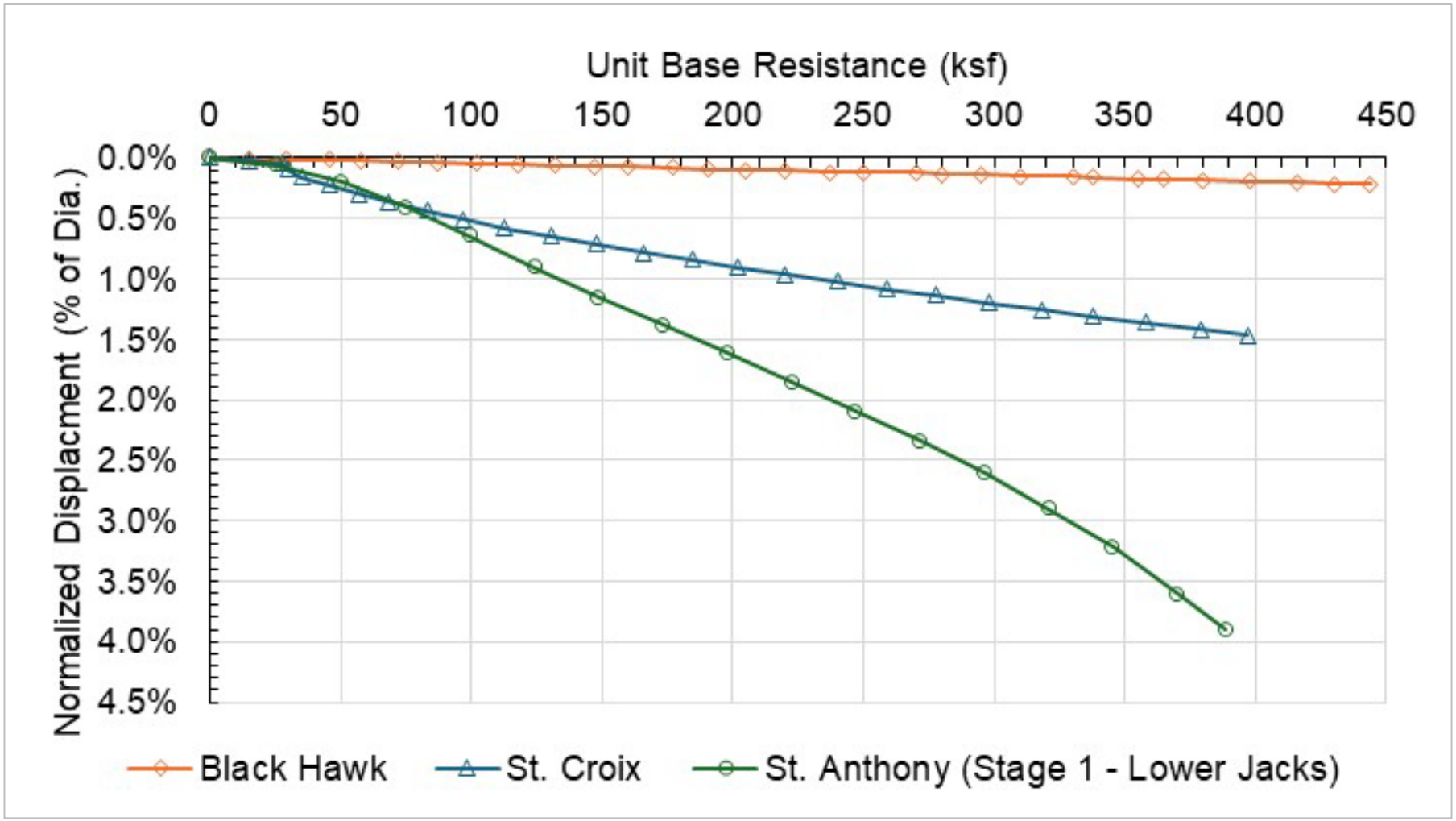




Load Test Results: Butterfly Graph



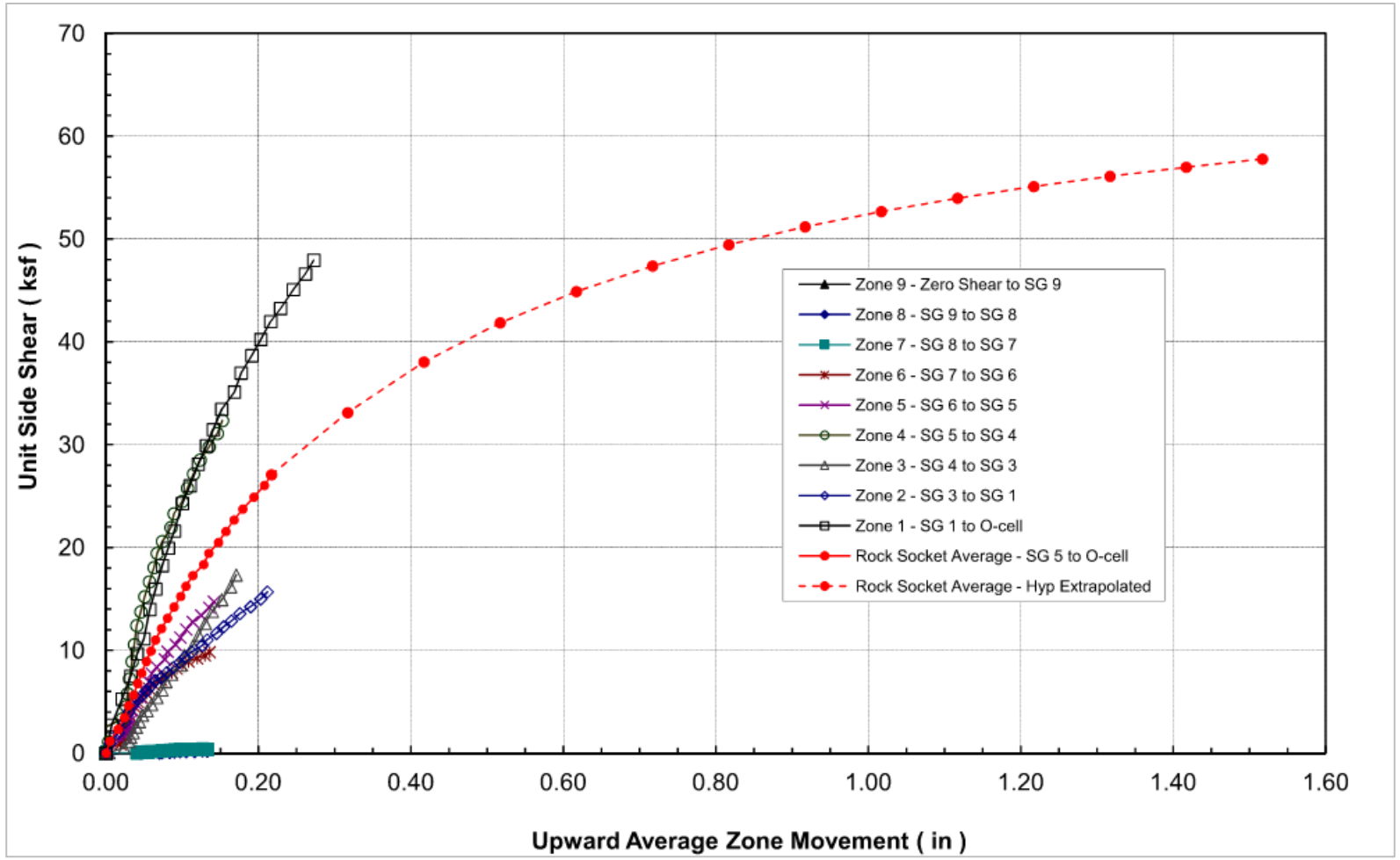
Load Test Results: Normalized q-z



Load Test Results: Unit Side Resistance

Project	Average Maximum Mobilized Unit Side Resistance (ksf)	Maximum Mobilize Unit Side Resistance, Single Segment (ksf)	Displacement at Maximum Mobilized Unit Side Resistance (in.)
St. Anthony Falls (Stage 2 – Upper Jacks)	31.3	54.2	0.64
St. Croix Crossing	25.2	47.9	0.27
Black Hawk Bridge	16.6	30.3	0.14

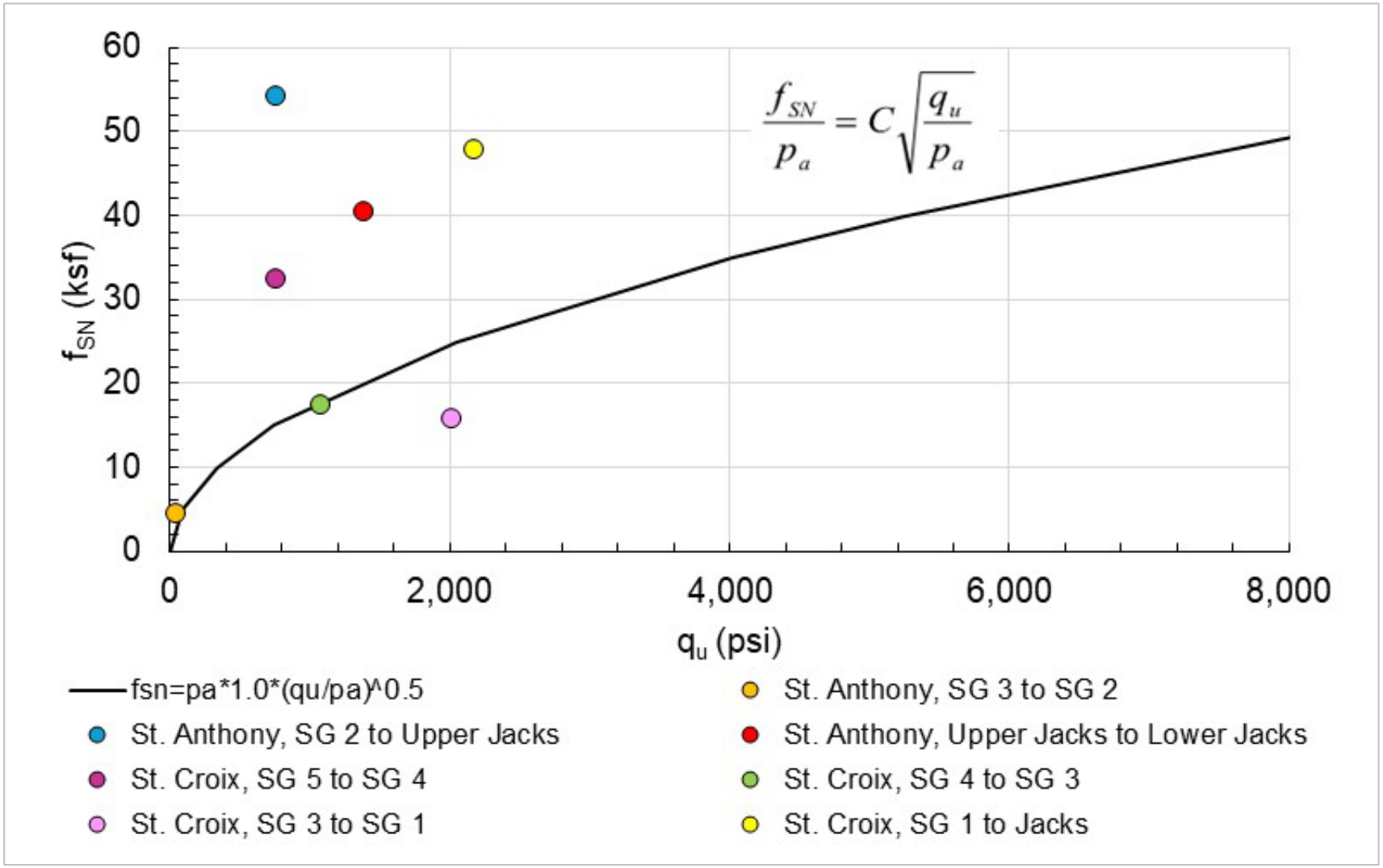
Load Test Results: t-z



St. Croix, others were similar

Comparison to Empirical Design Equations

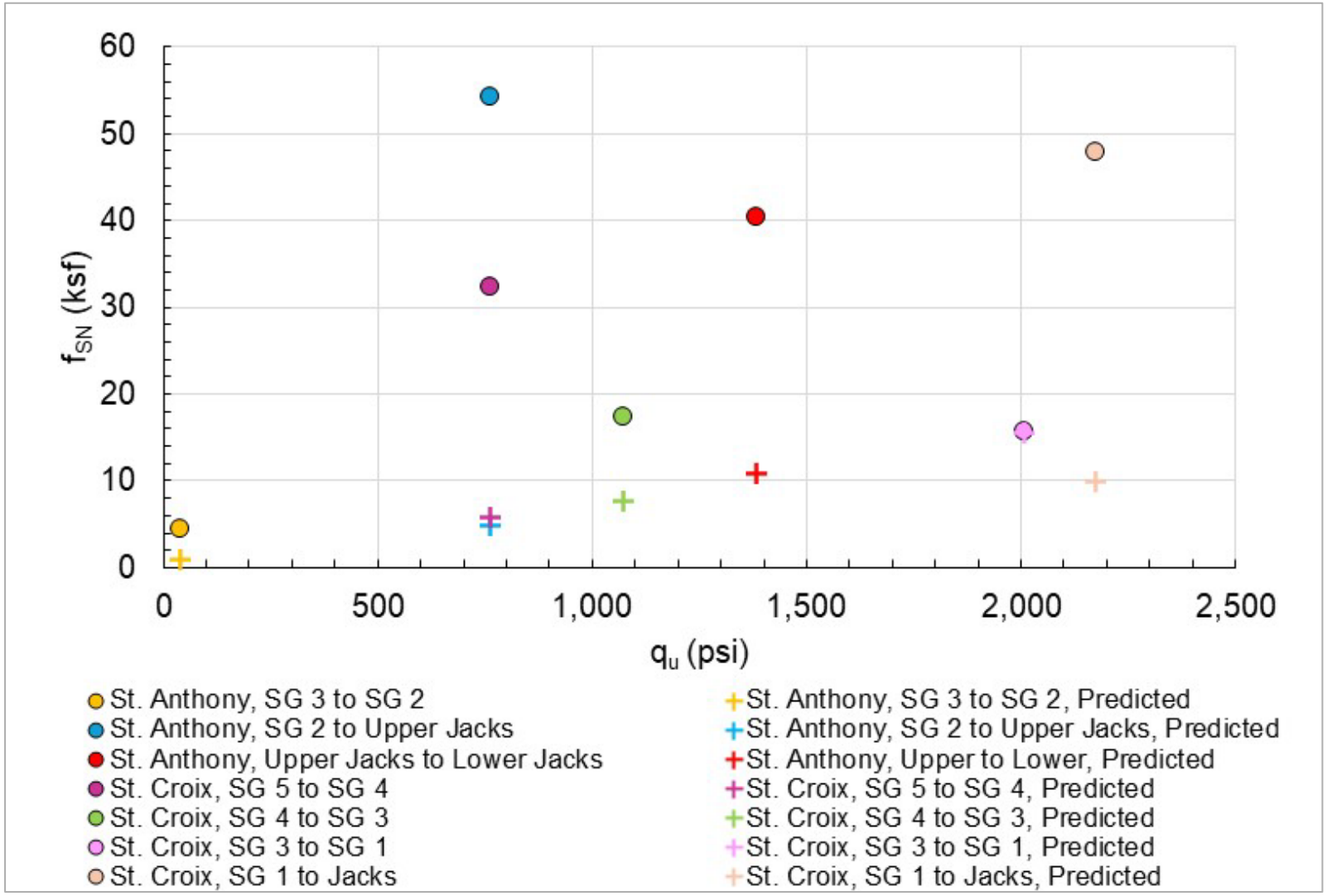
- “Normal Rock”
- Rock socket that is not susceptible to collapse
- C = 1.0
- No data for Lansing because there are no samples for q_u



Comparison to Empirical Design Equations

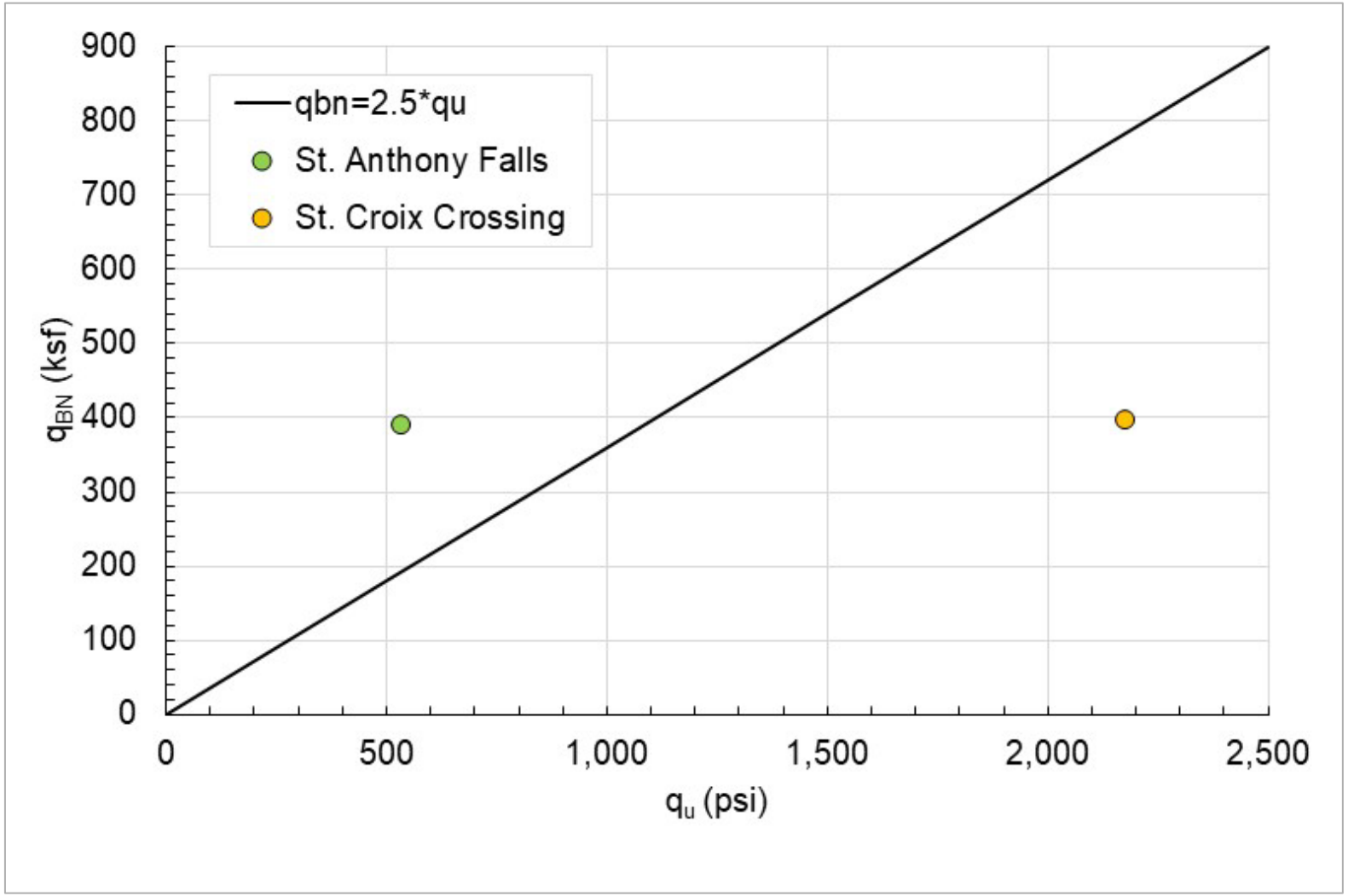
- Unstable rock
- C = 0.65
- $\alpha_E = f(\mathrm{RQD})$
- $\alpha_E = 0.38$ to 0.96

$$\frac{f_{SN}}{p_a} = 0.65\alpha_E\sqrt{\frac{q_u}{p_a}}$$



Comparison to Empirical Design Equations

- "Normal Rock"
- No data for Lansing because there are no samples for q_u
- Measured values are not fully mobilized



Discussion

- Construction practices have a significant influence on axial performance (Axtell et al. 2013).
- Managing groundwater is critical, particularly artesian conditions, axial performance can be impacted.
- Support fluid management, cleanout, concrete mix design, concrete placement, and quality management are critical to axial performance in weakly cemented sandstone.
- These load tests allowed production drilled shaft rock socket lengths to be reduced compared to what they would have been based on empirical design methods.
- The conditions at these sites favor load testing due to the difficulty in collecting bedrock data representative of the in-situ conditions.
- Because construction has a significant influence on performance in weakly cemented sandstone, constructing a non-production test shaft has the added benefit of serving as a demonstration shaft to validate means and methods.

Conclusions

- Weakly cemented sandstone is capable of resisting large axial loads in excess of the capacity of the bi-directional jacks.
- Side and base resistance are mobilized simultaneously at compatible displacements, a conclusion that is supported by Axtell et al. 2022.
- Traditional empirical design methods typically underestimate the unit side and base resistance in weakly cemented sandstone.
- Projects in similar bedrock may benefit from a load test.
- Proper construction and quality verification is necessary to achieve the results demonstrated by these load tests.



Thank you for your time & attention.

Questions?

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St. Croix Crossing Bridge, Oak Park Heights, MN

USACE-DFI Working Group on Seepage Control

DFI 50
Nashville, TN
October 21, 2025

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US Army Corps of Engineers.

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Teton Dam Failure
June 5, 1976



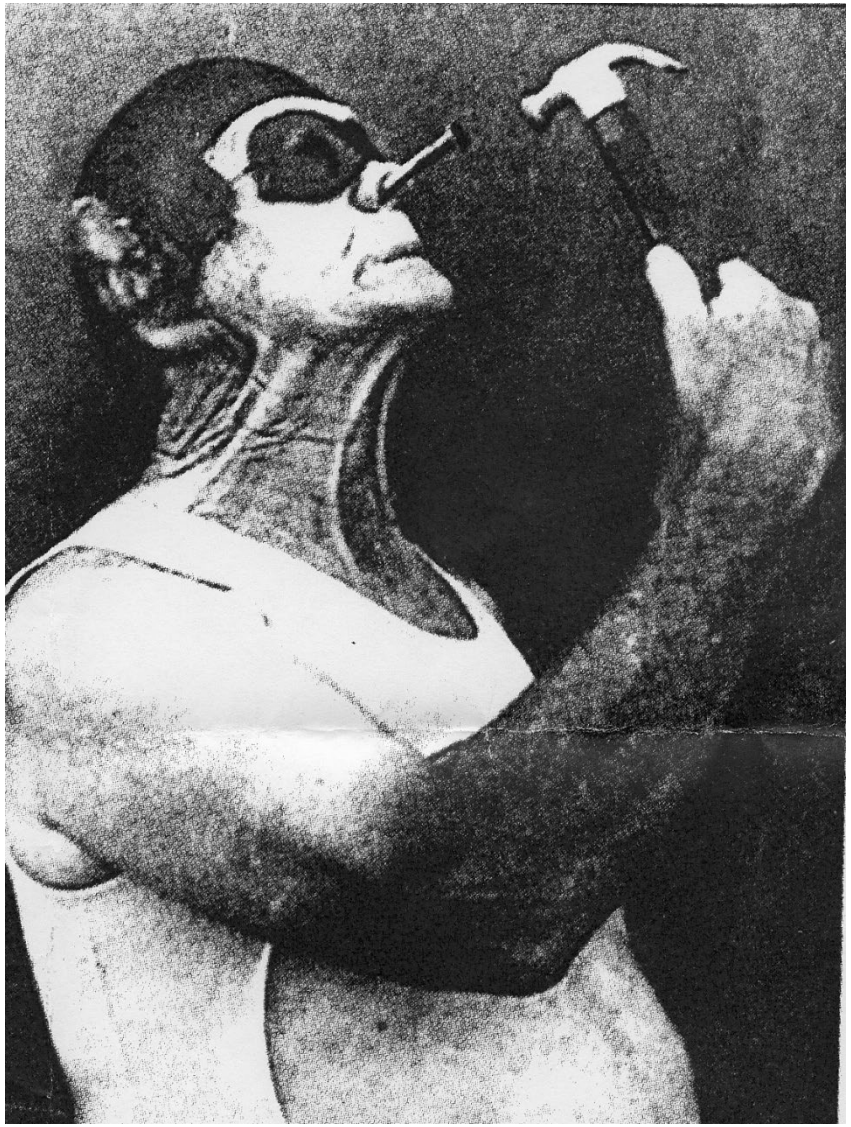
US Army Corps of Engineers

- As a result of assessment of USACE portfolio, over $3Billion was forecasted to be spent on cutoff walls in DSAC 1 dams
- USACE engaged with DFI in 2010
- Began development of the Engineering Manual (EM) on Seepage Control cutoffs

International Symposium on Seepage Barriers

- Last Symposium was in 1991 in Atlantic City, NJ sponsored by ASTM
- February 2014 in Miami, Florida
- Sponsored by Deep Foundations Institute

Modifications to Existing Embankment Dams



FIRST -
“DO NO HARM”

Navajo Dam – New Mexico

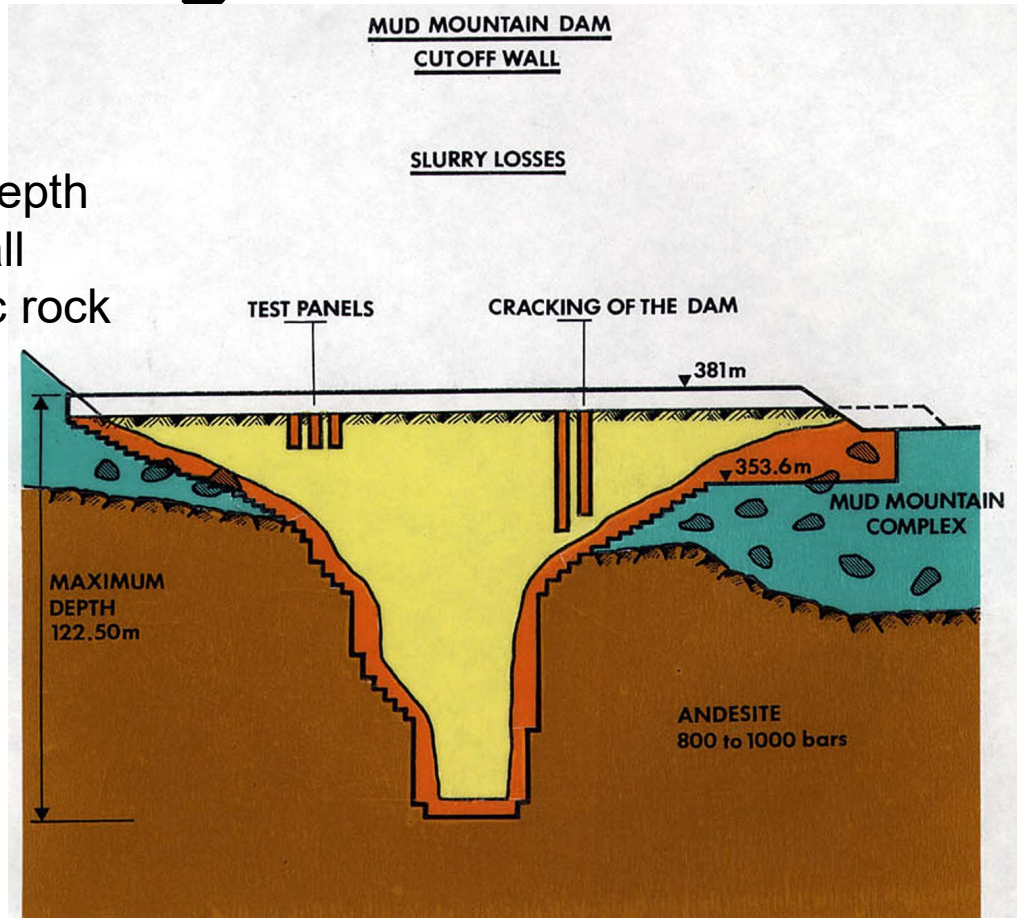




Mud Mountain Dam – Washington



700 LF of cutoff wall
- 400 FT maximum depth
- Hydro-mill slurry wall
- 20,000 psi andesitic rock
- Grouting

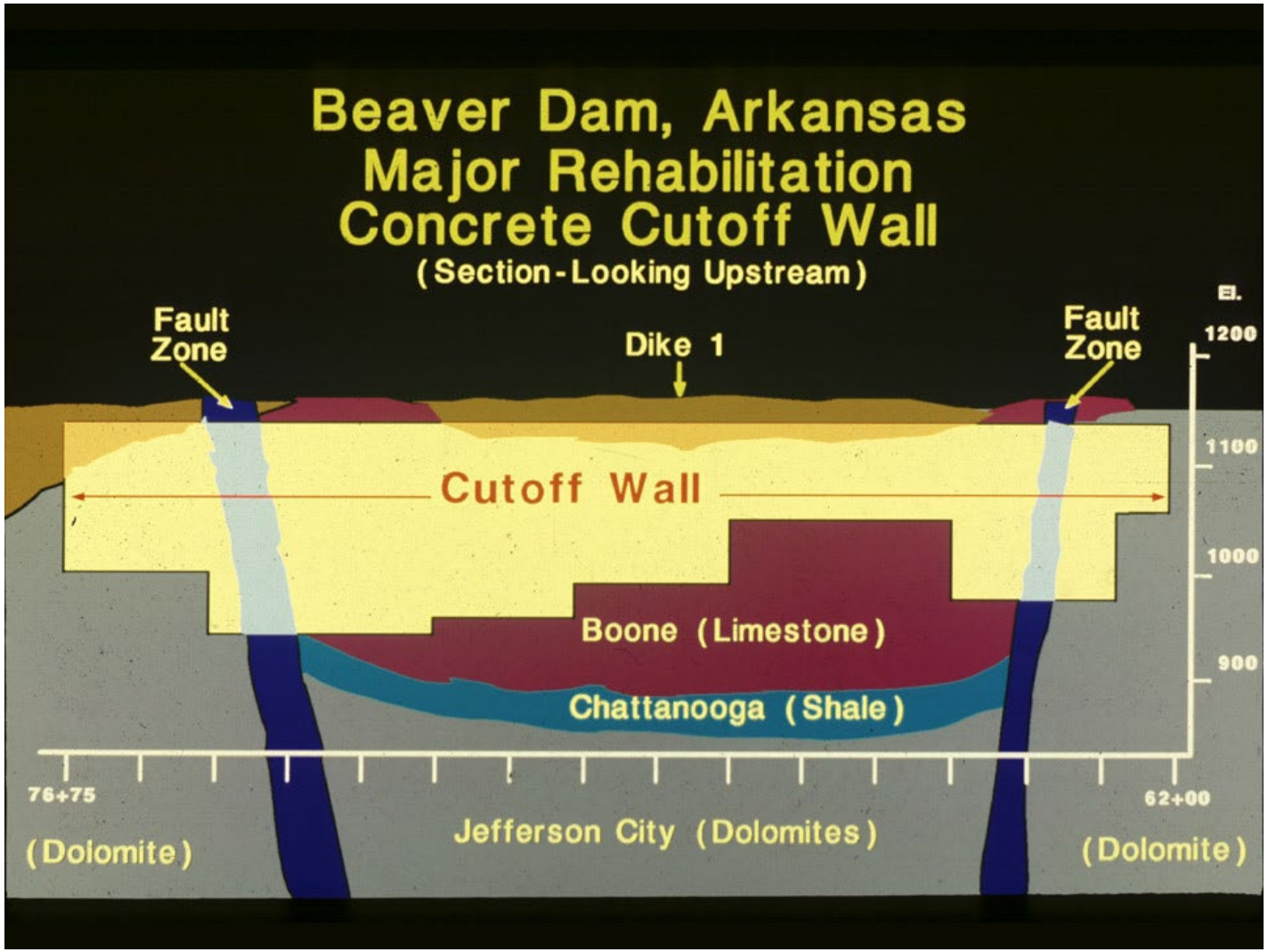


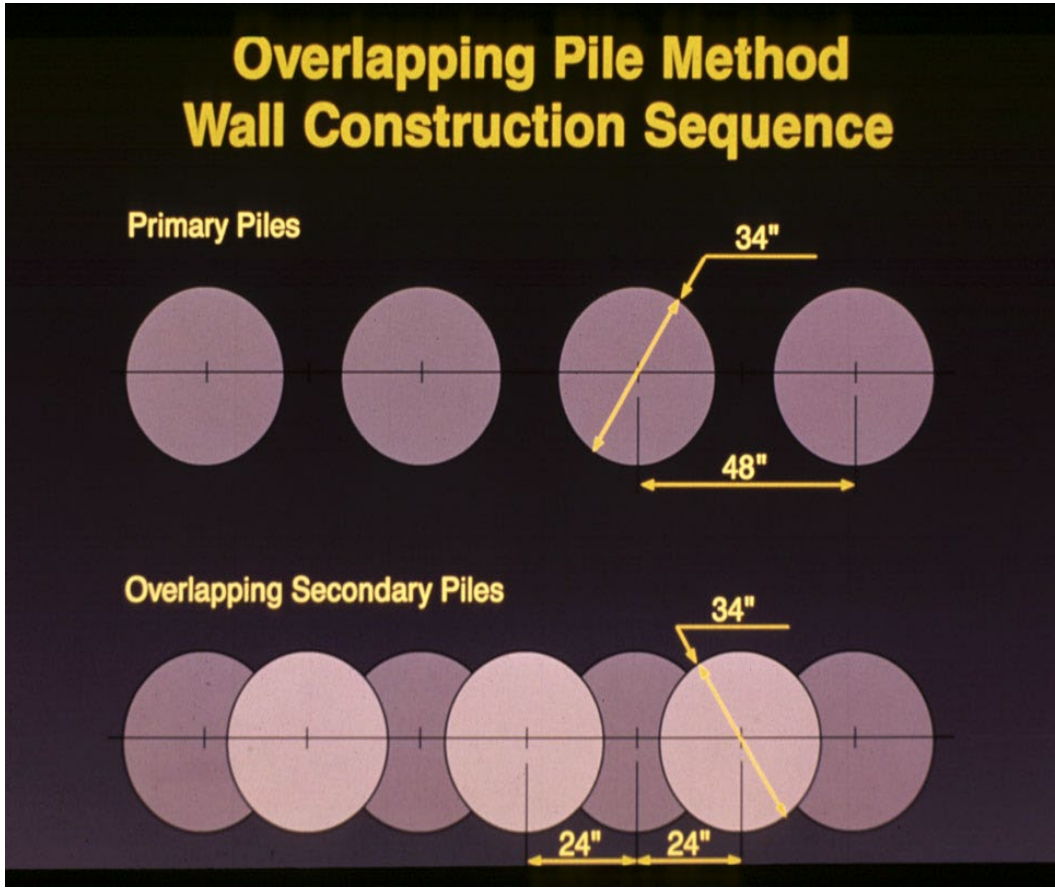
Mosul Dam- Iraq





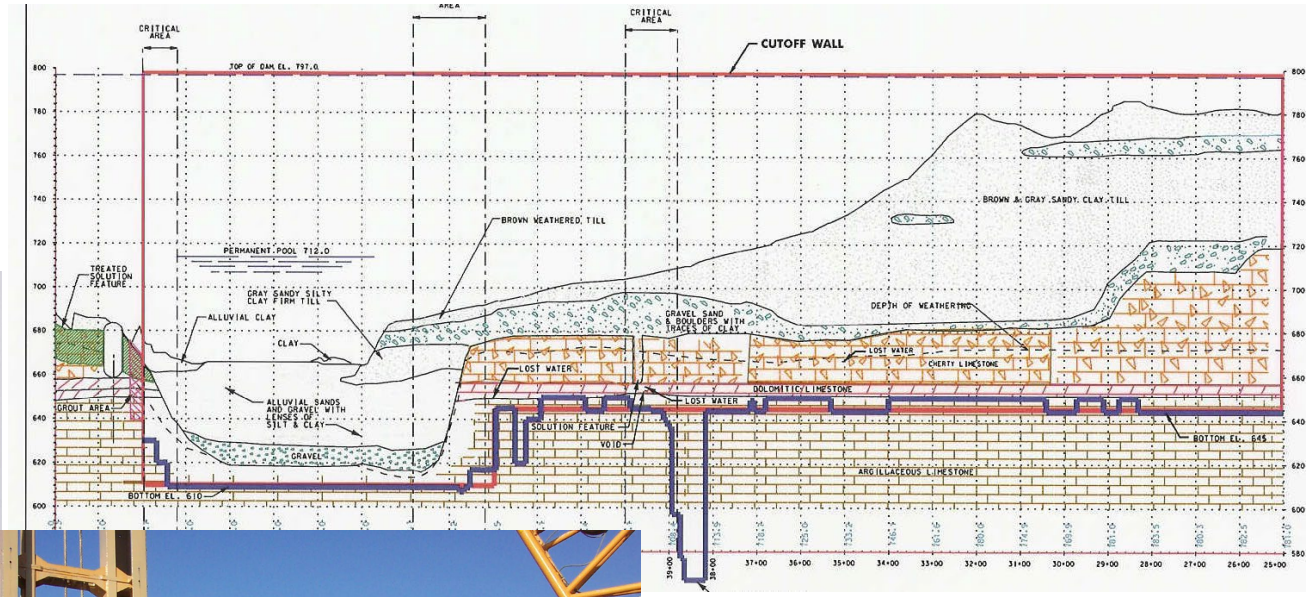
Beaver Dam - Arkansas





Mississenewa Dam - Indiana

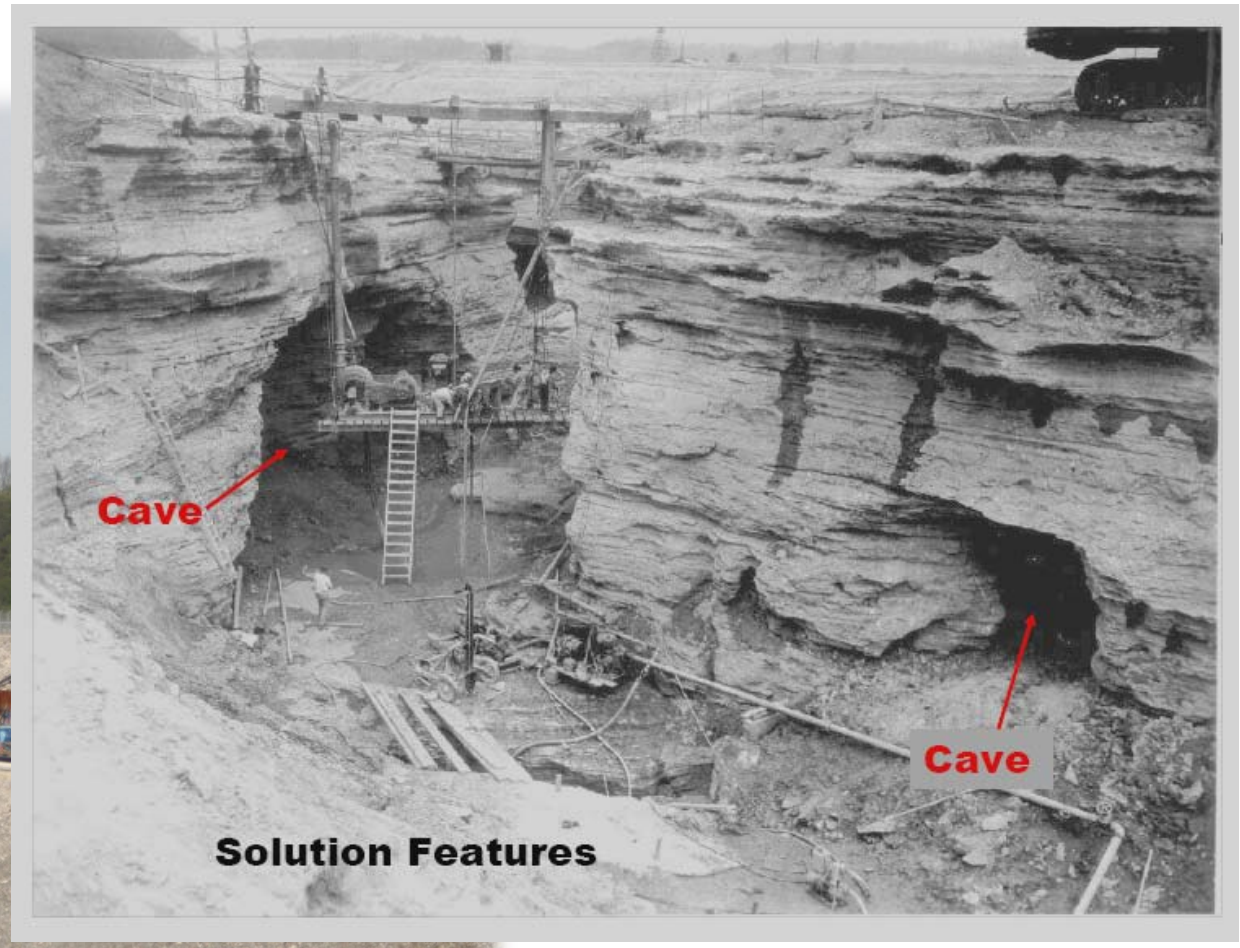




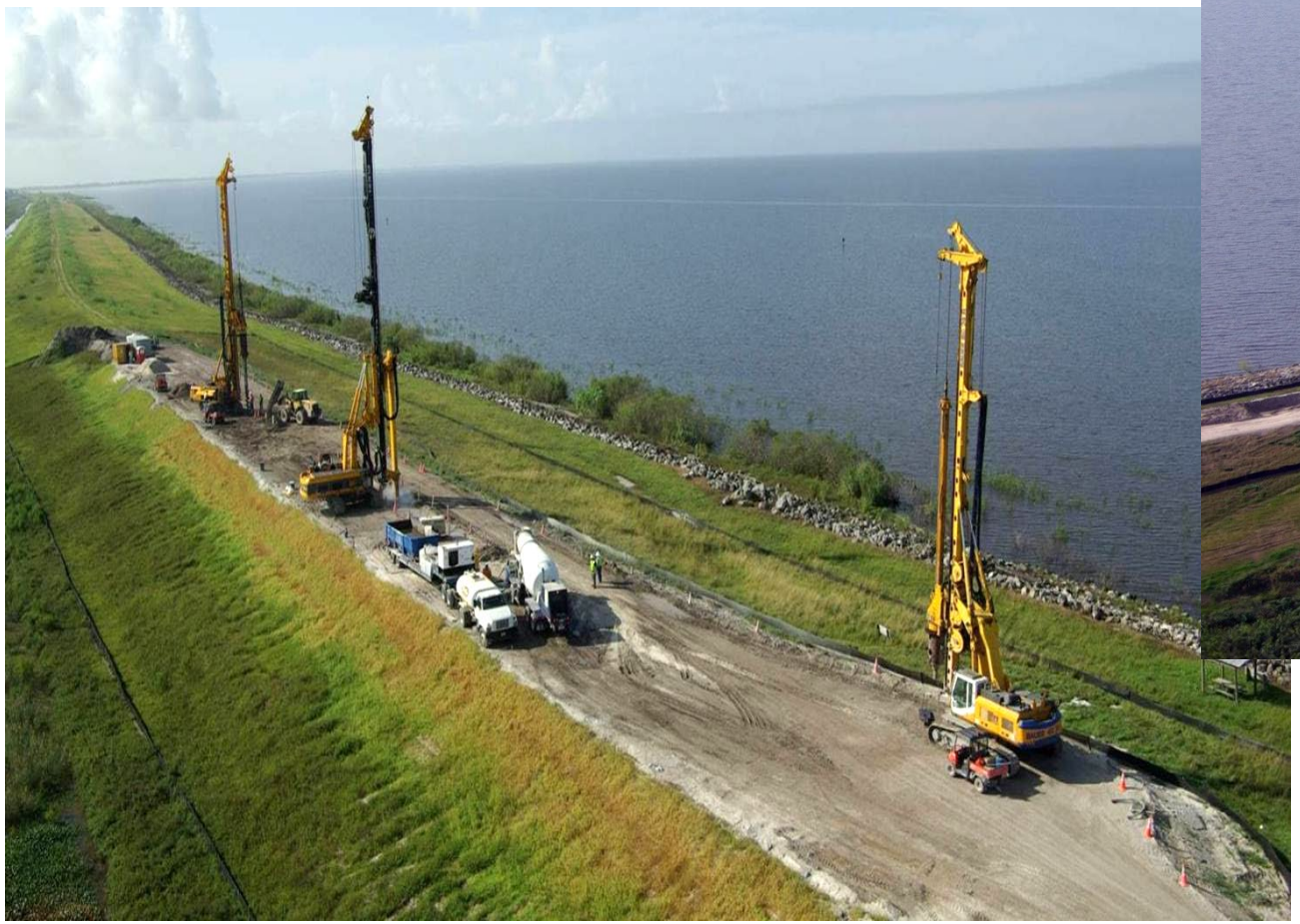


Wolf Creek Dam - Kentucky



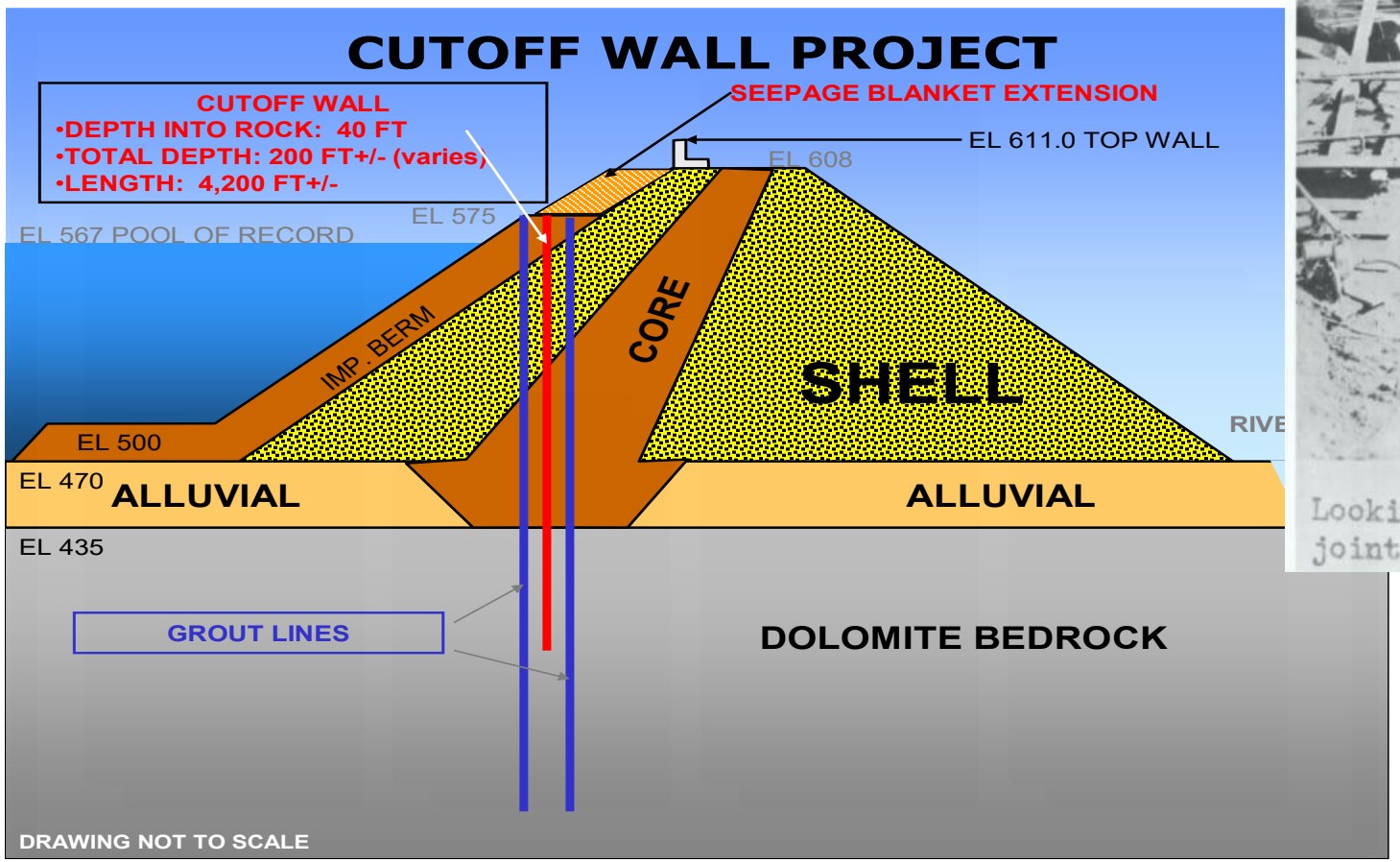


Herbert Hoover Dike - Florida



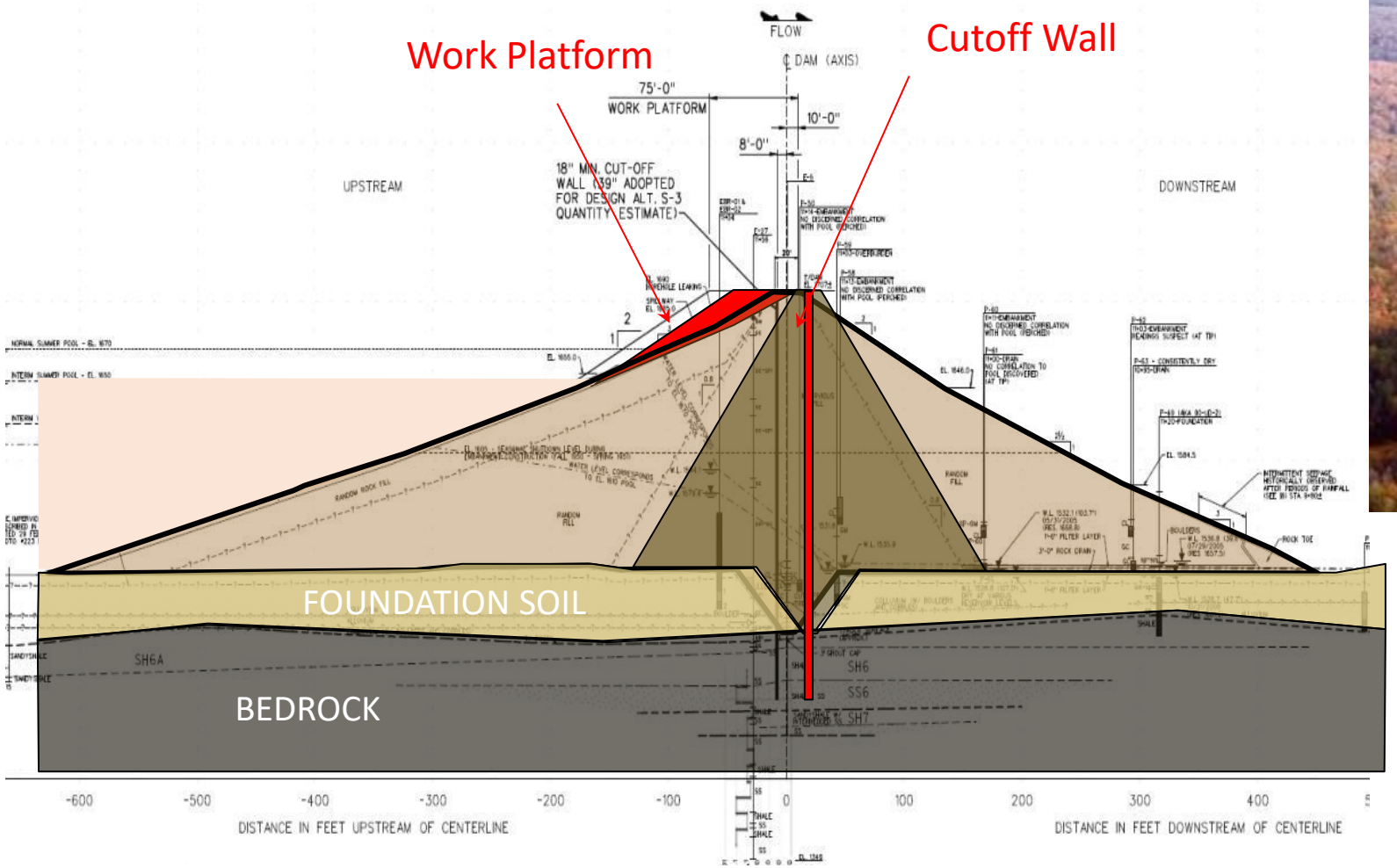


Clearwater Dam - Missouri





East Branch Dam - Pennsylvania





Center Hill Dam - Tennessee



AV Watkins Dam - Utah



First USACE-DFI Workshop
December 6-7, 2011, Deerfield Beach, FL



First USACE-DFI Workshop – 2011

- Agencies – USACE, USBR and NRCS
  - USACE Districts: RMC, Jacksonville, Pittsburgh, Nashville, New England, Seattle, Nashville, Kansas City, Little Rock, Chicago, Memphis, Tulsa, Huntington, Seattle, Sacramento, St. Louis
- Contractors – ACT, Bauer, Geo-Solutions, Inquip, Hayward Baker, Layne GeoConstruction, Nicholson, Raito, Treviicos
- Consultants and Researchers – URS, Geosystems, Sea Level Engineering, Utah State, Gannett Fleming

First USACE-DFI Workshop – 2011

- Breakout sessions
  - Issues related to cutoff techniques
  - QAQC and verification
  - Contractual and procurement practices
  - Research needs and future work
- Summary notes available

Second USACE-DFI Workshop
November 15-16, 2016, Sacramento, CA



Second USACE-DFI Workshop – 2016

- Agencies – USACE and DWR
  - USACE Districts: SPK, MVK, MVS, NWK, LRH
- Contractors – Bauer, Bencor, DeWind, Geo-Solutions, Great Lakes, Inquip, Hayward Baker, Malcolm, Nicholson
- Consultants and Researchers – AECOM, Blackburn, Environmental Geotechnics, GD Consulting, Gannett Fleming, Geosyntec, ILF, HDR, URS, GEI, Kiewit, Kleinfelder, Liquid Earth Support, Schnabel Engineering, Sea Level Engineering, Slurry Systems, U of CA Berkeley, Utah State, Gannett Fleming

Second USACE-DFI Workshop – 2016

- Design
- Roles and Responsibilities of Designs and Construction Managers
- Products
- QA/QC Testing Requirements
- Long-Term Performance
- Acceptance Criteria

Second USACE-DFI Workshop

- Presentations and Breakout sessions
- Notes summarized, not published

- Unresolved topics from this workshop formed basis of 2023 workshop

USACE-DFI Seepage Control Workshop 2023

How did we get here?

Expectations and goals

Logistics



Expectations – 2023 Workshop

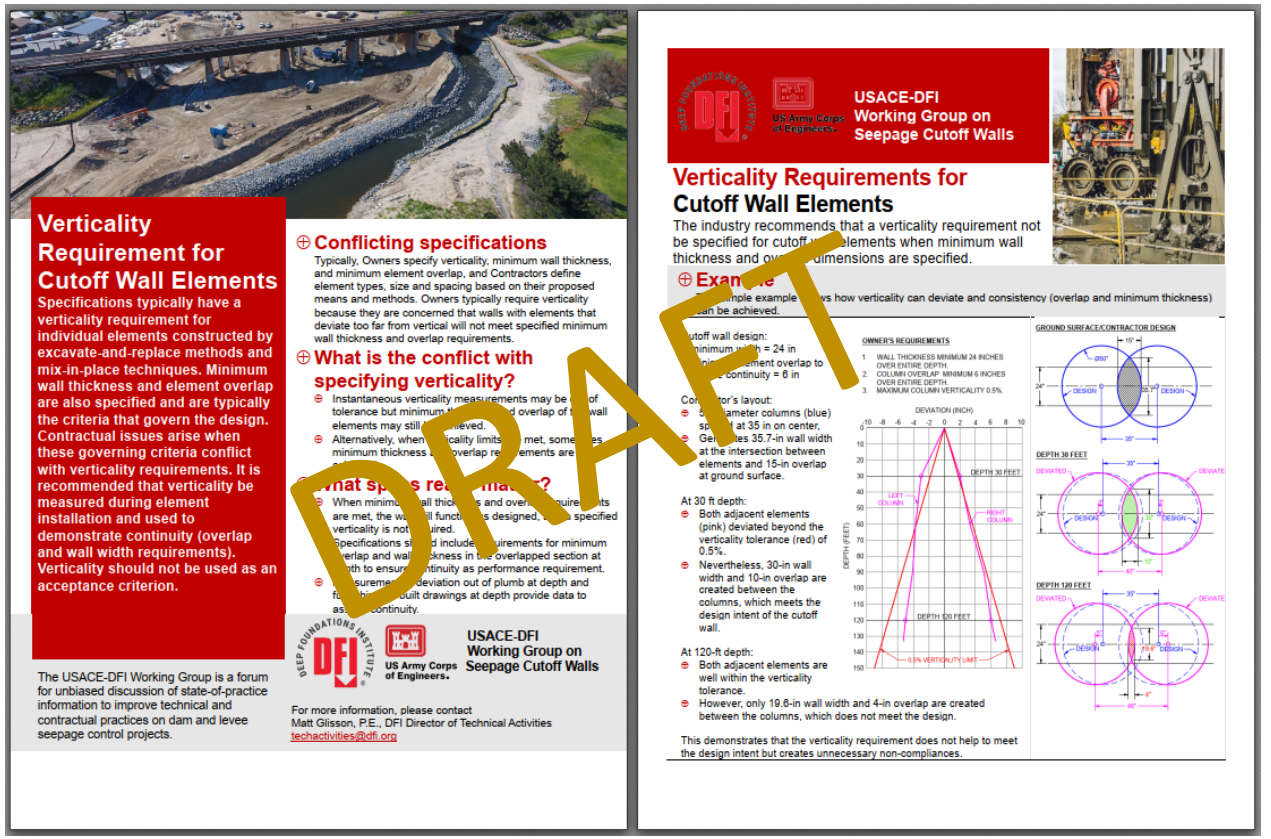
- Provide a forum for dam and levee engineering and construction Community of Practice to collaborate.
- For Industry to offer suggestions to the Government prior to solicitation on the construction of dam and levee seepage control projects.
- To develop and implement consistent, appropriate and effective technical specifications for each cutoff technology nationwide.
- To define required technical research areas and sources of funding.
- To plan future workshops and activities.

Goals – 2023 Workshop

- To ensure that safety for the personnel and for the structures (risk reduction) is always considered above any other circumstance.
- To avoid claims during the project execution.
- To optimize data management implementation to:
  - Maximize the QC/QA verification during construction (possibly, in real time).
  - Produce, collect, and store digital data obtained before, during, and after the project's life to be used for future expansion, rehabilitation and as a database for future new projects.
- To work towards defining state-of-practice processes that assure the quality and performance of cutoff projects.
- To build trust and partnering commitments between the Government and Industry.

Workshop Output

- Subcontractor Involvement in Prime Meetings
- Verticality
- Acceptance Criteria for Low Strength Material
- Data Specifications
- Telemetry
- SHS 28-day acceptance criterion



Mini-Workshop October 2024

Goals

- Reconnect the working group members
- Review USACE 65% Deep Mixing Specification topics
- Review draft content for first round of working group flyers
- Outline the next topics we are planning to address



Other Joint Activities

- USACE-DFI Memorandum of Understanding (signed 2013, 2018)
- Levee Safety Coalition (2013)
- Hurricane Sandy Meeting (2014, Ft. Hamilton, NY)
- Levee and Coastal Restoration Conference (2014, Miami)
- Midwest Levees and Rivers Technical Exchange (2014, St. Louis)
- Working Group Meeting at DFI Deep Mixing Conference (2015, SF)
- DFI Annual Conference Seepage Track (2016, NYC)

What Has Improved

- To ensure that safety for the personnel and for the structures (risk reduction) is always considered above any other circumstance
- To avoid claims during the project execution
- To optimize data management implementation to:
  - Maximize the QC/QA verification during construction (possibly, in real time)
  - Produce, collect, and store digital data obtained before, during, and after the project's life to be used for future expansion, rehabilitation and as a database for future new projects
- To work towards defining state-of-practice processes that assure the quality and performance of cutoff projects
- To build trust and partnering commitments between the Government and Industry

Opportunities Going Forward

- To work towards defining state-of-practice processes that assure the quality and performance of cutoff projects
- To build trust and partnering commitments between the Government and Industry
- Maintain an open dialogue
- Support understanding and seek mutual benefit
- Support development of new guide specifications
- Review specifications for projects prior to bid stage

Questions



US Army Corps of Engineers.

DEEP FOUNDATIONS INSTITUTE DFI



BAUER Group

BAUER Spezialtiefbau GmbH

Application of Diverse Ground Improvement Techniques for the Stabilization of Reinforced Earth Wall Approach Ramps

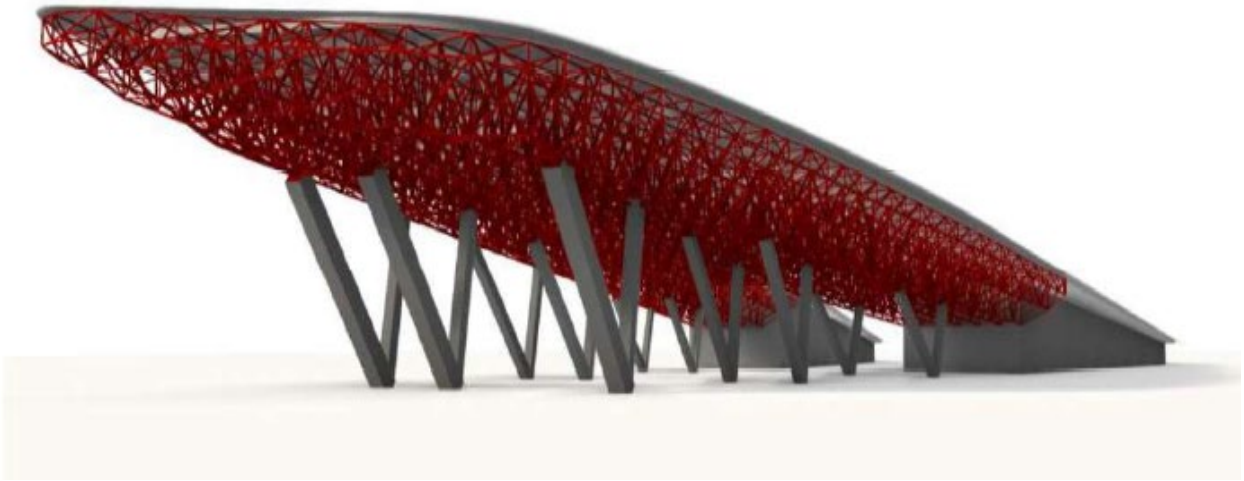
— "The opinions expressed in this presentation and on the following slides are solely those of the presenter and not necessarily those of DFI. DFI does not guarantee the accuracy or reliability of the information nor endorse the recommendations offered by the presenter".

JASON REDGERS– BAUER Spezialtiefbau GmbH,
21 October 2025

Introduction

Reason for work?

1. Total and differential settlement between piled and non piled area
2. Inundation collapse worry over wetting





Perspective view of the structure

INTRODUCTION:
Scope of Works

Soil Improvement under approach Ramps 1 and 2 (Blue Area) rising from 0 to 26m

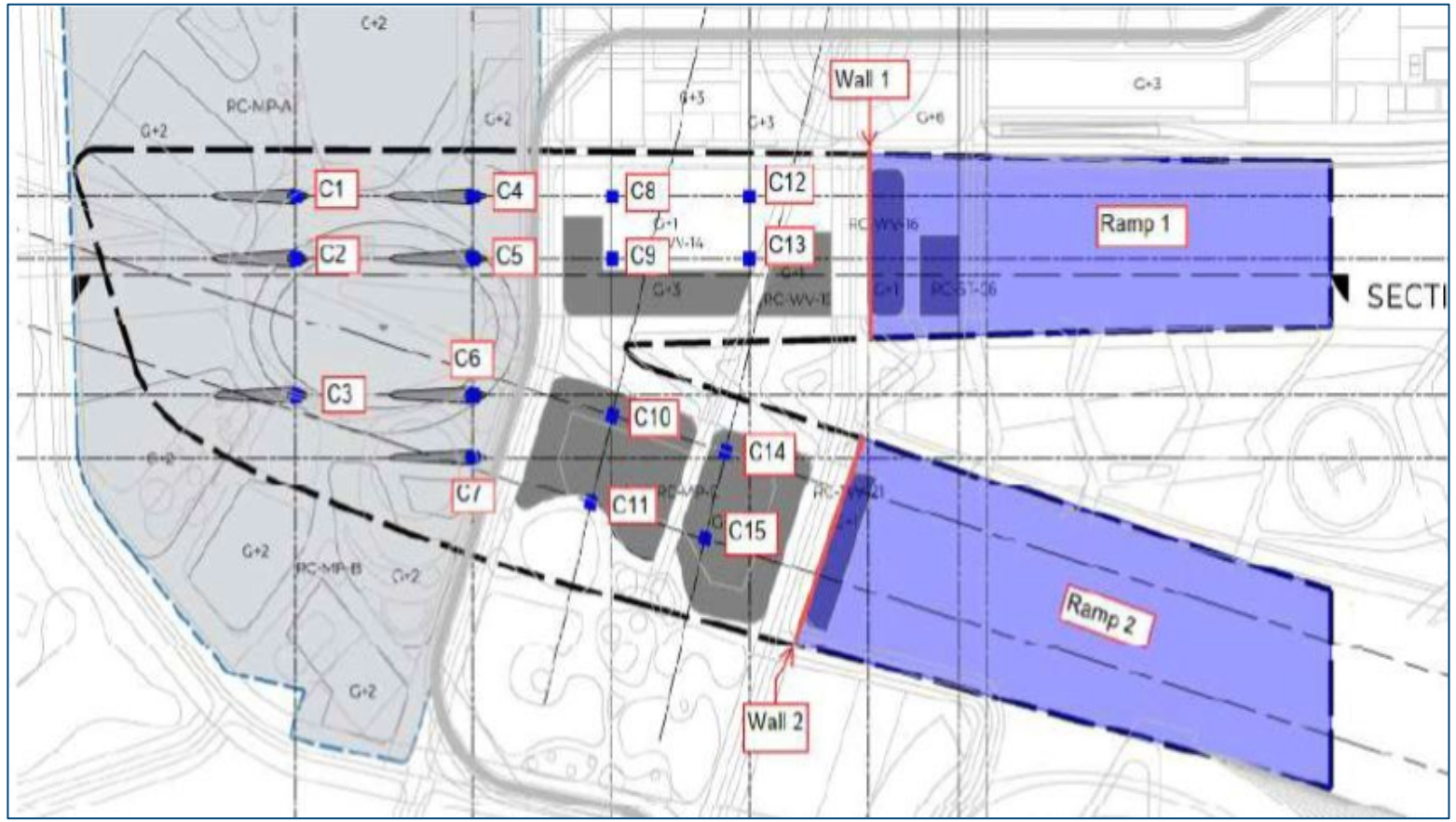
Ramp1 – Zone 1,2,3
Ramp2 – Zone 4,5

Zone 1 and 4 are maximum pressure

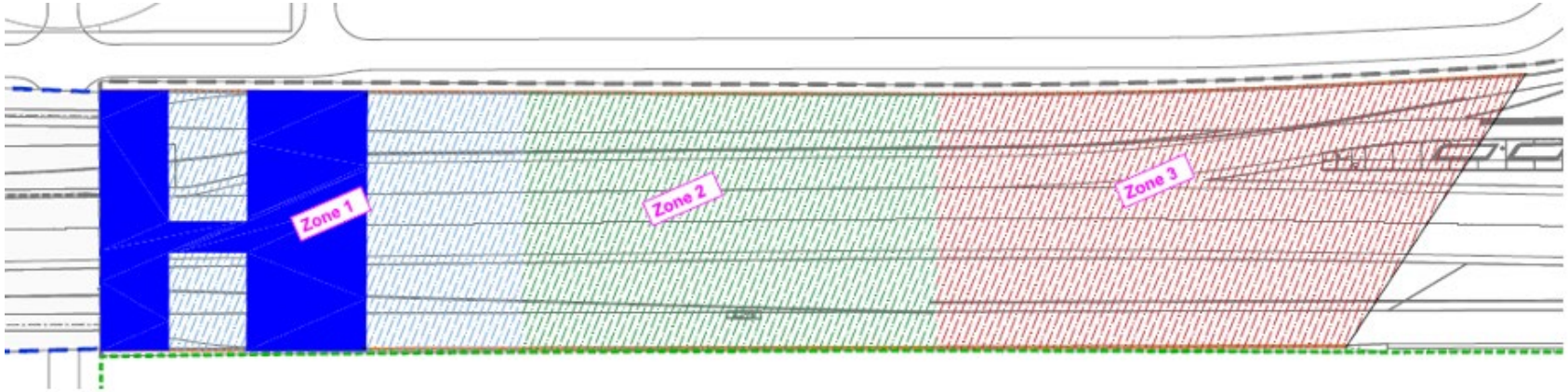
All Zone pre treated with WTF and then SCM for Zones 01 and 04

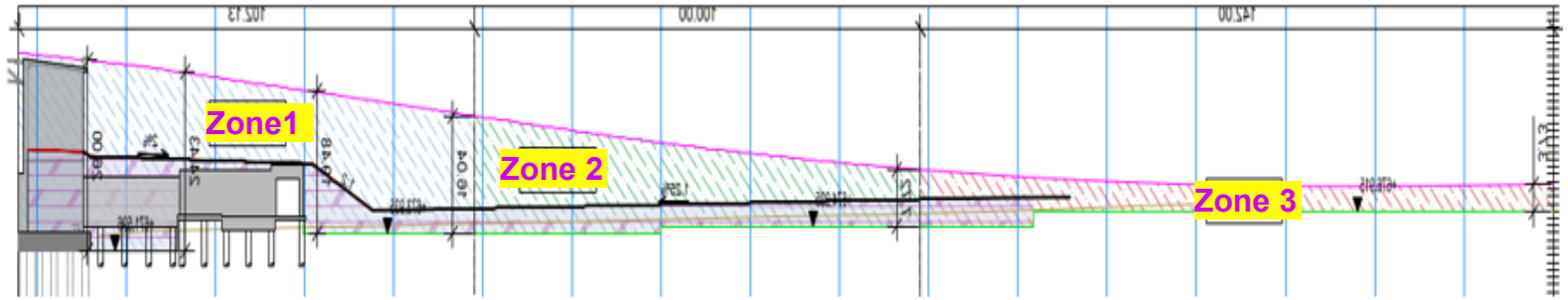
Then completed by RIC and HEIC

Total Area = 31,000m^2
SCM = 11,000m^2
Other = 20,000m^2



Zone Classification – North Ramp

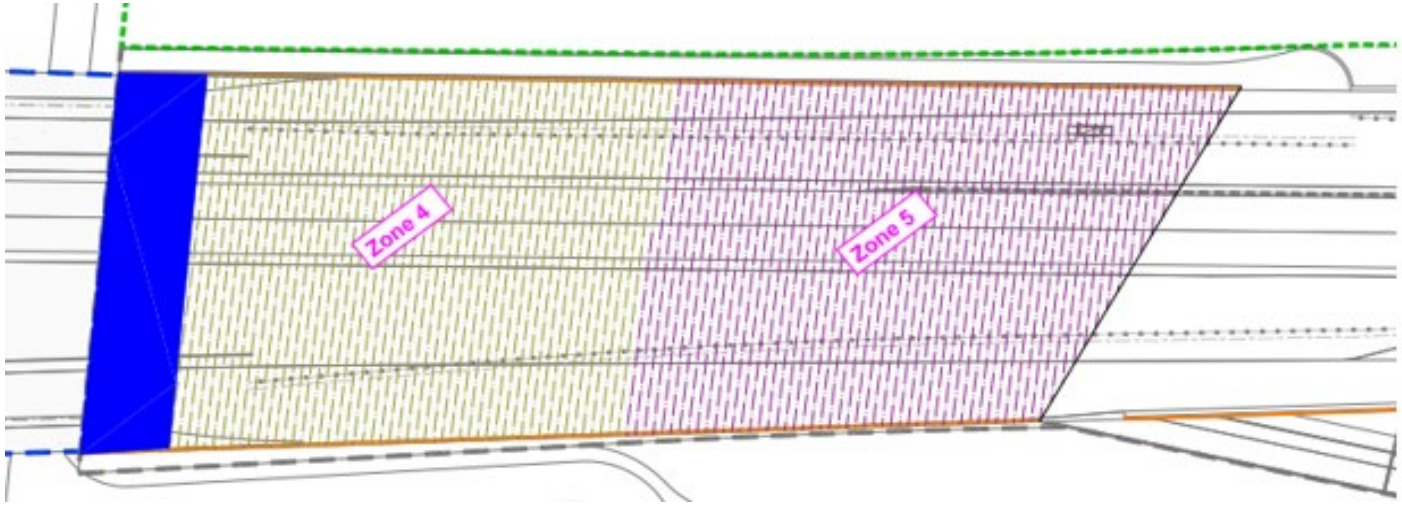


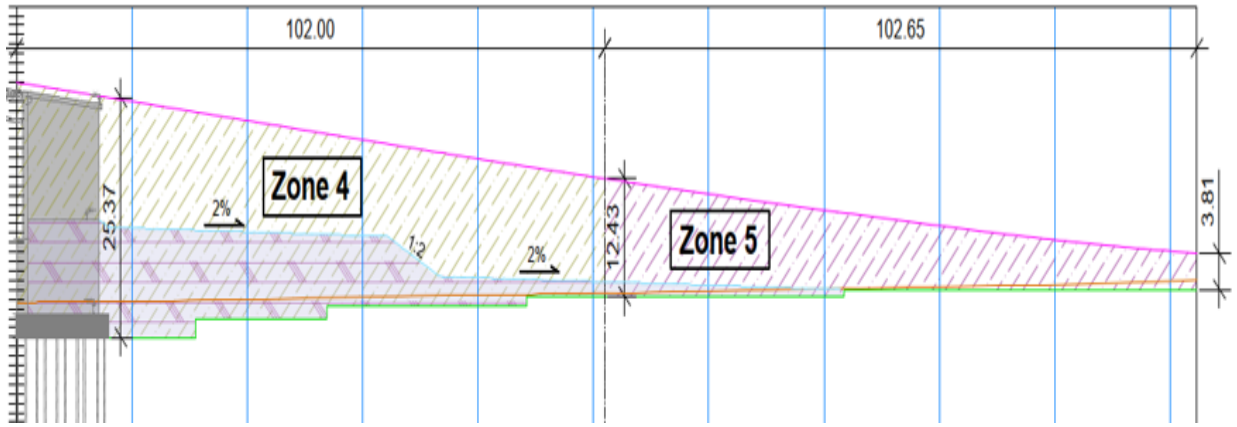


North Ramp:

- **Zone 1 (MSE Height 26 ~ 16m)**
- **Zone 2 (MSE Height 16 ~ 8m)**
- **Zone 3 (MSE Height 8 ~ 4m)**

Zone Classification – South Ramp



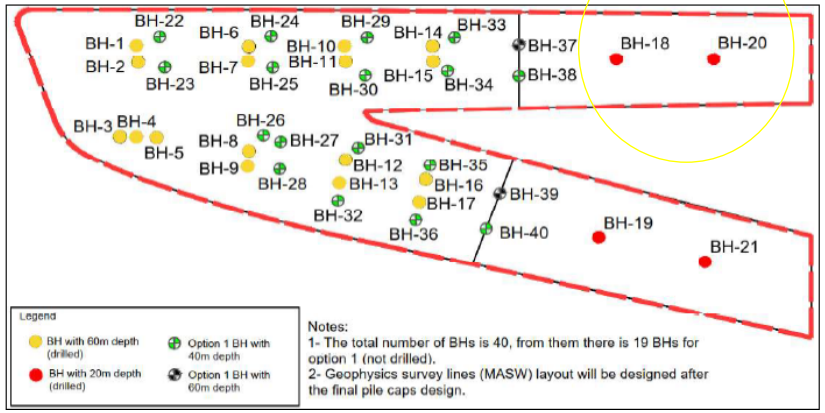


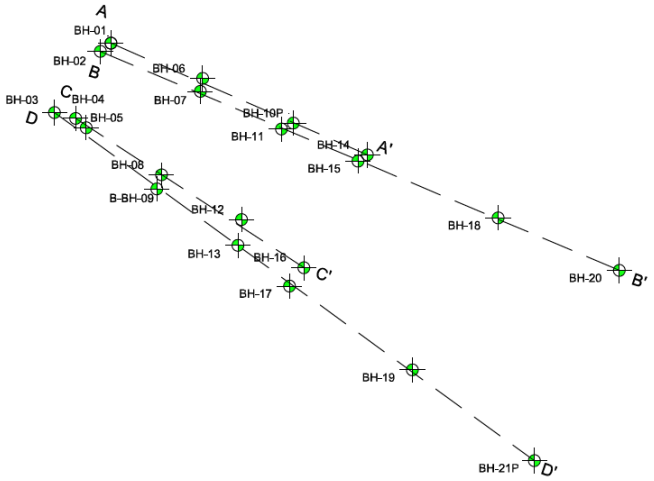
South Ramp:

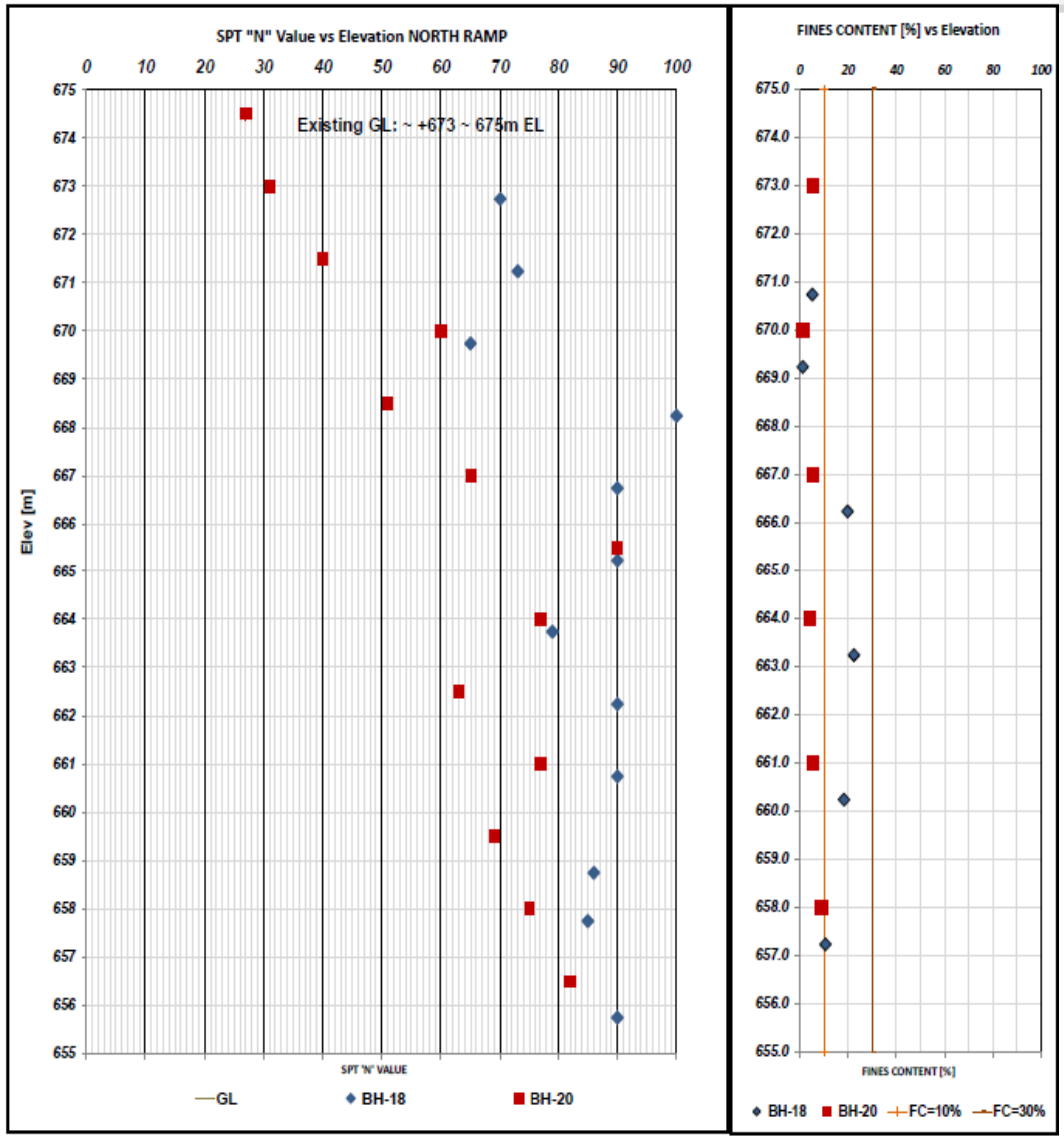
- **Zone 4 (MSE Height 26 ~ 16.5m)**
- **Zone 5 (MSE Height 16.5 ~ 9m)**

Application of Diverse Ground Improvement Techniques for the Stabilization of Reinforced Earth Wall Approach Ramps

Ground Conditions



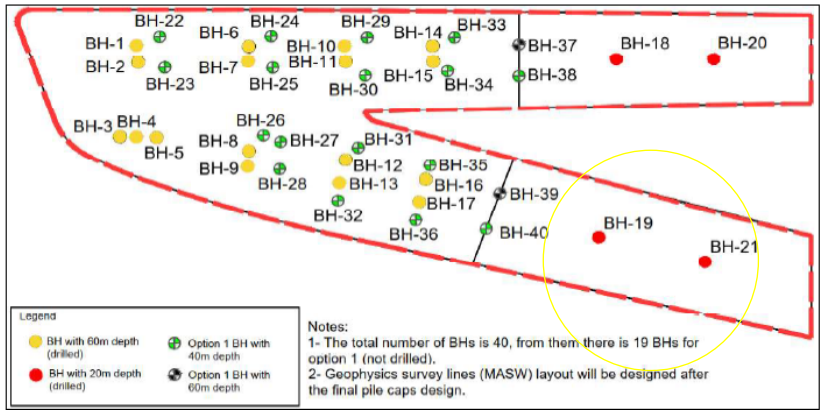


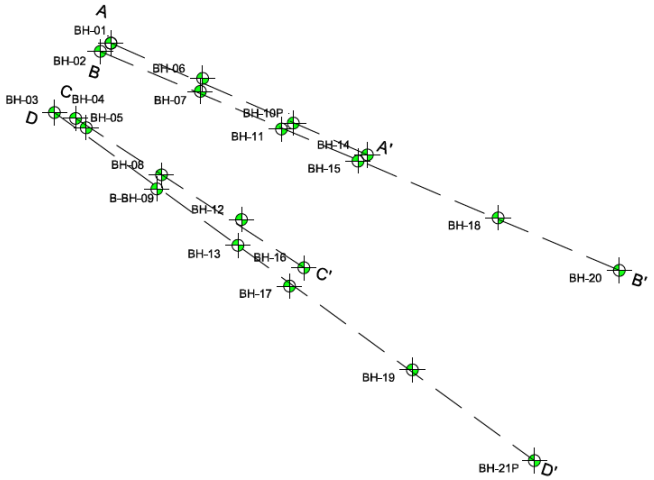


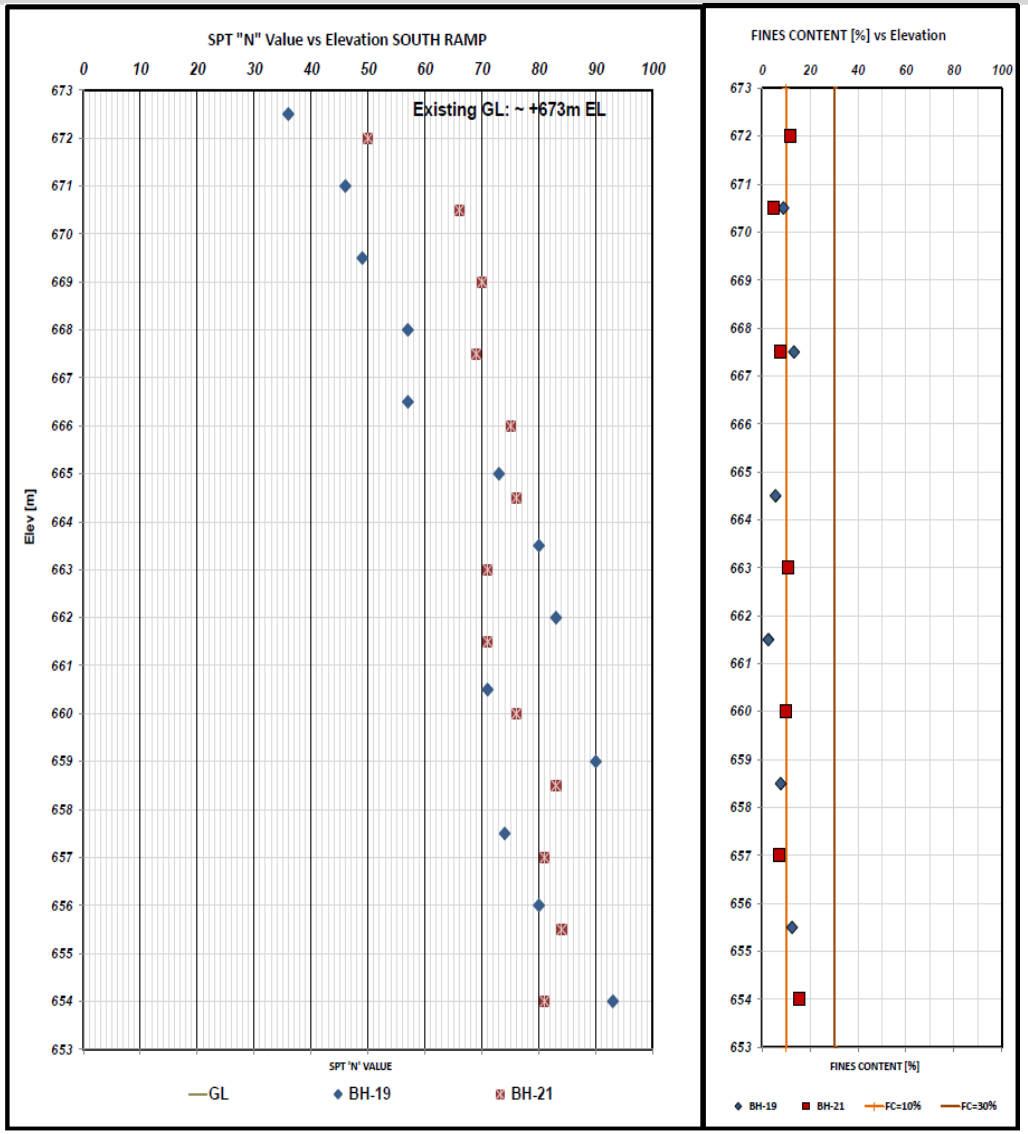
SPT N vs Fines Content vs Elevation at North Ramp

Application of Diverse Ground Improvement Techniques for the Stabilization of Reinforced Earth Wall Approach Ramps

Ground Conditions

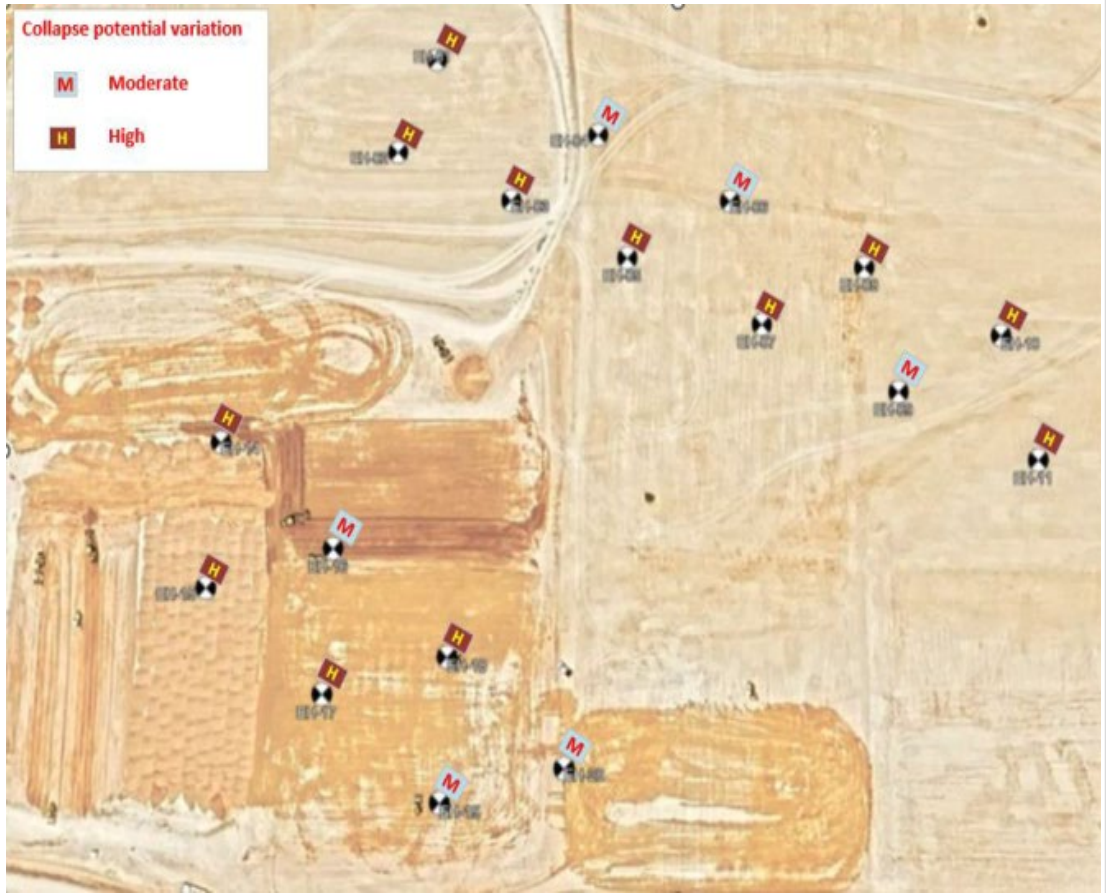


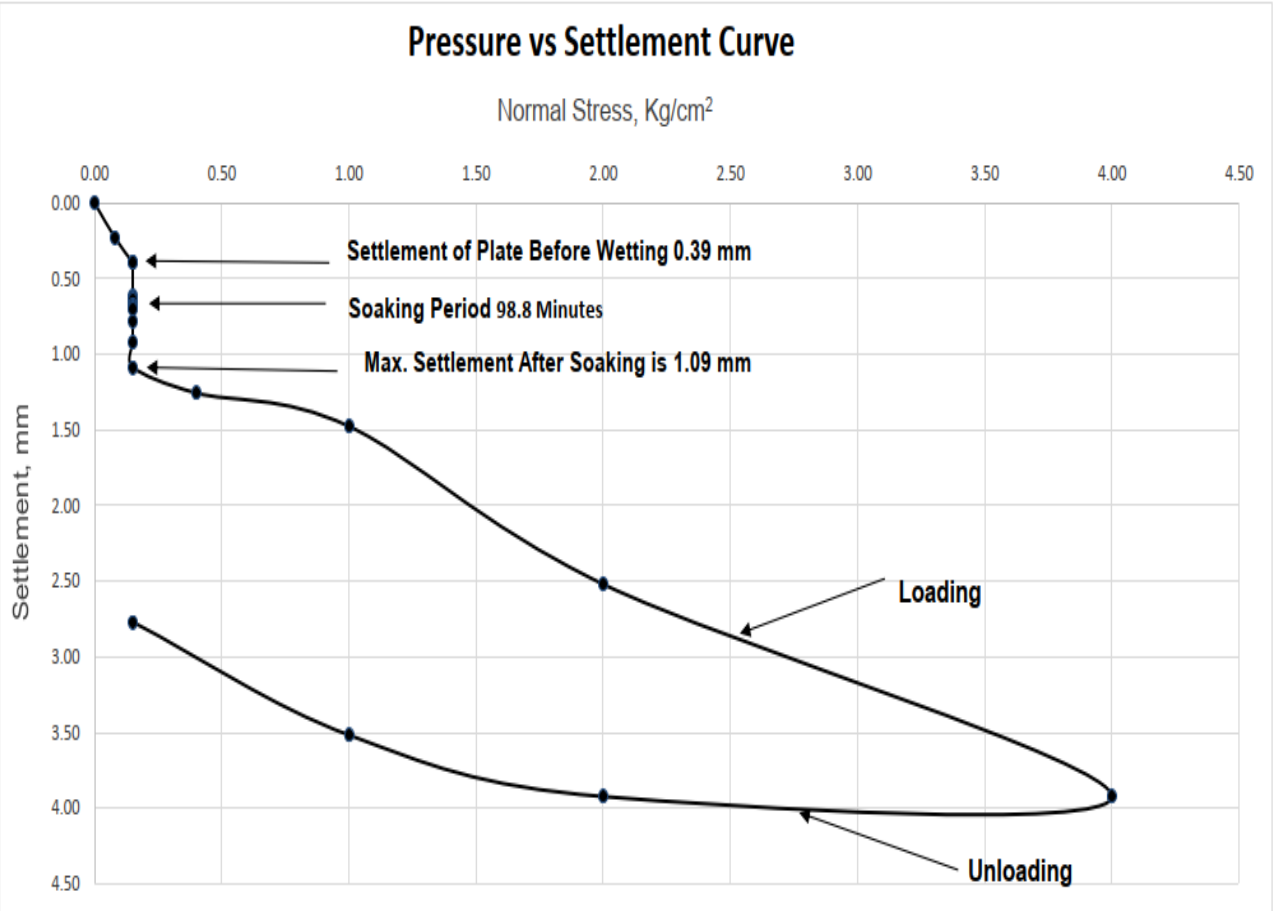




SPT N vs Fines Content vs Elevation at South Ramps

BREA Infiltration and Plate Load Test locations and Results





Ground Improvement Design

ZONE 1 AND ZONE 4:

Max Height of MSE walls, As per consultant specifications,

- **Average unconfined compressive strength, UCS shall be 1.5 MPa of the soil mix column**
- **And a minimum stiffness of 200 MPa on testing**
- **Zone Load Test, ZLT on 3x3m area for working load 575 kPa to verify**
- **Final settlement @ working load shall not exceed 12.5mm**

- **Ground improvement specified and implemented**

  Soil Mixing Columns, SCM with 2000mm Dia on 2.25 x 2.25 Triangular grid

Ground Improvement Design

ZONE 2, ZONE 3 AND ZONE 5

Relatively lower height of MSE walls, minimal treatment of ground:

- **To increase the stiffness of ground**
- **Limit post construction settlements on the surface**

- **Ground improvement proposed and implemented:**

  Pre-boring using 660mm Dia ϕ tool on 2.5 x 2.5m Triangular grid

  Vibro replacement/ Stone Column with 900mm Dia on 2.5 x 2.5m Triangular grid

  Supplementary Surface compaction techniques, RIC & HEIC

AVAILABLE DEEP SOIL MIXING TECHNIQUES

BAUER

Deep Mixing Methods within BAUER





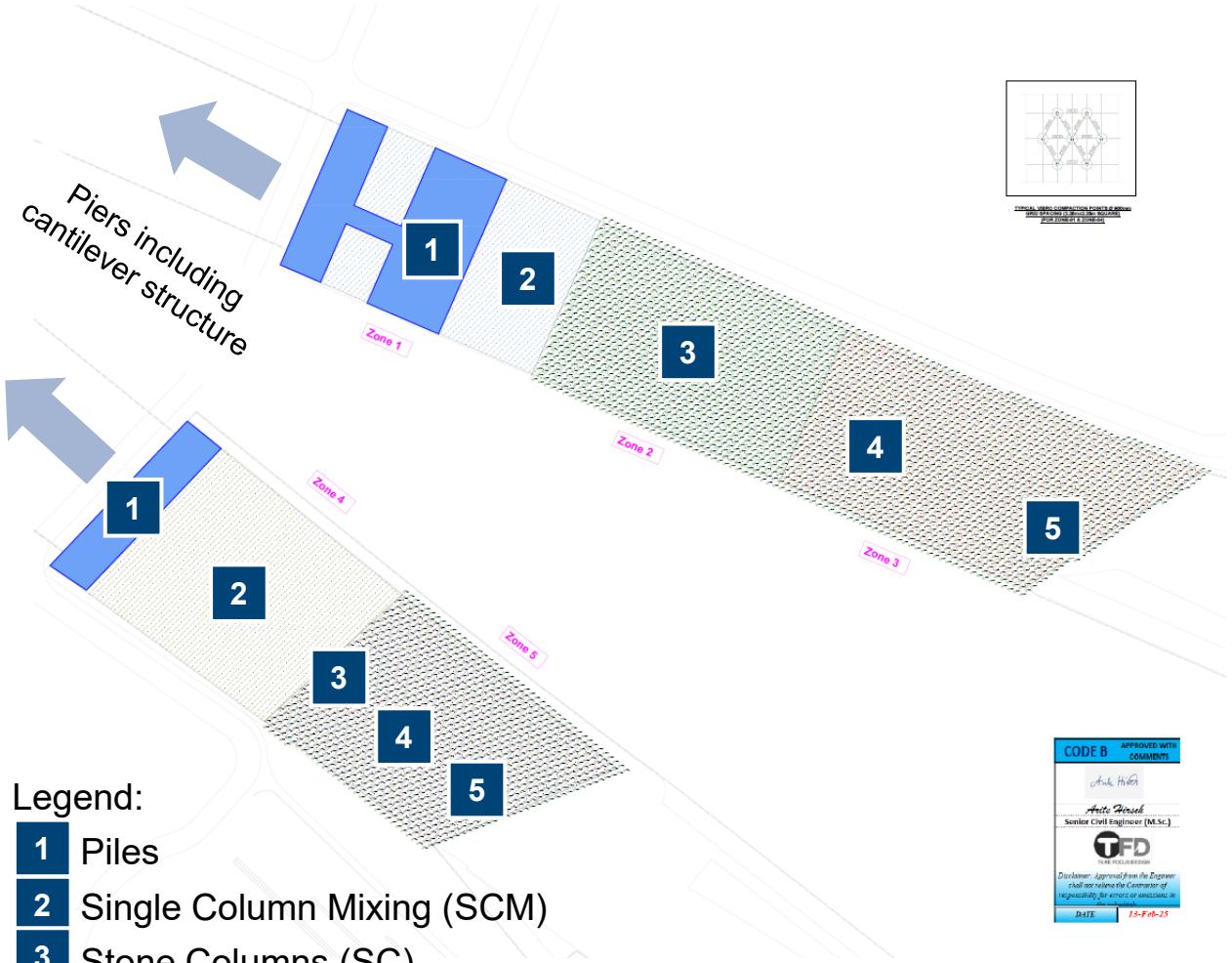






No.	Functional capabilities	Mixed-In-Place (MIP)	Single Column Mixing (SCM)	Cutter Soil Mixing (CSM)	Soil Mixing Wall (SMW)	Single Column - Double Head (SCM-DH)
1	First application	1977	1996	2003	2002	2012

Qiddiya Projects Executed by SAUDI BAUER
Speed Park Blade – Stone Columns & Rapid Impact Compaction



Legend:

1. Piles
2. Single Column Mixing (SCM)
3. Stone Columns (SC)
4. Rapid Impact Compaction (RIC)
5. High Energy Impact Compaction (HEIC)





Deep Soil Mixing, DSM Design Methodology

Federal Highway Administration Design Manual: Deep Mixing for Embankment and Foundation Support Publication No. FHWA-HRT-13-046

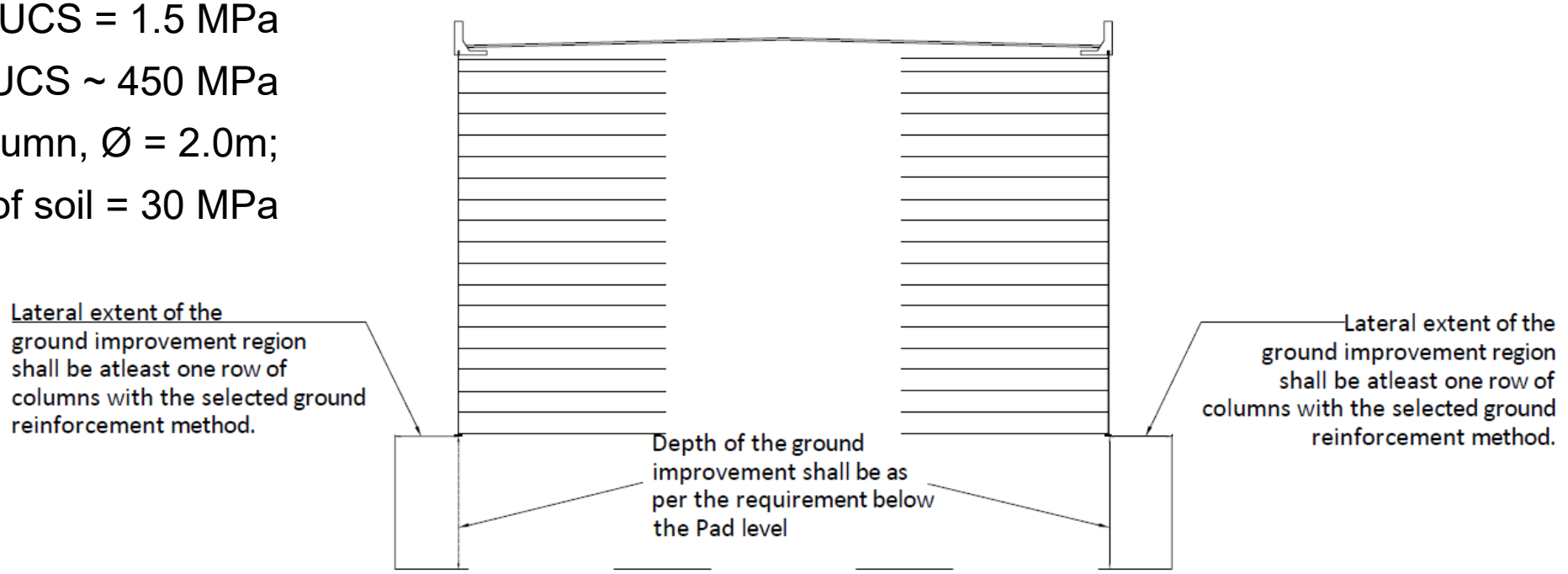
Isolated Columns Triangular grid configuration

Target UCS = 1.5 MPa

Stiffness/Modulus, E' = 300*UCS ~ 450 MPa

Diameter of DSM column, Ø = 2.0m;

Average Stiffness of soil = 30 MPa



Typical section A-A'

Deep Soil Mixing, DSM Design Methodology

Federal Highway Administration Design Manual: Deep Mixing for Embankment and Foundation Support Publication No. FHWA-HRT-13-046

Required Area Replacement Ratio, ARR [%]

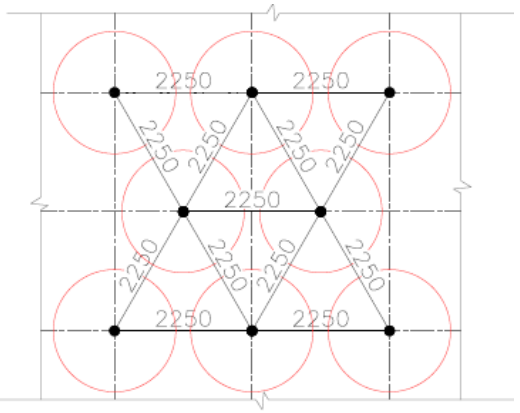
$$a_{s,center} \geq F_{cc} \frac{q}{2 s_{dm} f_v}$$

Fcc = Factor of safety against crushing of the center isolated columns

Sdm = Shear strength of deep mixed ground

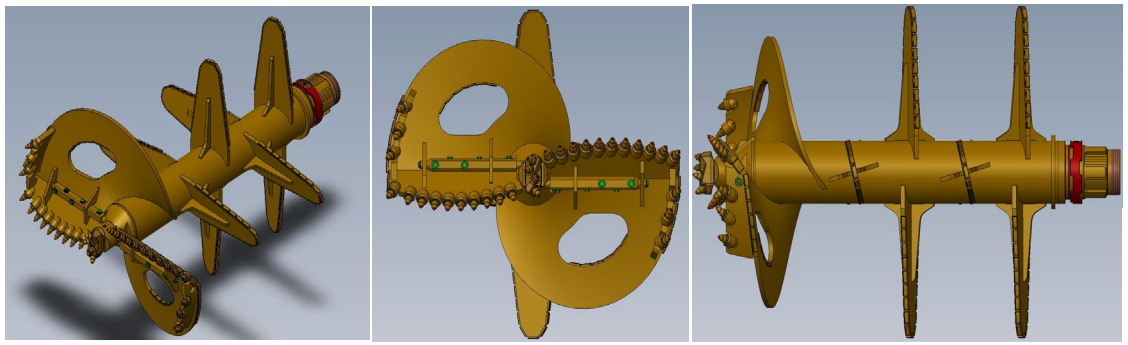
$$s_{dm} = \frac{1}{2} f_r f_c q_{dm,spec}$$

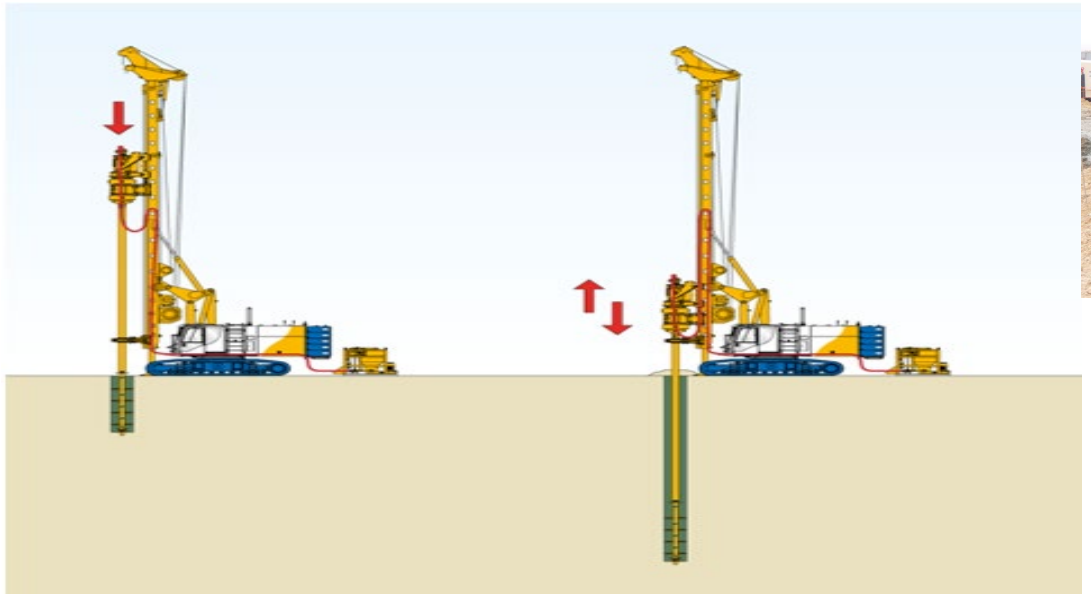
qdm = Unconfined Compressive Strength, UCS



Vertical Stress [kPa]	579
Required ARR [%] @ centre using UCS = 1.5 MPa	66.2%
Provided ARR [%]	71.7%
Proposed Grid Spacing , Triangular [m x m]	2.25 x 2.25
Composite Modulus, Dc [MPa]	330.4

Single Column Mixing Tool



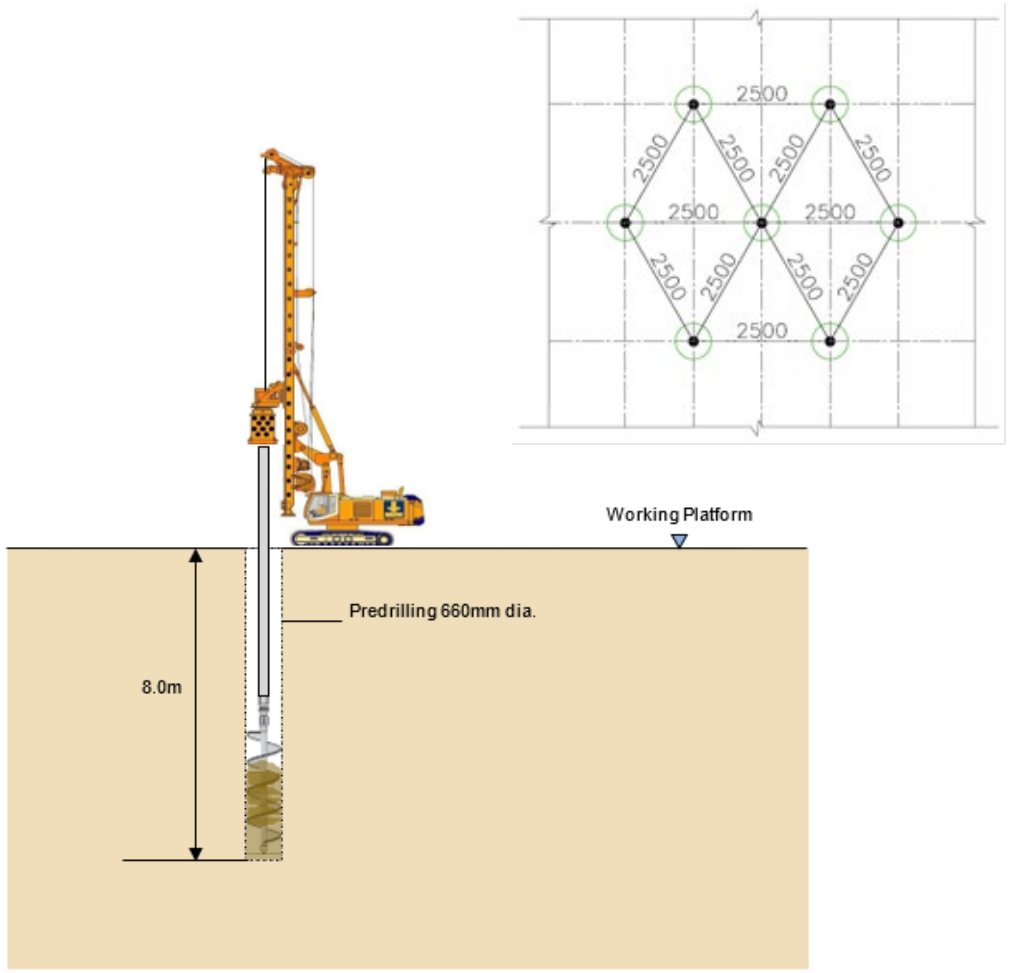


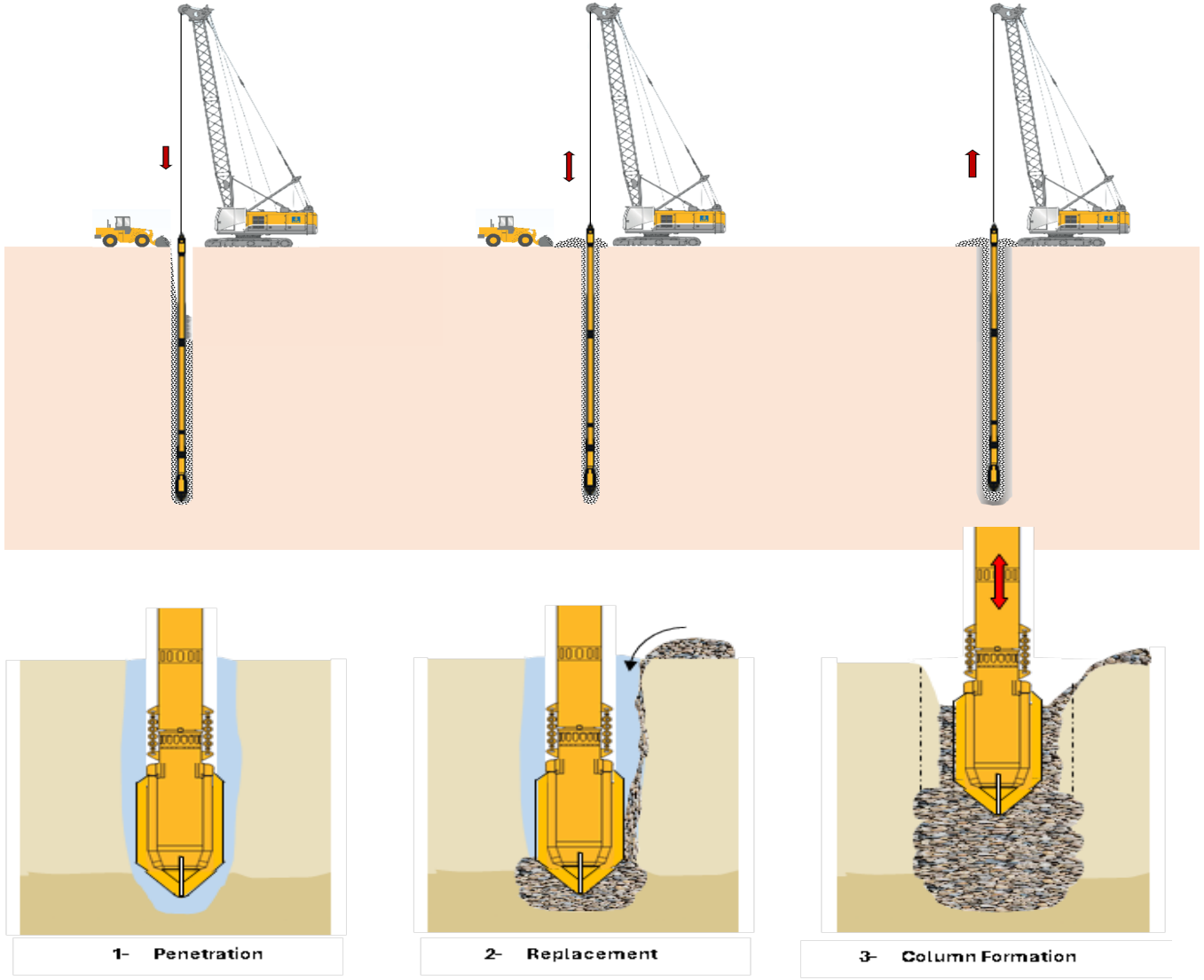




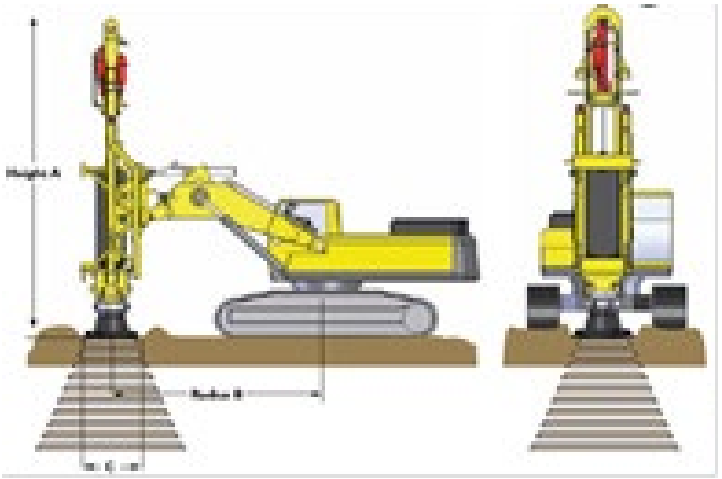
Application of Diverse Ground Improvement Techniques for the Stabilization of Reinforced Earth Wall Approach Ramps

Zone 2, Zone 3 & Zone 5



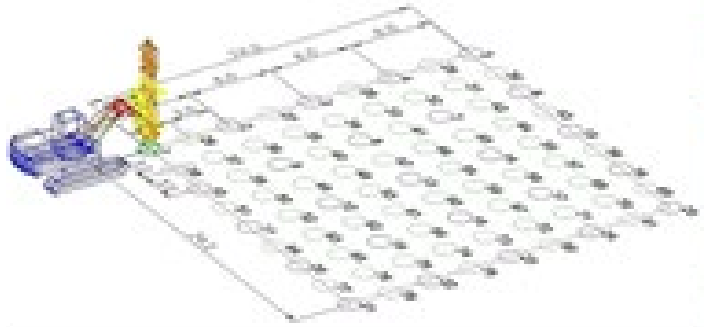


Zone 2, Zone 3 & Zone 5 Supplementary Surface compaction RIC & HEIC

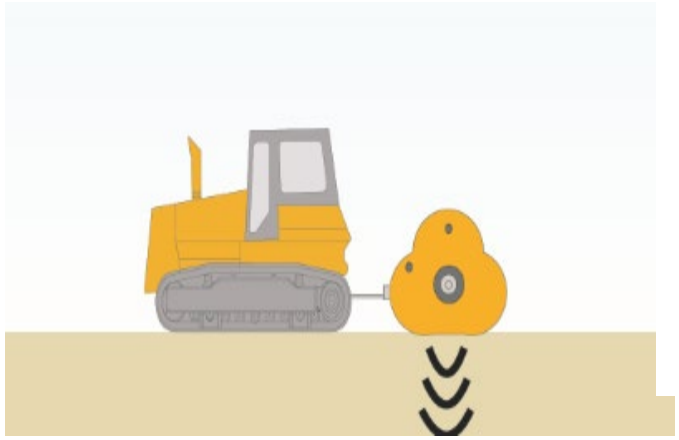


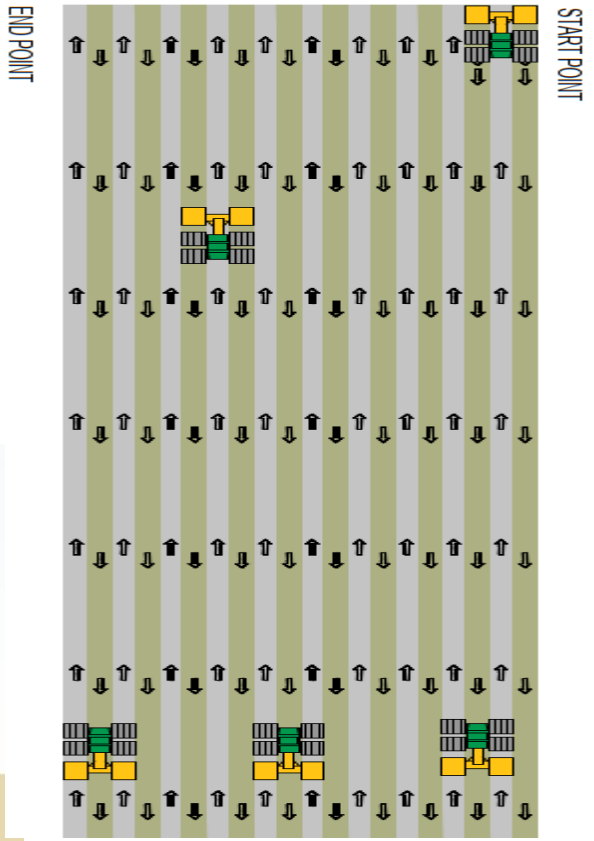












Application of Diverse Ground Improvement Techniques for the Stabilization of Reinforced Earth Wall Approach Ramps



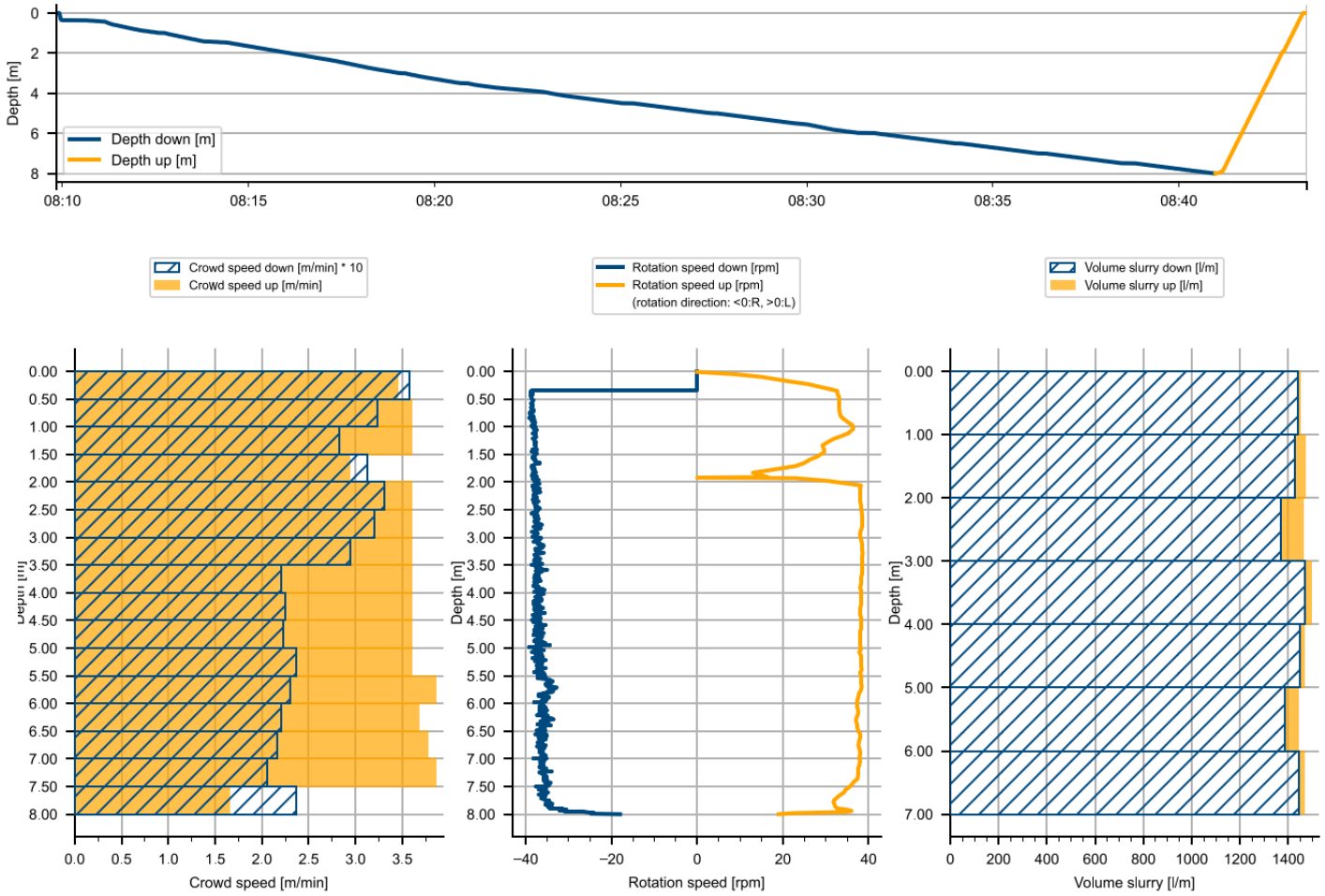
Application of Diverse Ground Improvement Techniques for the Stabilization of Reinforced Earth Wall Approach Ramps



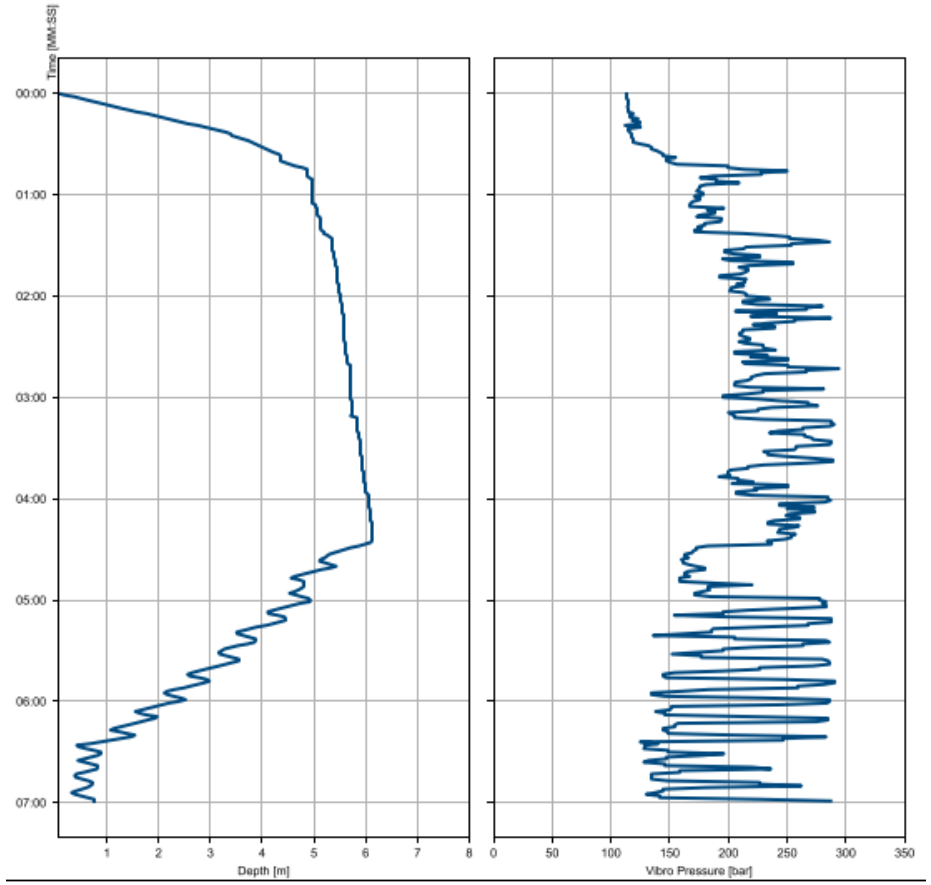




QUALITY ASSESSMENT:
Production Plots



Production Plot SCM Column

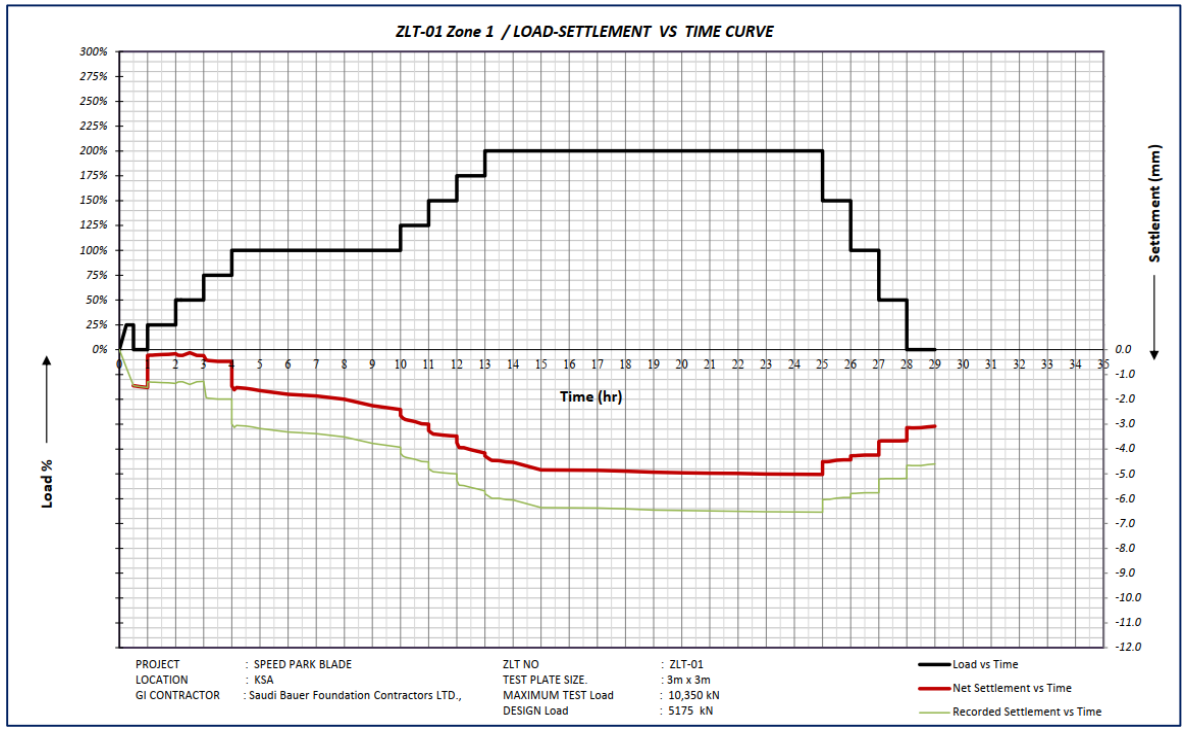


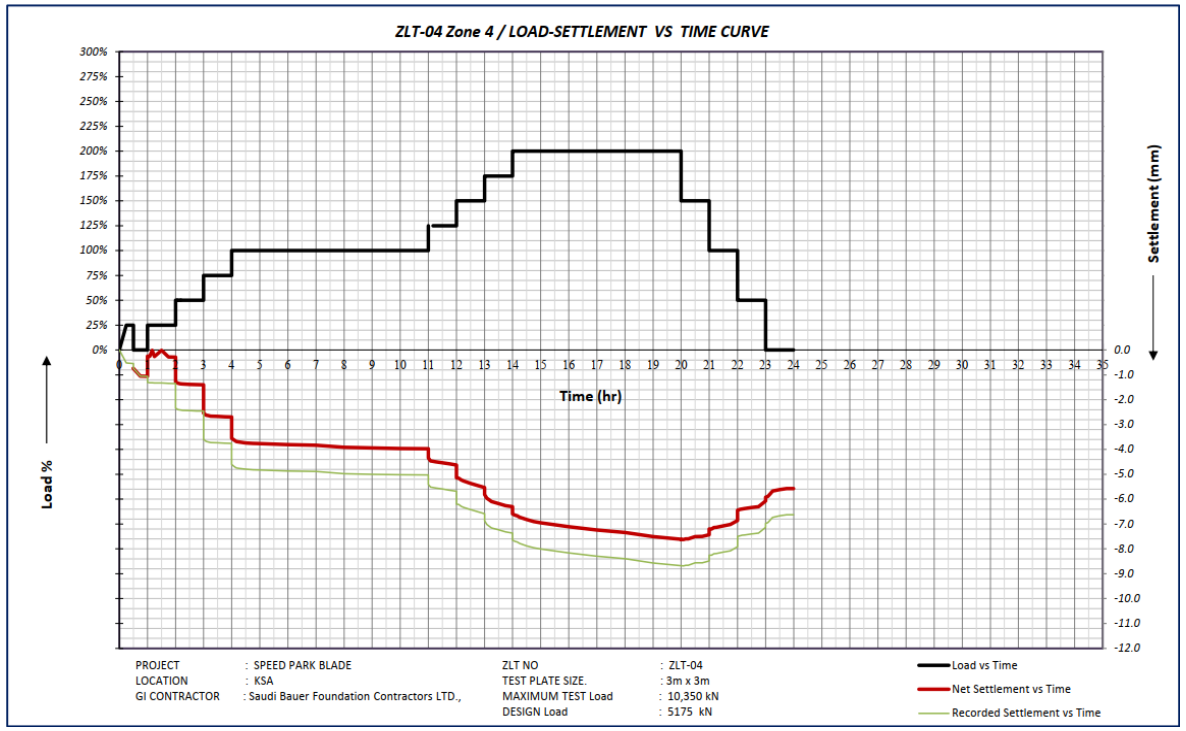
Production Plot Stone Column

Verification Testing – Zone Load Test, ZLT undertaking in Zone 1 & Zone 4



Verification Testing – ZLT Results in Zone 1 & Zone 4





Actual settlements in Zone 1 and Zone 4

- 3 to 5mm at working load, 575 kPa
- 5 to 8mm at maximum load i.e., 200% of working load, 1150 kPa
- < specified target settlement of 12.5mm at the working load

Zone	Test Load (% of Working Load)	Measured Settlement (mm)	Acceptance Criteria (mm @ 100%)	Remarks
Zone-1	200%	6.54	≤ 12	Accepted (Well within limit)
Zone-4	200%	8.68	≤ 12	Accepted (Well within limit)

Verification Testing – DSM Compressive Strength Tests undertaking



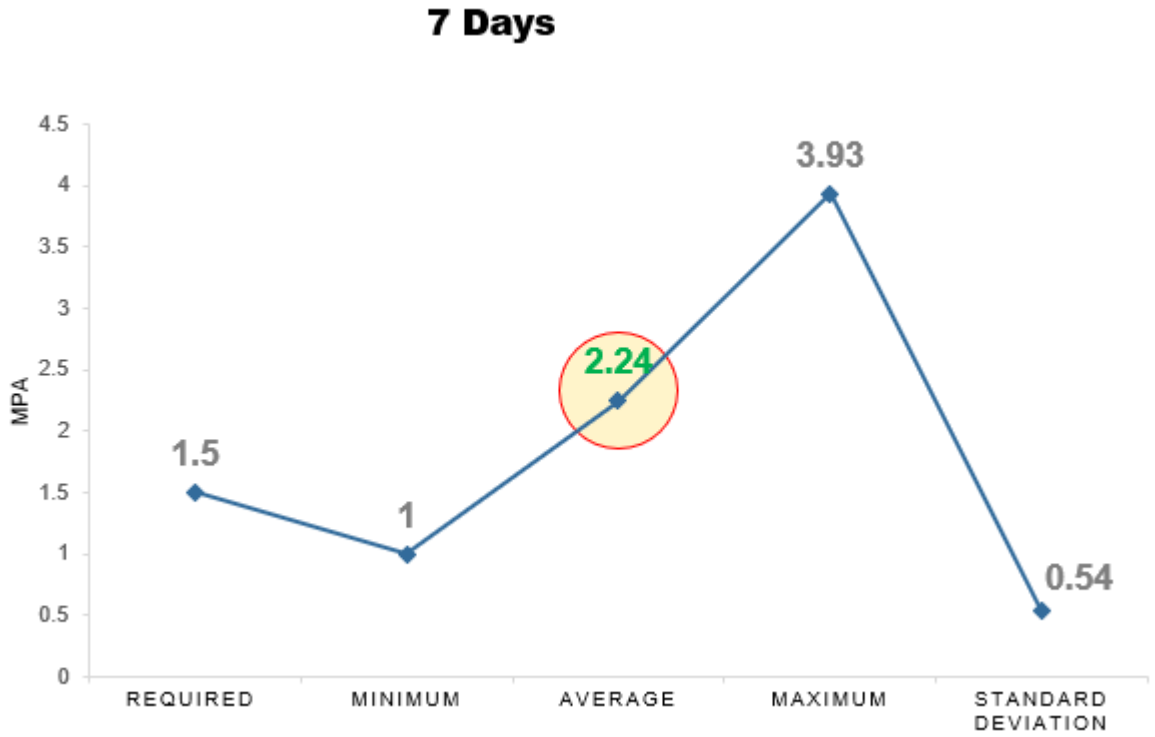


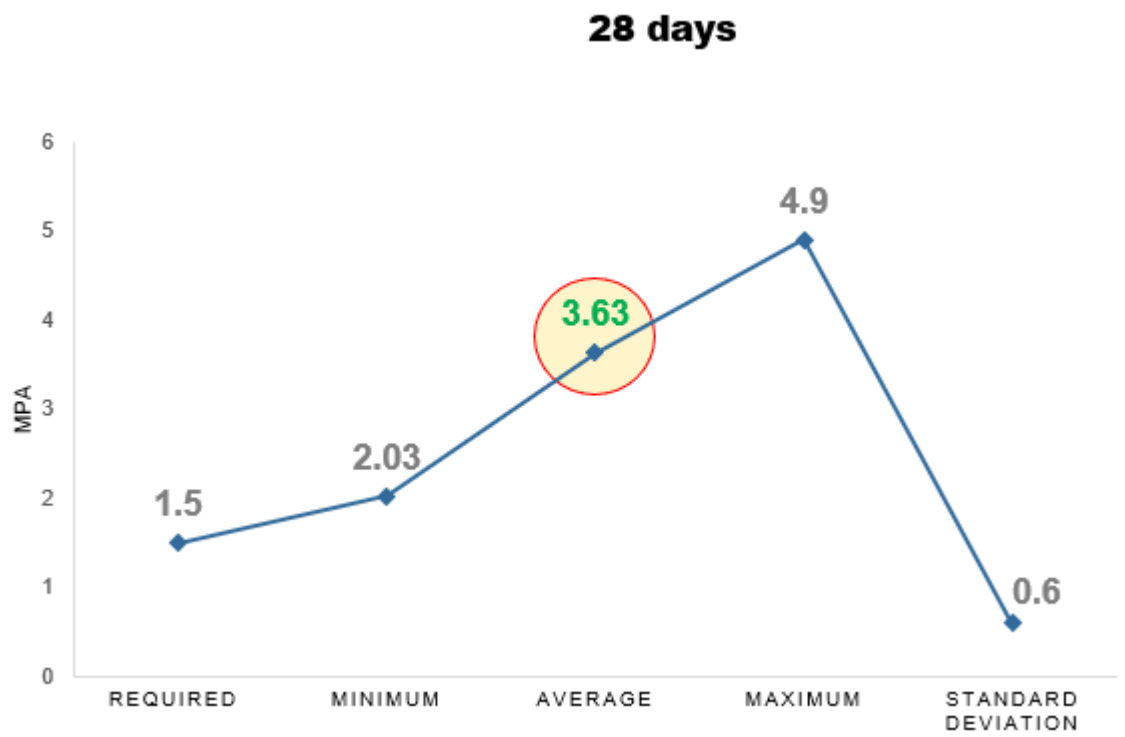






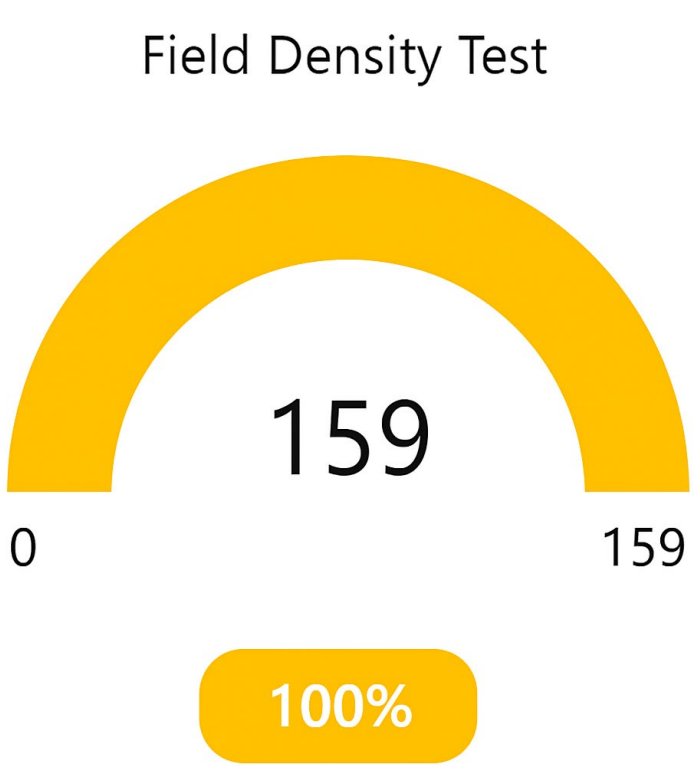
Verification Testing – DSM Compressive Strength Tests in Zone 1 & Zone 4





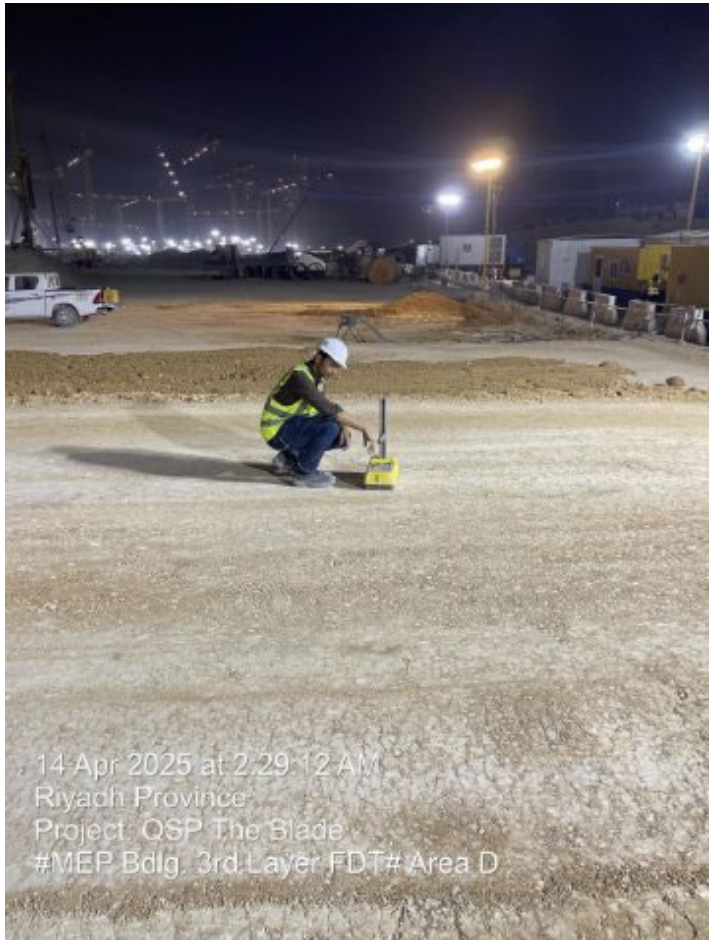
Total **360 Compressive Strength** Tests were performed and achieved the required strength

Verification Testing – Zone 2, Zone 3 & Zone 5



- **All Tests are passed as per project criteria**





Verification Testing – Zone 2, Zone 3 & Zone 5

PostCPT Test attempts made in the areas of Zone 2, Zone 3 and Zone 5 encountered Early Refusal due to Hard Ground. As these areas were not critical, and ground was already hard, minimum treatment performed to increase the ground stiffness and limit the differential settlement to allowable limits

No further verification testing was required.

WHAT WAS DONE:
Scope of Works

#	Area / Zone	SCM Dia. [mm]	Design [Nos]	As Built [Nos]	Design [m³]	As Built [m³]
1	Zone – 01	2000	834	834	19,352	19,366
2	Zone – 04	2000	1,437	1,437	36,518	36,512
	Total		2,287	2,287	55,870	55,878

#	Area / Zone	SC Dia. [mm]	Design [Nos]	As Built [Nos]	Design [m]	As Built [m]
1	Zone – 02	900	1240	1240	9,920	8,007
2	Zone – 03	900	1540	1540	12,320	10,285
3	Zone – 05	900	1212	1212	9,696	7,930
	Total		3,992	3,992	31,936	26,222

#	Area / Zone	Spacing [mm]	Design [Nos]	As Built [Nos]	Remarks
1	Zone – 02	3000	885	885	Qty includes the primary and secondary phase RIC compaction. Average – 04 blows were achieved on each print.
2	Zone – 03	3000	1,076	1,076	
3	Zone – 05	3000	859	858	
	Total		2,820	2,819	

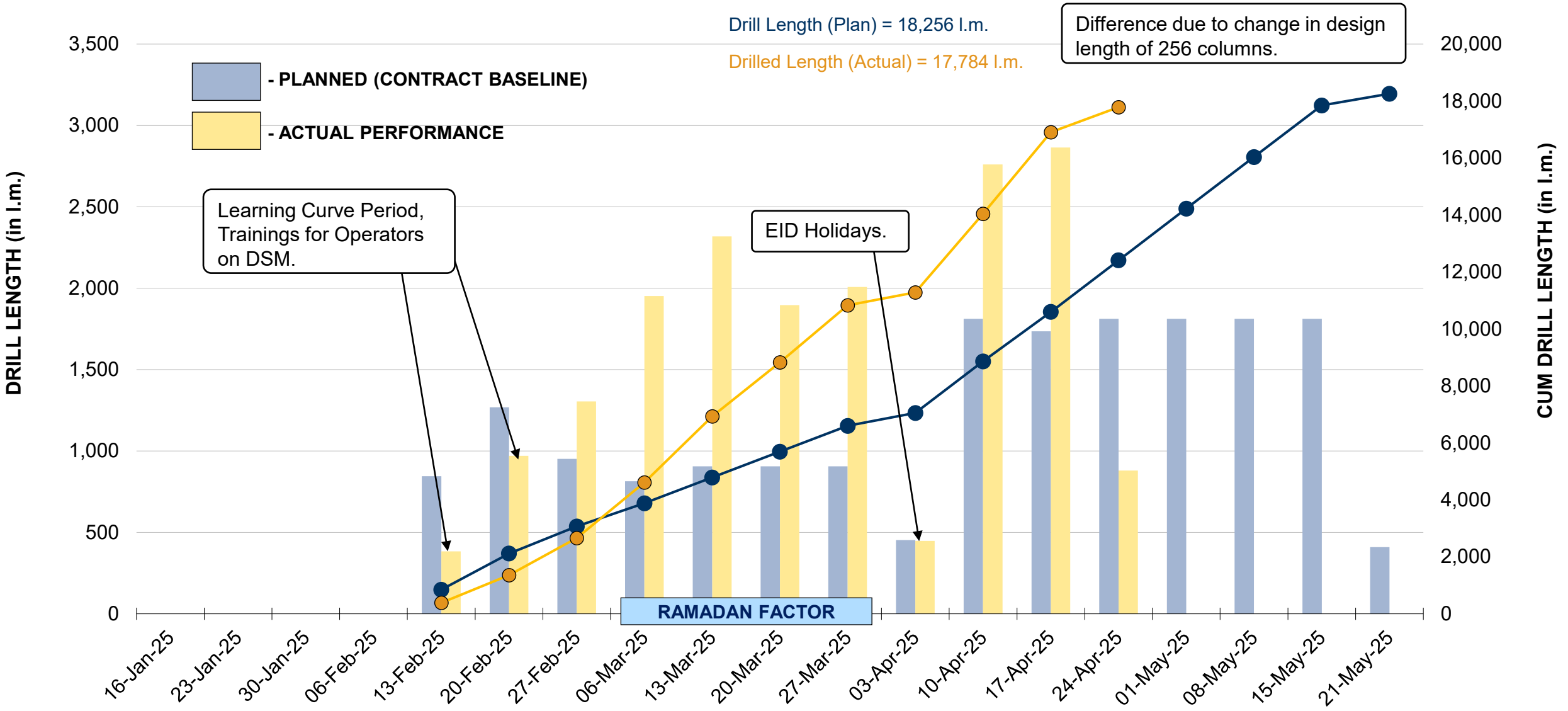
2.25 m spacing, achieving an Area Replacement Ratio (ARR) of 71.6%.

Target was 8m or refusal ; in order to uniform the soil and using the water to “pre collapse” and fill the void

RIC was final treatment to ensure maximum compaction. Typically 30/40 blows per point can be expected to achieve the criteria ; here was achieved after 4 blows!

Performance

Soil Mixing (Linear Meters)



Summary & Conclusions





PASSION for PROGRESS

Thank you

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Slurry Wall Design Techniques and Construction Practices Over the Last 50 Years in Northeastern United States

S. Y. Fantaye, P.E., Principal

Ketan H. Trivedi, P.E., Senior Associate

Alex Ciamei, P.E., Senior Associate

Lixun Sun, PhD., Lead Engineer

MRCE
built on firm foundations

Mueser Rutledge Consulting Engineers
14 Penn Plaza, 225 W. 34th Street
New York, NY 10122

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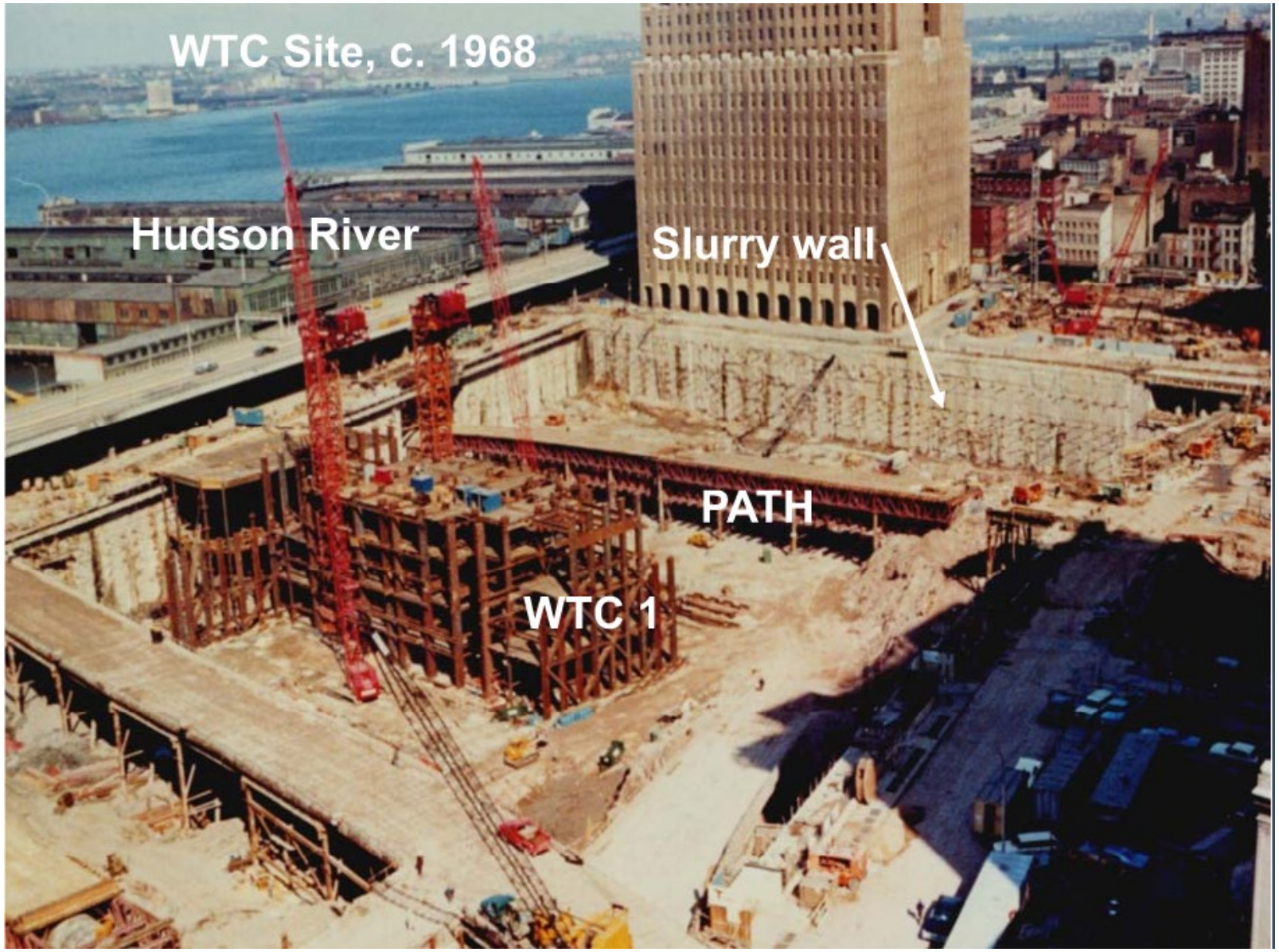
Outline

Design

- History and development
- Limit equilibrium method
- Finite element method
- Comparison of methods

Construction

- Panel Excavation
- Panel Joints
- Reinforcing
- Quality Control



History and Development

Methods of Analysis

- Limit equilibrium method.
- Finite element method.

Tools for Analysis

- Slide rule (mid-1800s)
- Hand Calculator (mid-1960s)
- Computer programs
  - Structural
    - FORTRAN (Language, 1957)
    - STRESS (1963)
    - STRUDL (1967)
    - STAAD-III (1980s)
    - RISA (Late 1980s)
    - Others
- Computer programs
  – Soil-Structure Interaction
    - BmCol (Late 1960s)
    - CivilTech Software (1990)
    - Plaxis (1987)
    - Many Others

Design Methods

- Working stress method
- Ultimate strength method (1971)

Limit Equilibrium Method

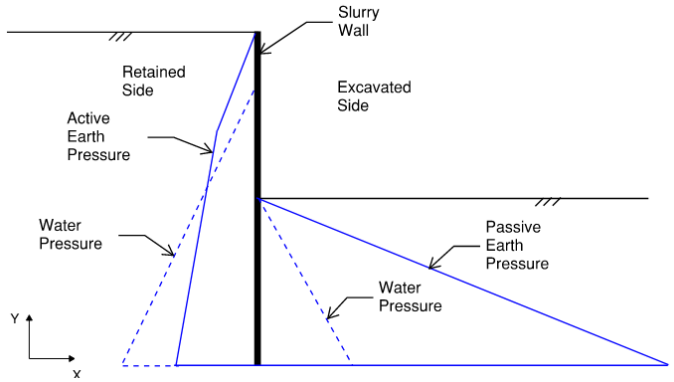


Fig. 1. – Pressure Diagrams for Limit Equilibrium Method

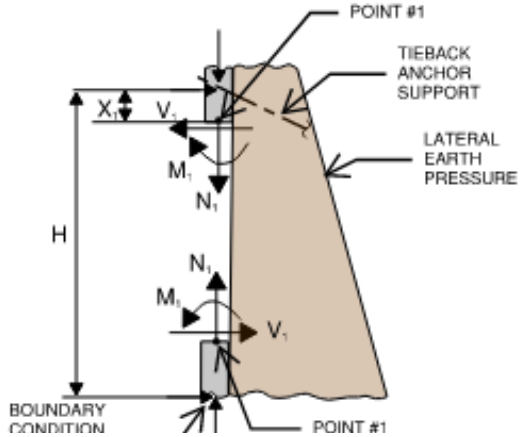


Fig. 2. – Free body diagram of slurry wall with one level of support

- Importance of staging analysis
- Location of “assumed pin support.”

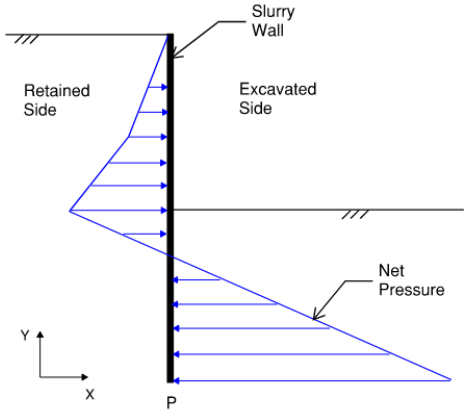


Fig. 3. – Net Pressure Diagram for 1st Excavation Stage (Cantilever)

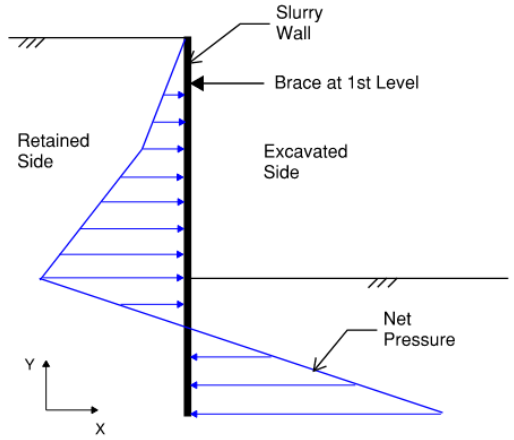


Fig. 4. – Net Pressure Diagram for 2nd Excavation Stage (One Level Brace)

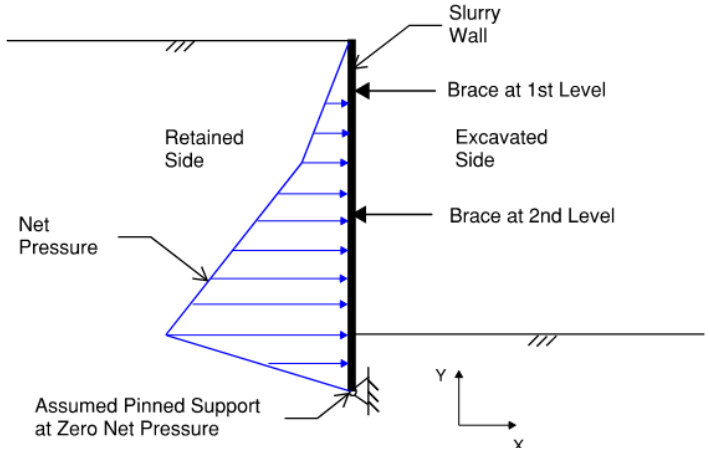


Fig. 5. – Net Pressure Diagram for 3rd Excavation Stage (Two Level Braces)

Finite Element Method

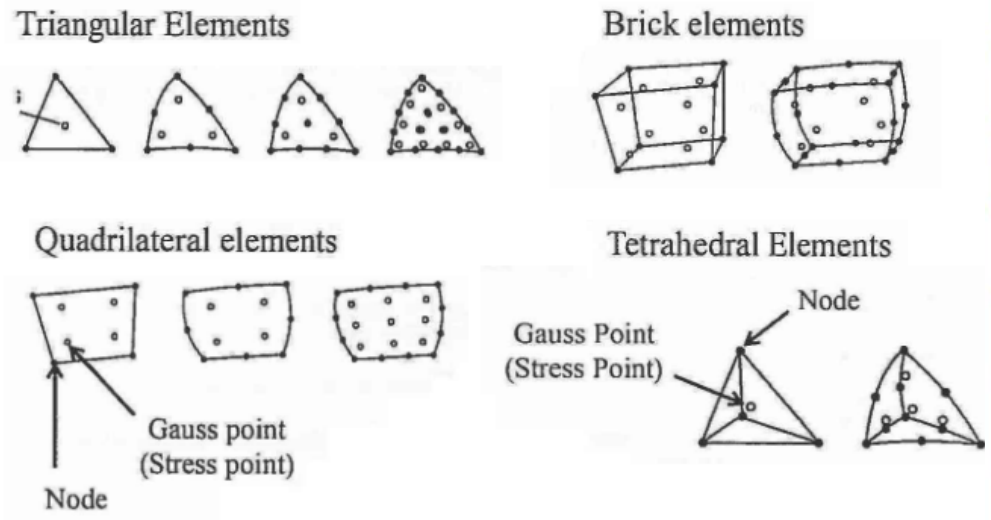


Fig. 6. – Types of Elements Used in Finite Element Analysis

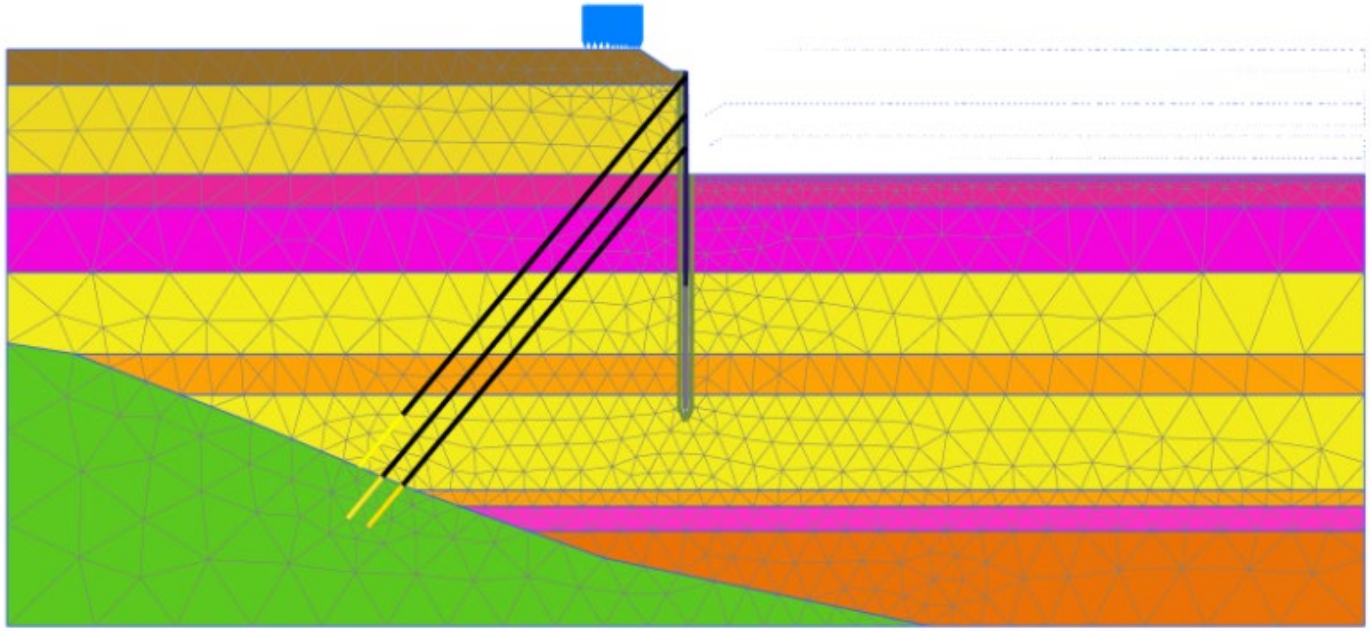
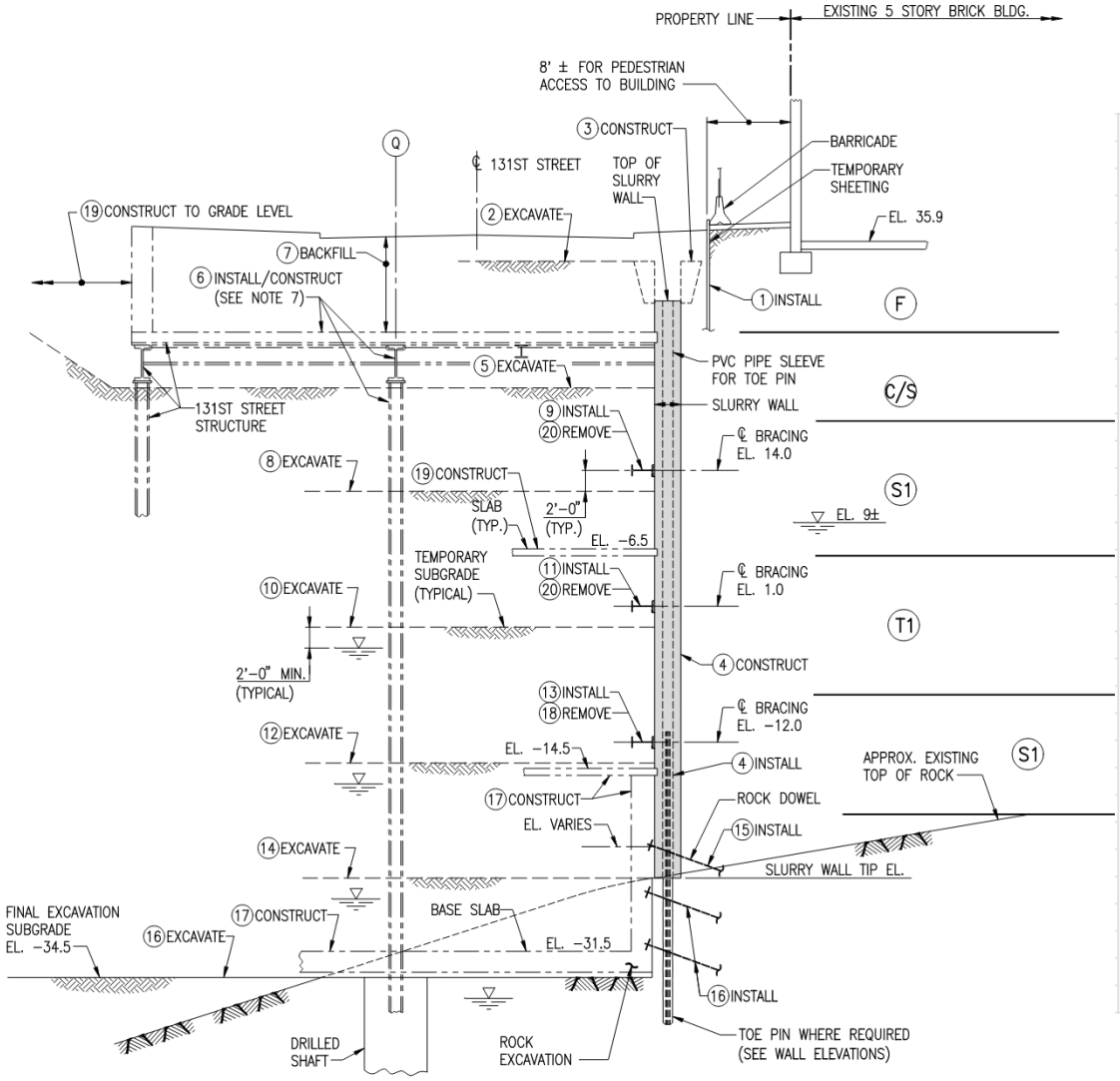
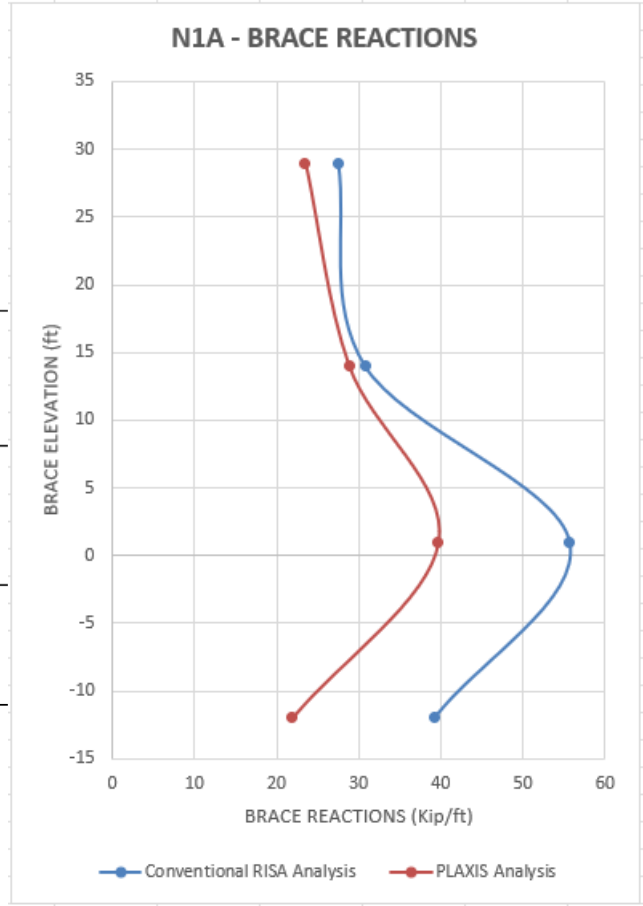


Fig. 7. – A Typical FEA/Plaxis 2D Mesh for SOE Analysis

- Allows determination of impact of slurry wall, bracing system and sequence of excavation on the movement of adjacent structures.

Comparison of Methods





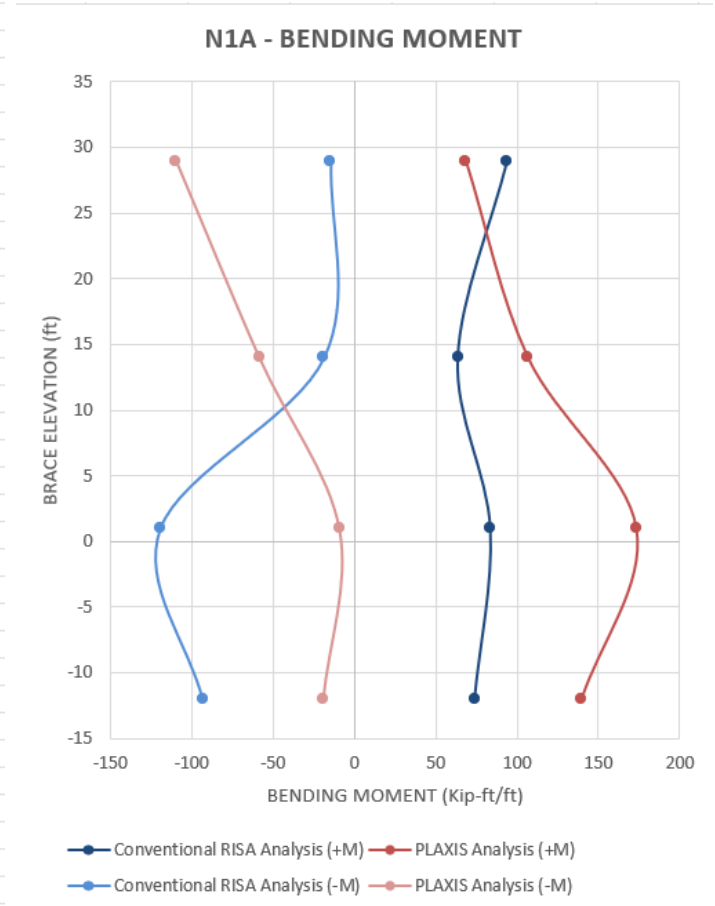
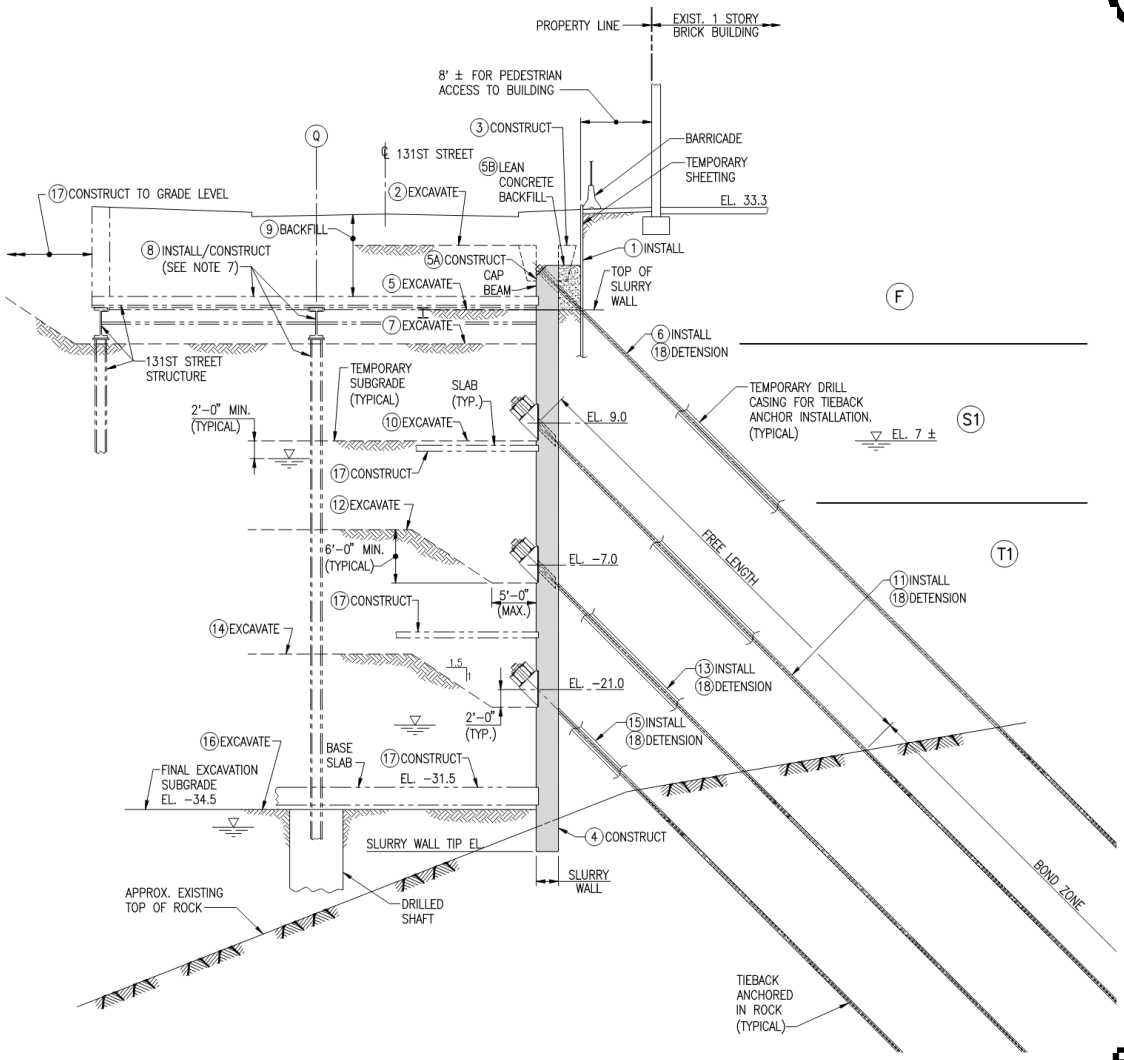
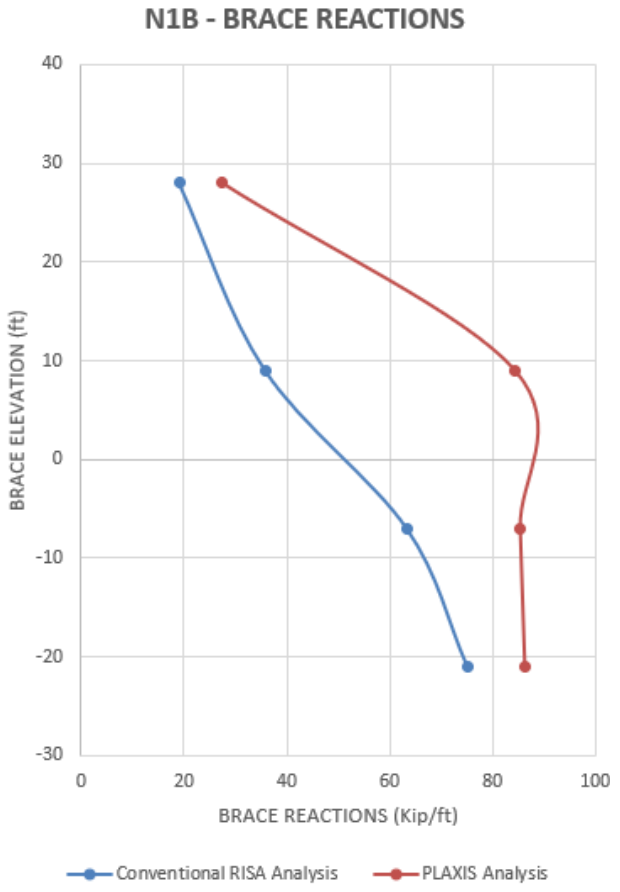


Fig. 8. Comparison of Brace Loads and Wall Moments – Case 1 (Note wall is supported by preloaded internal bracing)

Comparison of Methods (con't)





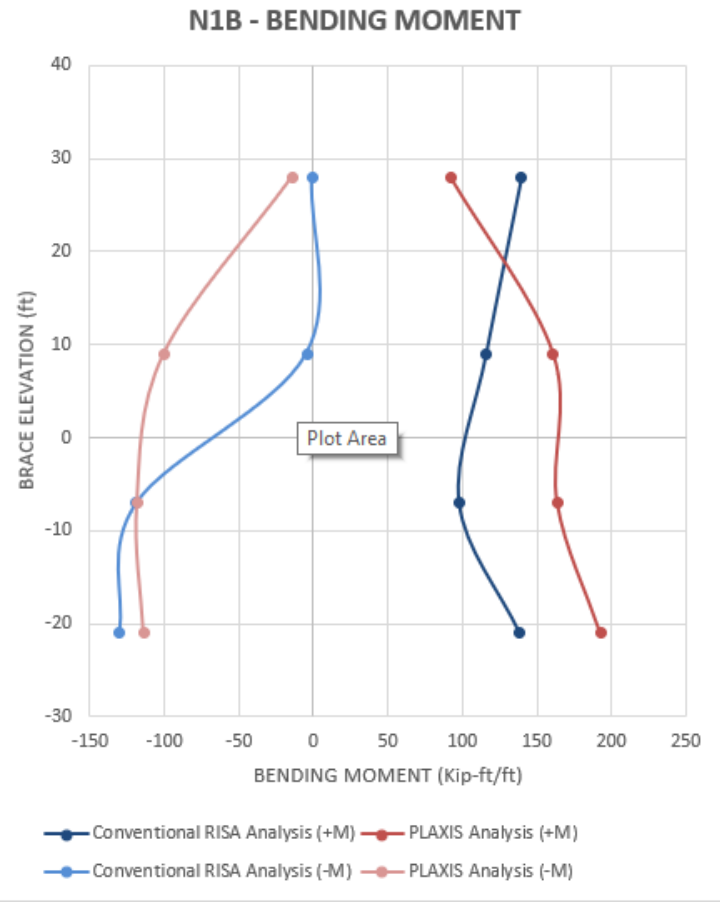


Fig. 9. Comparison of Brace Loads and Wall Moments – Case 2 (Note wall is supported by preloaded tieback anchors)

Construction Methods

Mechanical Clamshell

Hydraulic Clamshell

Hydromill/Hydrofraise

Mechanical Clamshell and Chisel

- Initial method of Excavation involved cable supended mechanical clamshell bucket
- Percission Chisels were used to penetrate obstructions
- Impresa Costruziono Opere Specializzate (ICOS) credited with invention of the mechanical chisel in the early 1950s.





Equipment

- Hydraulic Clamshell System introduced in the early t mid 1970s
- Hydrofraise developed in Equrope by Soletanch-Bachy in the early 1970s



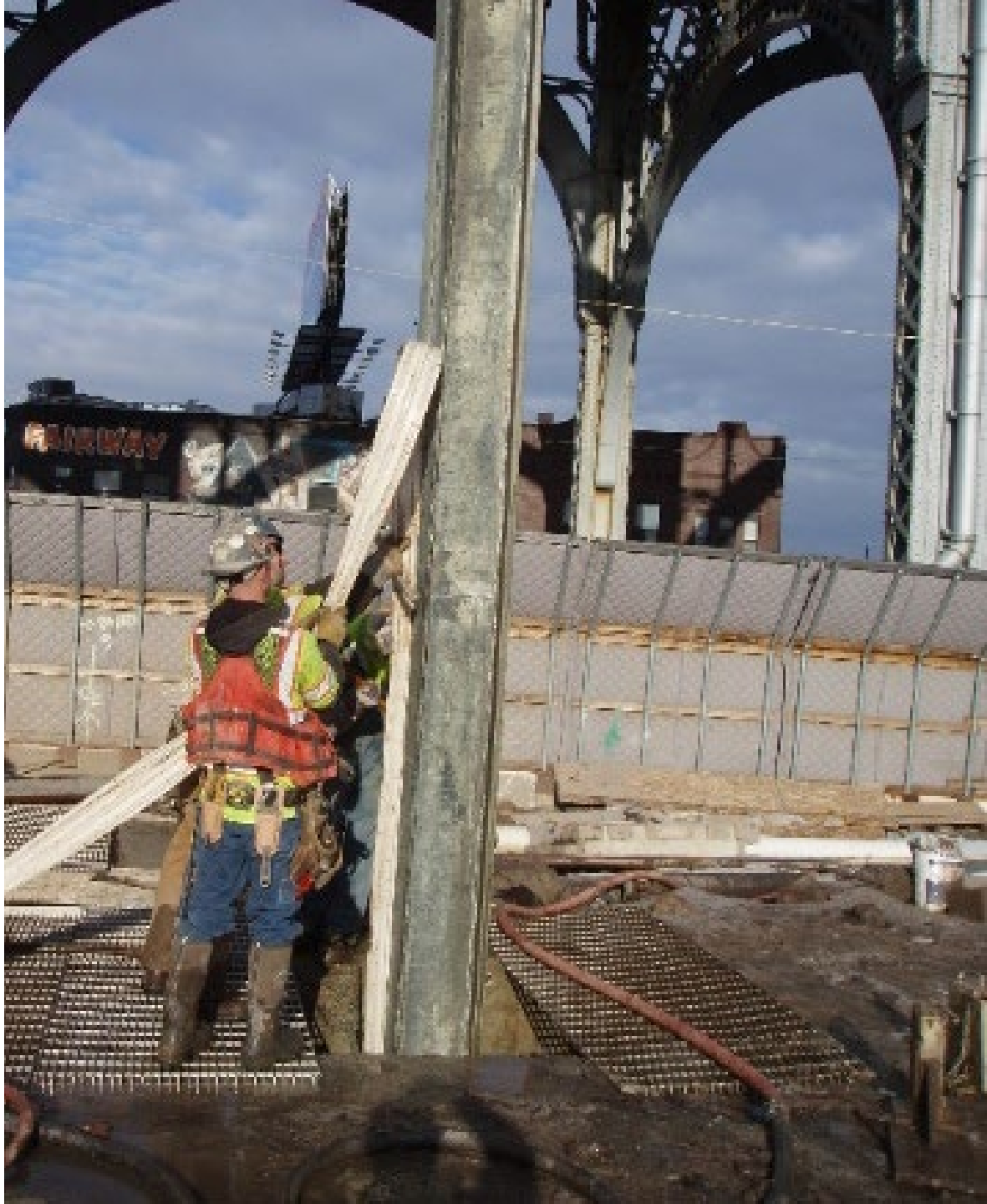




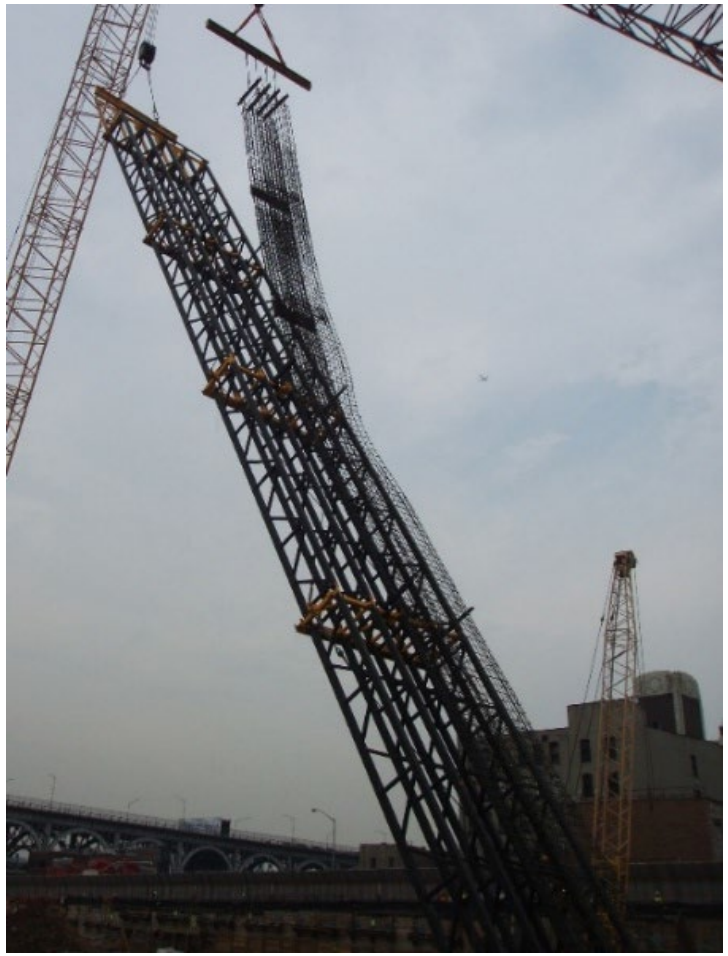
Diaphragm wall panel joint construction methods

- Wide Flange Beam End Stops
- Continuous Wayerstop (CWS) Joint
- “Overbite” Construction Joint





Slurry wall reinforcing





- Reinforcing Cage
  - Tilt Table
  - Rigging

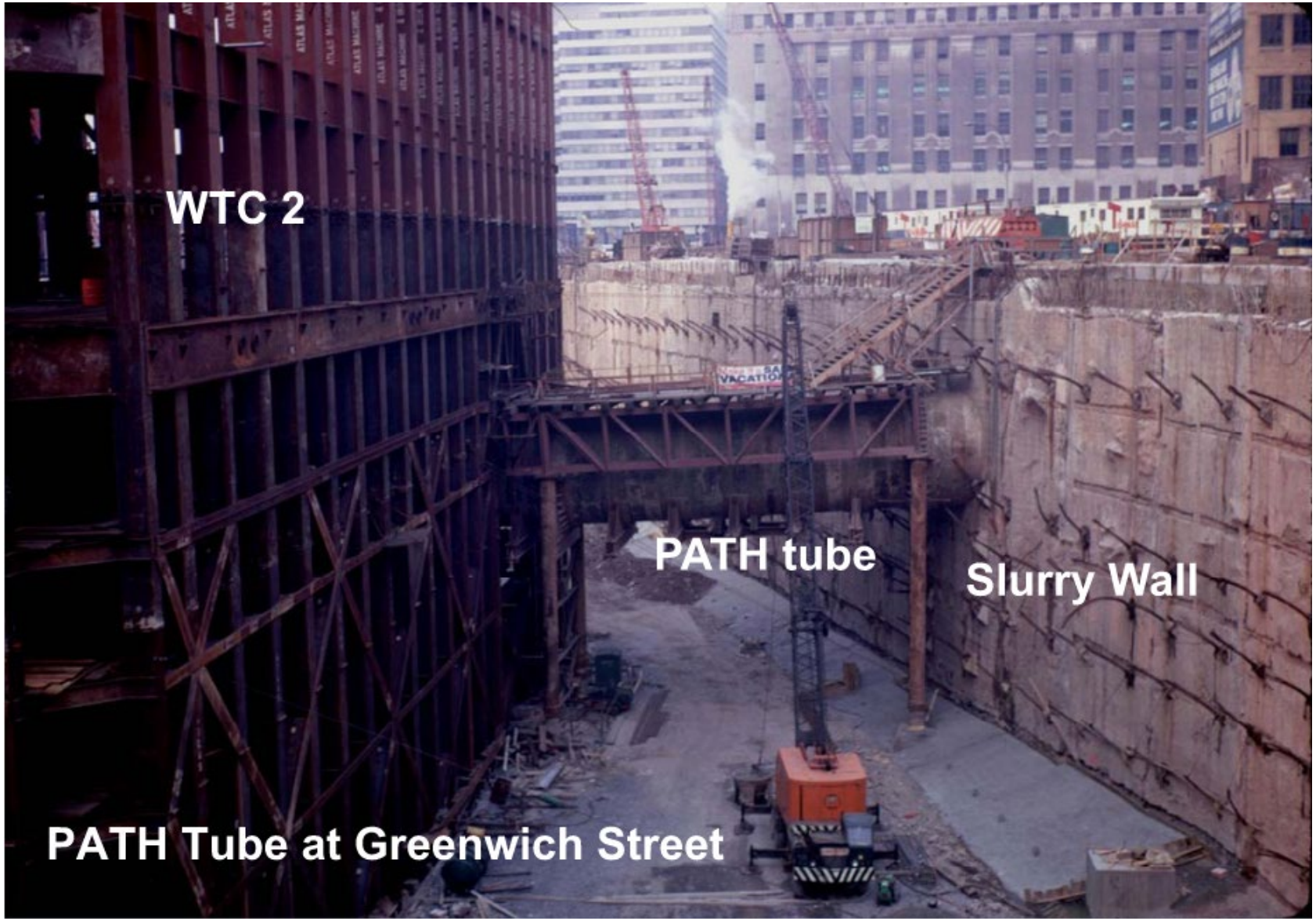
QUALITY CONTROL AND ASSURANCE

- Review of slurry wall submittals
- Careful and detailed daily logging of activities.
- Daily sampling and testing of the slurry mix to measure density, viscosity, pH and filtrate loss
- Inspecting/observing panel excavation including proper documentation of method of excavation, quality and level of slurry mix within the trench, type and quantity of material being excavated, verticality, sounding of panel bottom with a weighted tape.
- Reinforcing cages inspection. Spacing between reinforcing bars is critical for concrete placement.
- Accurate surveying of reference elevations. Typically, the top of guide wall.
- Concrete sampling and testing
- Inspecting/observing concrete placement.
    - Tracking of Tremie Pipe Embedment (Prior/Post removal of sections)

Representative Slurry Wall Projects

NAME[1]	PERIOD	LOCATION	PURPOSE	GEOLOGICAL CONDITIONS	WALL PROPERTIES			LATERAL WALL SUPPORT	CONSTR. METHOD	ANALYSIS METHOD
					THICK.	DEPTH				
					PANEL FT (m)	PANEL FT (m)	EXCAV. FT (m)			
Manhattanville, Columbia University	2008-2025	NY, NY	Excavation Support & Perm. Wall	Fill, Organic, Sand, Varved, Till, Decomposed/Weathered Rock, Bedrock	Rectangular: 3 to 4 (0.9 to 1.2), T-panels: 3 (0.9)	140 (42.7)	40 (12.2)	Permanent Floor Slabs, Tiebacks, Cross Lot Bracing	Top-Down / Conventional	Limit Equilibrium, FEM
World Trade Center Post 9/11 Recovery	2001-2006	NY, NY	Excavation Support & Perm. Wall	Fill, Organic, Sand, Till, Decomposed Rock, Bedrock	3 (0.9)	100 (30.5)	80 (24.4)	Tiebacks	Conventional	Limit Equilibrium, FEM
World Trade Center Vehicle Security Center	2007-2014	NY, NY	Excavation Support & Perm. Wall	Fill, Organic, Sand, Decomposed Rock, Bedrock	3 (0.9)	100 (30.5)	60 to 100 (18.3 to 30.5)	Tiebacks, Raker / Butress panel, Permanent Floor Slabs	Conventional	Limit Equilibrium, FEM
Gowanus Canal CSO	2019-2025	NY, NY	Excavation Support & Perm. Wall	Fill, Organic, Sand, Till, Decomposed/Weathered Rock, Bedrock	T-panels: 3 (0.9)	210 (64.0)	30 (9.1)	Cross Lot Bracing, Permanent Floor Slabs	Conventional	Limit Equilibrium, FEM
New York Law School	2004-2008	NY, NY	Excavation Support & Perm. Wall	Fill, Sand, Till, Decomposed Rock, Bedrock	3 (0.9)	100 (30.5)	65 (19.8)	Permanent Floor Slabs	Top-Down	Limit Equilibrium
63rd St. Line to Queens Blvd. Subway Extension	1994-1996	NY, NY	Excavation Support	Fill, Varved, Sand, Bedrock	3 (0.9)	35 to 80 (10.7 to 24.4)	35 to 80 (10.7 to 24.4)	Tiebacks	Conventional	Limit Equilibrium
National WWII Monument	1999-2002	Washington, DC	Excavation Support & Perm. Wall	Fill, Sand, Bedrock	3 (0.9)	30 to 40 (9.1 to 12.2)	15 to 30 (4.6 to 9.1)	None	Conventional	Limit Equilibrium
Boston Marine Industrial Park Tunnel	1991-1992	Boston, MA	Excavation Support	Fill, Blue Clay, Bedrock	3 (0.9)	70 to 100 (21.3 to 30.5)	40 to 60 (12.2 to 18.3)	Cross Lot Struts / Pin Piles	Conventional	Limit Equilibrium
Four Seasons Hotel and Legg Mason Headquarters	2006-2008	Baltimore, MD	Excavation Support & Perm. Wall	Fill, Alluvial Silt/Clay, Potomac Sediments, Residual Soil, Weathered Rock, Bedrock	2.5 (0.8)	70 to 100 (21.3 to 30.5)	55 (16.8)	Tiebacks, Permanent Floor Slabs	Conventional	Limit Equilibrium, FEM
30 Hudson Street Office Complex	1999-2002	Jersey City, NJ	Excavation Support & Perm. Wall	Fill, Organic, Sand, Till, Decomposed Rock, Bedrock	2.5 (0.8)	35 to 65 (10.7 to 19.8)	45 (13.7)	Tiebacks, Permanent Floor Slabs	Conventional	Limit Equilibrium

Thank You!



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PARALLEL SESSION B1
Gold Innovations I



Subsurface Exploration for I-40 Reconstruction in the Pigeon River Gorge Post Hurricane Helene

Chris Ramsey, PE and Michael Senior, PE

DFI Annual Conference 2025

October 22, 2025

Schnabel ENGINEERING

Build Better. Together.

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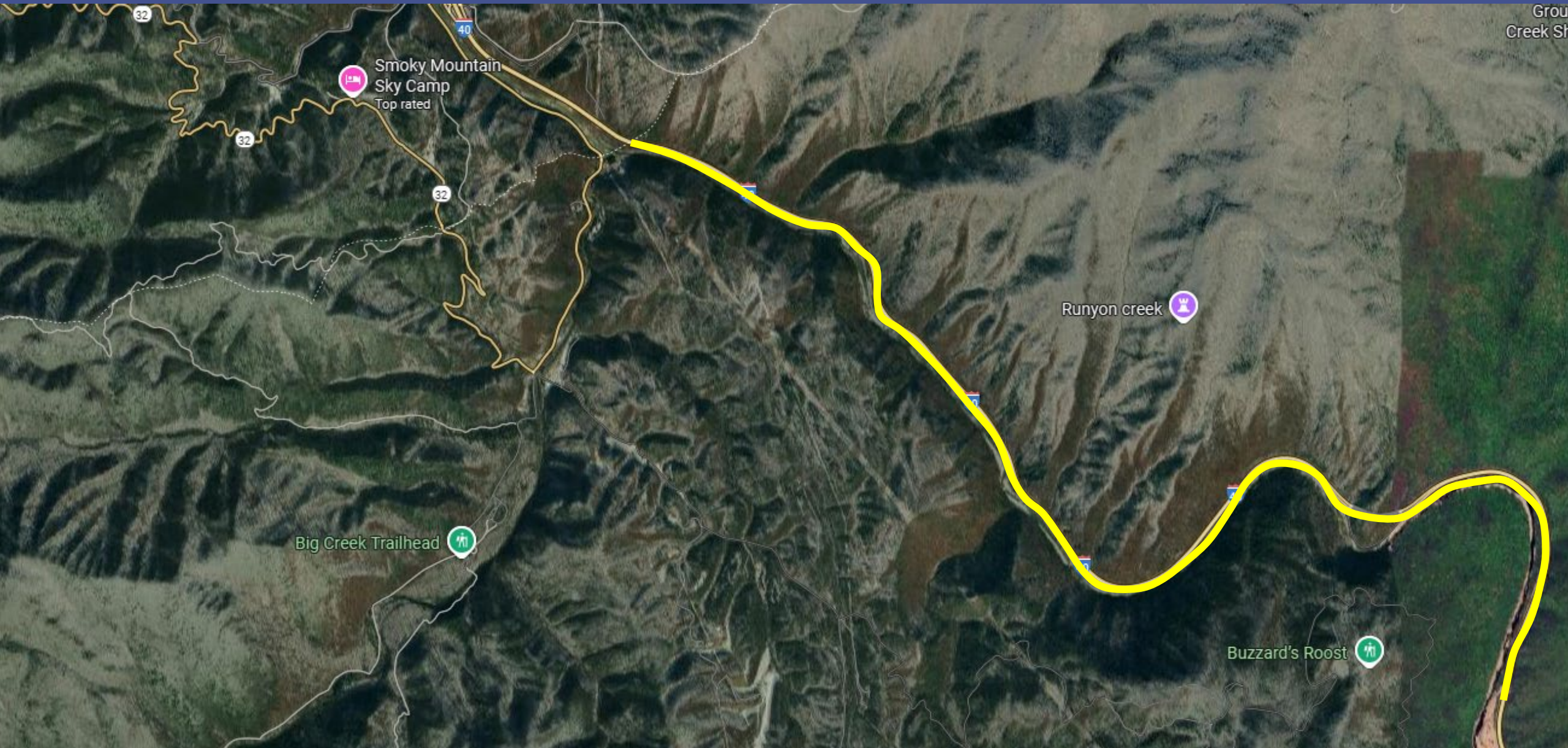
PRESENTATION OUTLINE

- History of I-40 through Pigeon River Gorge
- Damage from Hurricane Helene
- Geotechnical Exploration
- Data, Data, Data!

History of I-40 through Pigeon River Gorge

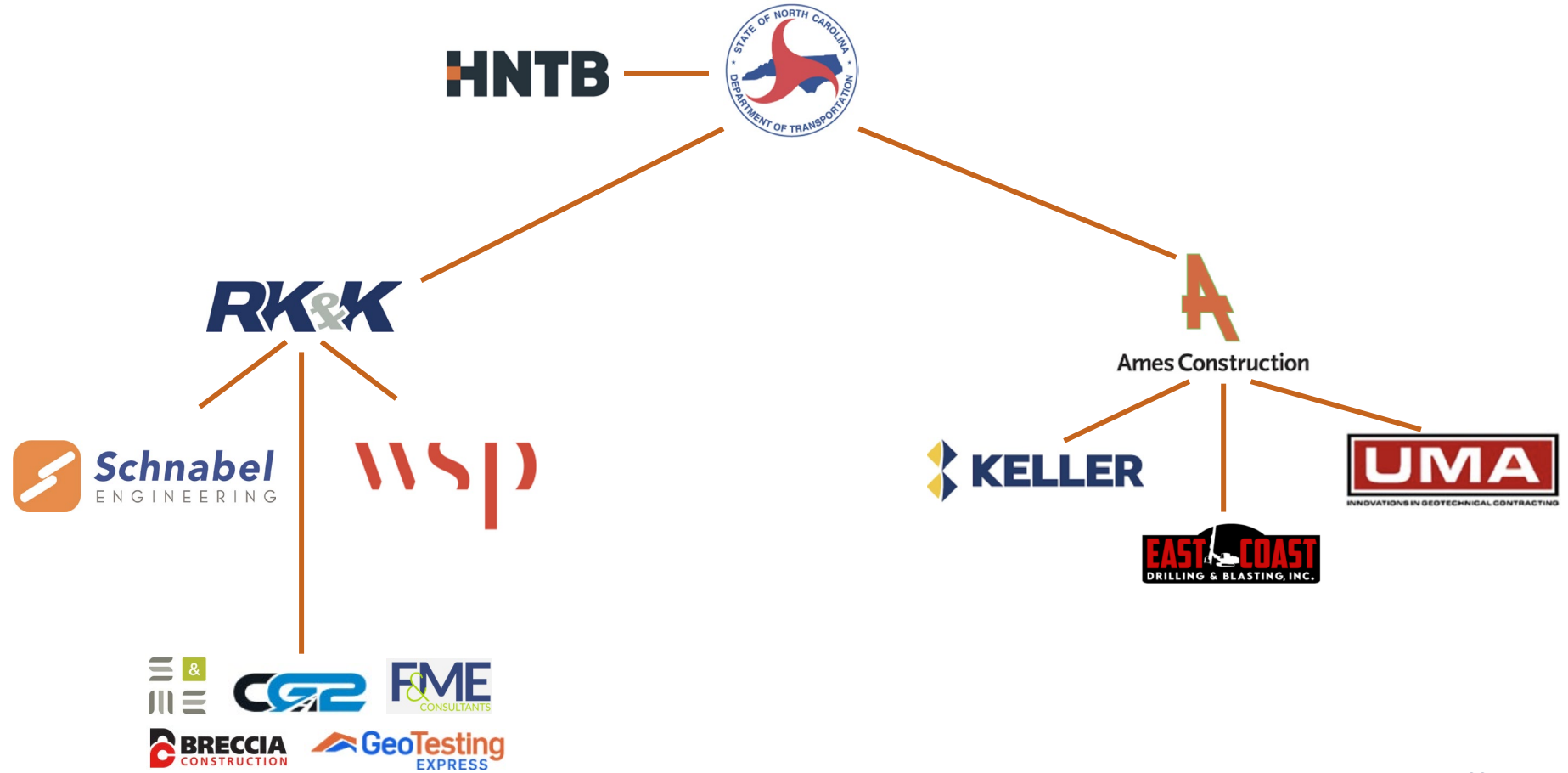


History of I-40 through Pigeon River Gorge





Overall Project Team – CMGC Procurement



Parallel Emergency Stabilization Efforts

WRIGHT BROTHERS

GSI GeoStabilization International

ACCESS LIMITED

Project Goals

Per FHWA via NCDOT

- Resilient, Redundant, Reliable
- I-40 Corridor from MM 0.0 to MM 5.0 has a history of closure/repairs recurring every ~10 years
- Provide a system that will be resilient in storm events – generally withstand a storm event and not have extended road closures

- From FHWA-HIF-23-008, 2023:

Resilience	With respect to a project, "resilience" means a project with the ability to anticipate, prepare for, and/or adapt to changing conditions and/or withstand, respond to, and/or recover rapidly from disruptions, including the ability: (A) to resist hazards or withstand impacts from weather events and natural disasters, or reduce the magnitude or duration of impacts of a disruptive weather event or natural disaster on a project; and (B) to have the absorptive capacity, adaptive capacity, and recoverability to decrease project vulnerability to weather events or other natural disasters.	23 U.S.C. § 101(a)(24)[4] (See also definition in TEACR Synthesis Report)

Excerpt from: Table 2-1, "Geohazards, Extreme Weather Events, and Climate Change Resilience Manual", B. Zelenko et al, (2023)

GEOTECHNICAL INVESTIGATIONS

- What has already been used?
  - Horizontal drains
  - Mass Rip Rap Embankment
  - Soldier Pile Walls
- What type and length of structure is anticipated?
  - Tunnel?
  - Walls & Viaducts?
  - Reinforced Buttress?
  - Continuous Walls?
- Determine investigation methods

Previous Repair Methods used at I-40 & Pigeon River



GEOTECHNICAL INVESTIGATION

- 5 miles of stabilization
- Find depth to bedrock
- Overburden is very variable – silts & sands up to 20 foot boulders
  - Observation of temp. soil nails used for stabilization
  - Multichannel Analysis of Subsurface Waves (MASW) Geophysical investigation
  - Air track probes ~200 feet
  - Traditional rock coring ~500 feet
  - Include televiewer of rock cores

GEOTECHNICAL INVESTIGATION

- Over 600 investigation data points, split into 3 Phases
  - Phase I: WB Lanes while no live traffic
  - Phase II: EB Lanes
  - Phase III: At river level from causeway
- Over 15 miles of MASW
- Lab sample data (UCS, Point Load, Direct Shear, etc.)

Geotechnical Investigation
Soil Nail Installation

Temp. Soil Nail Drilling Observation

- Depth to rock
  - Range from 2 to 38 feet
  - Rock not encountered in ~25% of observed nails
- Penetration Rates
  - Range from 0.1 to 4.1 minutes per foot (min/ft)

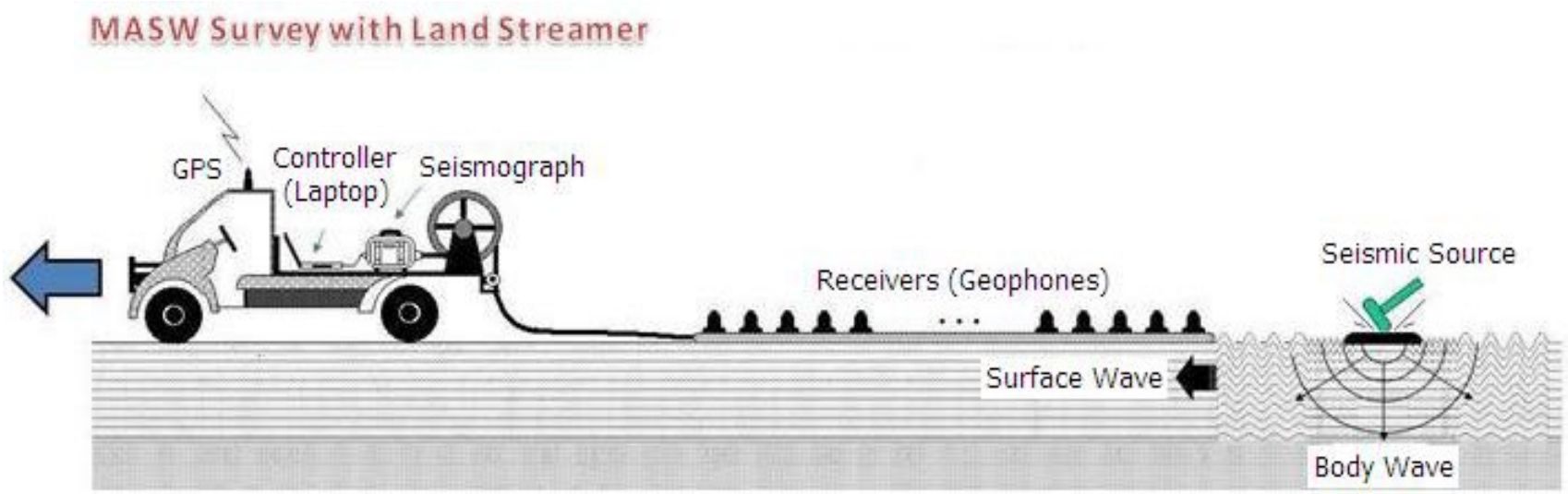


Geotechnical Investigation
MASW Surveys

Geophysics

- 3 crews working multiple shifts
- ~15 miles of data collected

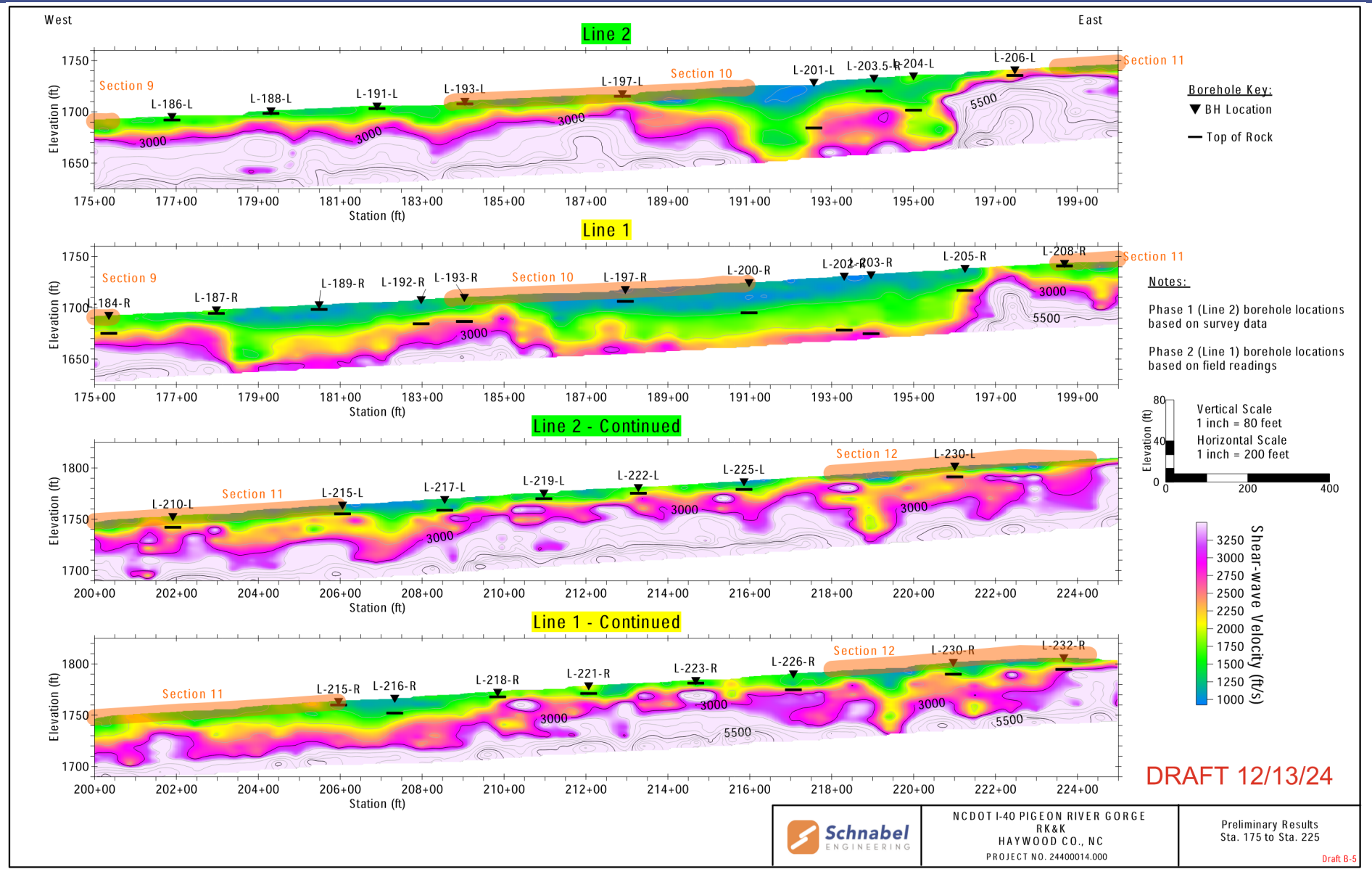




Geotechnical Investigation
Geophysics – High Resolution Imagery with MASW



Geotechnical Investigation
Geophysics – MASW Profiles



Geotechnical Investigation
Conventional Drilling Methods

Geotechnical borings

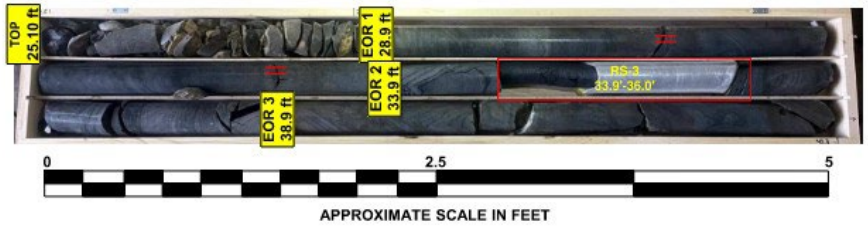
- Probe to rock
  - Casing advancer on Geotech drill
  - Micropile drill
- Core bedrock 30 feet
- Televiewer core length

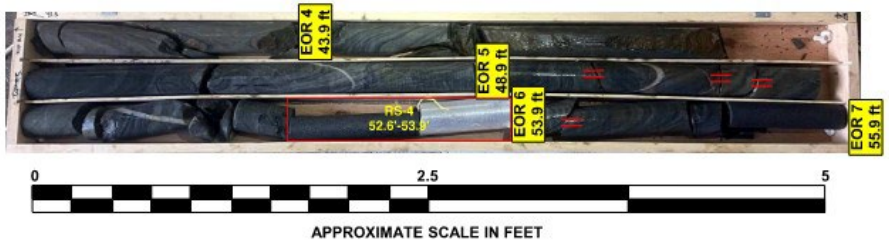




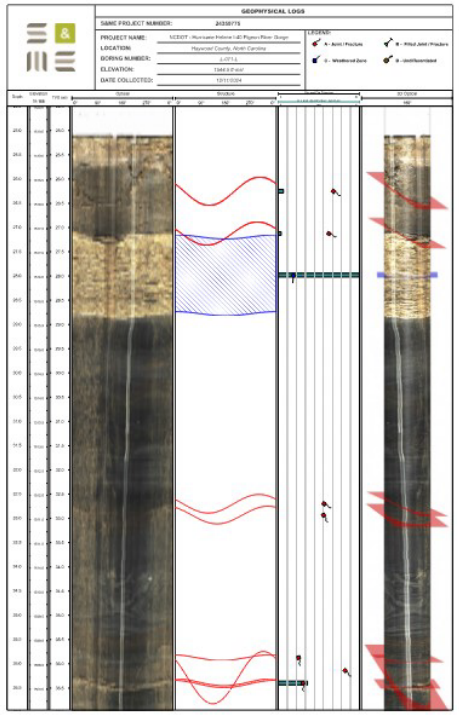


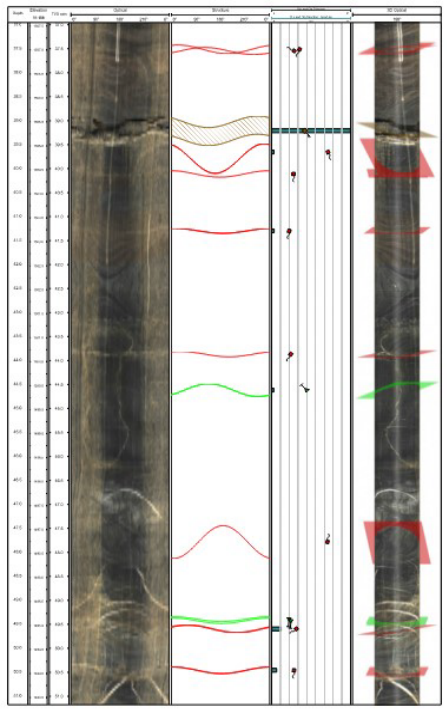
Geotechnical Investigation
Rock Core & Optical Televiewer Data Comparison











Geotechnical Investigation
Air Track Methods

Air Track Probes

- Up to 3 rigs at a time
- Depth to rock
  - Range from 3 to 58 feet
  - Rock not encountered in ~7% of probes











Geotechnical Investigation
Considerations

Top of Rock

- Steep sloping bedrock surface
- Variability in rock surface elevation
- Hard rock – siltstone, sandstone, meta graywacke, quartzite

Rock Bedding / Fractures Planes

- Dip toward the river

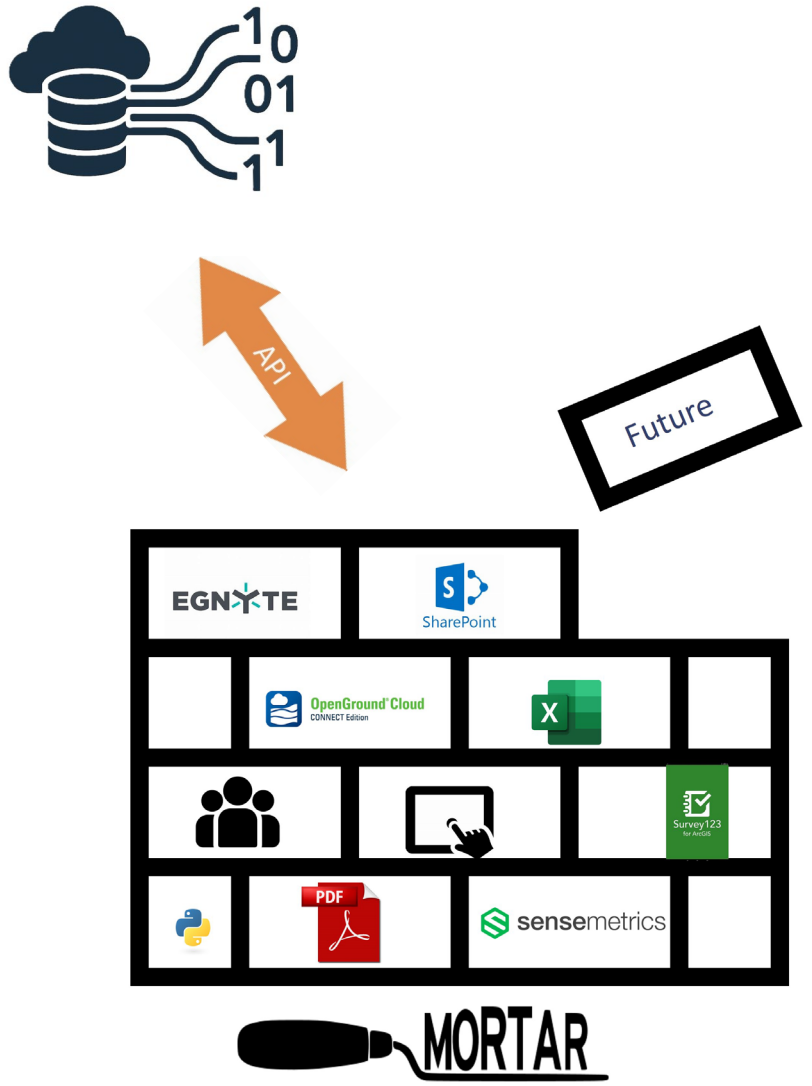


DATA MANAGEMENT WITH MORTAR

- Internally developed database system (MORTAR)
- Geo-referenced data
- Data mapping and integration
- Facilitates a more informed design process

Data Management with Mortar
Next Generation of Project Delivery

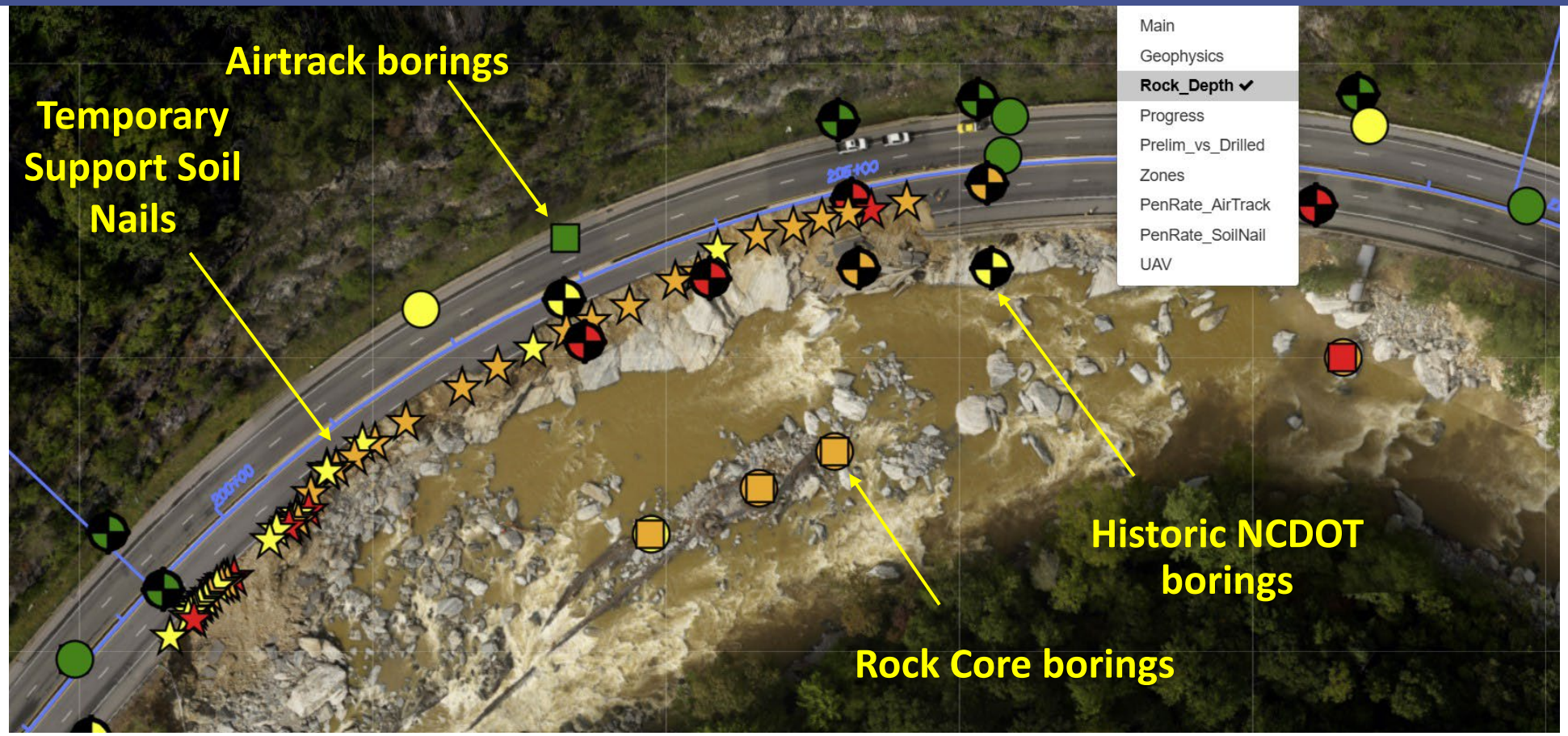
- Centralized Data Hub
- Built-in QA/QC
- Dynamic Visualization
- Linked Content
- Integration & Portals
- Advanced Outputs

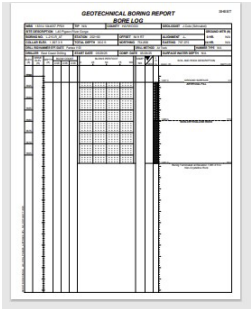


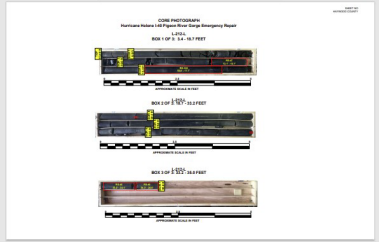
MAPPING & VISUALIZATION

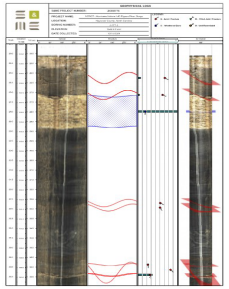
- Internally developed database system (MORTAR)
- Data mapping and integration
- Facilitates a more informed design process

Mapping and Visualization
Mortar Portal



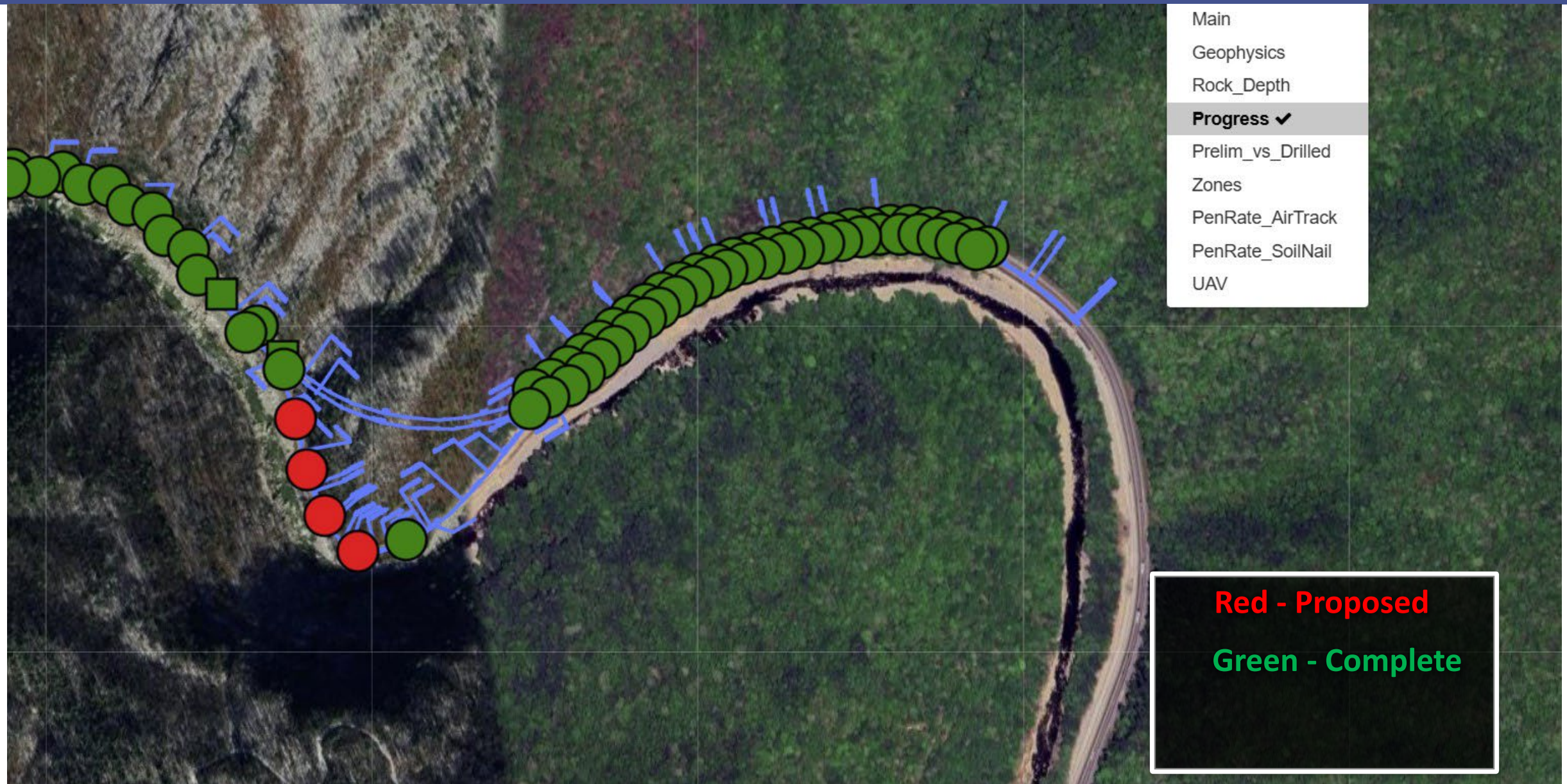






- >30 feet
- 30 feet > 20 feet
- 20 feet > 10 feet
- < 10 feet

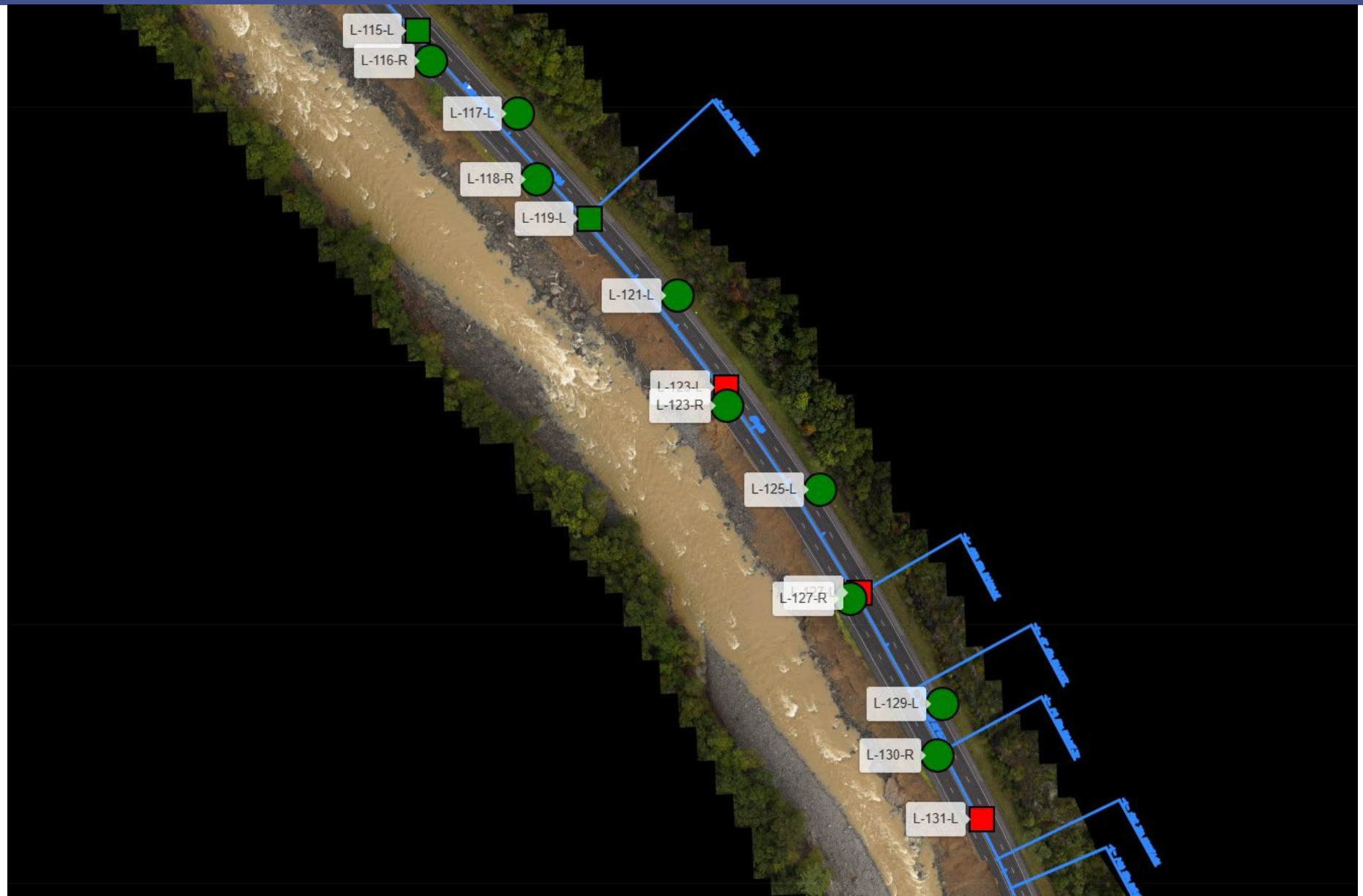
Mapping & Visualization
Mortar Progress Example



Pre-Storm Satellite Aerial Imagery



Post Storm Satellite Aerial Imagery



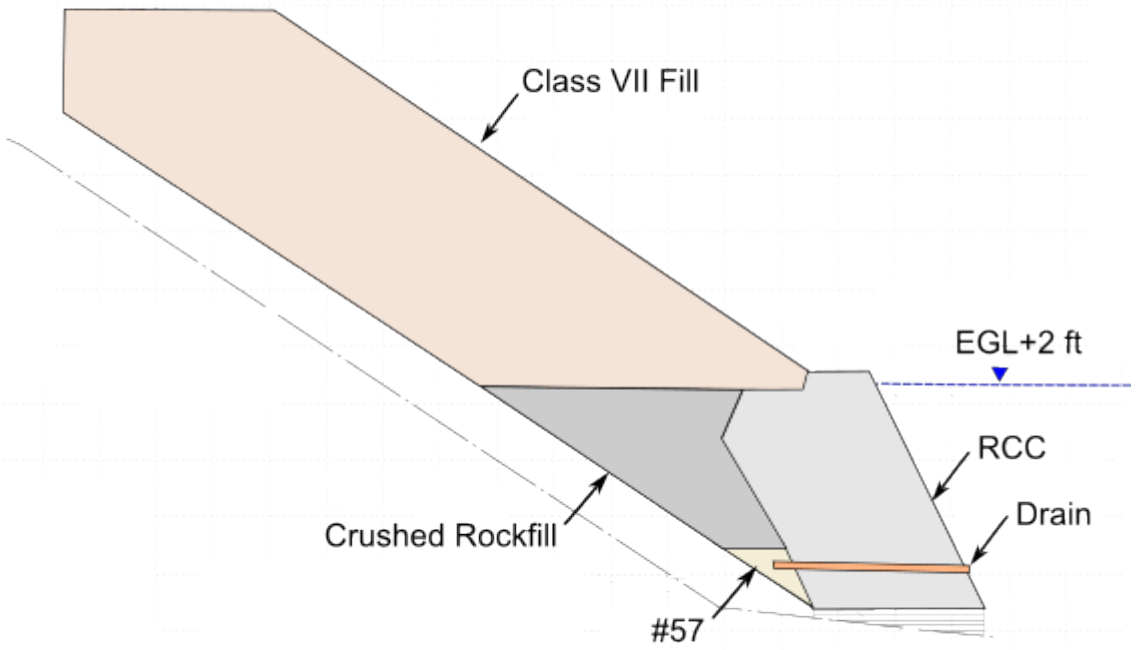
GEOSTRUCTURAL CONCEPTS

- TUNNEL (no-go)
- VIADUCT WITH WALL AT CENTERLINE (no-go)
- RCC BUTTRESS
- WALL AT SHOULDER WITH BACKFILL OPTIONS
- WALL AT TOE TO RE-ESTABLISH EMBANKMENT

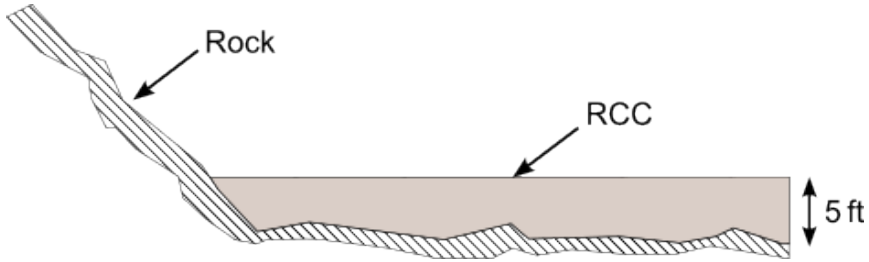
RCC Buttress Concept

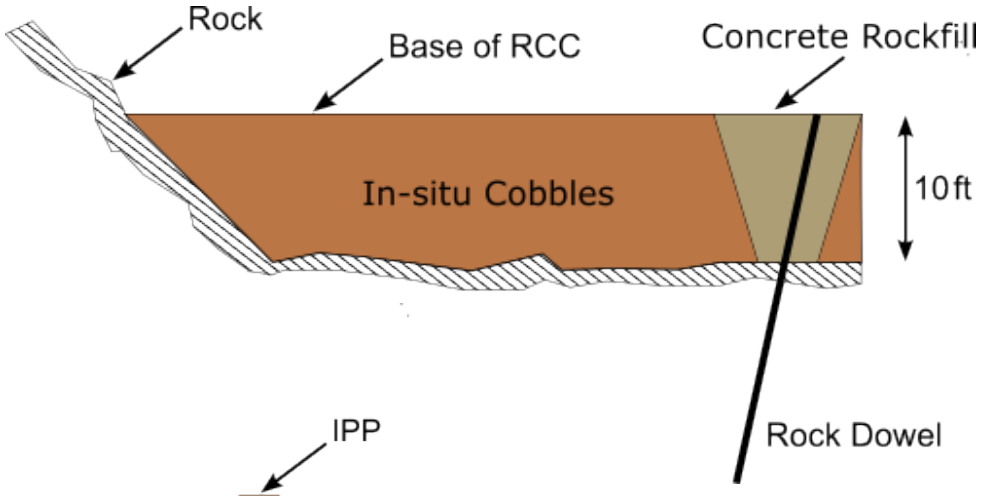
Roller-Compacted Concrete

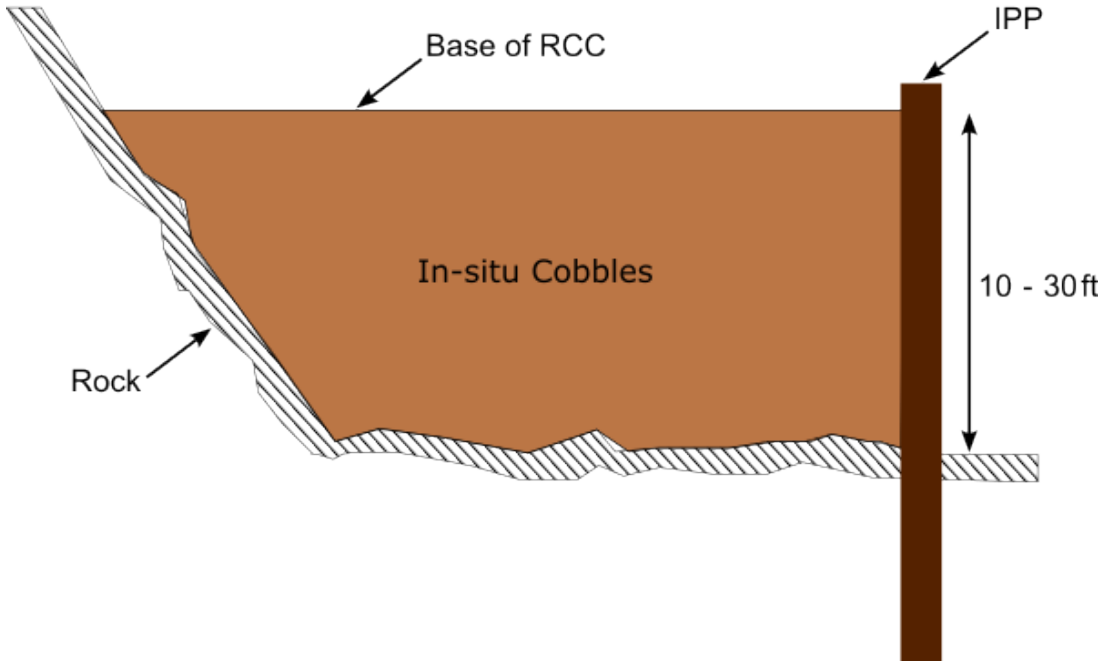
- No Slump Concrete (2,000 psi)
- Embankment above Energy Grade Line (EGL)
- Toe Treatment for Scour Protection



Toe Treatment for Scour Protection



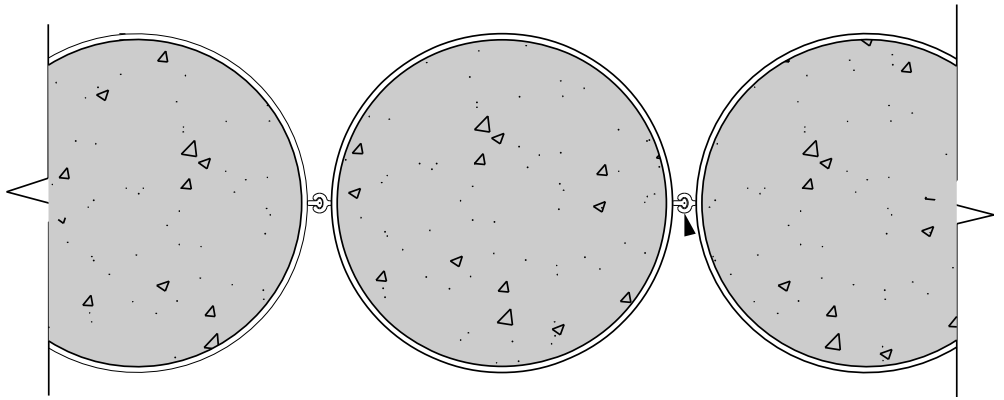


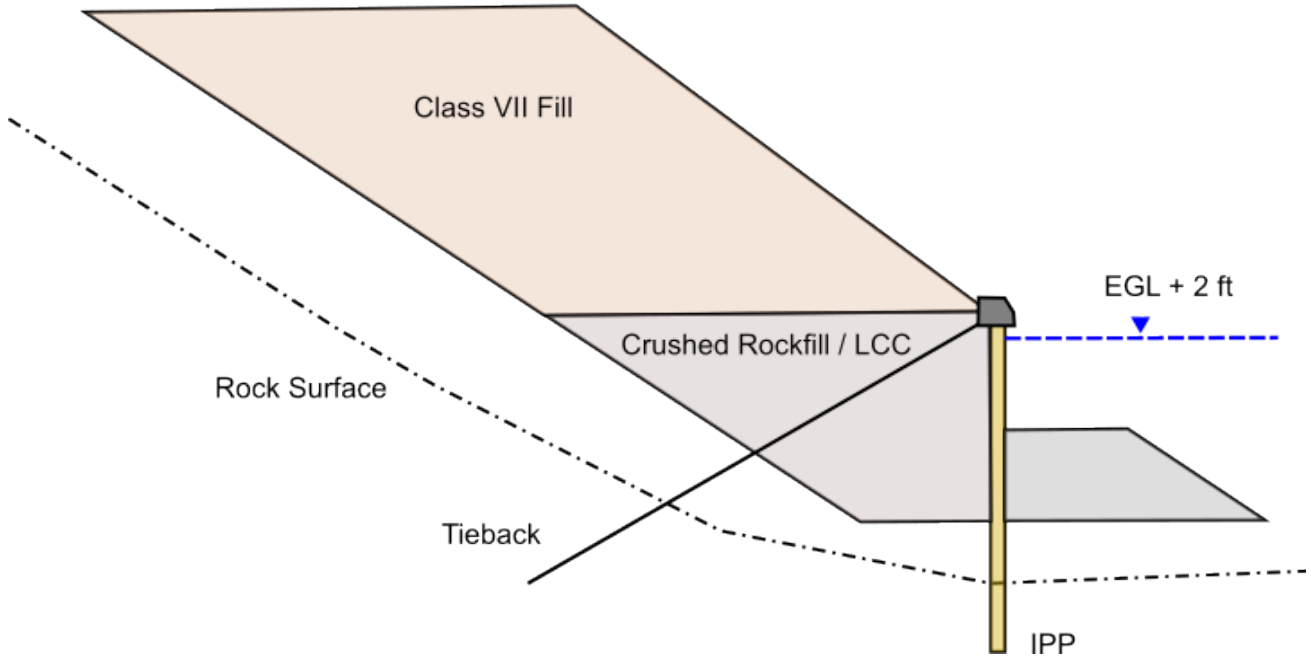


Interlocking Pipe Pile Wall Concept



Keller - North America





LESSONS LEARNED

- Need to be adaptable to get information quickly and inform timely decisions on an emergency project
  - Ground conditions vs. design concepts
  - Air tracks vs. traditional borings
- Data management helps with large volumes of data to review/reduce

QUESTIONS?

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Schnabel ENGINEERING

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Design, Construction, and Performance of a Sensitive Braced Excavation in Boston Blue Clay

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COLLIN E. SWEENEY, P.E., GEOENGINEERS USA, P.C.

JEFFREY M. TAYLOR, TREVIICOS

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Project Background

Construction of a 14-story building with 3 levels of below-grade parking.

- Cuts of up to 44 feet below grade.

Site located in the Fenway neighborhood of Boston, Massachusetts.

Tight urban environment with a historic building and light rail subway tracks in close proximity to the proposed building.

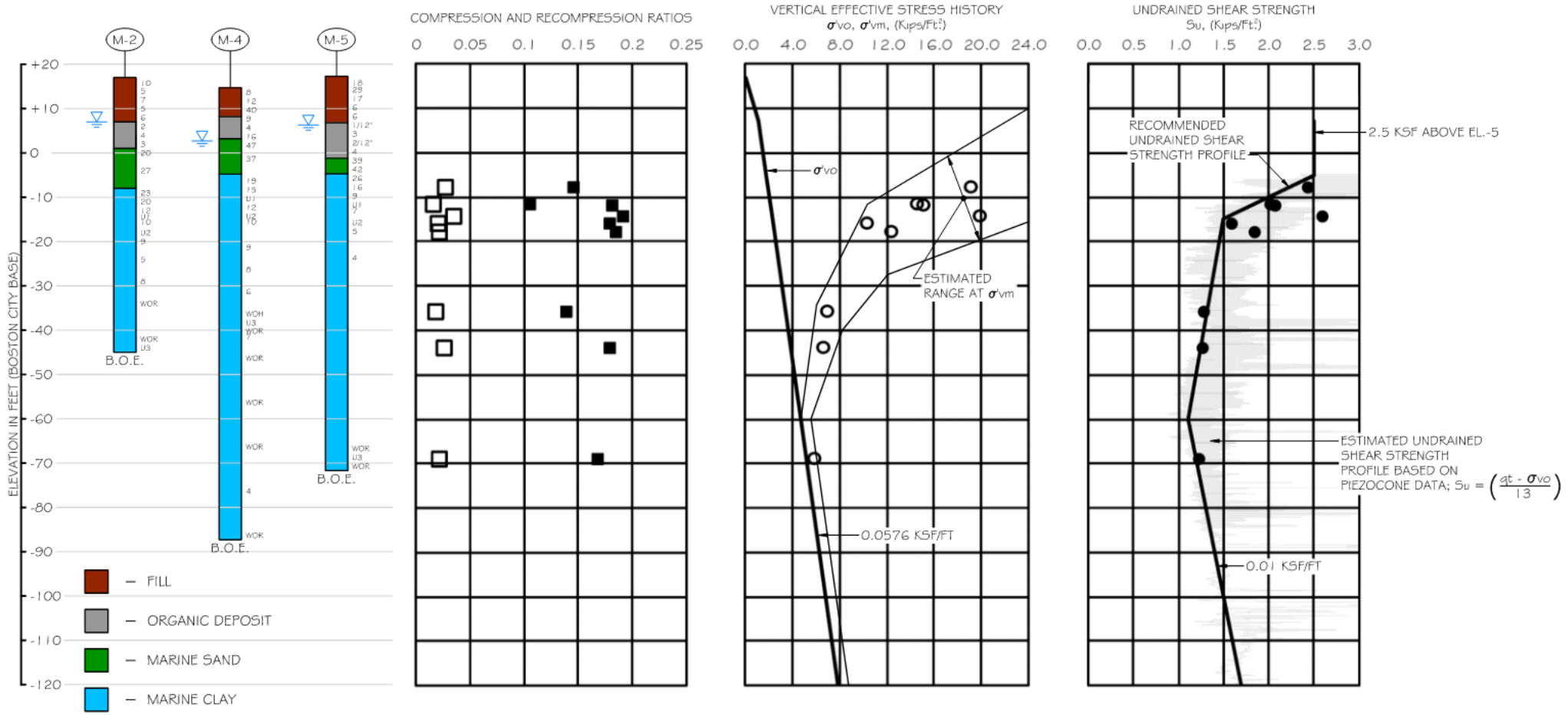
Phased excavation due to site constraints.

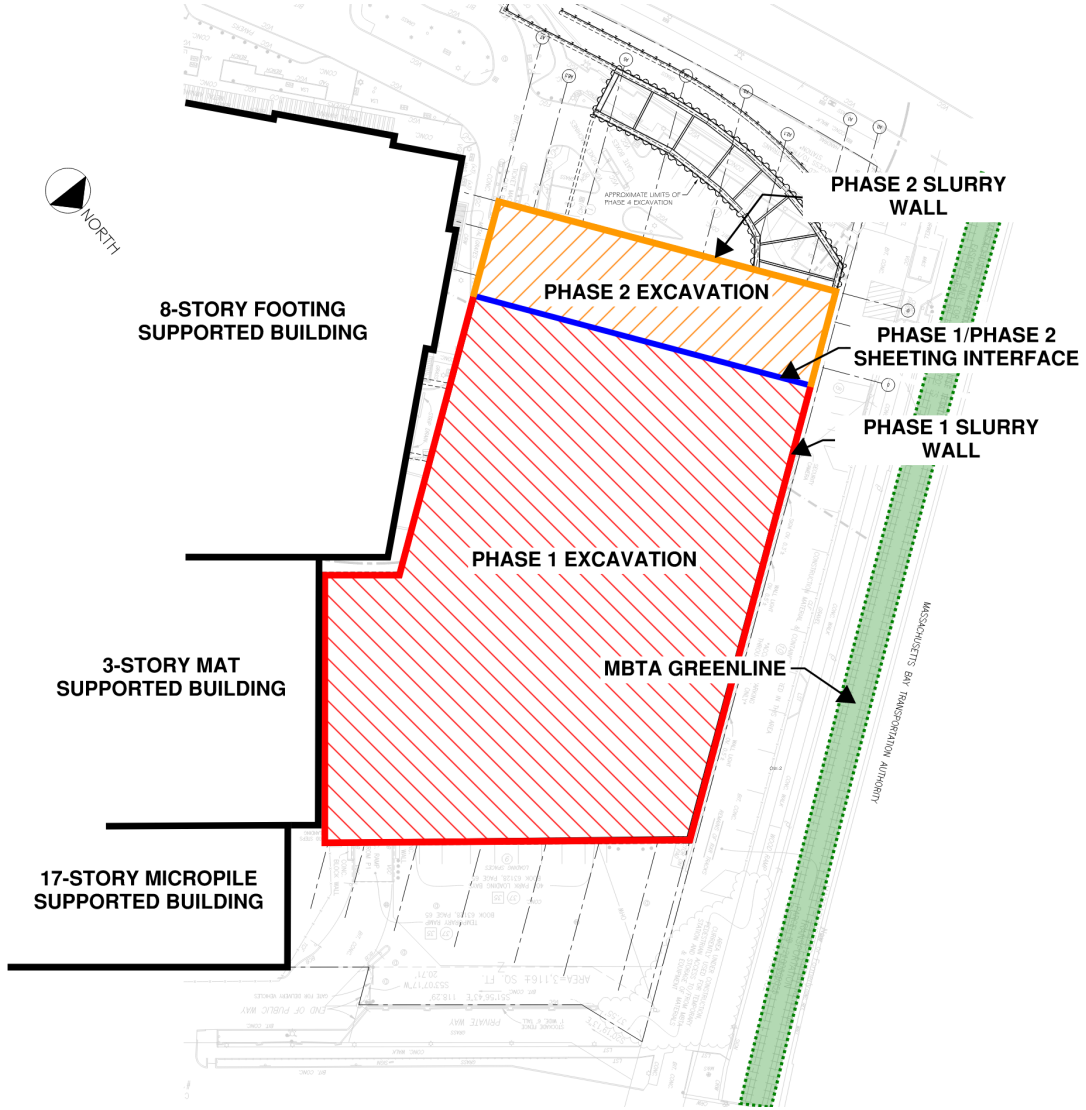
Geotechnical Overview

Generalized Subsurface Strata	Approximate Thickness (Feet)
Fill	4 to 21 (1.2 to 6.4 m)
Organics	1 to 18 (0.3 to 5.5 m)
Marine Sand	4 to 16.5 (1.2 to 5 m)
Marine Clay	135 to 173 (41.1 to 52.7 m)
Glacial Deposits / Weathered Bedrock	10 to 18 (3 to 5.5 m)
Bedrock	N/A

Groundwater located ~8 to 11 feet below existing grades.

Geotechnical Overview





Project Setting

8-story building (1928):

- Founded on spread footings bearing on marine sand at a depth of approximately 16 feet (4.9 m).
- Single below-grade level.

3-story building (1965):

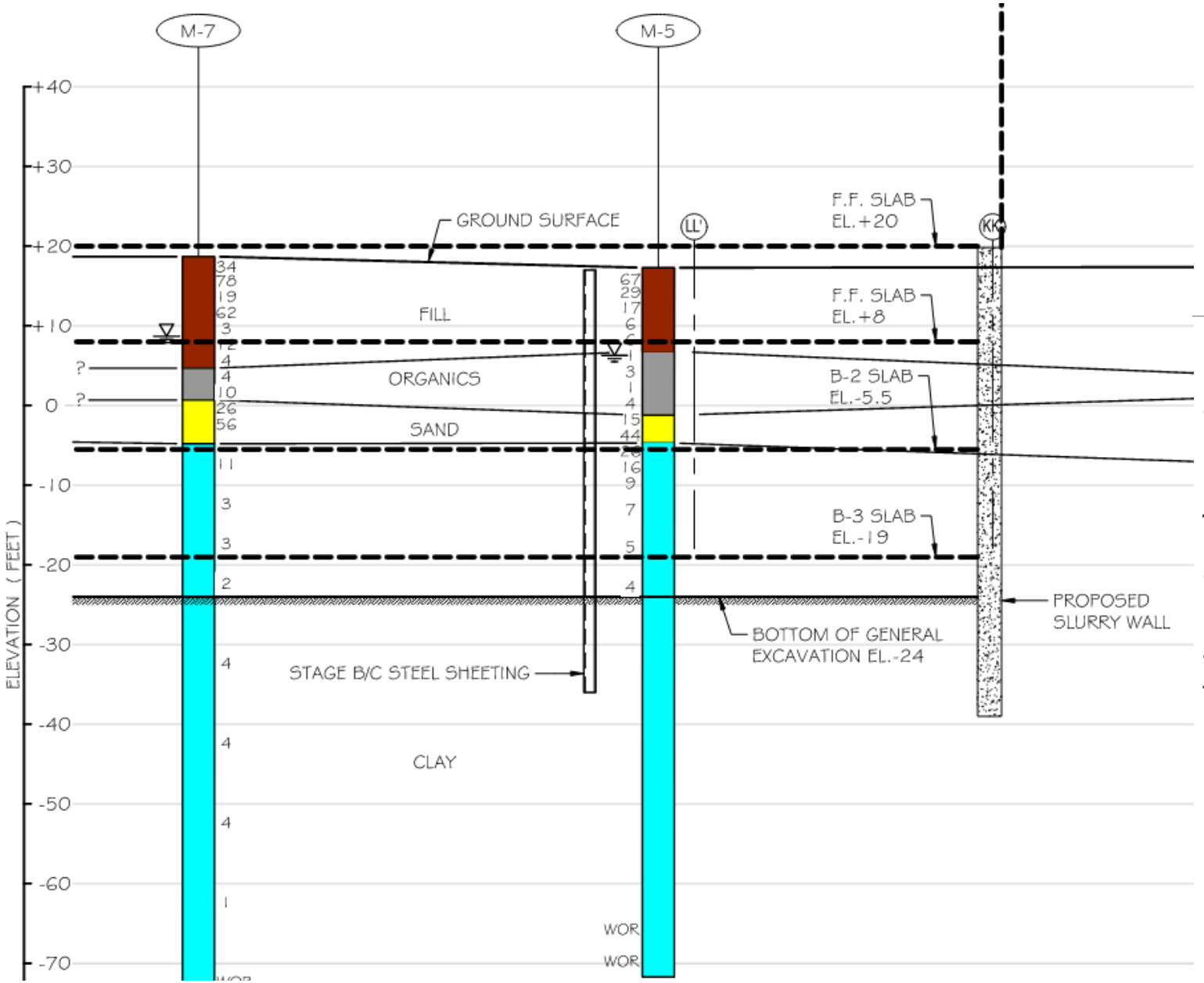
- Founded on a structural mat bearing on marine sand at a depth of approximately 14 feet (4.3 m).
- Single below-grade level.

17-story building (2019):

- Founded on micropiles bearing in bedrock extending to depths of approximately 240 to 250 feet (73.2 to 76.2 m).
- No below-grade levels.

MBTA Green Line Light Rail

- Founded on a mat foundation bearing on marine clay and marine sand at depths of approximately 6.5 to 20 feet (2 to 6.1 m).
- Drops below-grade as the tracks move from south to north.



Project Site Considerations

Deep excavations below overly consolidated portions of the Boston Blue Clay deposit.

Thick miscellaneous urban fill and organic silt deposit.

Sensitive structures adjacent to the excavation bearing above the proposed bottom of excavation.

Support of Excavation Design

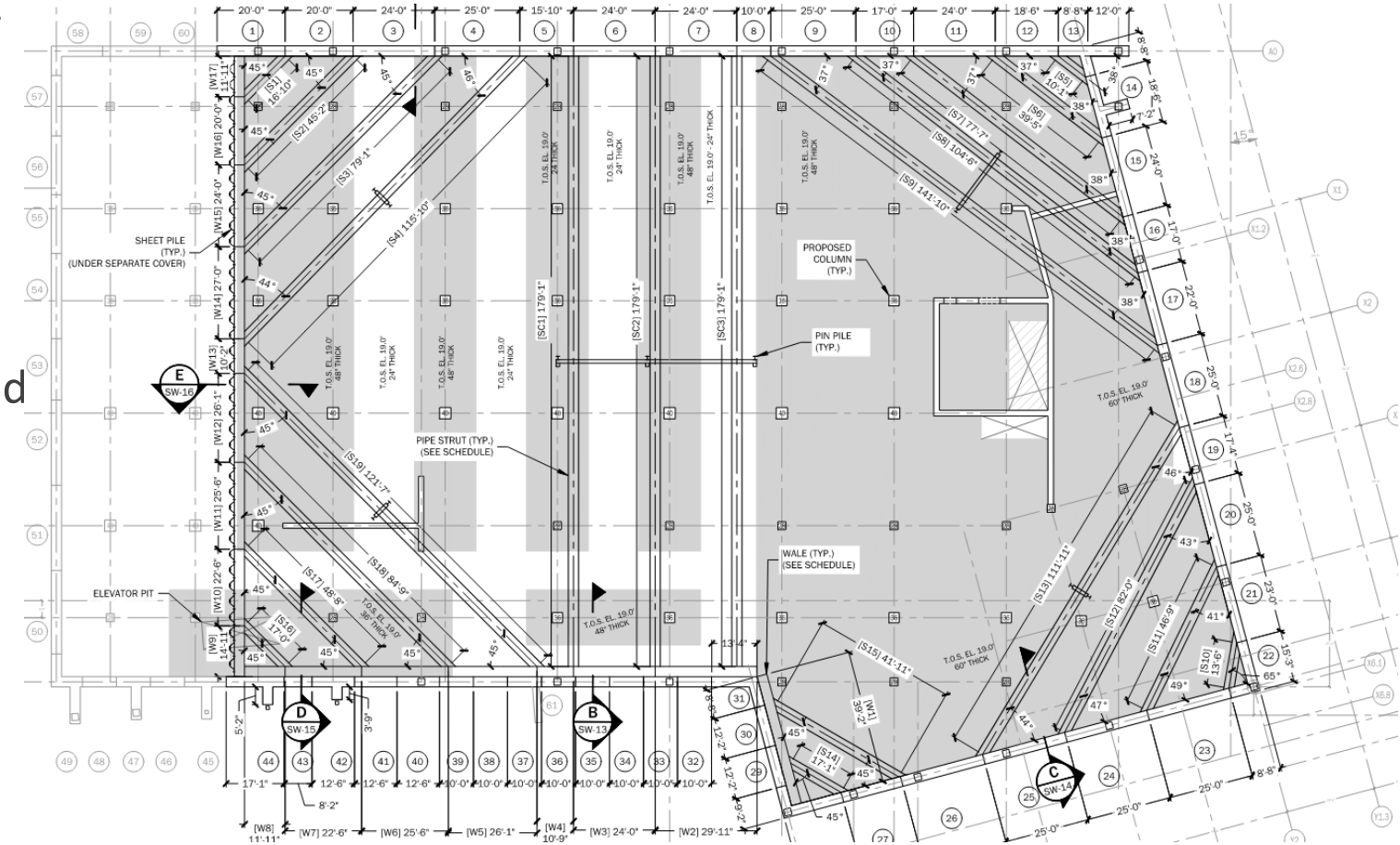
SOE required to construct 3 levels of below-grade parking

Braced slurry wall chosen for SOE

- Stiff system to limit wall deflections
- Groundwater cutoff

Excavation used “bottom up” method

Approx. excavation size: 350’ x 200’



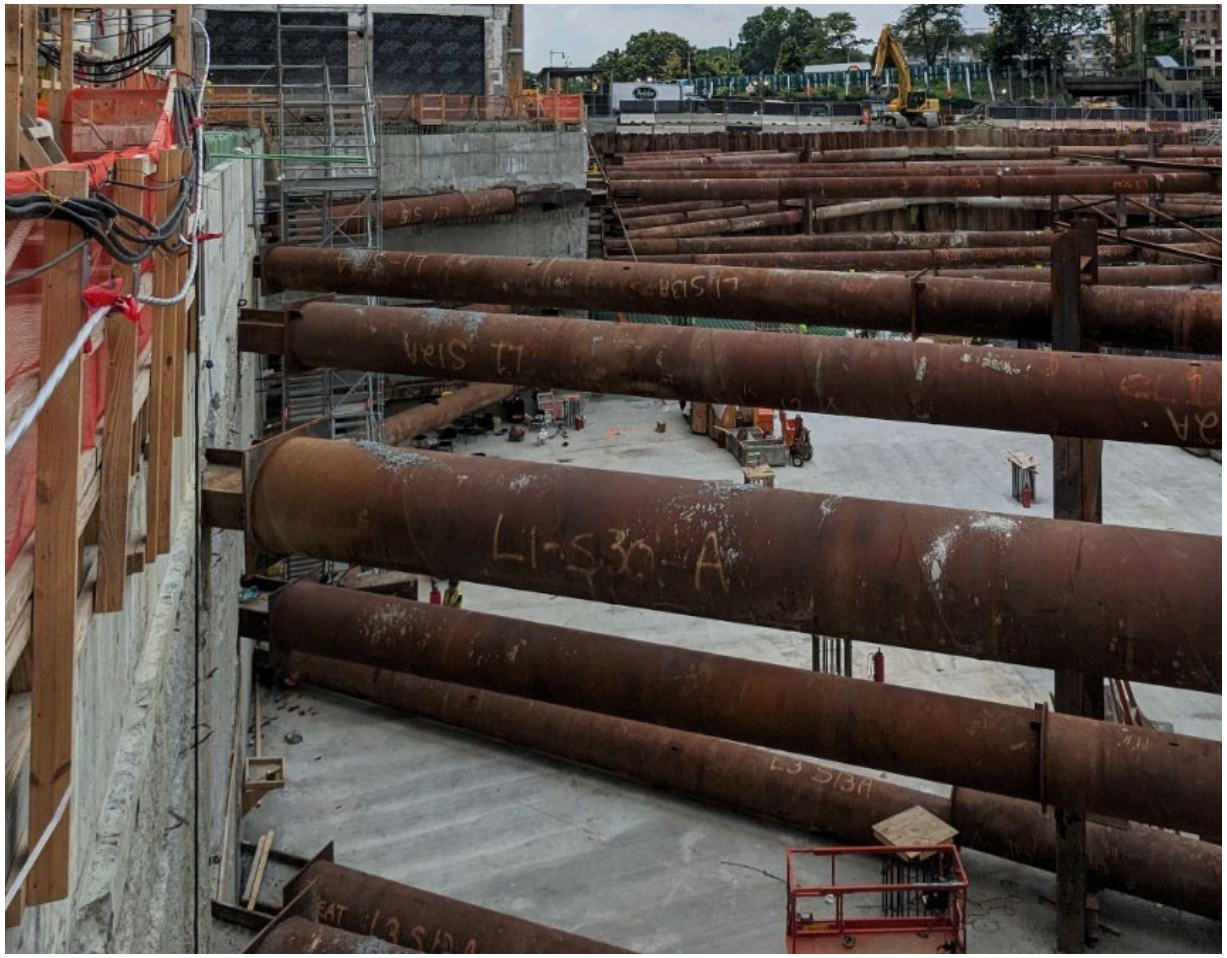
Support of Excavation Design

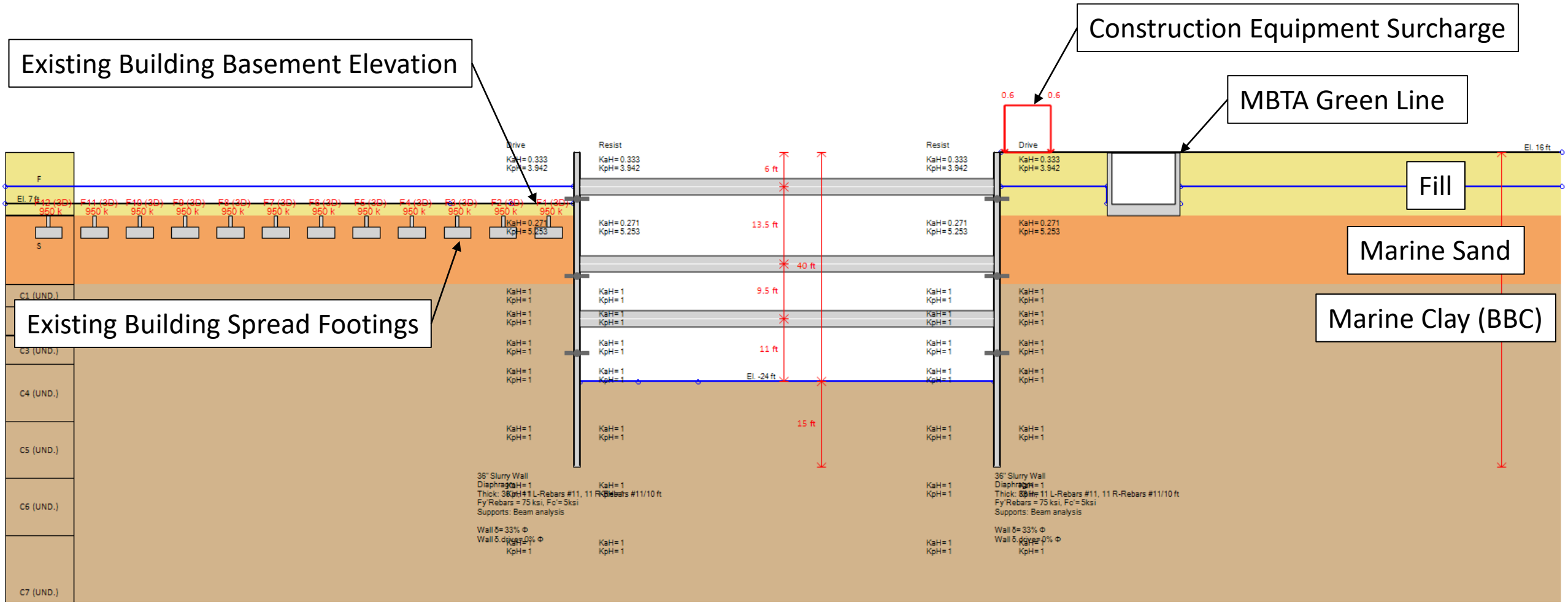
Lateral Earth Analyses performed using DeepEX

- 2D analysis
- Apparent earth pressures of 24H/12H
- 4 design sections to model adjacent surcharges
- Finite Element Analysis used to predict wall deflections

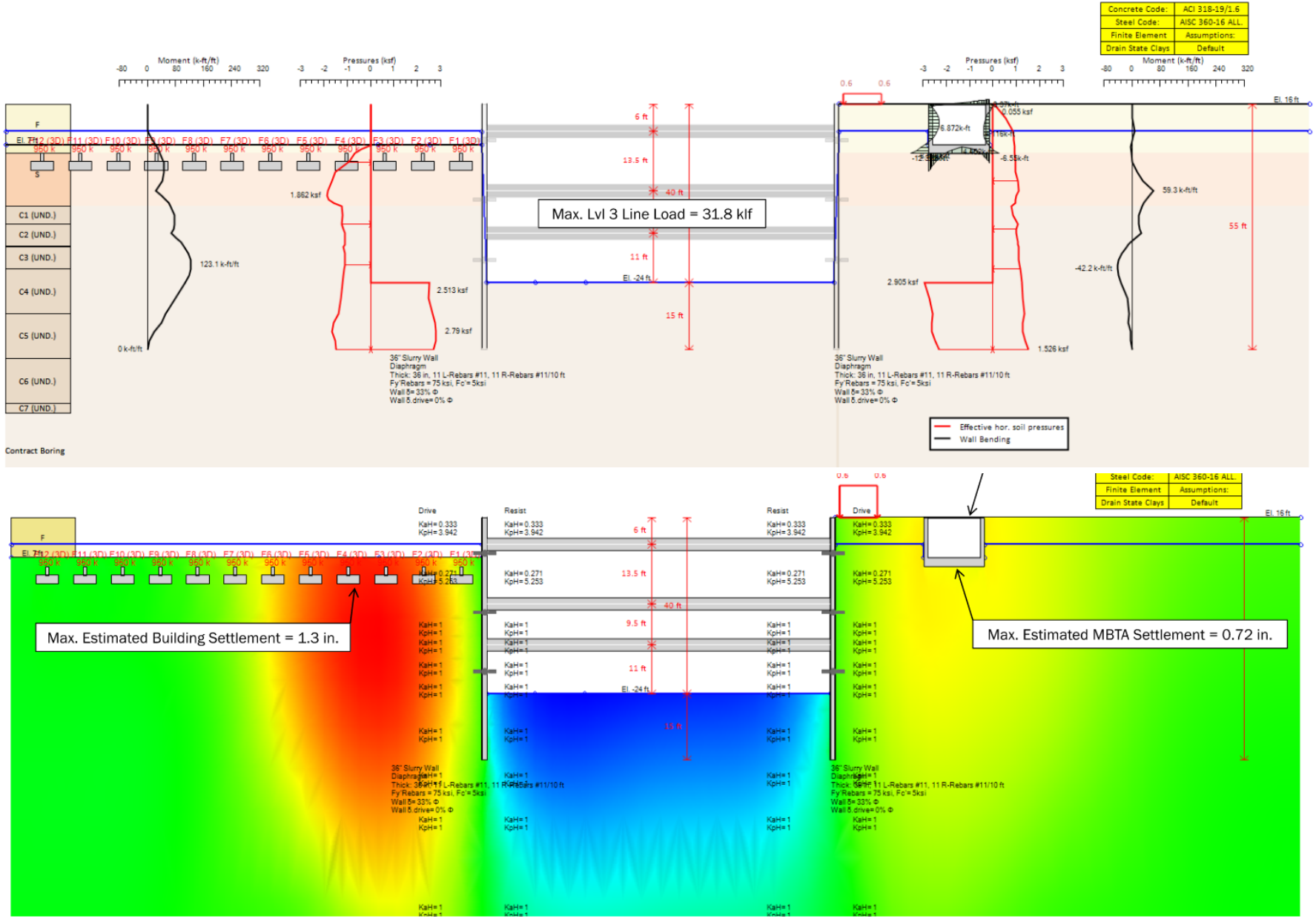
Structural calculations

- Bracing sizes/geometry and slurry wall reinforcement designed based on lateral earth analysis





DeepEX Lateral Earth Analysis



DeepEX Lateral Earth Analysis – Design Loads; Estimated Vertical Displacements

Subgrade Struts

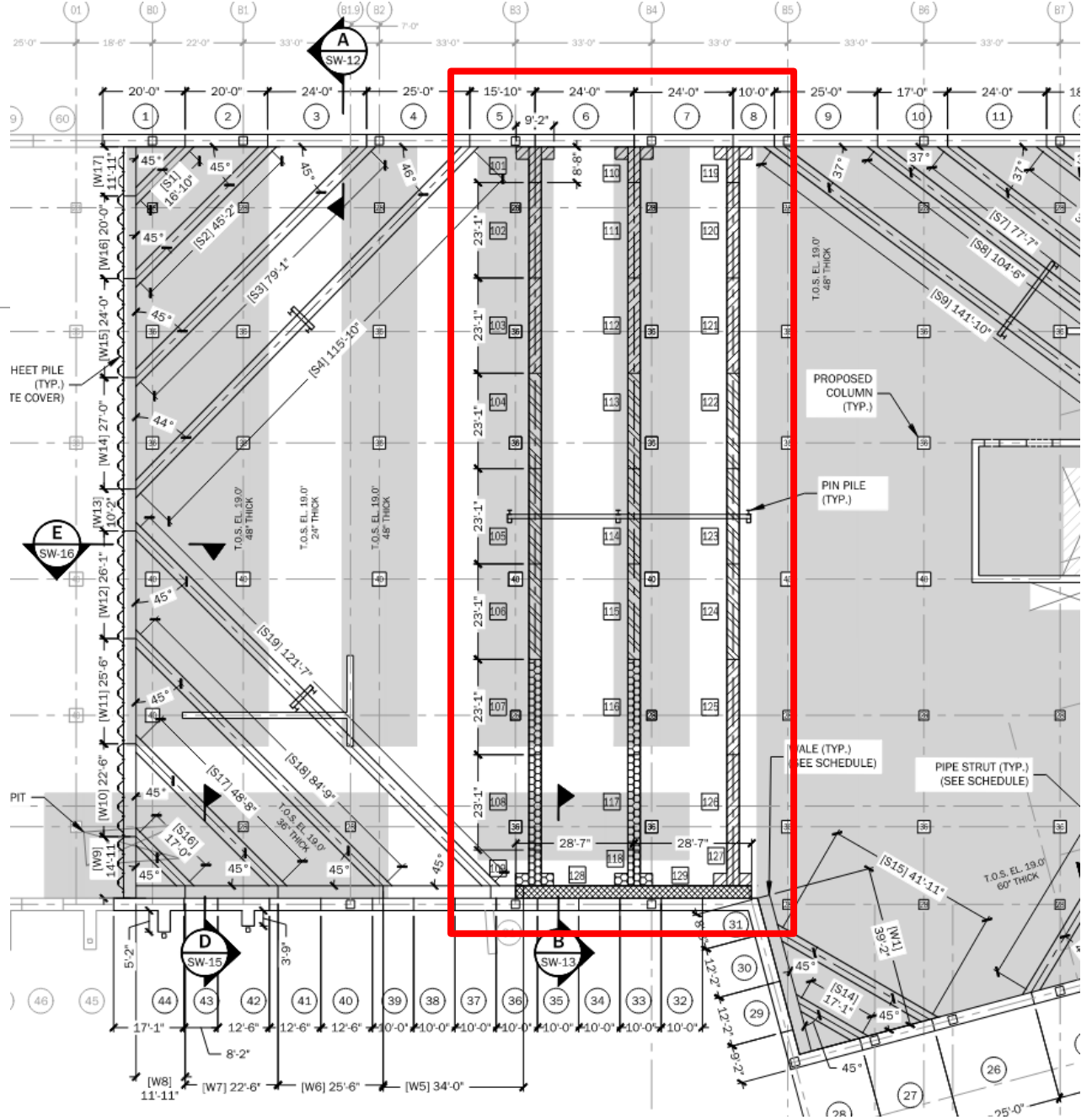
Additional below-grade supports to mitigate effects of soft clay

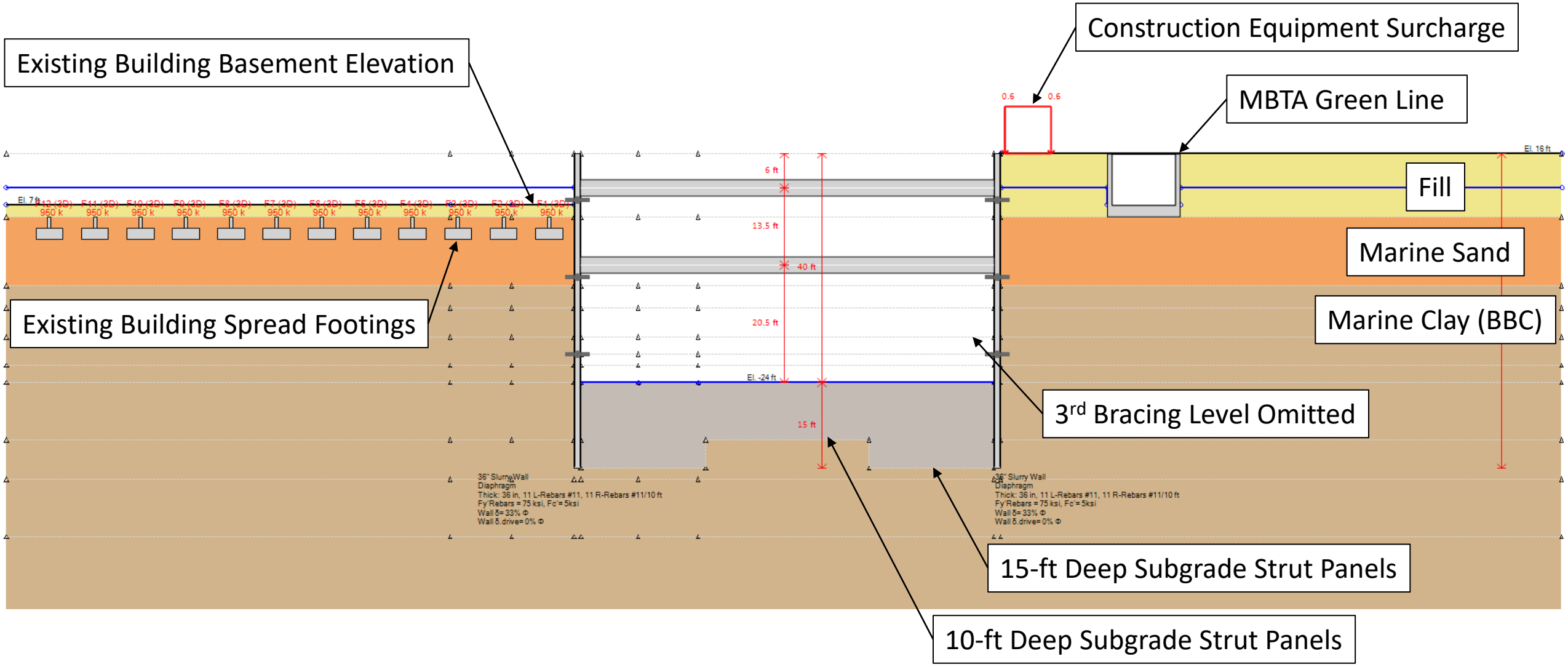
- Prevent “kick-in” wall toe movements

Cost-efficient solution of three subgrade struts across excavation

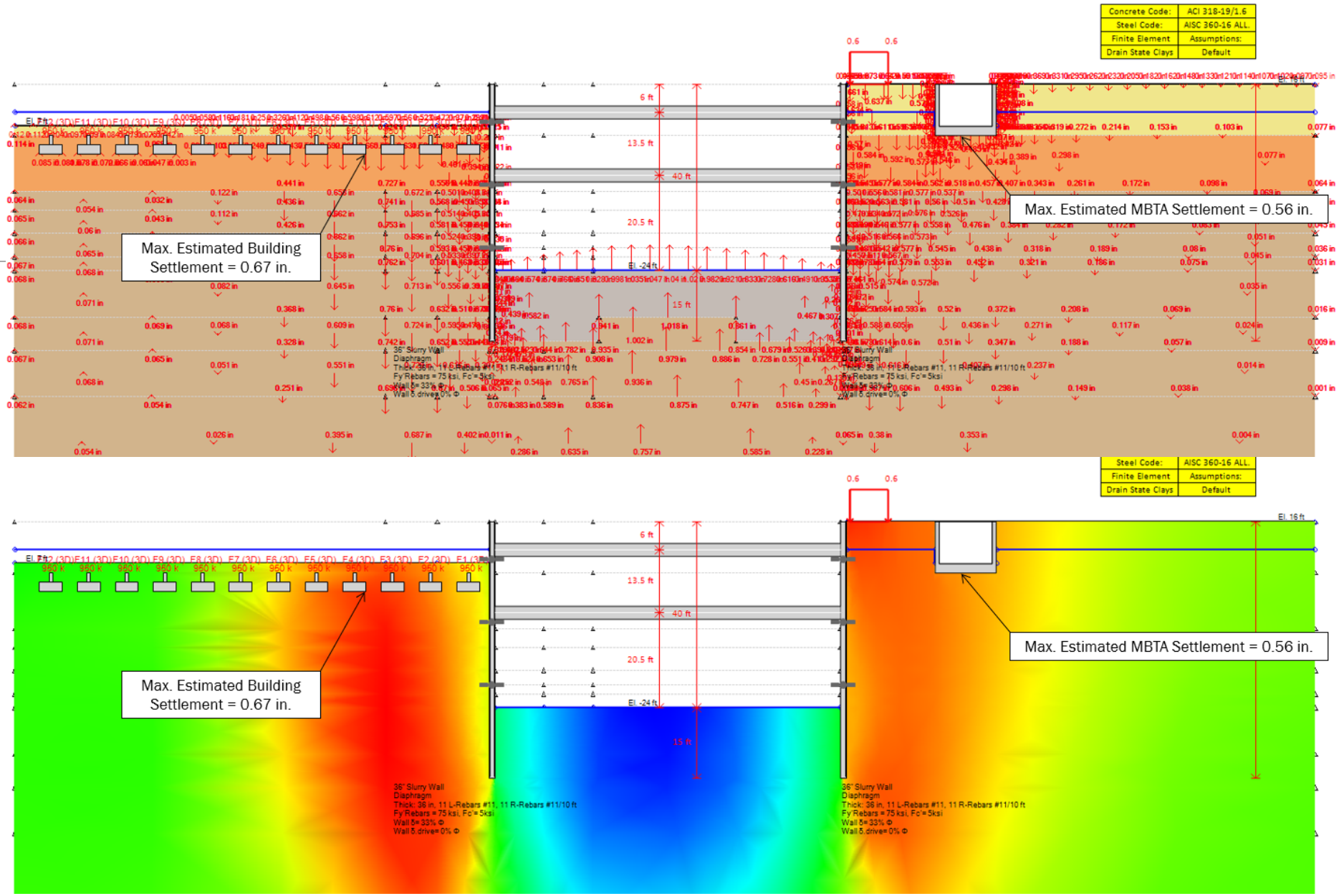
- Protected abutting building and MBTA line

Installed using slurry wall techniques





DeepEX Lateral Earth Analysis – Subgrade Struts



DeepEX Lateral Earth Analysis Subgrade Struts – Estimated Vertical Displacements



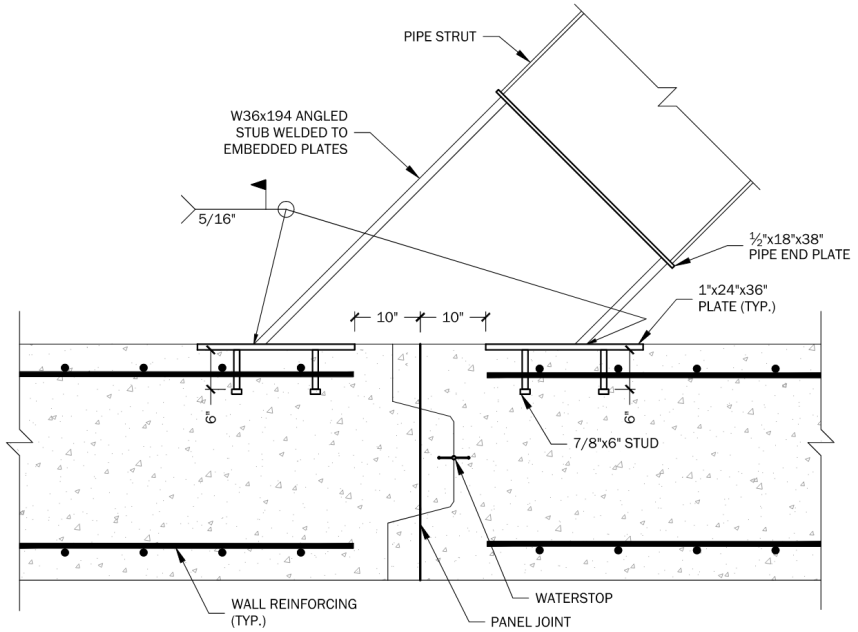
Stitched panorama of full excavation stage

Phase 2 Excavation

Subgrade struts not deemed cost-effective

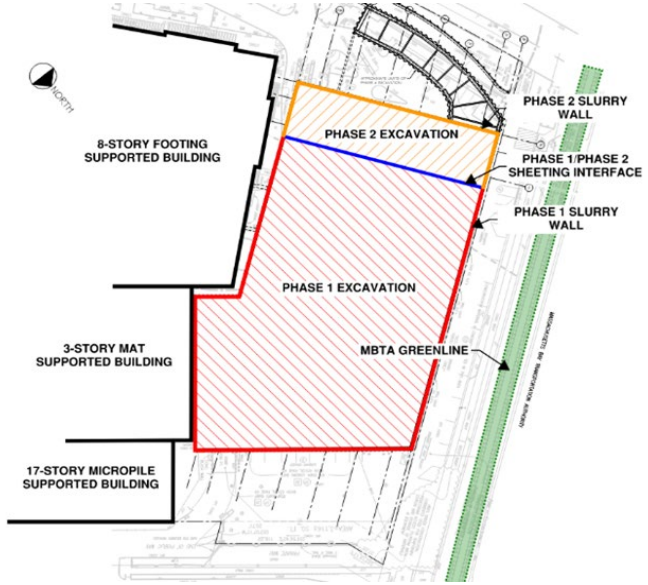
Geometry challenges

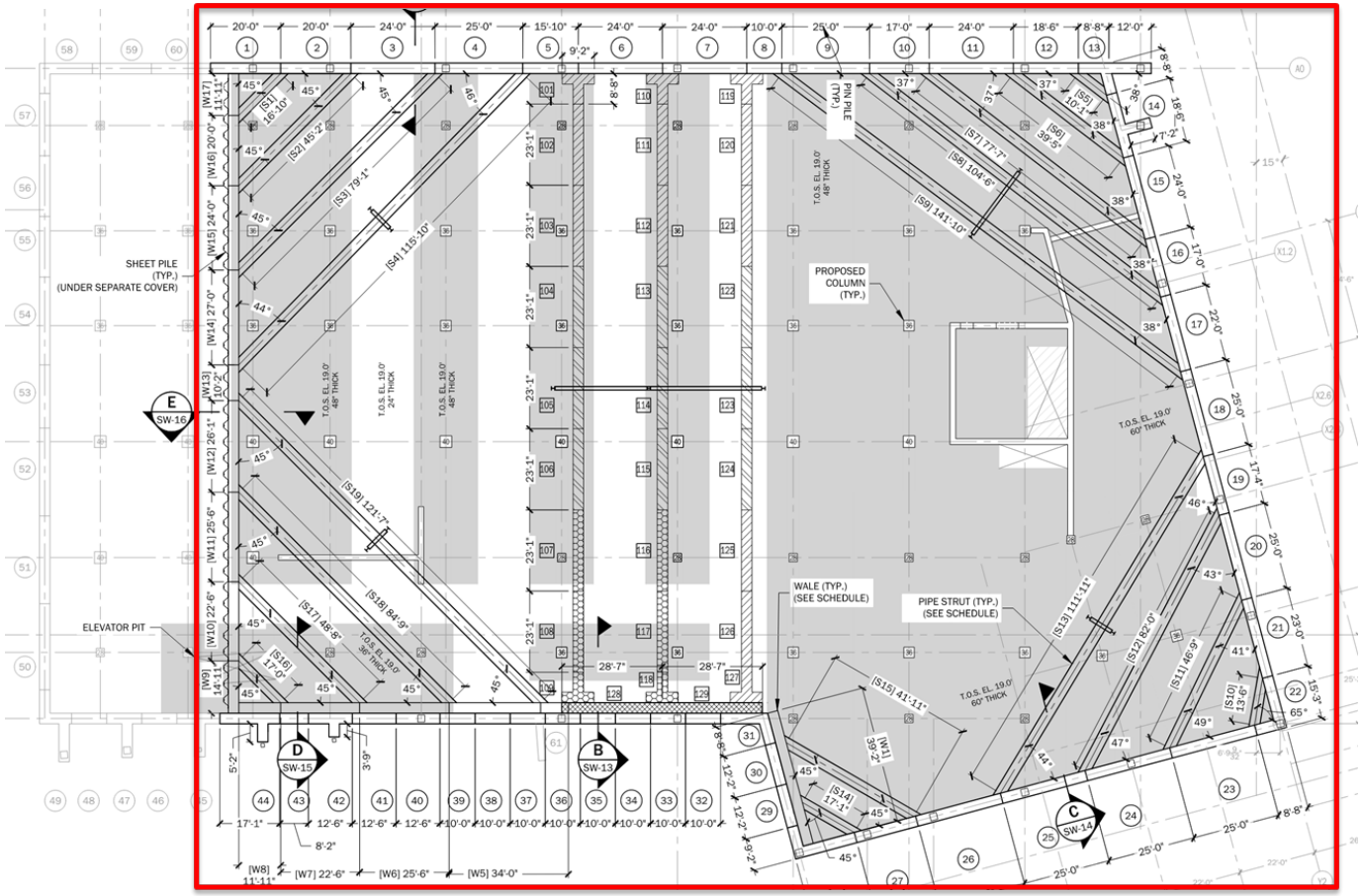
- Struts extended into Phase 1 construction
- Steep bracing angles made connections critical





Slurry Wall Construction

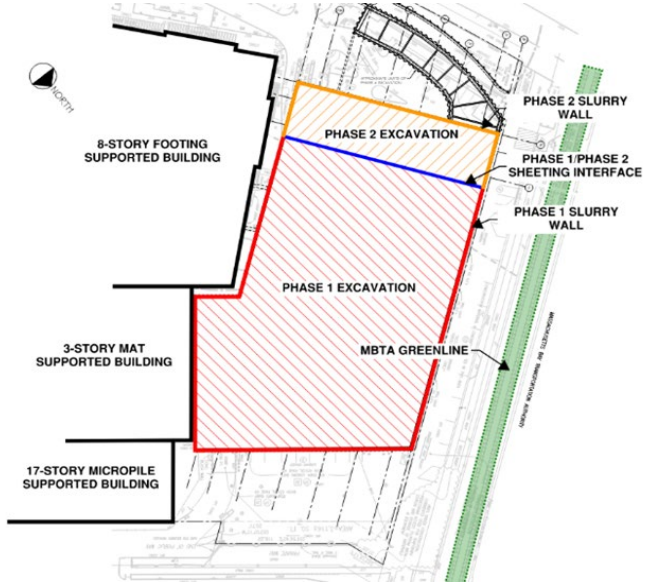


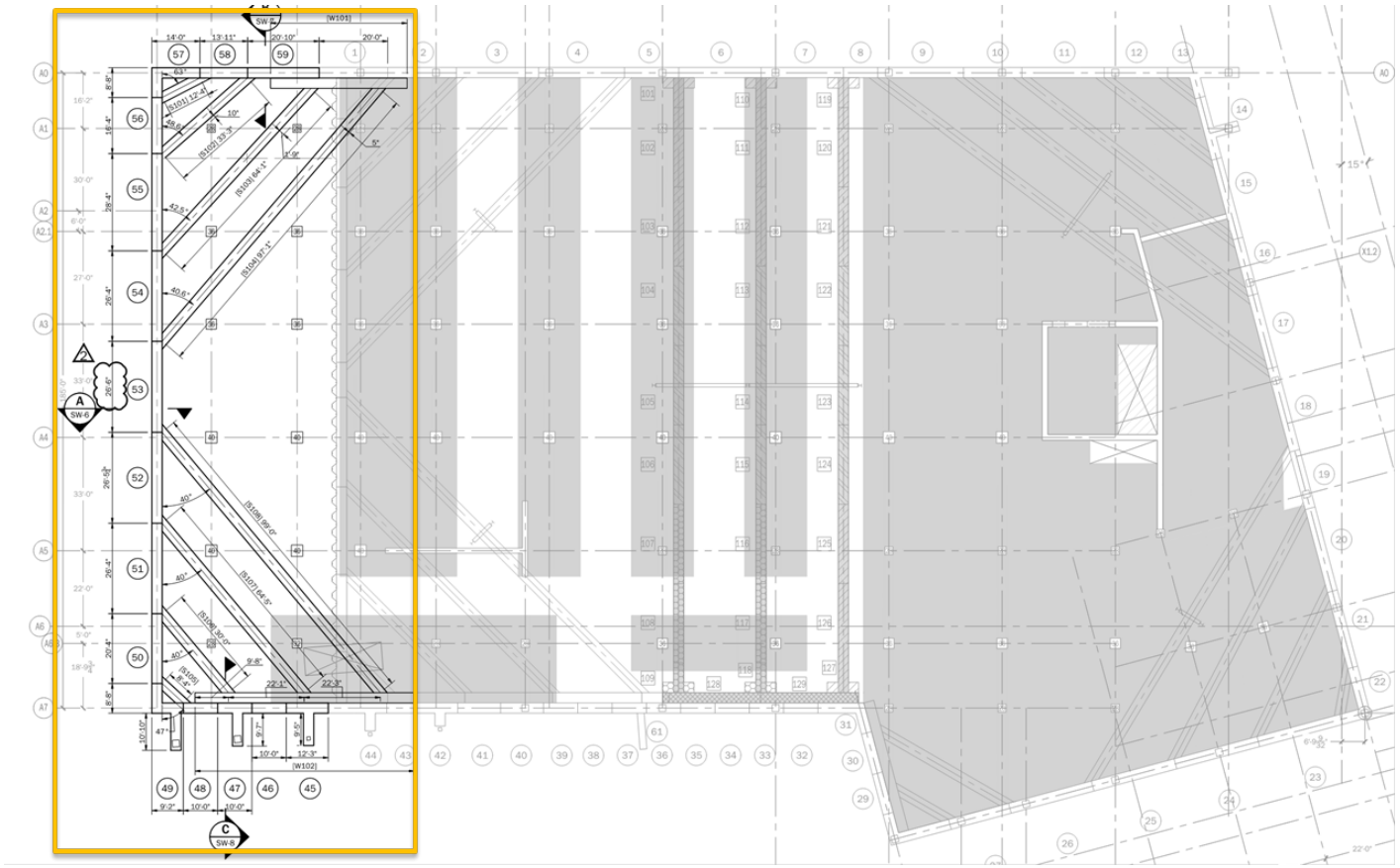


Phase 1 SW stats (2022)

- 3 ft thick slurry wall
- 811 LF / 41,300 SF
- 44 Panels

Slurry Wall Construction





Phase 2 SW stats (2024)

- 3 ft thick slurry wall
- 280 LF / 14,250 SF
- 15 Panels

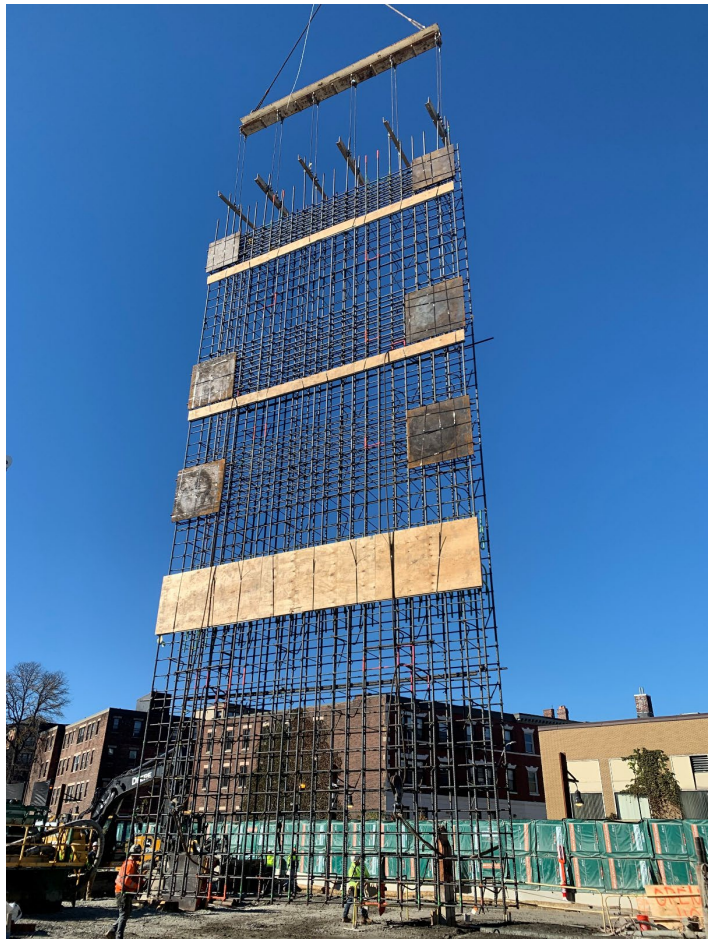
Slurry Wall Construction



- 3 ft thick x 9.25 ft long clamshell
- Max depth: 56 ft
- Bentonite slurry stabilizing fluid

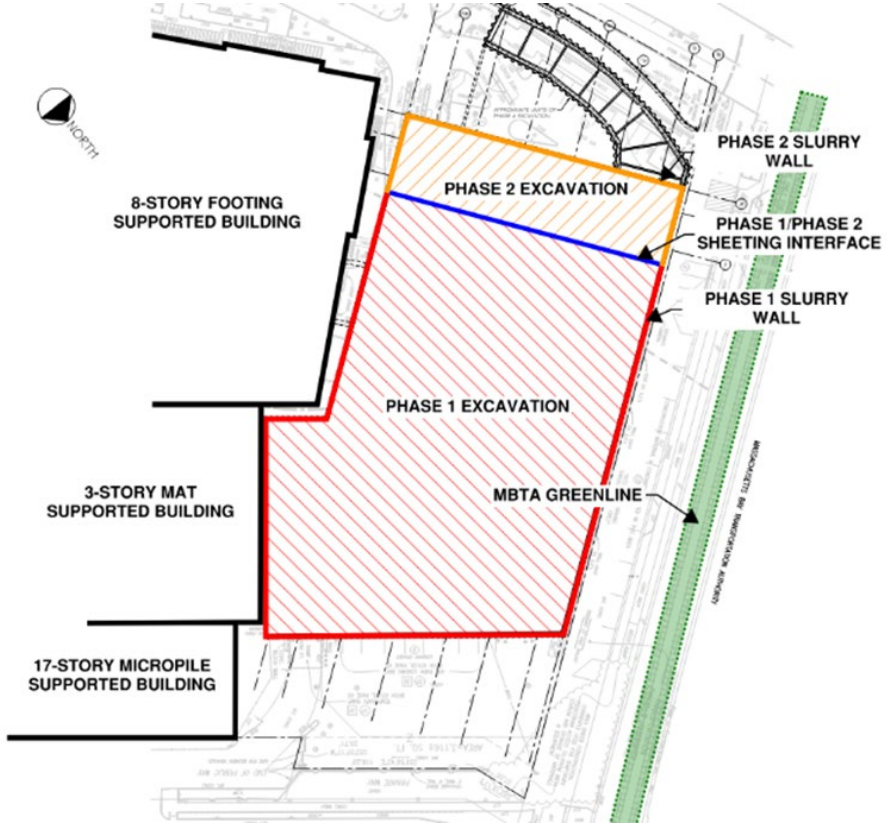


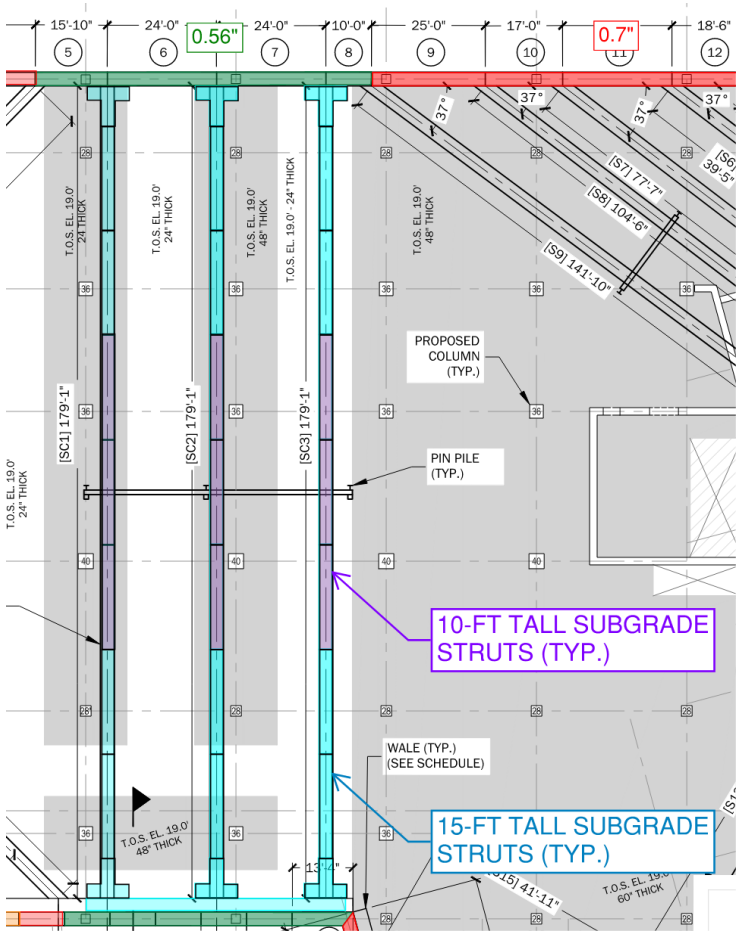
Slurry Wall Construction





Subgrade Struts

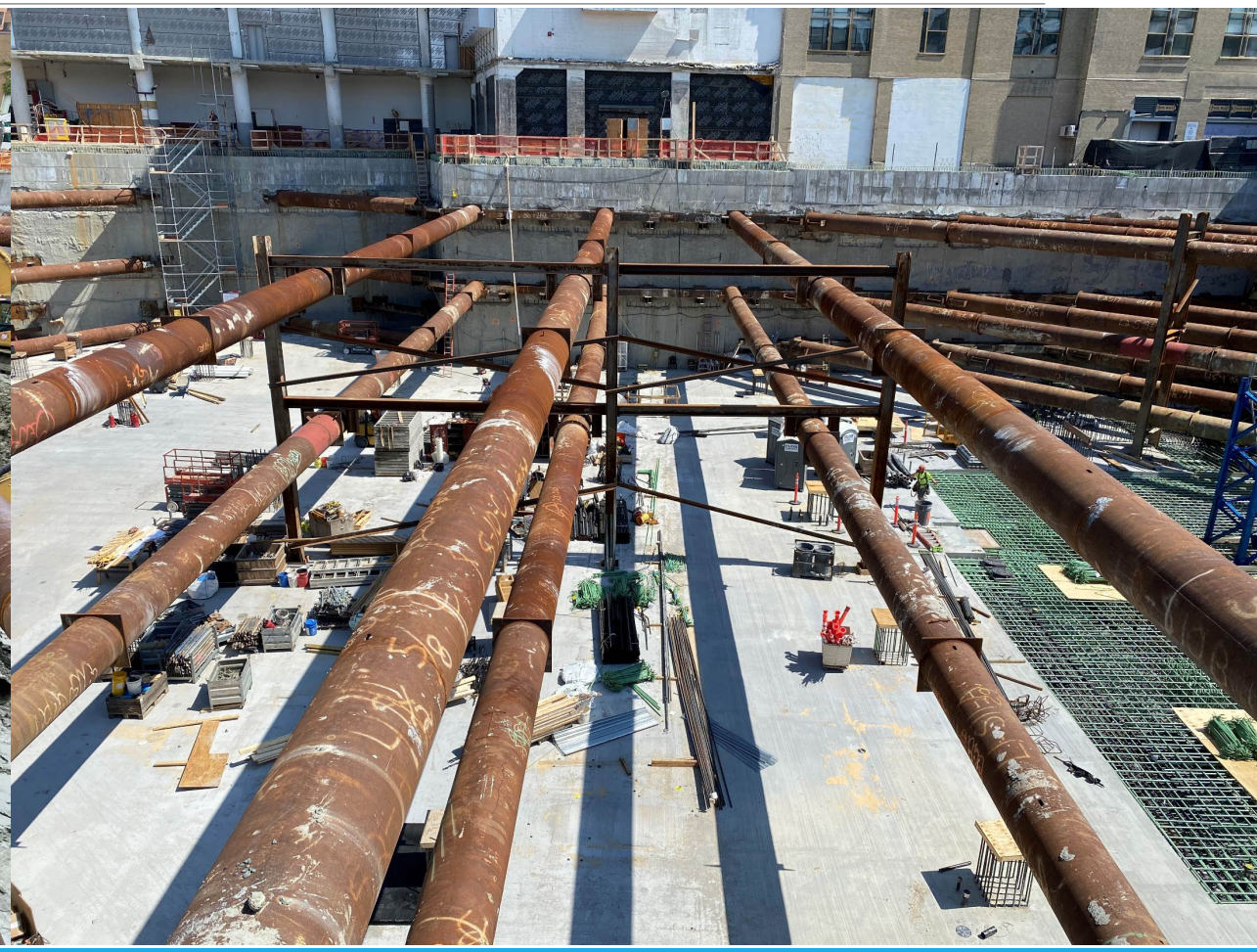




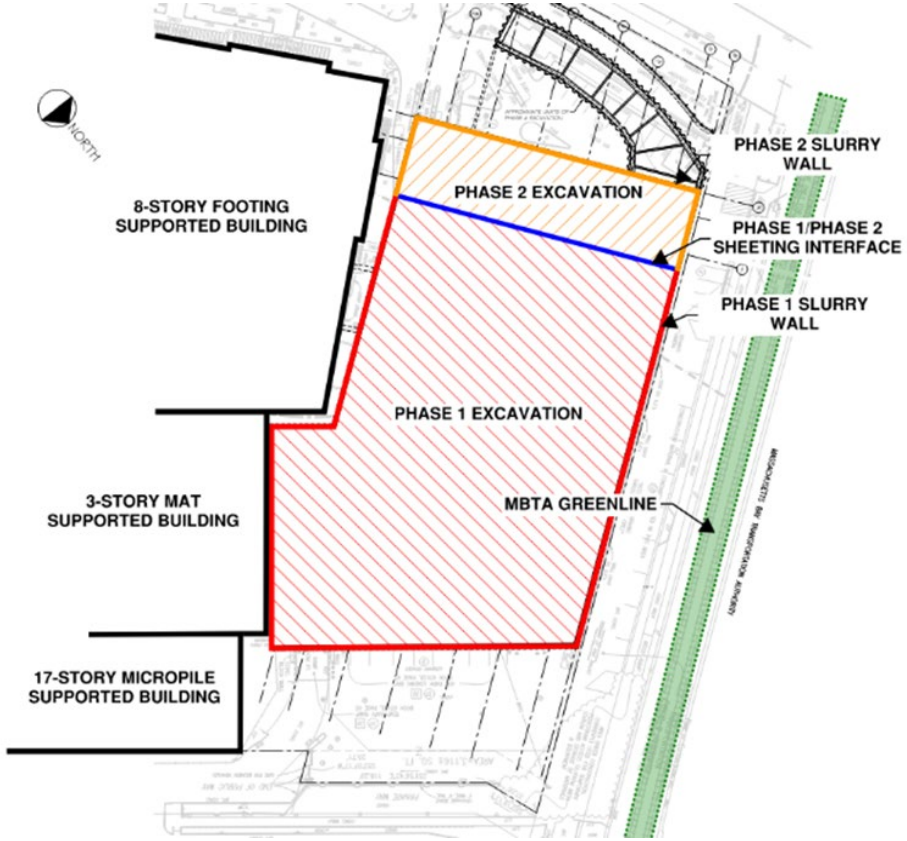
- 2.5 ft thick subgrade struts x 180 ft long
- Excavated with bentonite slurry as stabilizing fluid
- Max depth: 55 ft
- Cement/bentonite above structural concrete

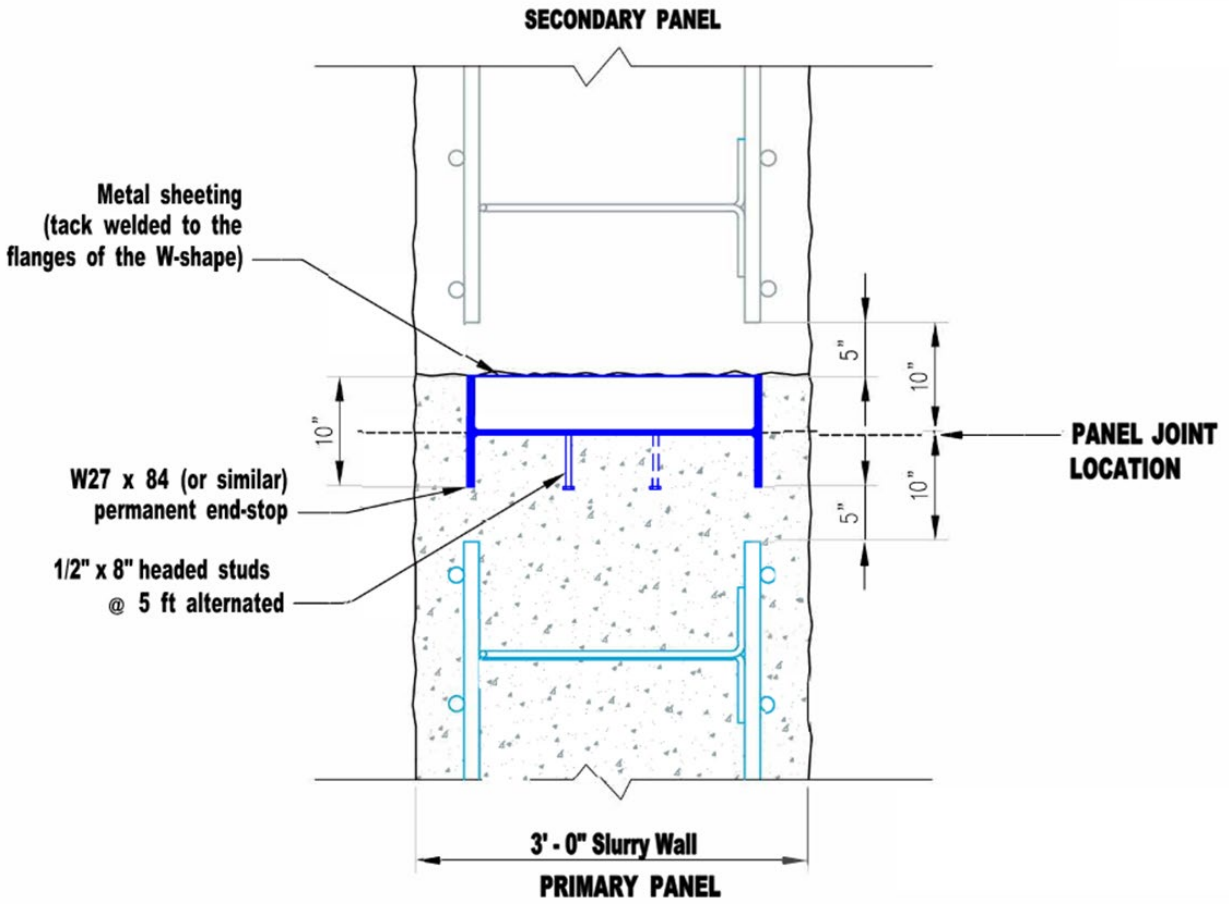
Subgrade Struts





End Stop Connection Between Phases



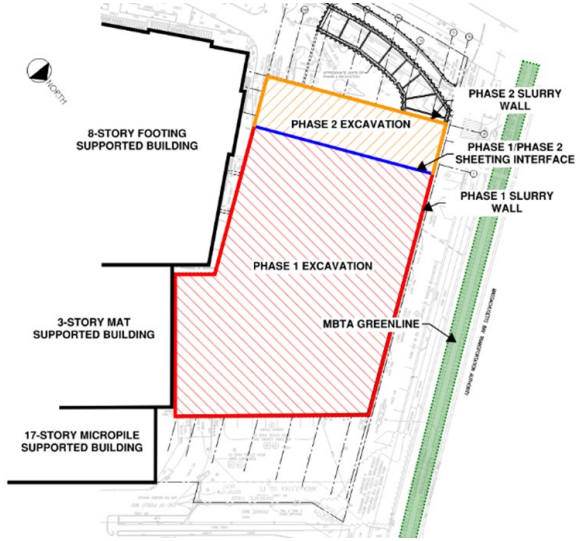


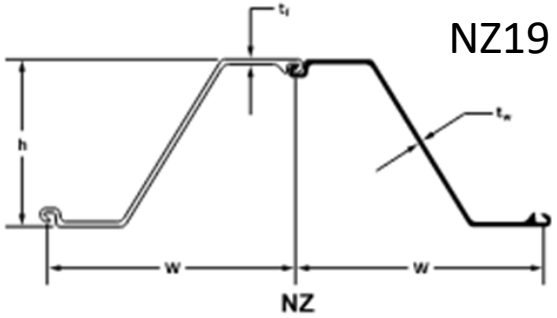
End Stop Connection Between Phases





Intermediate Steel Sheet Pile Wall

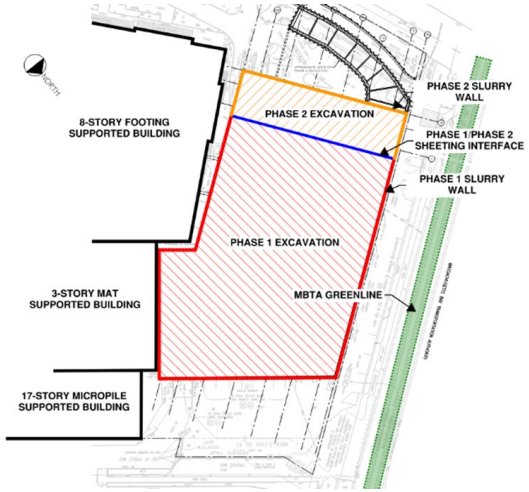


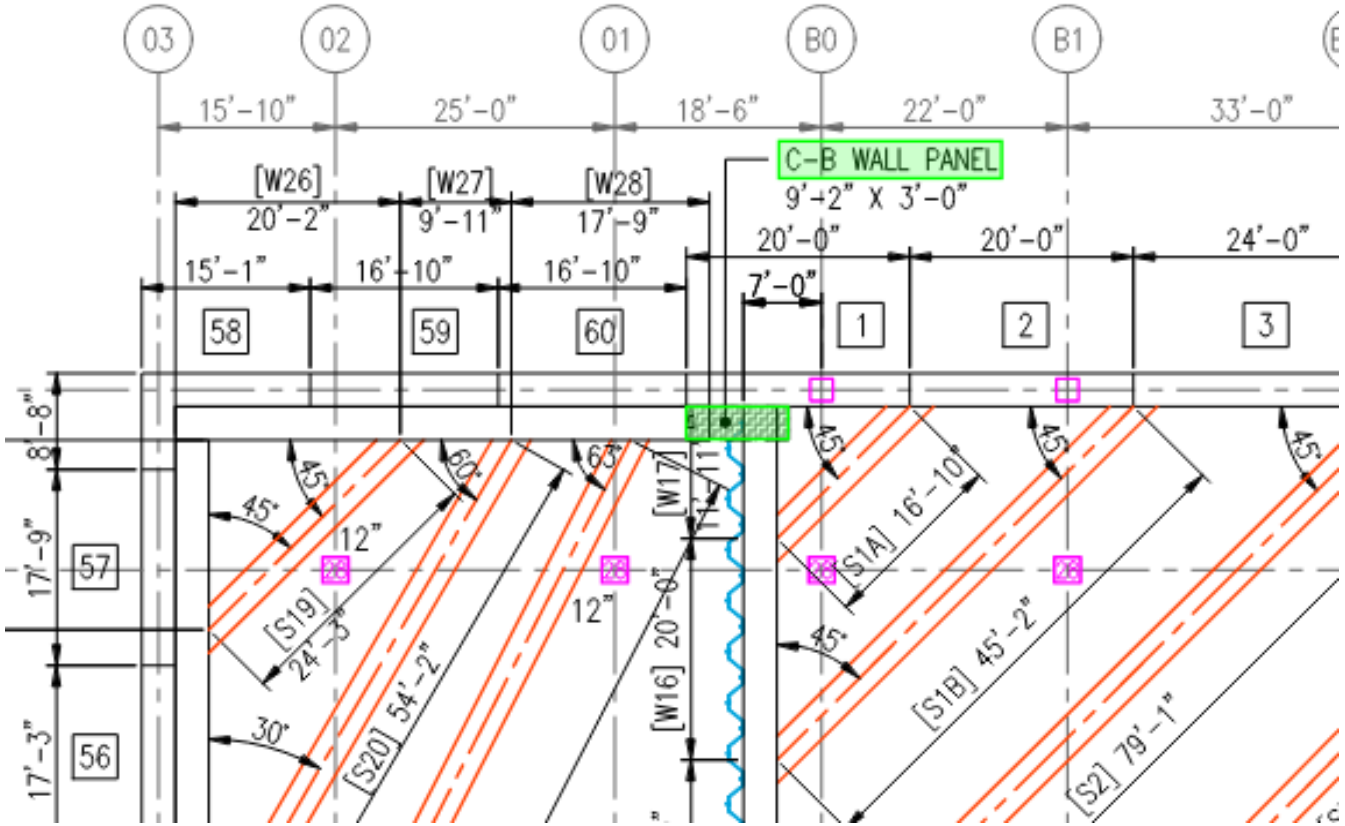




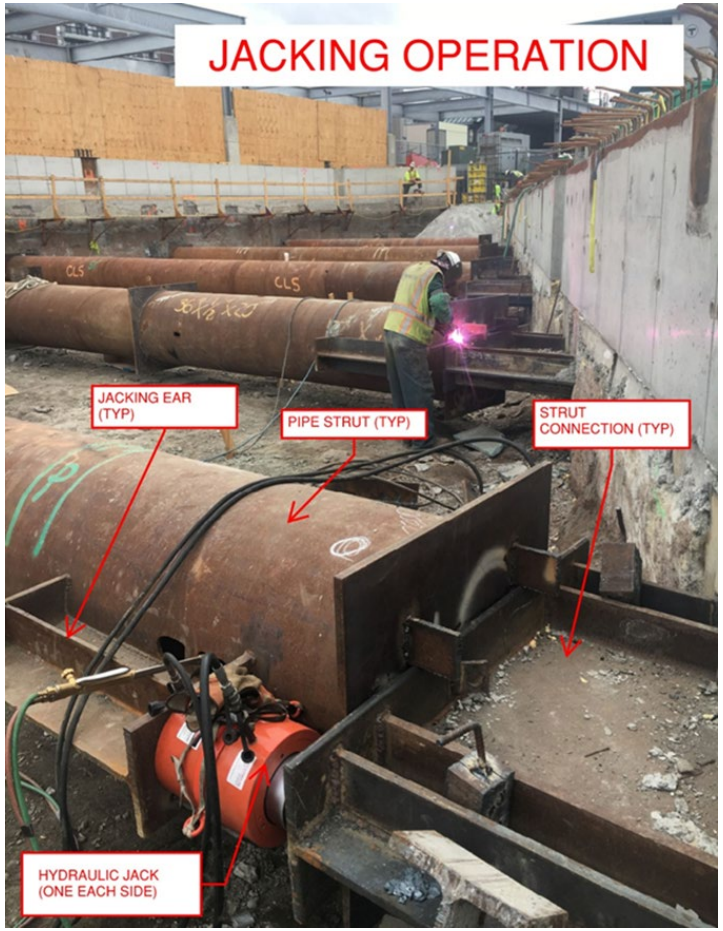
Intermediate Steel Sheet Pile Wall

- Single cement-bentonite (C-B) panel element installed at interface of slurry wall and sheet pile wall to control water and/or soil material ingress



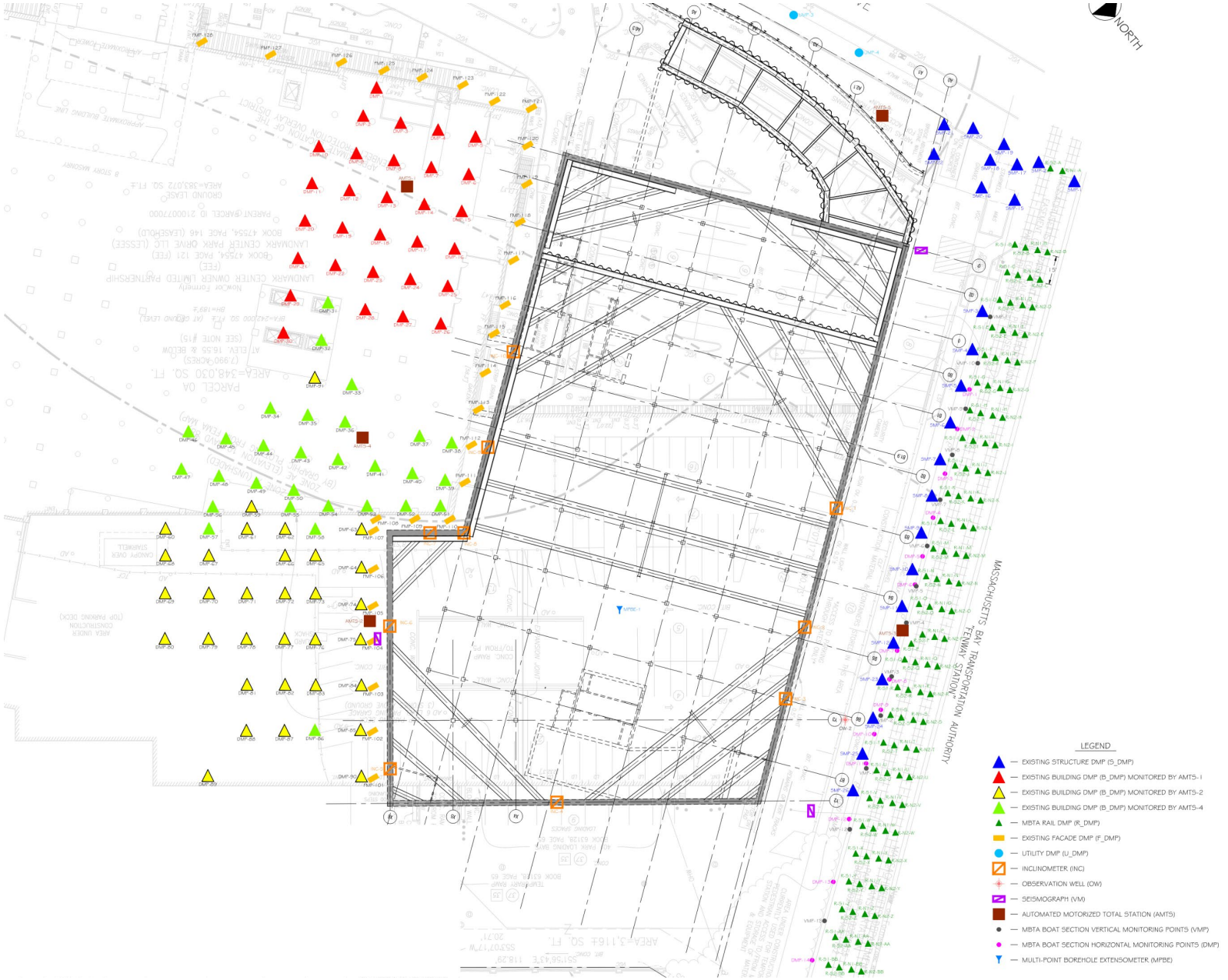


Bracing Install





LEVEL 2 BRACE SCHEDULE (GRADE 3 STEEL, 45 KSI)					
BRACE	LENGTH	DIAMETER	WALL THICKNESS	PINNED LOCATIONS	PRELOAD (KIPS)
	FT	IN	IN		
SC1	179.25	36	1/2	1	318
SC2	179.25	36	5/8	1	382
SC3	179.25	36	1/2	1	334



Geotechnical Instrumentation

14 inclinometers installed within the slurry wall

90 monitoring points on columns within the existing basements

59 monitoring points on the building facades (3 levels)

110 monitoring points installed on rail tracks

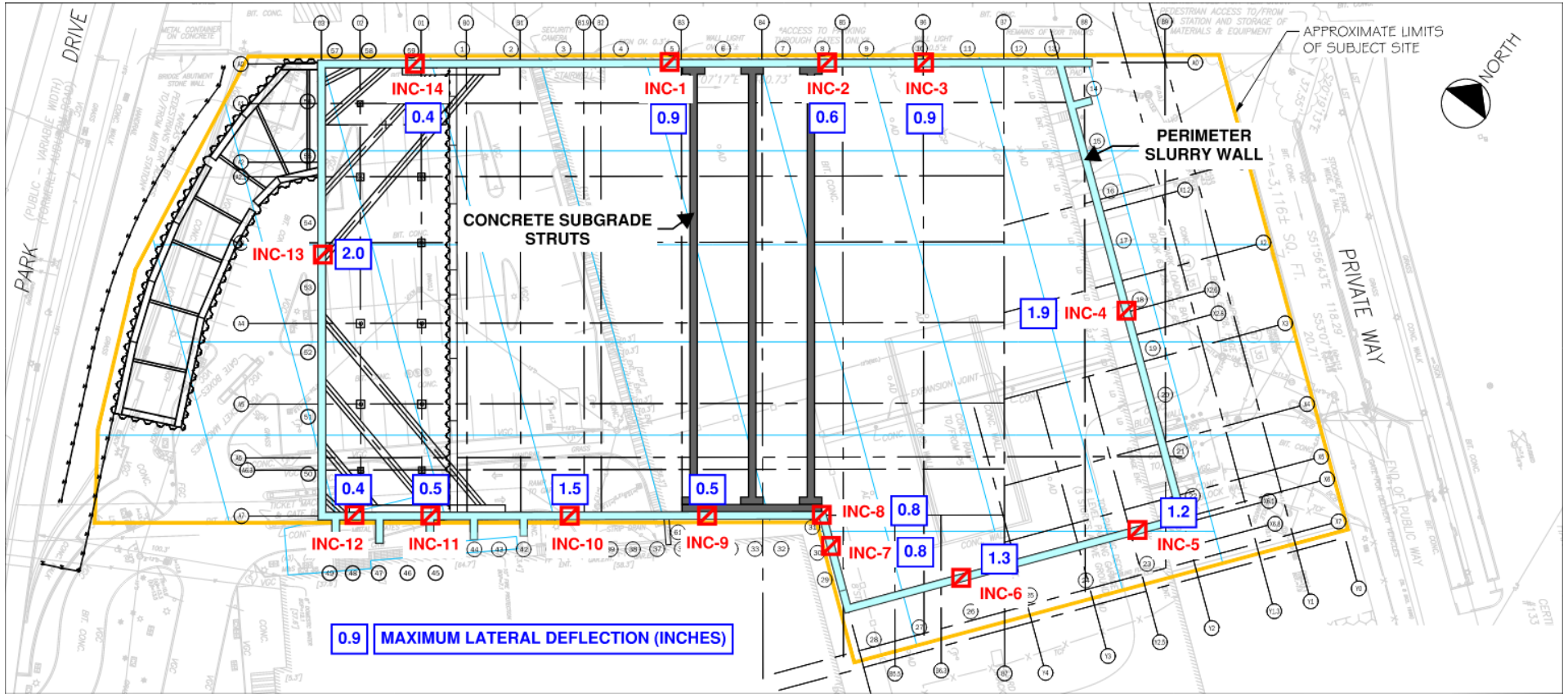
3 groundwater observation wells

30 monitoring points installed on rail infrastructure (platforms, boat section foundations, and stairways)

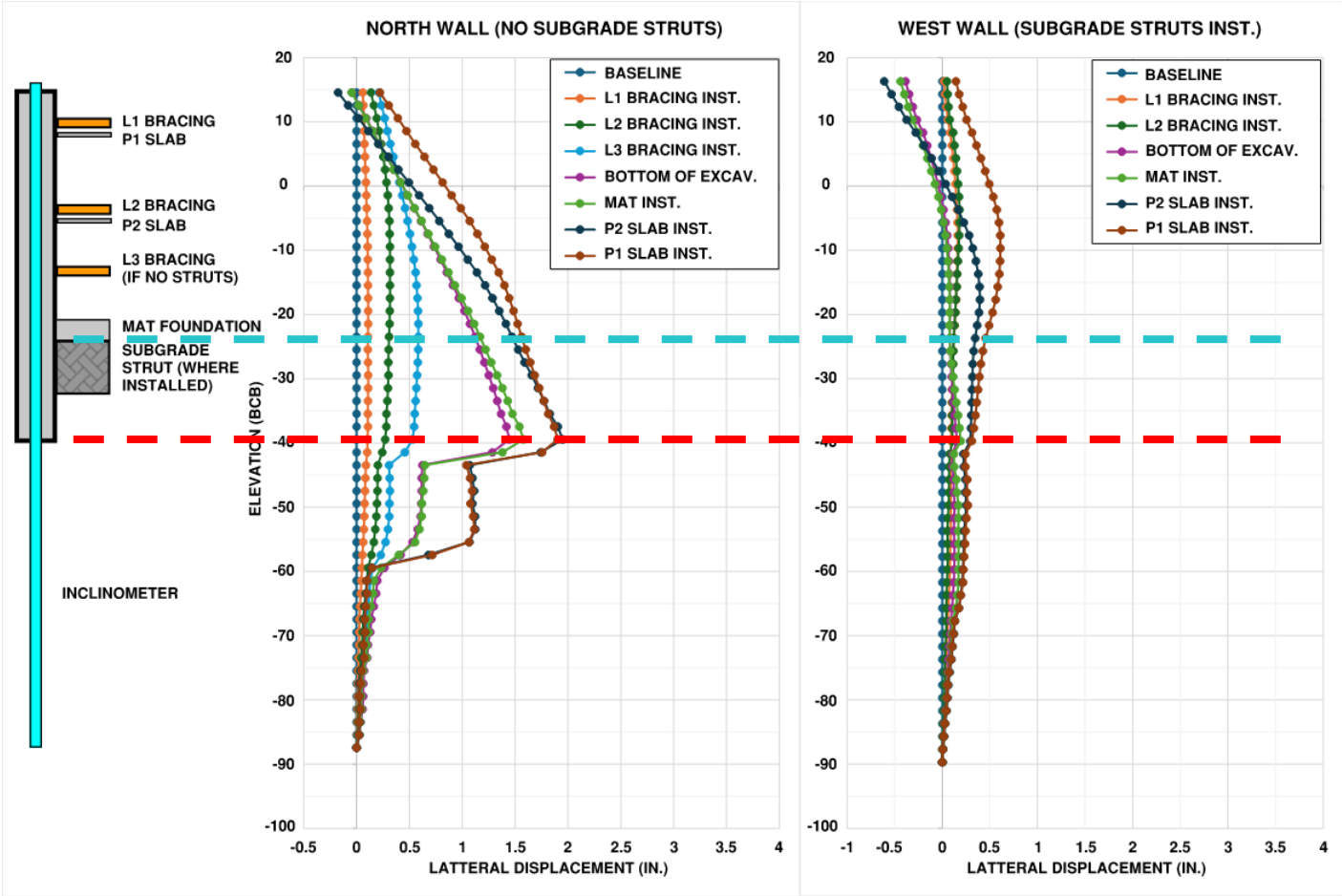
32 surface and utility monitoring points

1 magnet extensometer (heave gauge)

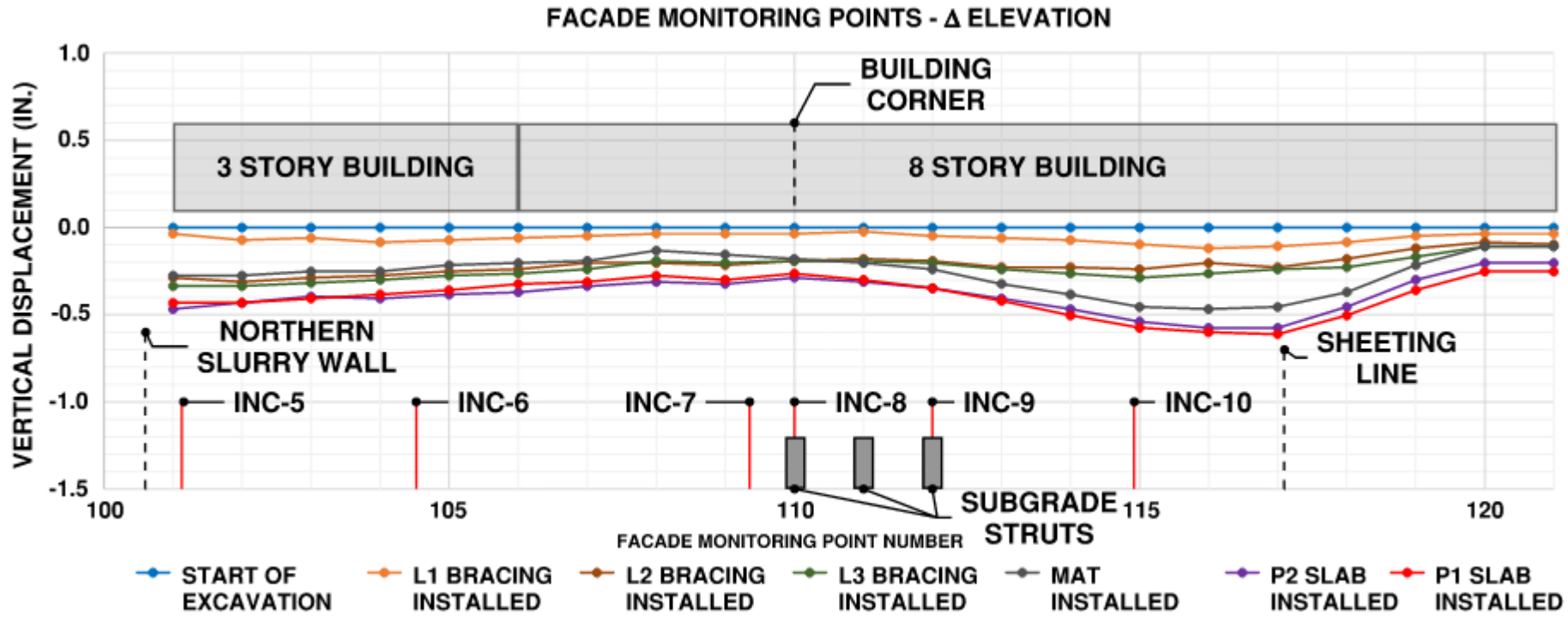
Lateral Performance of Slurry Wall



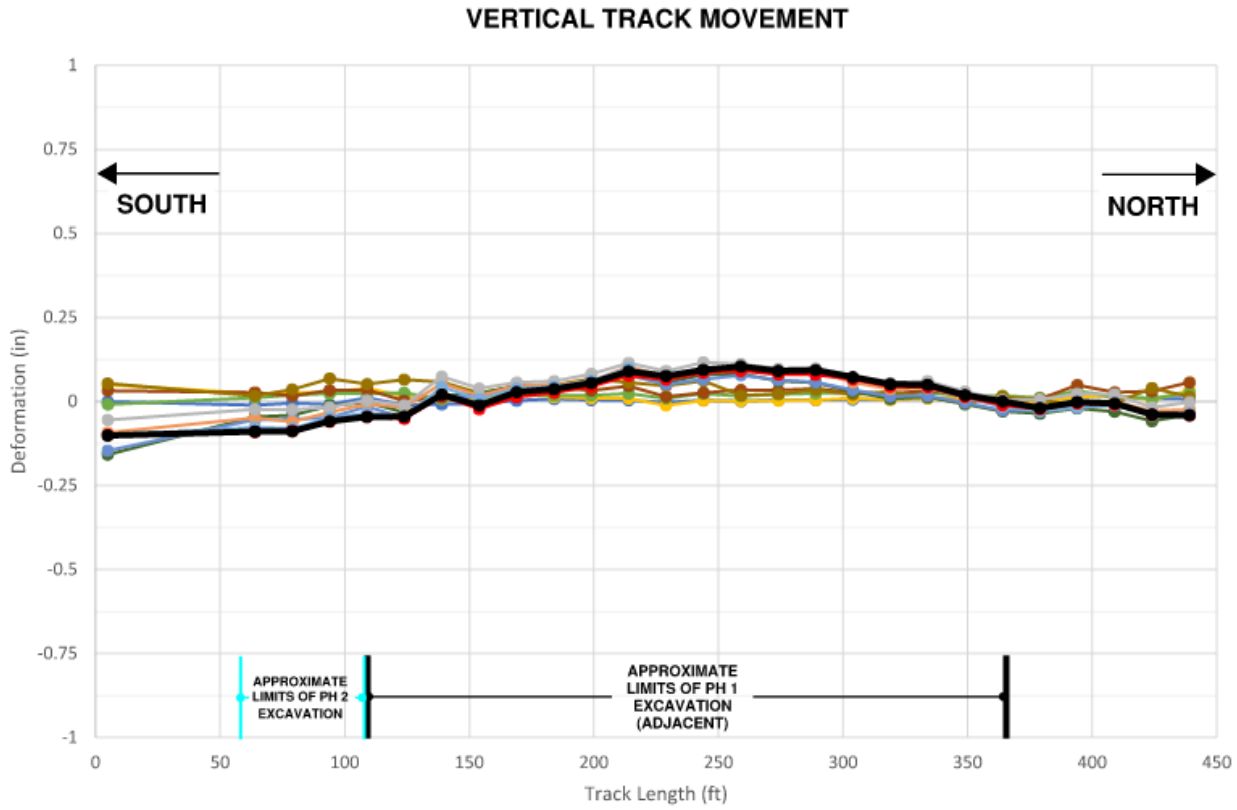
Lateral Performance of Slurry Wall



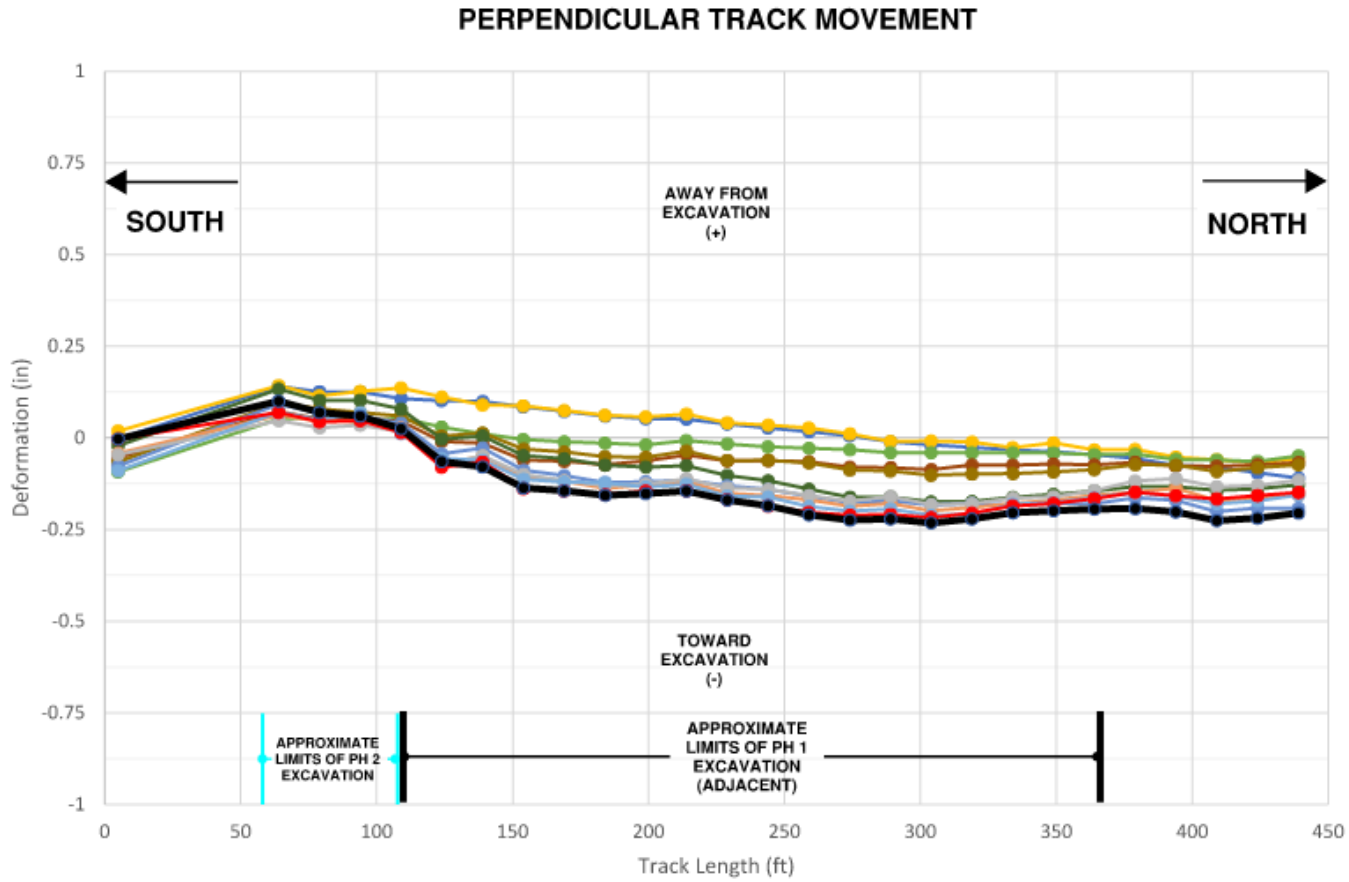
Vertical Movement of Adjacent Buildings



Vertical & Lateral Movement of MBTA Tracks



Vertical & Lateral Movement of MBTA Tracks



Thank You

Project Team

- Developer: Samuels & Associates
- General Contractor: Suffolk Construction Company, Inc.
- Architect: Elkus Manfredi
- Structural Engineer: McNamara Salvia
- Geotechnical Engineer: McPhail Associates, LLC
- Support of Excavation Designer: GeoEngineers USA, P.C.
- Support of Excavation Installer: TREVIICOS
- Earthwork Contractor: The Welch Corp.



Highway 417 Bridge Replacements

Reuse of Existing Piles Study

Tony Sangiuliano

October 22, 2025

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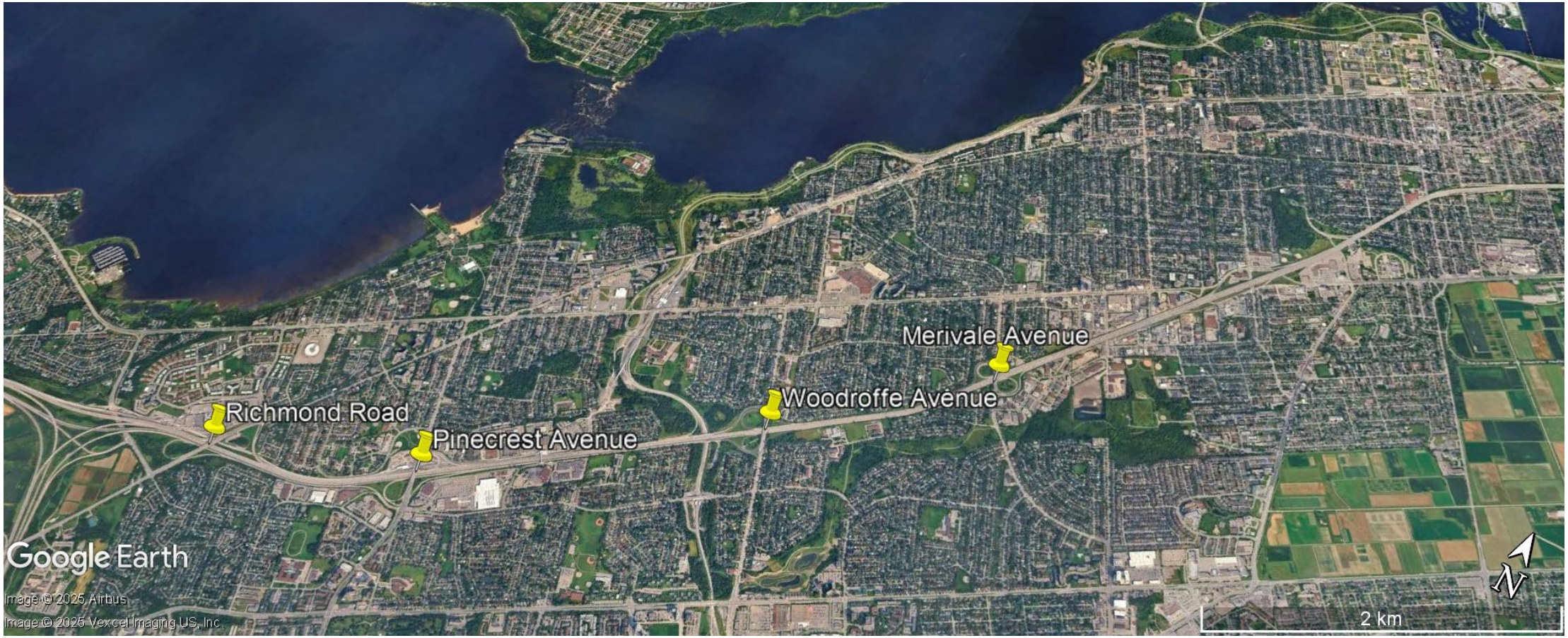
Rapid Bridge Replacements along Ontario Highway 417

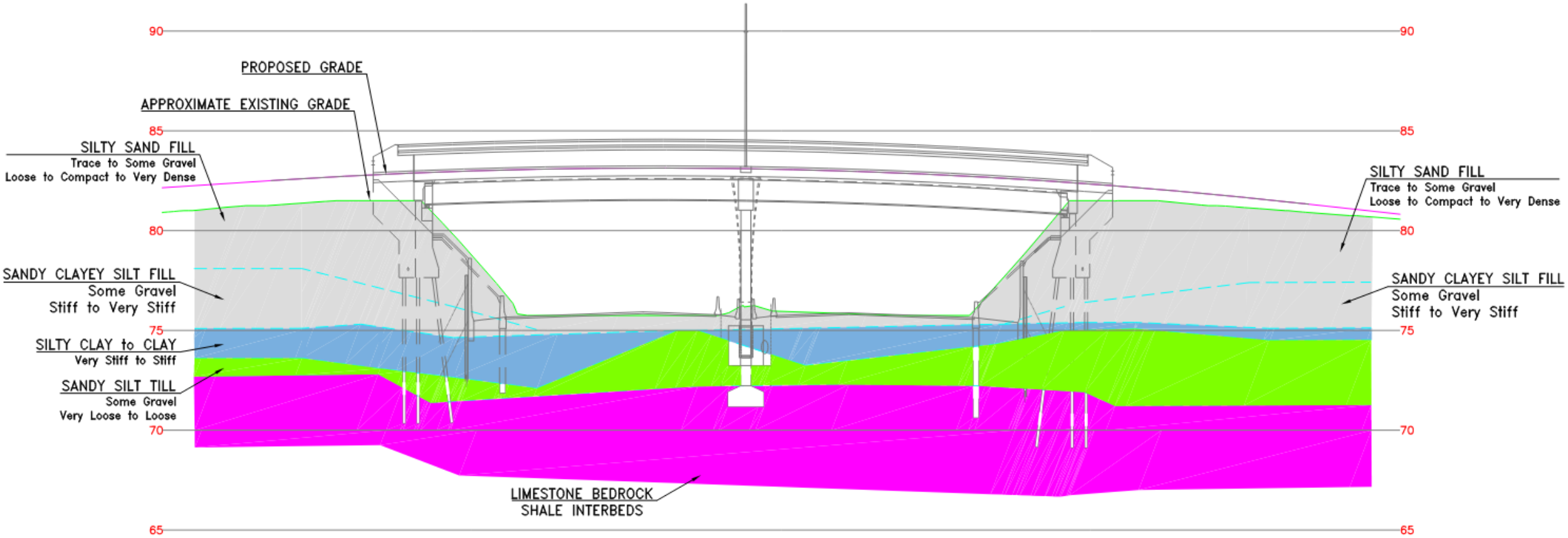


Rapid Bridge Replacements along Ontario Highway 417

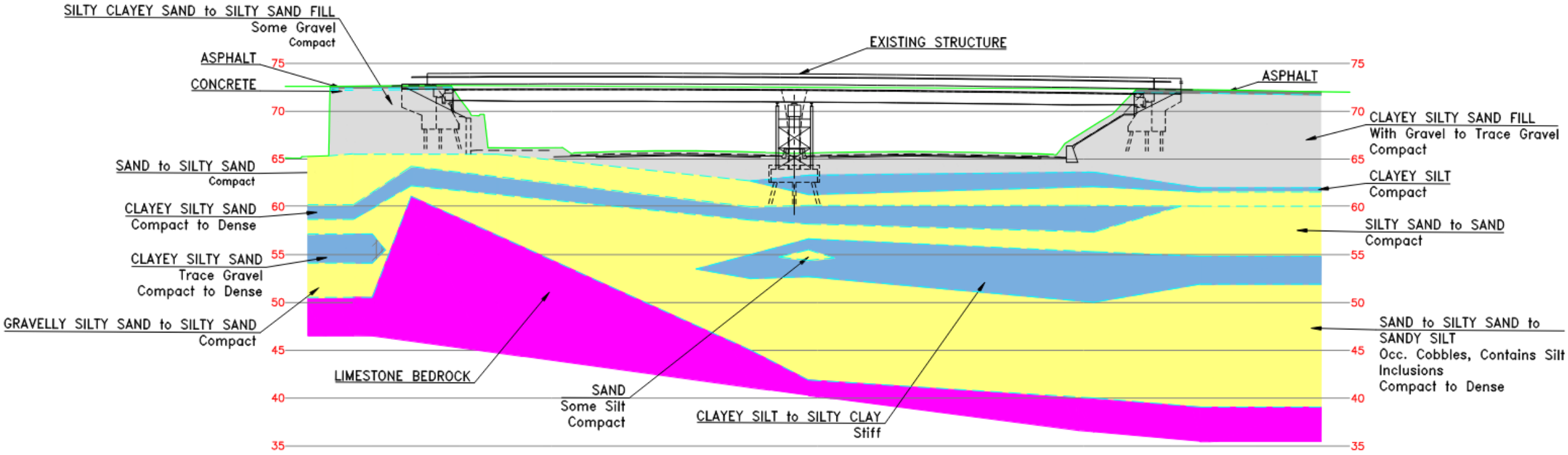


Richmond Road structure from Southeast Wingwall

Subsurface Conditions
Woodroffe Avenue



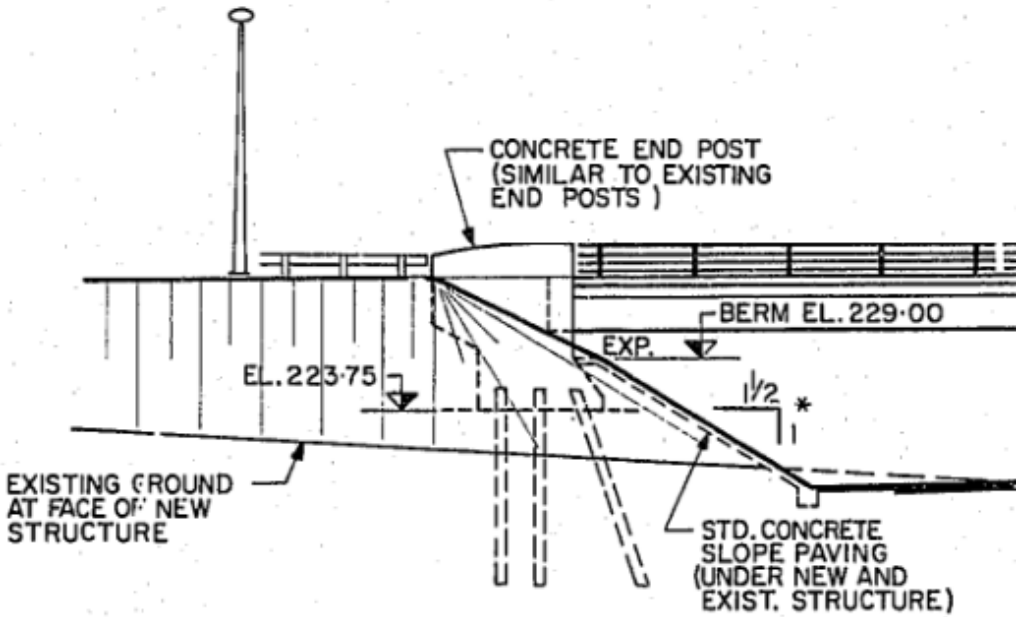
Subsurface Conditions
Richmond Road



Structural Capacity

- **Corrosion**
  - CALTRANS 0.025 mm/yr
  - Eurocode 0.015 mm/yr
  - BS8002 0.015 mm/yr

- CHBDC seismic performance criteria not met for > 0.015 mm/yr
- Supplemental deep foundation elements
  - More cost
  - More time



Supplemental Site Investigations

- Excavation and trench box installation
- Partial exposure of piles under wingwalls
  - Hydrovac
  - Abrasive blast cleaning
- Unshrinkable fill, backfill, and site restoration



Abutment Pile Exposure
Woodroffe Avenue



Woodroffe Avenue –
before blasting of piles



Woodroffe Avenue –
after blasting of piles



Close-up of pile surface

Abutment Pile Exposure
Richmond Road

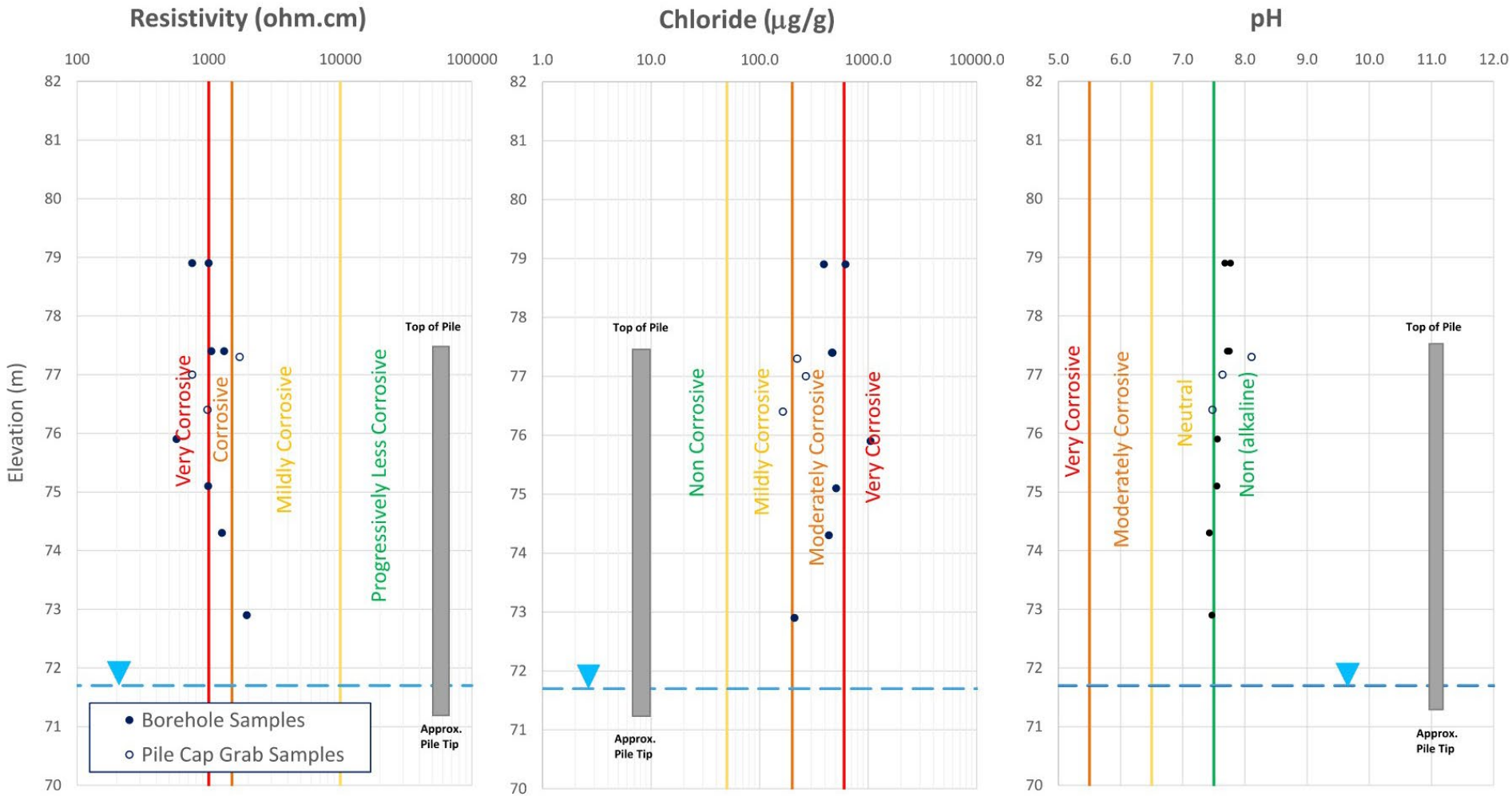


Richmond Road – under southeast wingwall



Close-up of pile surface – corrosion pits and flange thinning

Soil Analyses



Soil Parameters at Woodroffe Avenue

Non-Destructive Testing
Ultrasonic and Visual Inspection

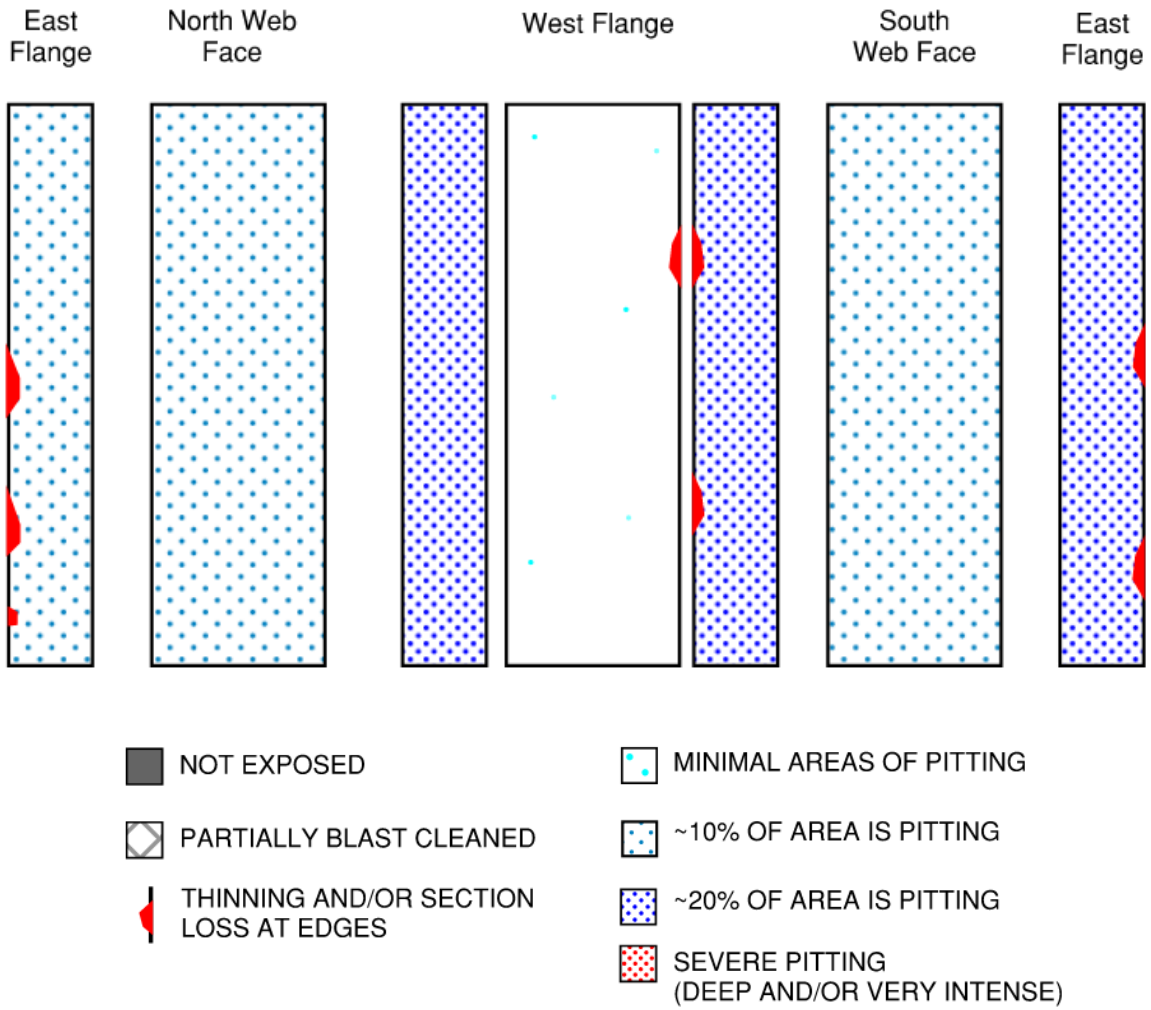
- Pitting depth measurements
  - Ultrasonic testing
  - External pit gauge

- Average deepest pit and average corrosion pitting rate
  - Woodroffe 0.11 mm/yr
  - Richmond 0.07 mm/yr
  - Assuming constant rate



Ultrasonic testing at Woodroffe Avenue

Deterioration Mapping







Percent Area of Pitting

- Percent area experiencing pitting corrosion on each pile

$$\frac{SA_{pitting}}{SA_{total}} = \sum I_{pit,i} \cdot \overline{SA}_{pit,i} \cdot \frac{SA_i}{SA_{total}}$$

$SA_{pitting}$ = total area experiencing pitting corrosion
SA_{total} = total surface area of the pile
$I_{pit,i}$ = intensity of pits in the region under study
$\overline{SA}_{pit,i}$ = average surface area of one pit in the region under study
SA_i = total surface area of the region under study

Percent Area of Pitting

- Percent area experiencing pitting corrosion on each pile

$$\frac{SA_{pitting}}{SA_{total}} = 500\ \text{pit/m}^2 \cdot 200\ \text{mm}^2 \cdot 0.40 = 4\%$$

$SA_{pitting}$ = total area experiencing pitting corrosion
SA_{total} = total surface area of the pile
$I_{pit,i}$ = intensity of pits in the region under study
$\overline{SA}_{pit,i}$ = average surface area of one pit in the region under study
SA_i = total surface area of the region under study

Cross-Sectional Area Loss

- Total cross-sectional area loss including flange thinning

$$A_{combined} = \frac{SA_{pitting}}{SA_{total}} \cdot \bar{d} \cdot P_{pile} + 2\, t_f \cdot b_{loss}$$

$A_{combined}$ = total cross-sectional area loss
$\bar{d}$ = average of the three deepest pits as measured by the NDT specialist
P_{pile} = theoretical pile perimeter
t_f = flange thickness
b_{loss} = flange width loss due to corrosion

Average Site Corrosion Rate

- Average corrosion rate for each site considering the age of the piles (in years) at the time of the investigation

$$\bar{r}_{site} = \frac{1}{3} \sum \frac{A_{combined,i}}{N \cdot P_{pile,i}}$$

$\bar{r}_{site}$ = average site corrosion rate
$A_{combined,i}$ = total cross-sectional area loss for each pile at the site
N = age of each pile in years
P_{pile} = theoretical pile perimeter for each pile at the site

Section Loss Analysis

Site	Pile	Percent area of pitting corrosion [%]	Cross-Sectional Area Loss [mm^2]	Average Site Corrosion Rate [mm/yr]	Average Site Area Loss [%]
Woodroffe	W1	10%	2131	0.015	14.6
	W2	11%	1841		
	W3	15%	2149		
Richmond	R1	5%	793	0.007	7.11
	R2	5%	948		
	R3	10%	1203		

- Uniform corrosion rate represents a conservative estimate of future corrosion

Benefit Cost

Supplementary Corrosion Investigation

- Exposure of piles at two sites
- Site restoration
- Engineering fees
  - Site measurements
  - Corrosion assessment
  - Field sampling and testing
  - Reporting

$400,000

Projected Construction Cost Savings

- Supplementary piles at abutments
- Abutment modification
- Does not include
  - Construction staging
  - Increased construction duration

> $1,650,000

Summary

- Corrosion rate of 0.015 mm/yr per pile surface is appropriate for use at both bridge sites, as well as Pinecrest and Maitland
- In-situ corrosion investigations allowed design team to validate predicted service life of structures (75 years)
- Significant cost savings (> $1.25M)
- Lower impacts to public and environment



Thank You

Bridging Challenges: Creative Solutions for the I-81 Bridge Foundation Design

By:
Ram Kasturi, PE
Kamal Regmi, PE

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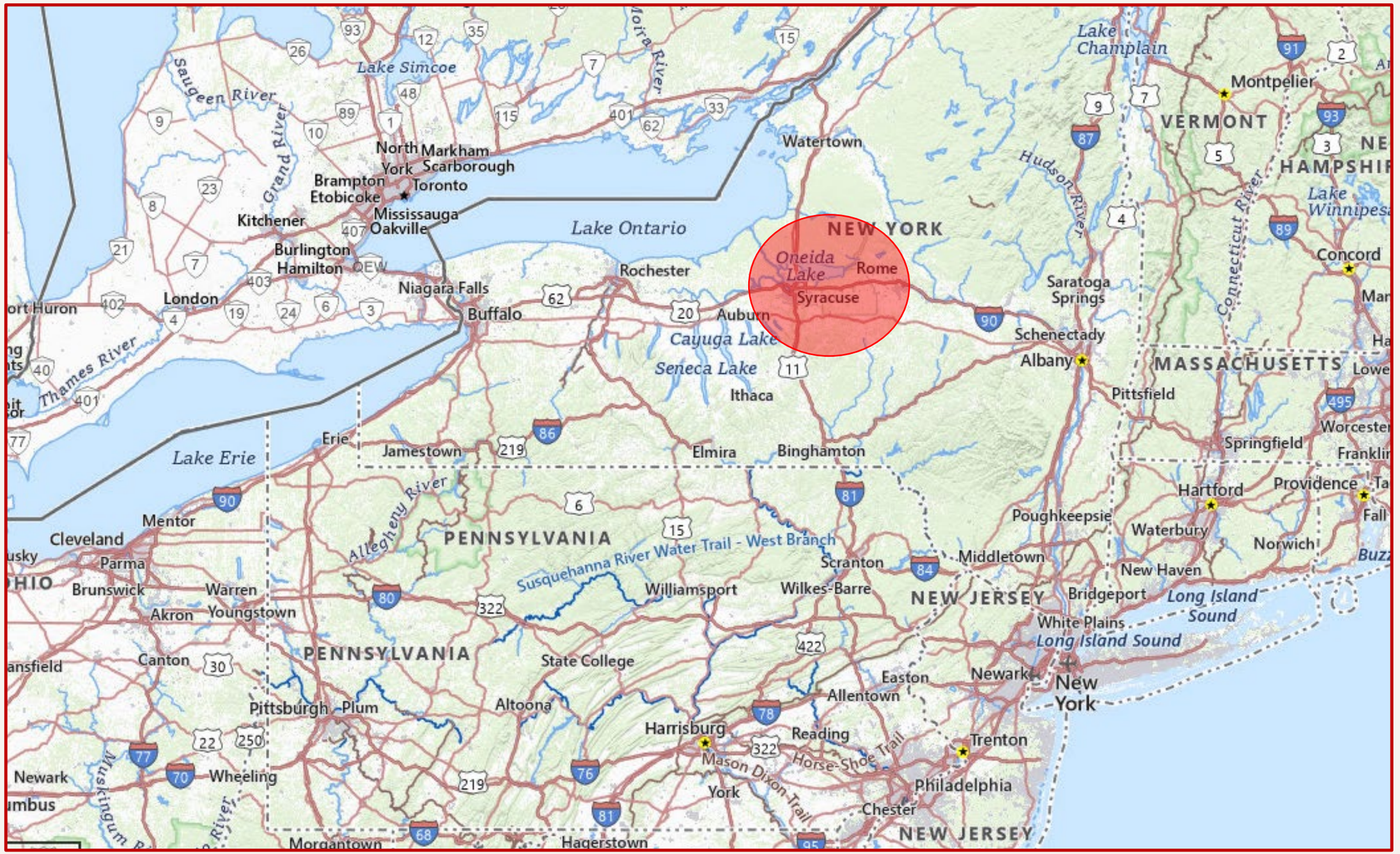
KC Engineering and Land Surveying, P.C.



PROJECT LOCATION

2 Phases → 8 Contracts → Estimated completion date 2029

Key Improvements:

Transportation & Mobility

- Replaces aging I-81/I-690 infrastructure,
- Creates Business Loop 81
- Upgrades key interchanges.

Environmental Justice Commitments

- Ensures fair distribution of benefits
- Improves access for underserved communities
- Mitigates noise, visual, and construction impacts

Pedestrian, Bicycle & Transit Enhancements

- Adds 12.5 miles of sidewalks, 5.7 miles of bike paths
- Safer crossings and better transit access

Environmental, Social & Economic Benefits

- Reduces emissions, restores wetlands, reconnects neighborhoods, supports local hiring, and minimizes displacement.



PROJECT OVERVIEW

Southern Interchange Reconstruction

- Reconfigures I-481/I-81 for new I-81 designation
- Improves traffic flow and safety

Exit 3 (DeWitt) Redesign

- Updates ramps and intersections at NY Routes 5 & 92
- Enhances access and reduces congestion

Corridor Expansion (I-690 to Exit 5)

- Widens and realigns roadways
- Supports higher traffic volumes and smoother transitions

Brighton Avenue and East Glen Avenue Roundabouts

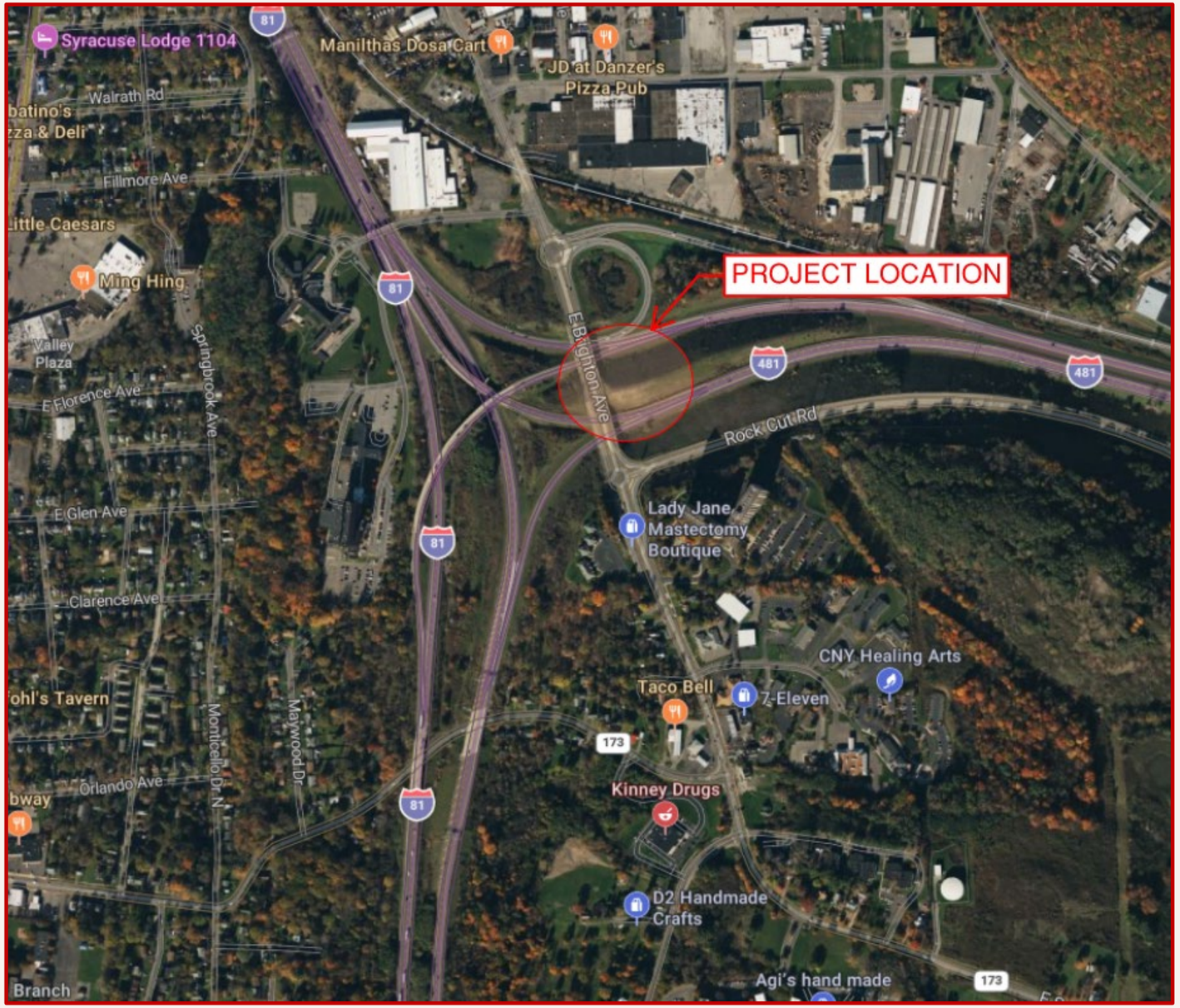
- Adds modern roundabouts in the Valley neighborhood
- Improves circulation and reduces delays

Major Infrastructure & Community Enhancements

- Nearly $400 million total contract cost
- Includes new bridges, bridge replacements/rehabs, retaining walls, sign structures, and culverts
- Adds noise walls, visual barriers, landscaping, and air quality monitoring

I-81 Phase 1, Contract 2





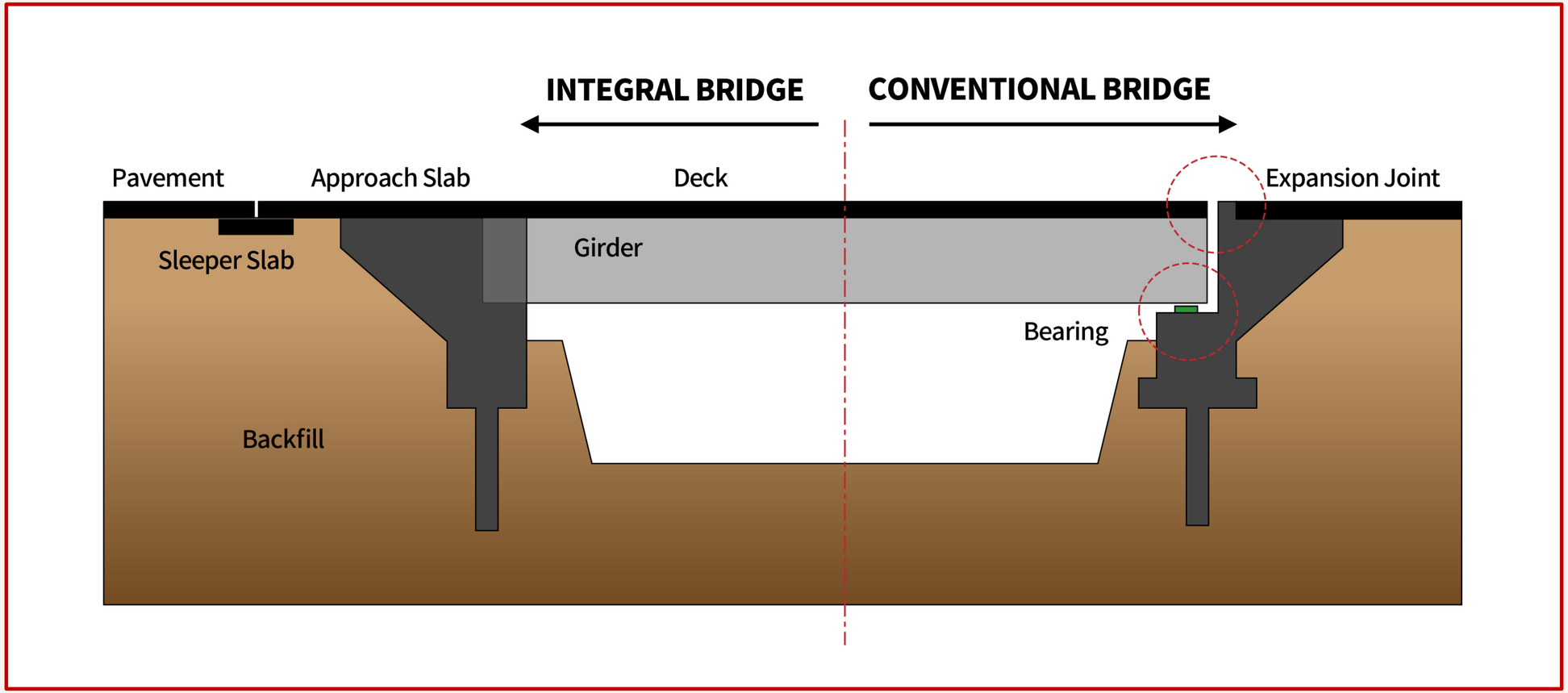
Southern Interchange Reconstruction

PROJECT REQUIREMENT

NYSDOT REQUIRES ALL BRIDGES TO BE DESIGNED WITH INTEGRAL ABUTMENTS UNLESS IT IS JUSTIFIED TO USE OTHER TYPES.

- **LONG SPANS**
- **SUBSURFACE SOIL / ROCK CONDITIONS**
- **TALL ABUTMENTS**

INTEGRAL Vs CONVENTIONAL



Integral Abutment Advantages

Maintenance Savings

No expansion joints means reduced maintenance costs

Prevents Water Damage

Eliminates joint leakage issues

Simplified Construction

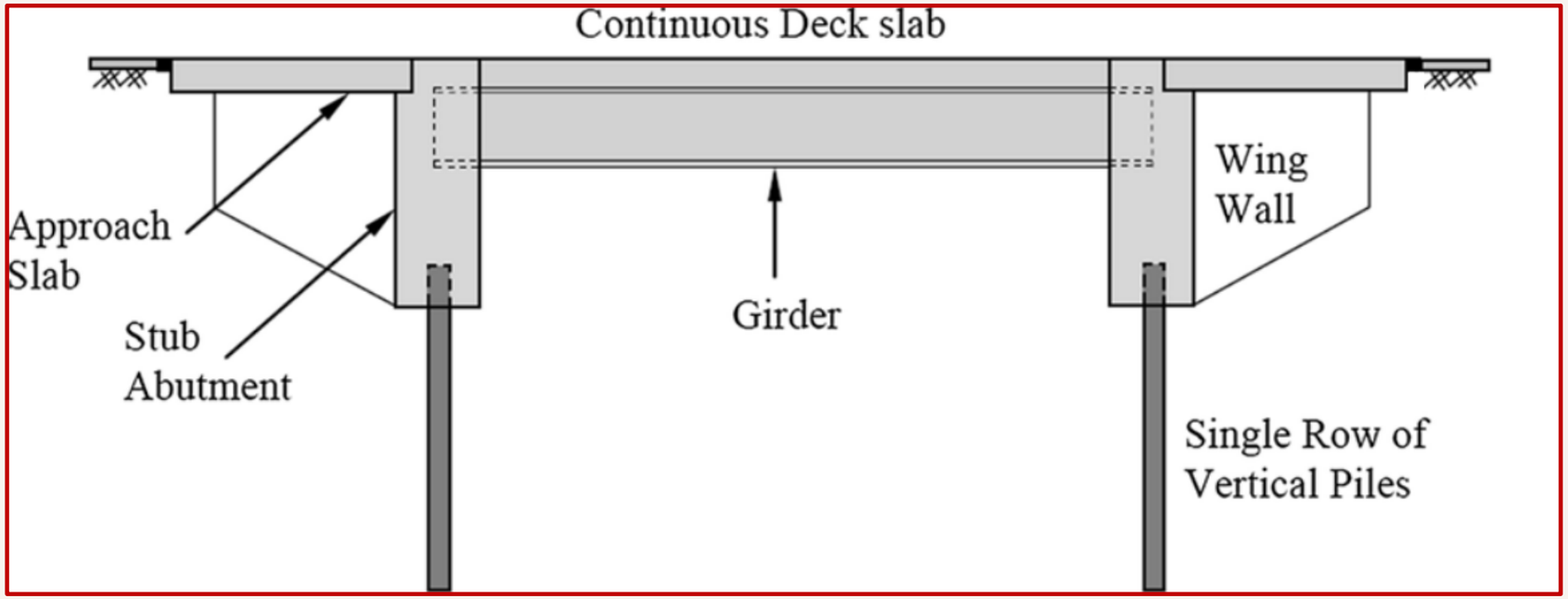
Fewer components to install

Extended Lifespan

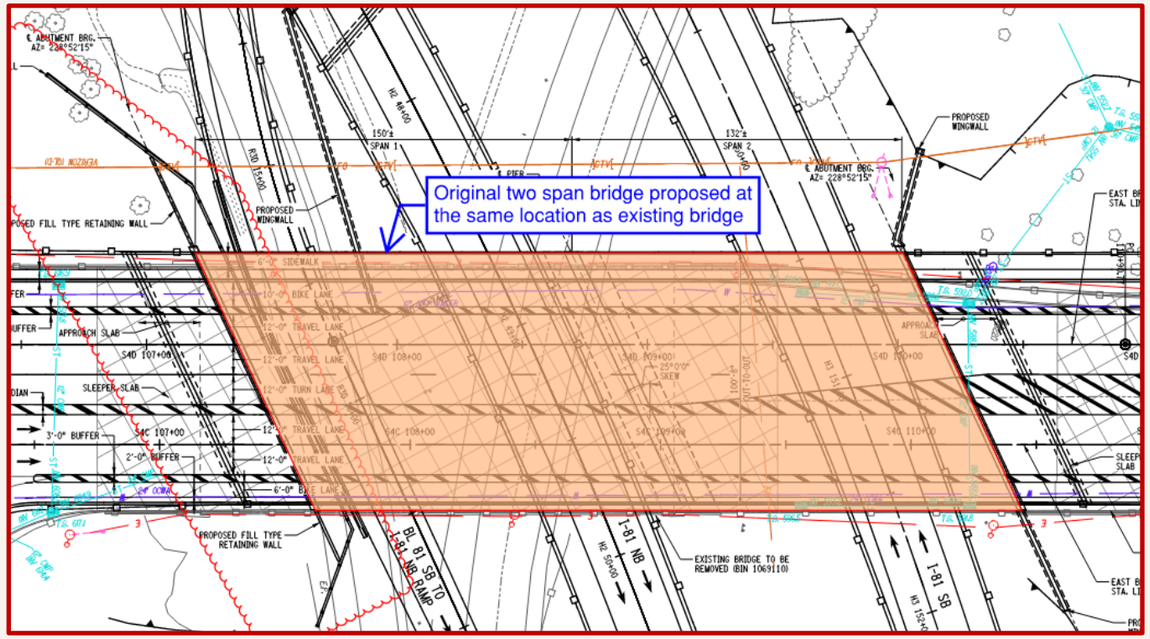
Improved durability of critical components

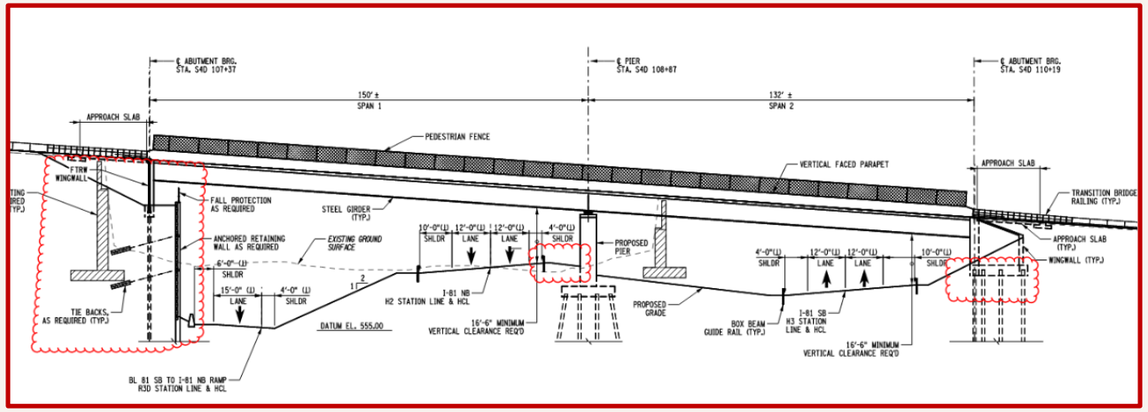
Lower Construction Cost

Fewer Piles required and no battered piles are needed



PRELIMINARY DESIGN





NYSDOT provided preliminary design as part of Request for Proposal (RFP) documents

New bridge proposed at the same location as the existing bridge

282 ft.
Bridge Length with two spans

25 deg.
Bridge Skew

101 ft.
MAX. Bridge Width

EXISTING GRIDGE



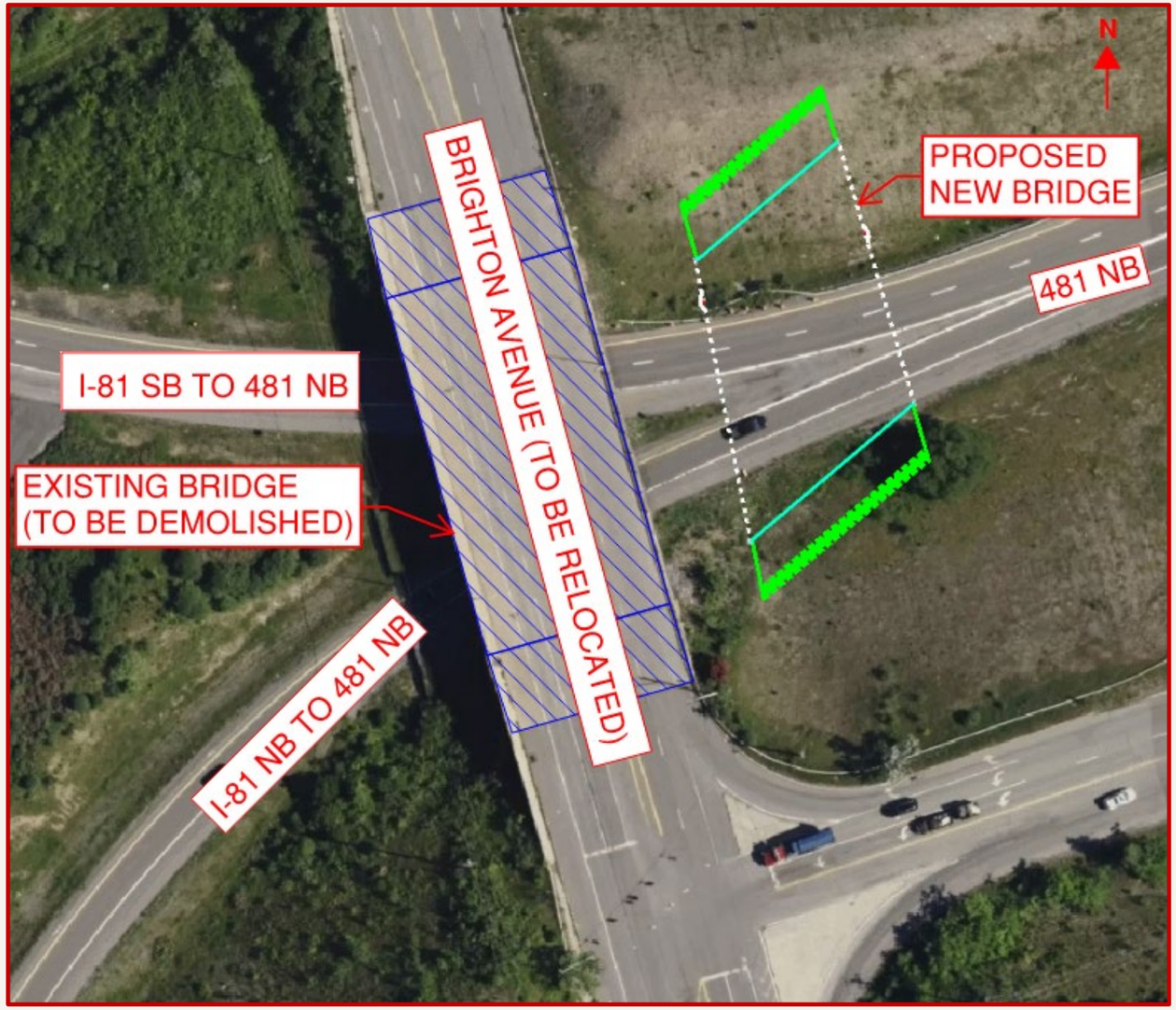
MAIN FEATURES OF THE BRIDGE DESIGN

- INTEGRAL ABUTMENT REQUIREMENT
- CONSTRUCTION CHALLENGES
- PRESENCE OF KARST FORMATIONS
- DESIGN CHALLENGES
- SITE CONSTRAINTS

BRIDGE LOCATION

Reconstruction of East Brighton Avenue in Syracuse, NY

A new East Brighton Avenue bridge proposed as part of the reconstruction



Construction Challenges

1

Temporary Relocation of Watermains

Existing bridge carries two large watermains that must remain functional during construction.

2

Maintaining Existing Traffic

Requires complex construction staging

Requires many traffic shifts

3

Mass Concrete Issues

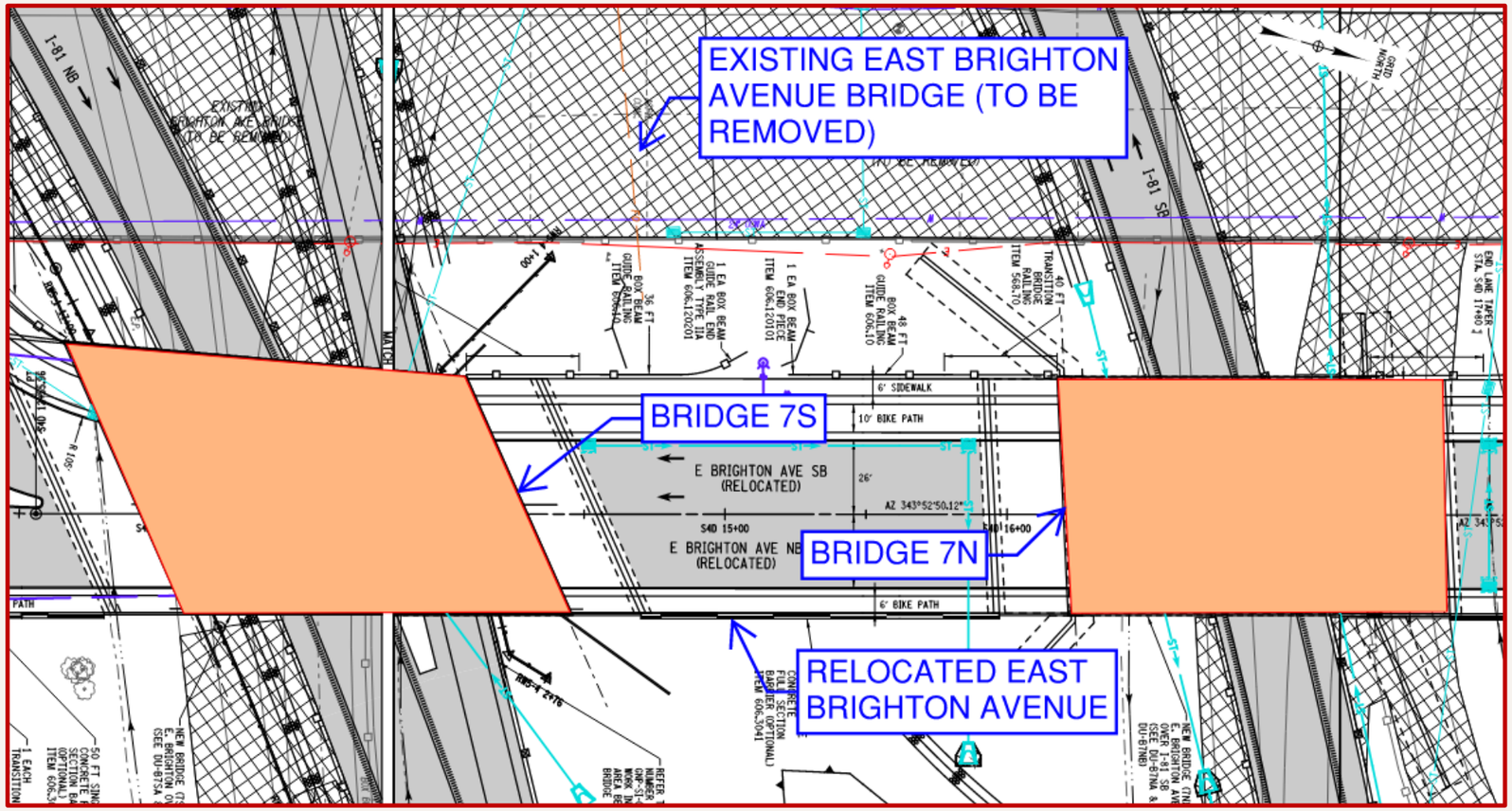
Potential quality control problems with large pours

Potential delays due to complexity and concrete curing

ALTERNATE DESIGN

(Alternative Technical Concept)

- **Bridge alignment shifted Eastward**
  - Allowed existing traffic to be maintained
  - Eliminated complex construction staging
  - Eliminated the need to relocate watermain lines
- **Bridge Split into 2 single span bridges**
  - Reduced the bridge footprint area
  - Reduced construction cost
  - Reduced future maintenance cost



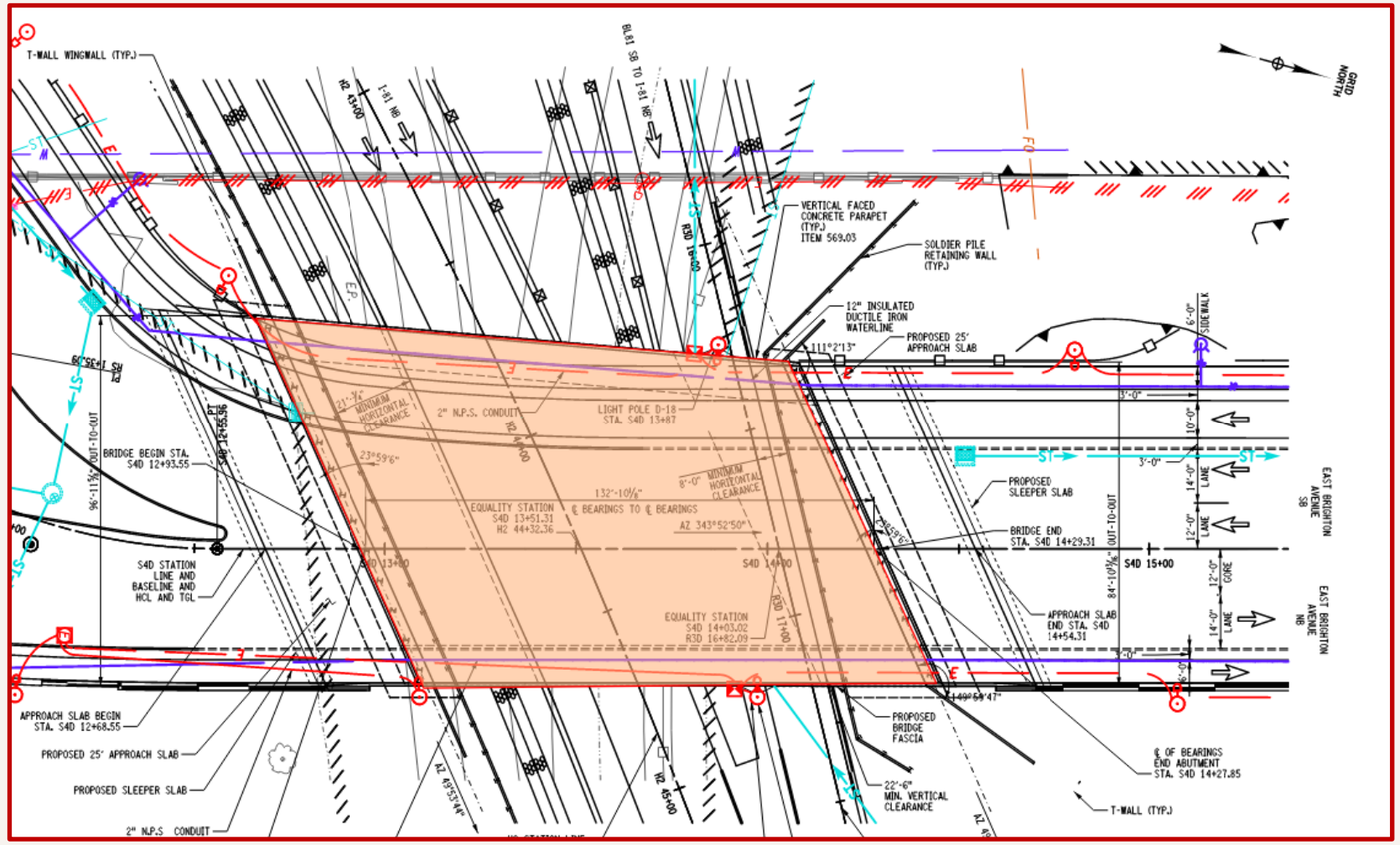
BRIDGE 7S

133 ft.
Bridge Length
Single-span design

~42 ft.
Initial Bridge Height

24 deg.
Bridge Skew

97 ft.
MAX. Bridge Width



Abutments > 15 ft requires complex soil structure interaction FEM modeling

15 ft
NYSDOT Limit
Standard design height limit

SOIL PROFILE

Bottom El. Of Abutment= 596’ ±

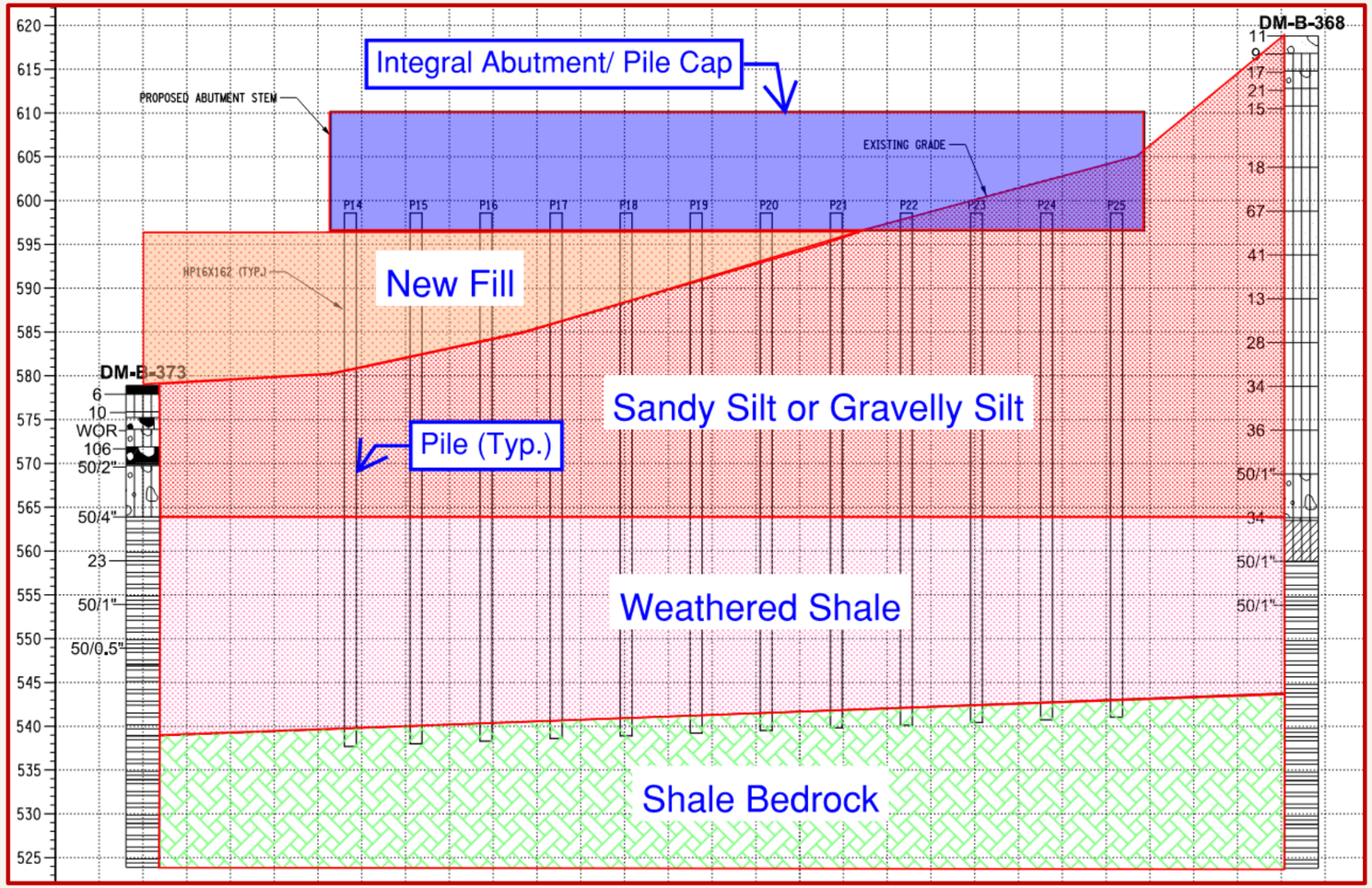
Top of Weathered Rock= 564’ ±

Avg. Top of Sound Rock= 540’±

➡ 24 ft. ± thick weathered rock Layer

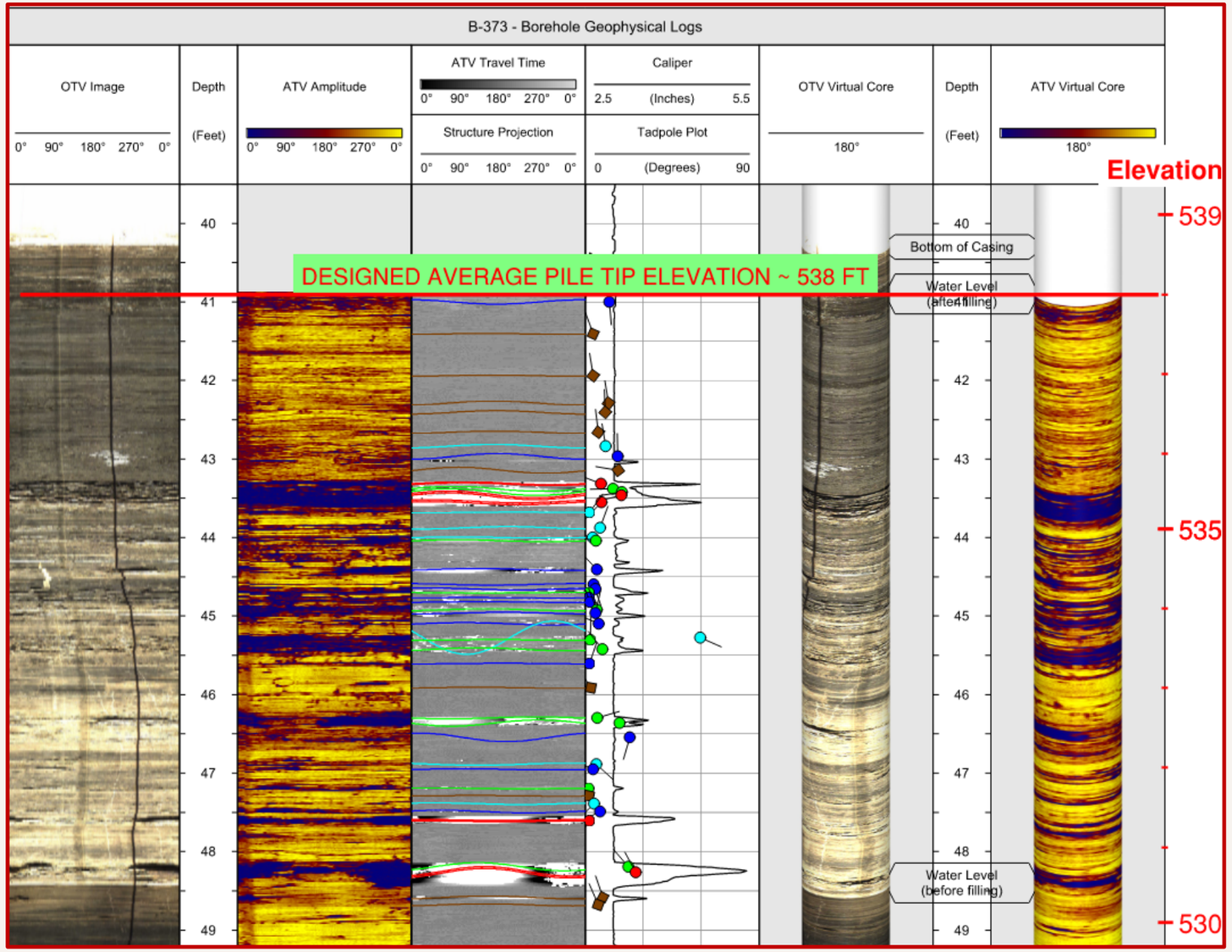
Erratic rock profile
with weathered rock.

Possible karst formations
at localized locations.

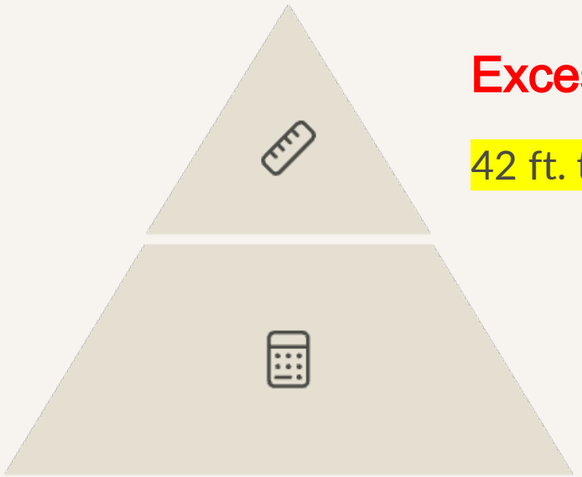


GEOPHYSICAL SURVEY

Geophysical survey indicated
no major voids
below the tip elevation of piles.



DESIGN CHALLENGES



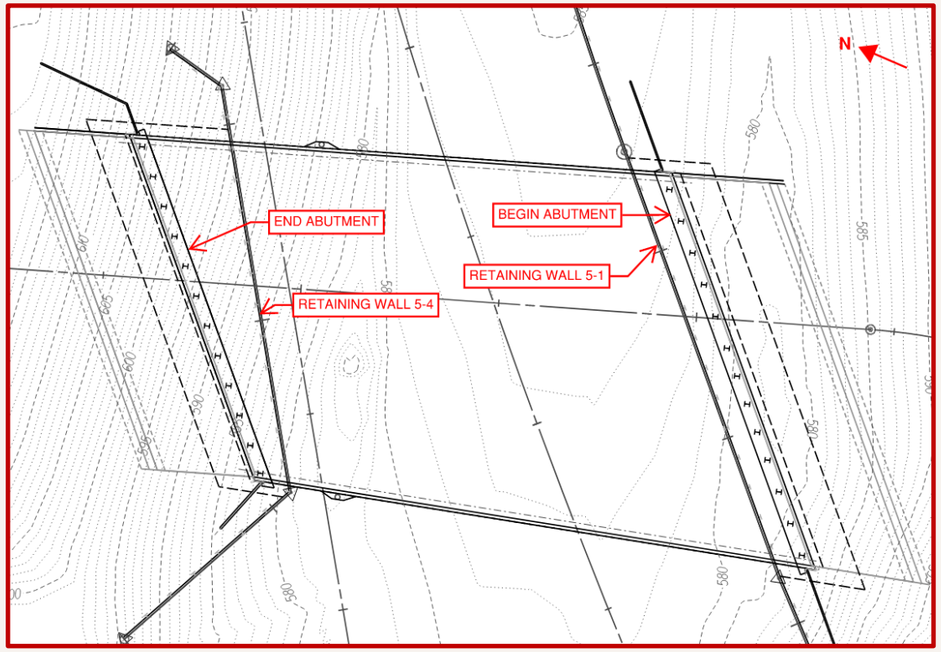
Excessive Height

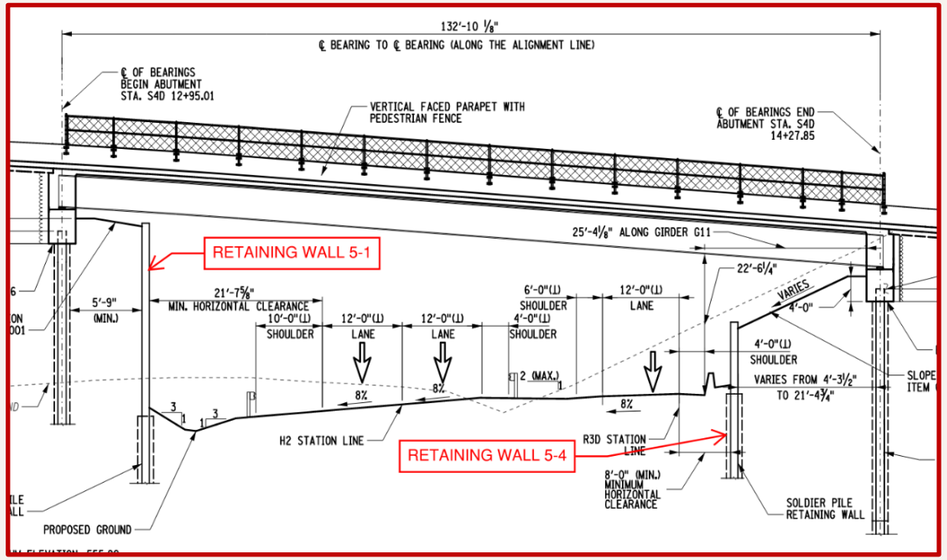
42 ft. tall abutment exceeds standard 15 ft. NYSDOT limit

Complex Analysis

Tall abutment design requires finite element modeling

BRIDGE 7S





Introduced Retaining Walls

- ❑ Retaining Walls 5-1 and 5-4 added in front of the Abutments
  - Reduced abutment height
    - Permitted the use of Standard NYSDOT Design
    - Eliminated the need for complex finite element modeling
    - Significantly reduced concrete volume for abutments
    - Allowed better Quality Control for concreting

Soldier Pile Wall Design

Wall Height

Variable from 1 ft to 42 ft

Up to four levels of anchors

Anchor Types

Grouted tiebacks in excavation areas

Deadman anchors in embankment areas

Anchor Specifications

Grouted anchors: 22-100 ft long

Bond length up to 60 ft

Installed Capacity up to 210 kips

Deadman Anchor System

Steel Sheet Pile

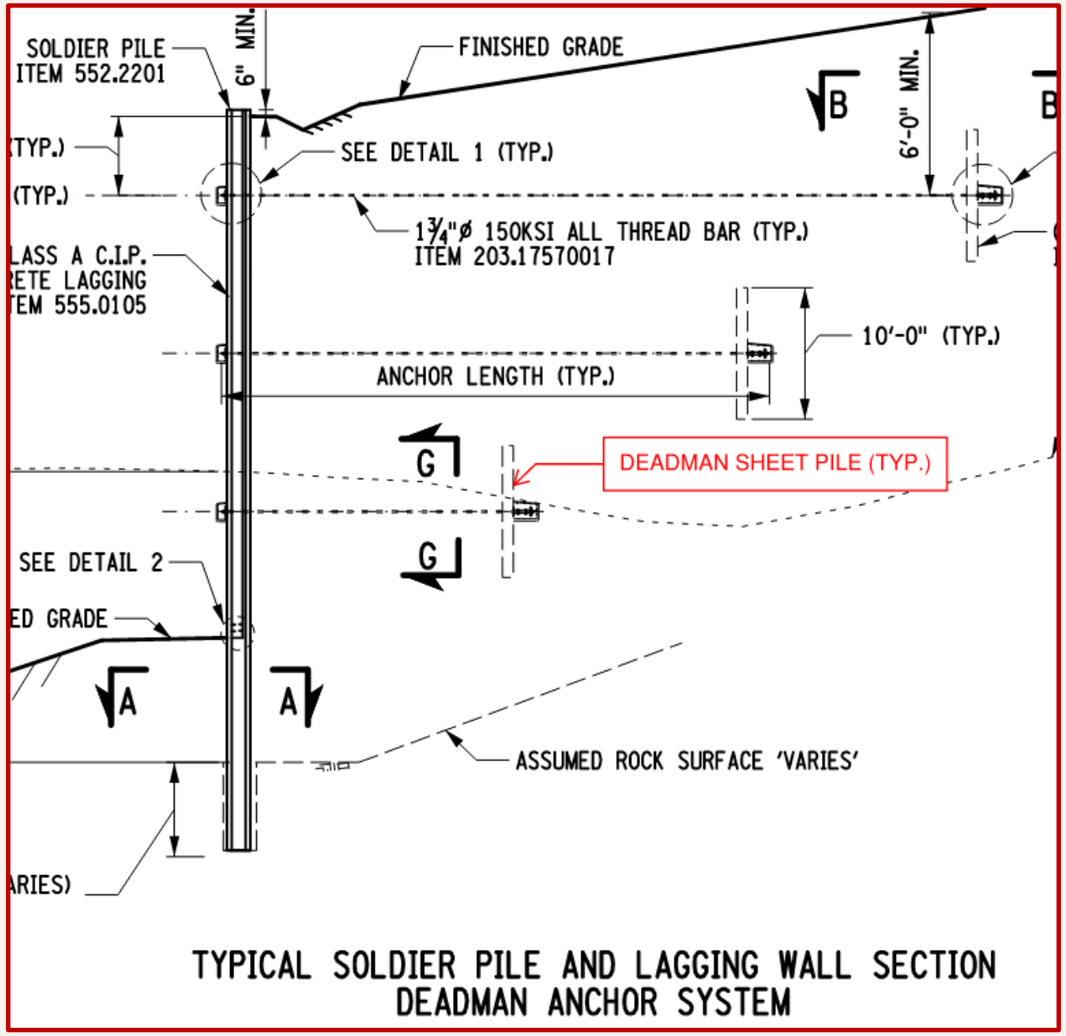
SCZ12 type behind wall

Steel Tie Rod

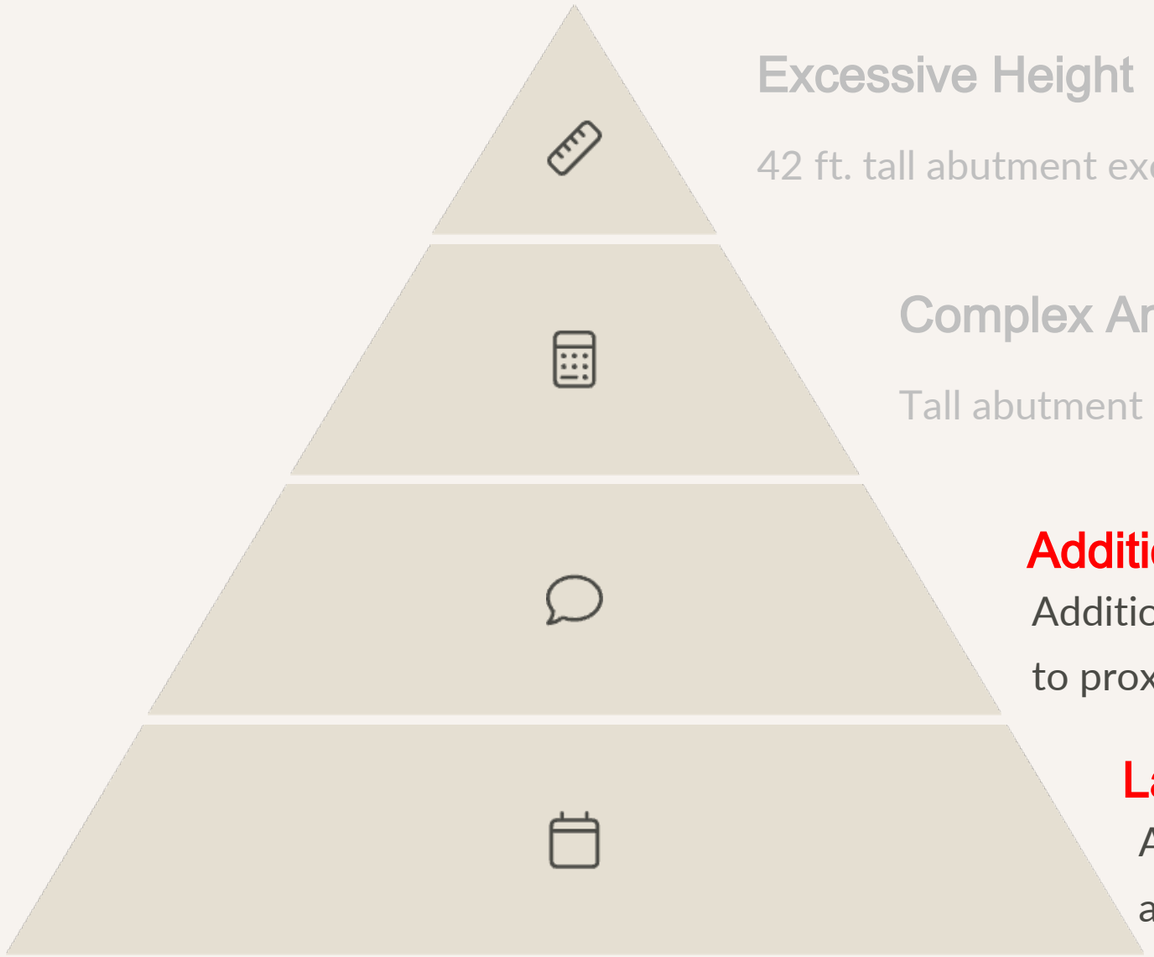
20-55 ft long

Design

Based on NAVFAC DM7.2



DESIGN CHALLENGES



Excessive Height

42 ft. tall abutment exceeds standard 15 ft. NYSDOT limit

Complex Analysis

Tall abutment design requires finite element modeling

Additional load on Retaining Wall

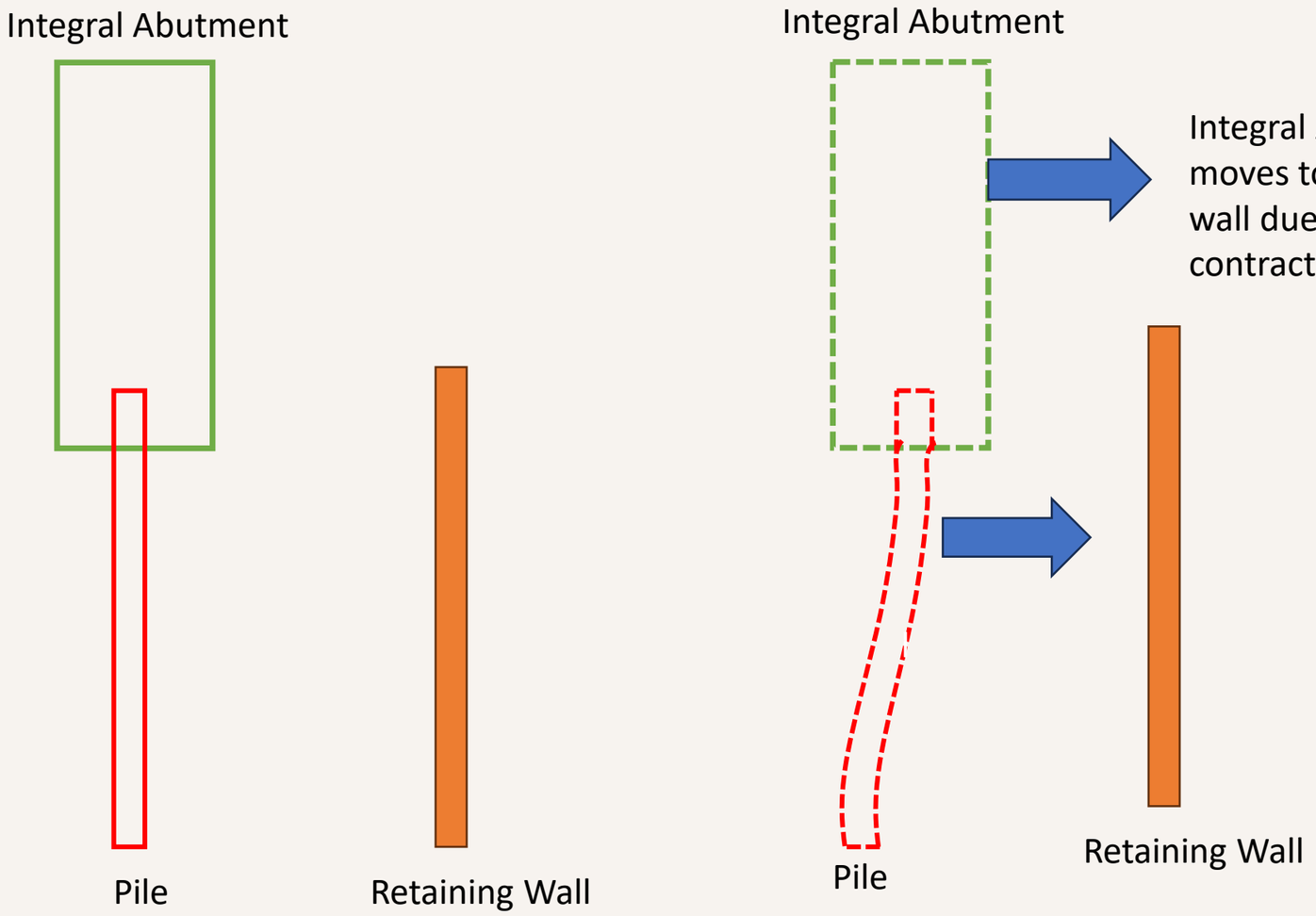
Additional load on retaining wall due to proximity with the integral abutments

Layout of Abutment Piles

Abutment piles layout revised to avoid conflict with tieback anchors and redesigned with bigger pile section and larger spacing

DESIGN CHALLENGES

Additional load on Retaining Wall



Design of any structures within 15ft needs to account for the influence of the movement of abutment and piles (NYSDOT Bridge Manual)

Design Challenges

Layout of Abutment Piles

Bridge Pile Redesign

Bridge piles redesigned using bigger section (HP16X162) and larger spacing (8’ c/c ±) to provide sufficient space for Anchors



SITE CONSTRAINTS

Tight Spacing

Min. 4 ft between wall and Abutment

Anchors between bridge piles

Anchors from Soldier pile walls had to accommodated between abutment piles



Crisscrossing Anchors

Wall 5-4 wraps around the abutment causing crisscrossing

Installation challenges

Both abutment piles and soldier piles installed in augured holes to minimize the impact due to vibration

SITE CONSTRAINTS

Tight Spacing

Challenging construction and field control due to tight spacing between abutment and soldier pile

Abutment and wall piles were installed in Augered holes to minimize the impact due to vibration

Site Constraints

Anchors Between Bridge Piles

3D CAD Model

3D CAD model utilized to carefully place tieback anchors between bridge piles to avoid conflicts



Site Constraints

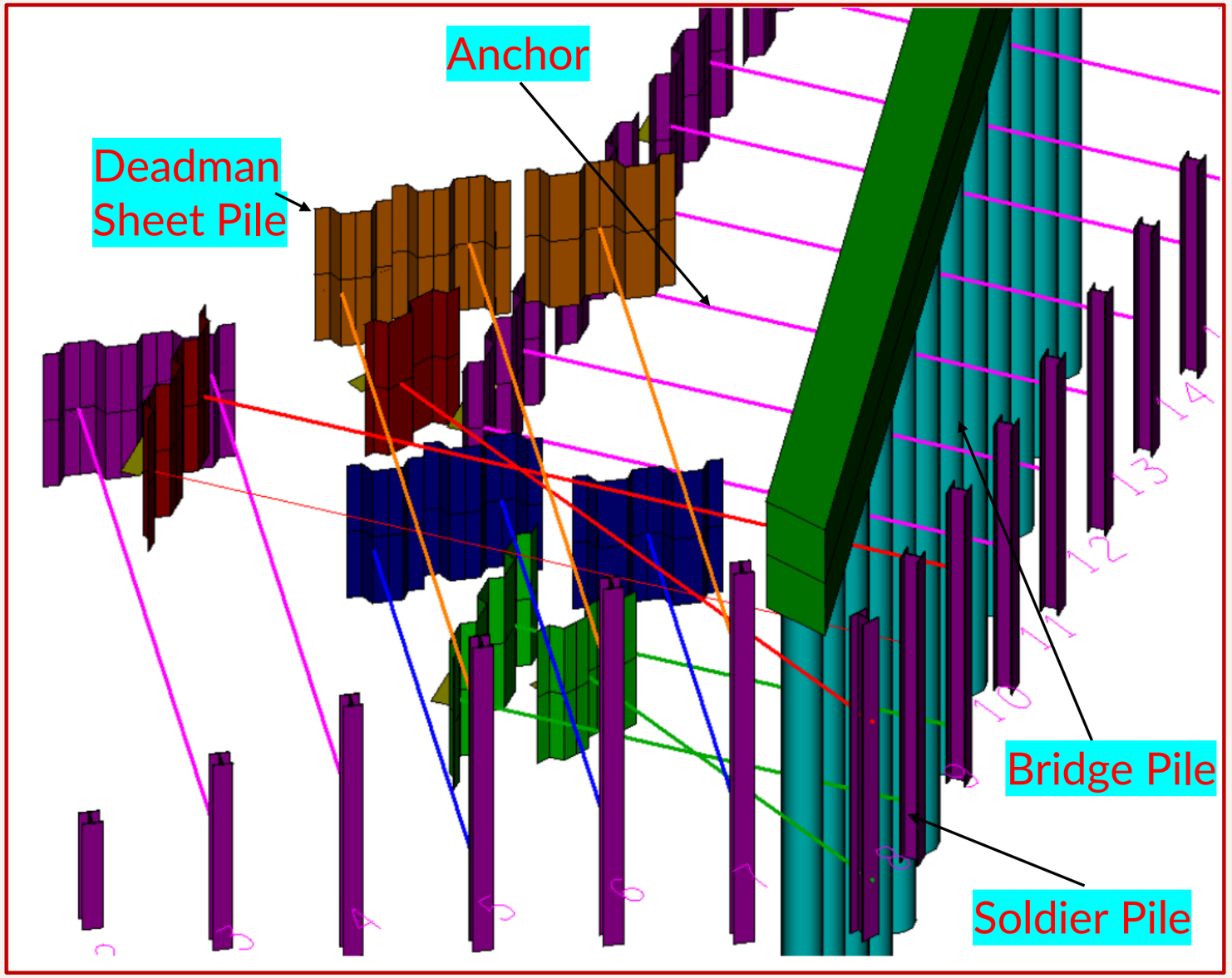
Crisscrossing of Anchors

3D CAD Model

3D CAD model used to identify the conflicts between anchors due to crisscrossing.

Design Modification

To resolve the conflicts, the design was modified by locating some anchors further back and developed suitable construction staging.



INSTALLATION ISSUES

Some piles extended almost 14 ft. deeper .

PDA tests to confirm the capacity of the installed piles.

Some piles deviated ~ 11 inches from intended location.

Pile tip damage



Construction in progress





Construction in progress



Key Takeaways

Innovative Solutions

Modified bridge alignment and layout to reduce complexity

Combined soldier pile walls with integral abutments to simplify design and construction

Careful Planning

3D modeling used to resolve construction conflicts

Detailed Geotechnical Investigation to evaluate subsurface conditions with up to 20 ft. deep rock coring

Geophysical surveys performed to confirm rock quality below pile tip elevation

Long-Term Benefits

Reduced maintenance costs

Enhanced durability of critical components

Lower lifecycle costs justify design complexity

Questions?



KC Engineering and Land Surveying, P.C.



Innovative Bridge Pier Installation with Super Reach Attachment System

Insights from the Jimmie Davis Bridge Replacement Project

Jewels Stover – Bauer Equipment

Sean Pruet – R.R. Cassidy Inc.

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Agenda

No.	Agenda topic
1	Introduction & Project Background
2	Scope of Work and Construction Sequence
3	Technical Challenges and Drilling Equipment
4	Conclusions

1 INTRODUCTION & PROJECT BACKGROUND

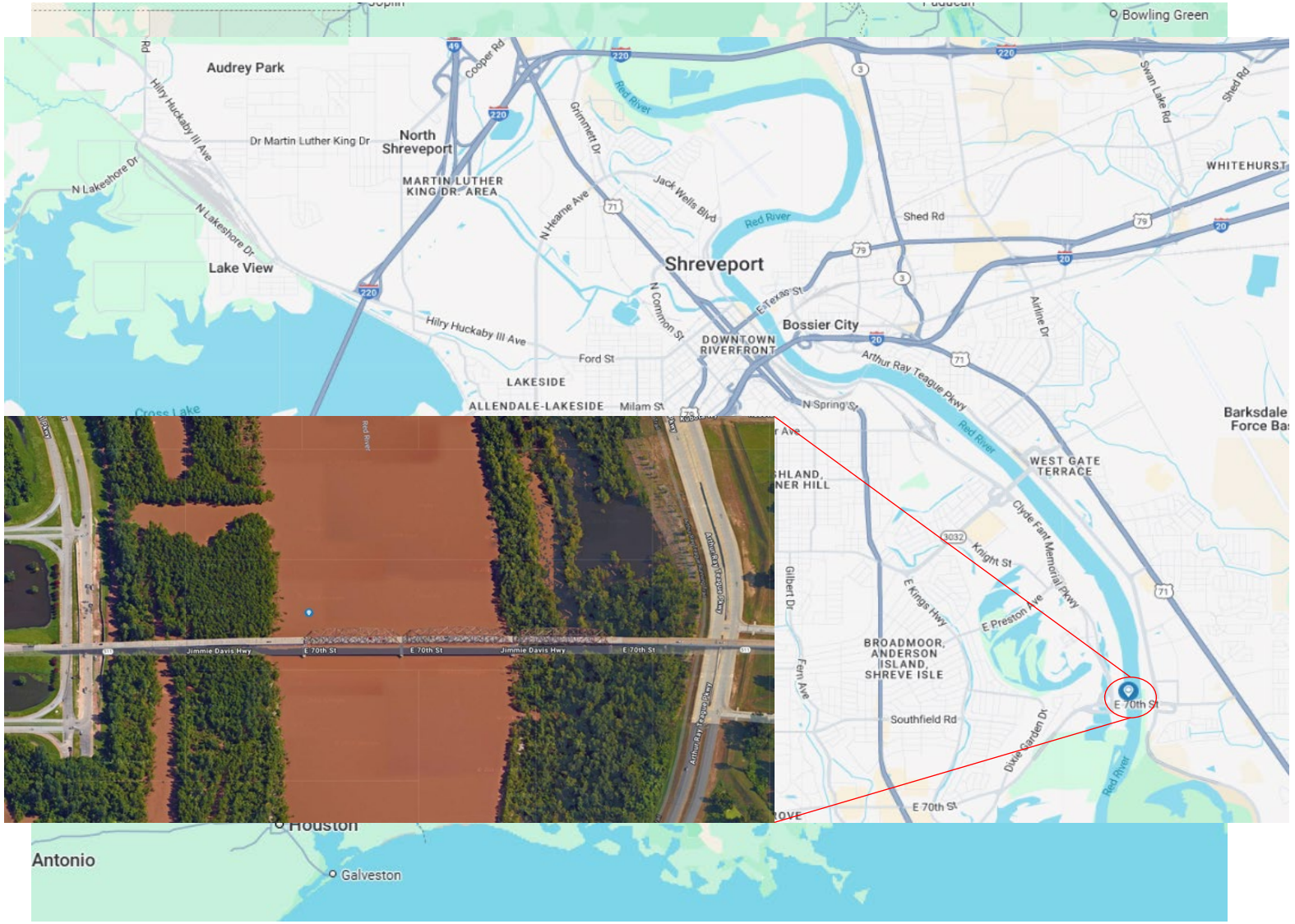


Photo: Primoris Heavy Civil

INTRODUCTION

LOCATION

- Northwestern Louisiana
- Spans the Red River
- Connects Shreveport and Bossier City



INTRODUCTION

JIMMIE DAVIS MEMORIAL BRIDGE

- Constructed in 1968
- Metal Truss Bridge – Spans 1.1 miles (1.8 km) over the Red River
- Known for impressive 300 ft (91.44 m) spans between bridge piers



PROJECT BACKGROUND

JIMMIE DAVIS BRIDGE REPLACEMENT

- $360 M Design-Build Project
- Construction of NEW 4-lane bridge adjacent to existing structure
- Original 2-lane structure repurposed for pedestrians and bicycles





2 SCOPE & CONSTRUCTION SEQUENCE

SCOPE & CONSTRUCTION SEQUENCE

R.R. CASSIDY SCOPE

- Foundations for 3 New Bridge Piers
- 45 Deep Drilled Shafts in the Red River
  - 3 rows of 5 shafts per pier
- Depths range from 100 ft (30.48 m) to appx 150 ft (45.72 m)
- Non-standard 58.5 inch (1486 mm) Drilled Shaft diameter



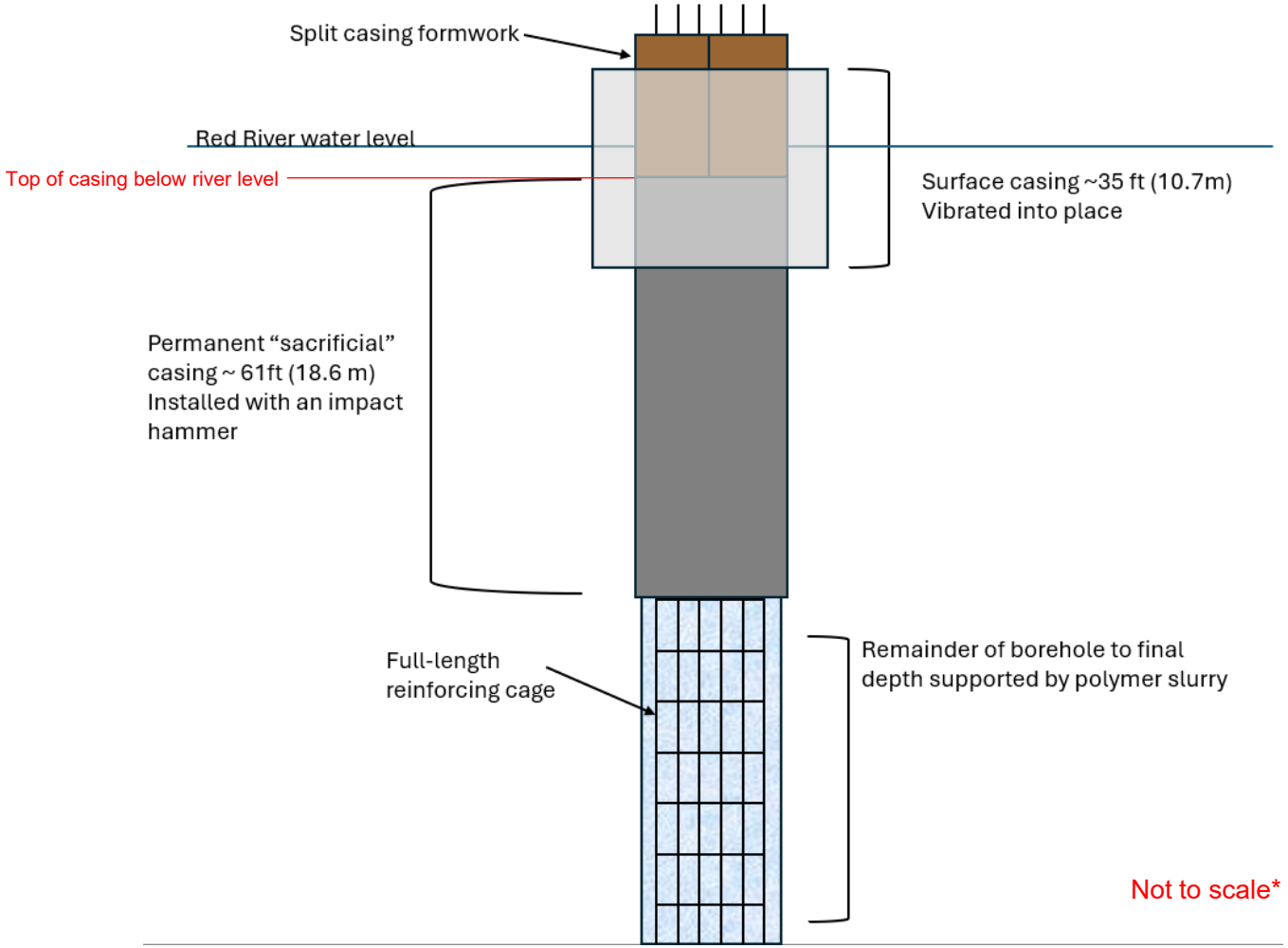
Photo: Primoris Heavy Civil

SCOPE & CONSTRUCTION SEQUENCE

DRILLED SHAFT OVERVIEW

- Non-standard diameter required custom fabricated drill tools





SCOPE & CONSTRUCTION SEQUENCE

FORMWORK

- Special template built for drilled shaft layout in the river





SCOPE & CONSTRUCTION SEQUENCE

SURFACE CASING

- 96-inch (2438 mm) ID Surface Casing
- 35 ft (10.7 m) in length
- Installed using crane-suspended free hanging vibrator

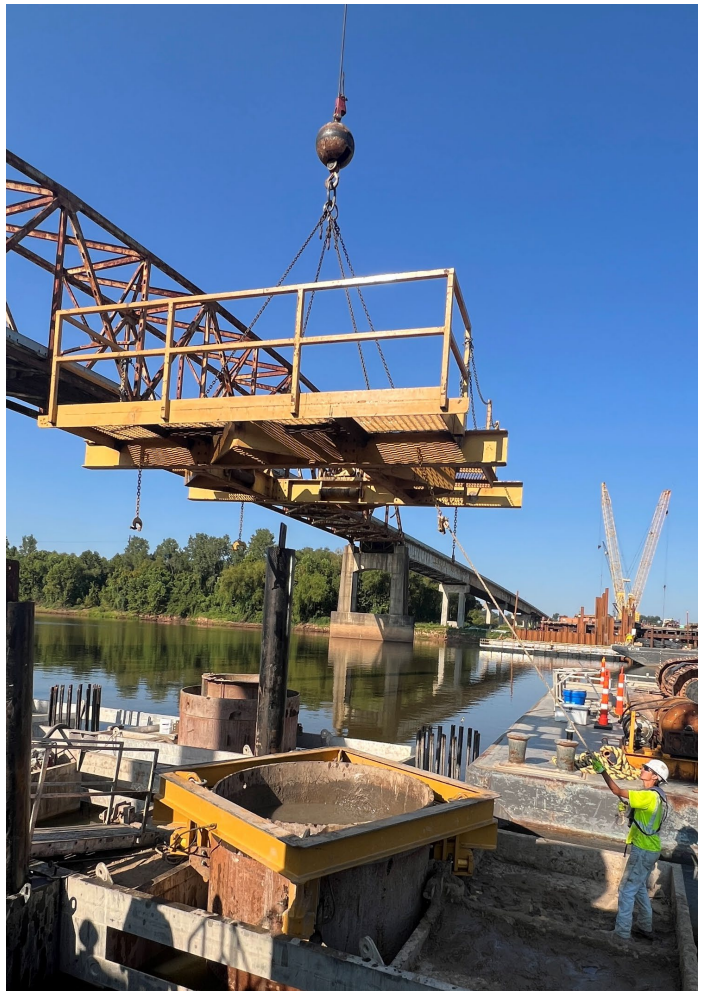




SCOPE & CONSTRUCTION SEQUENCE

PERMANENT SACRIFICIAL CASING

- 2-part Frame to set permanent sacrificial casing





SCOPE & CONSTRUCTION SEQUENCE

PERMANENT SACRIFICIAL CASING

- Appx 61 ft (18.6 m) in length
- 60-inch (1524 mm) ID casing
- Driven inside surface casing using an impact hammer





SCOPE & CONSTRUCTION SEQUENCE

SPLIT CASING

- Top of sacrificial casing terminates at El. 140.5 ft
  - ~4.5 ft (1.4m) underwater at normal pool levels
  - 1.5 ft (.46m) below top of concrete
- Requires split casing as formwork for final 1.5 ft (.46m) of the drilled shaft





SCOPE & CONSTRUCTION SEQUENCE

SLURRY SUPPORTED DRILL HOLE

- Below sacrificial permanent casing
  - drill hole is stabilized using polymer slurry





SCOPE & CONSTRUCTION SEQUENCE

SPECIAL SPECIFICATION PROVISION

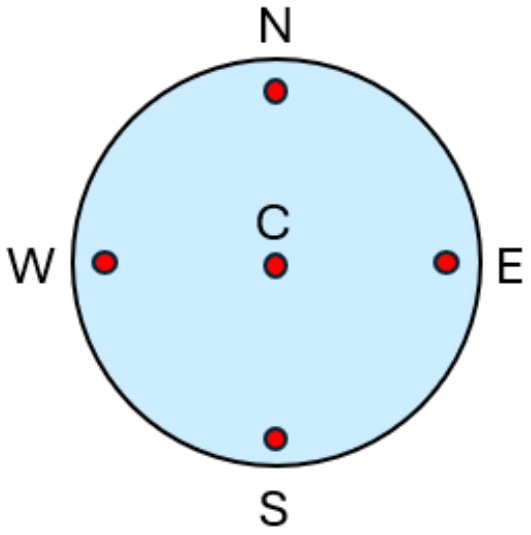
- Once excavation extends below sacrificial permanent casing, all subsequent operations must be completed in 36 hours
- If condition not met, then drill hole must be over reamed using a belling bucket
  - Increases to Ø 63-inch (1600 mm)



SCOPE & CONSTRUCTION SEQUENCE

BOTTOM CLEANING INSPECTION & CONFIRMATION

- SID camera measures remaining sediment on the bottom of the hole
- Sediment must NOT exceed:
  - ½ inch (12.7 mm) at any 3 of 5 locations
  - No single measurement >1.5 inch (38.2 mm)





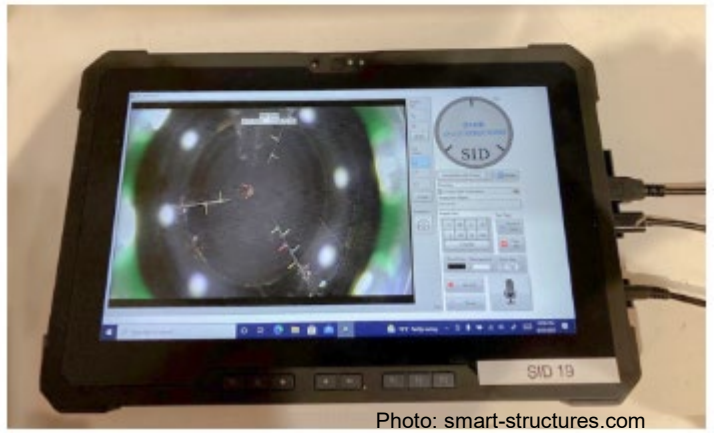


Photo: smart-structures.com

SCOPE & CONSTRUCTION SEQUENCE

REBAR INSTALLATION

- Full-length rebar cages
- Special tilting frame



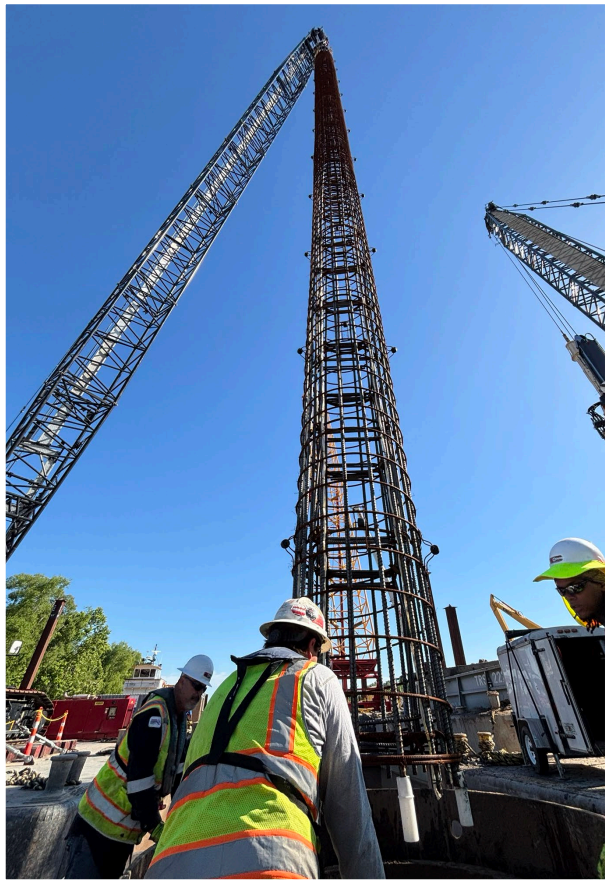


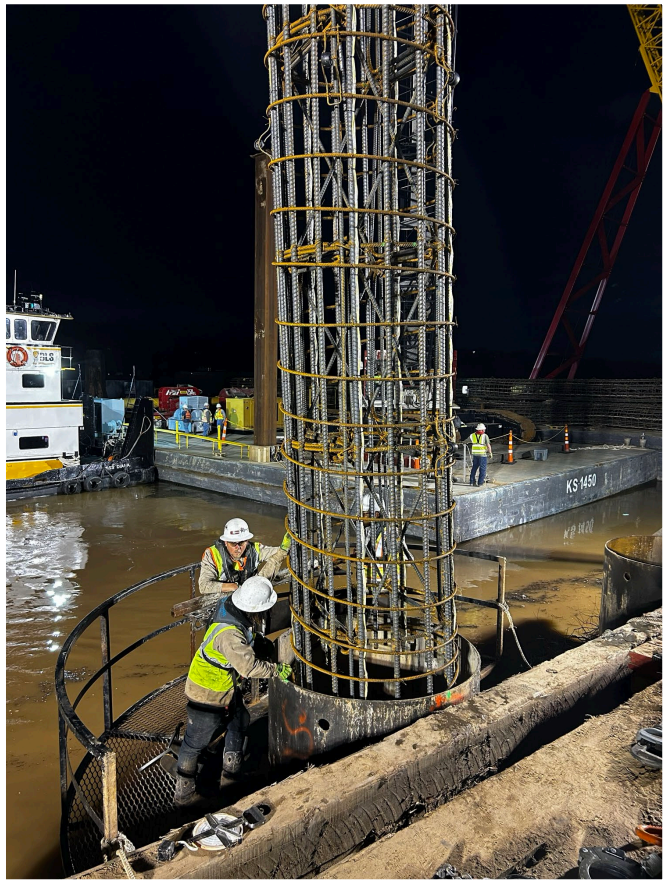
Scope and Construction Sequence

REBAR INSTALLATION







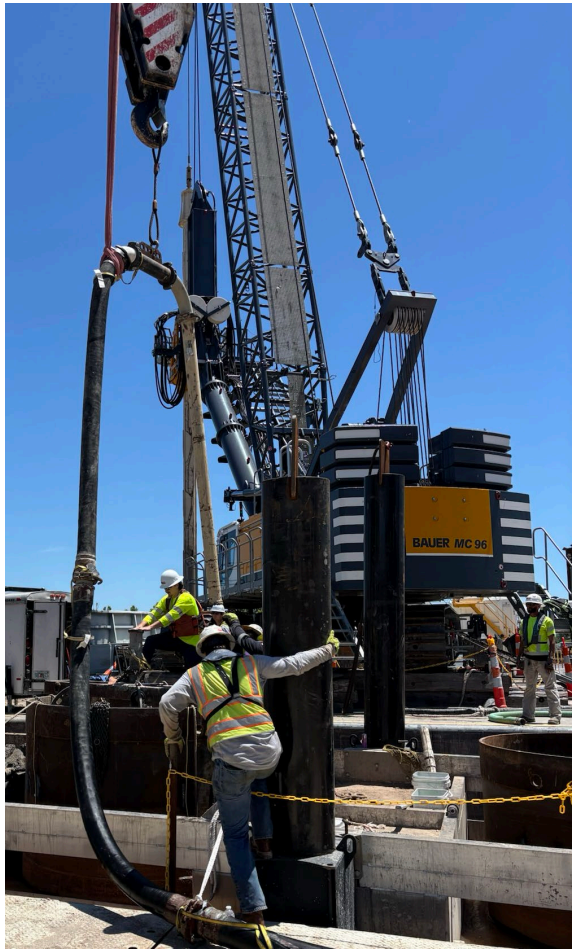


SCOPE & CONSTRUCTION SEQUENCE

CONCRETE PLACEMENT

- Concrete placed via tremie method
- Concrete trucks and pumps remain on land-based trestle

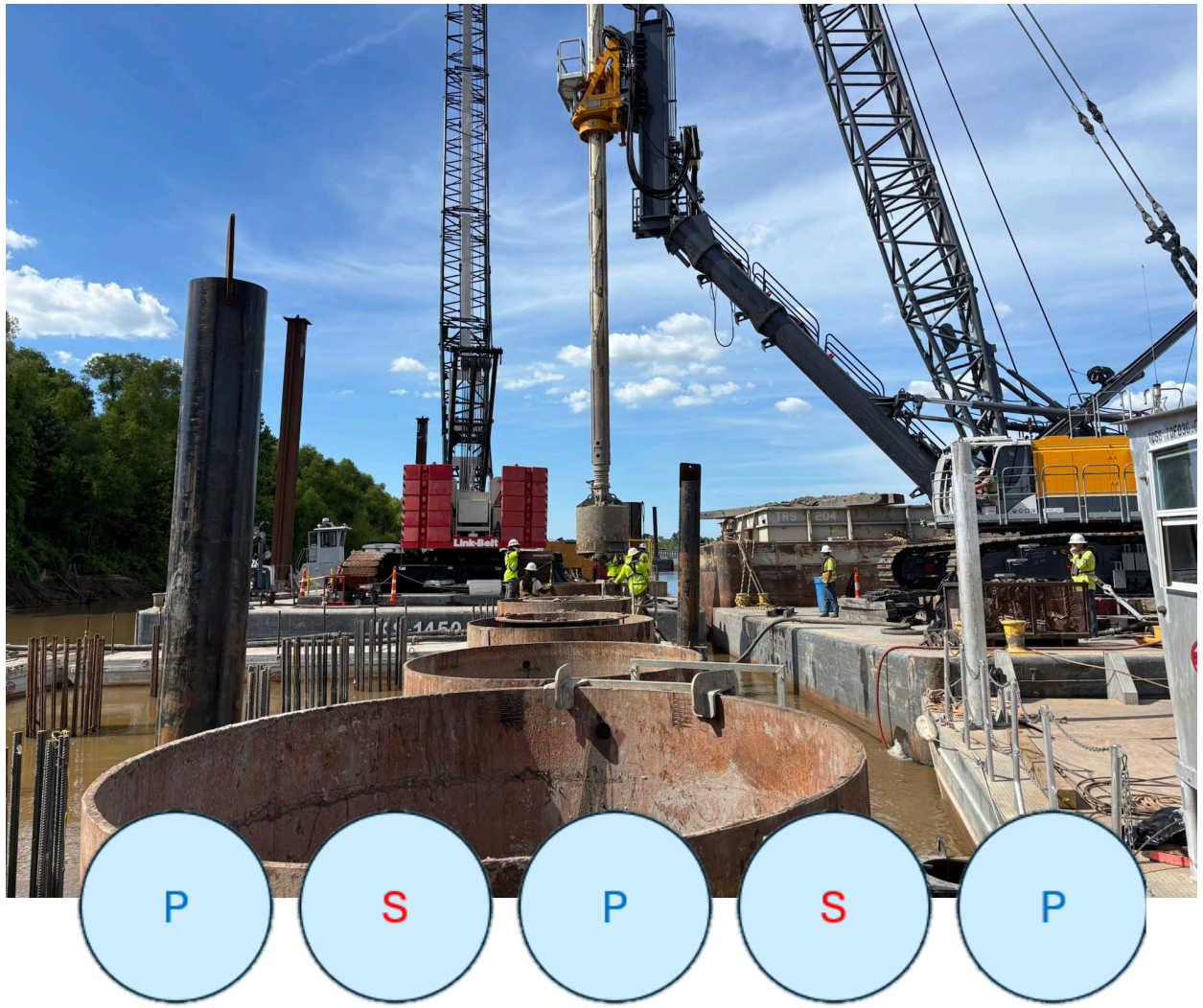


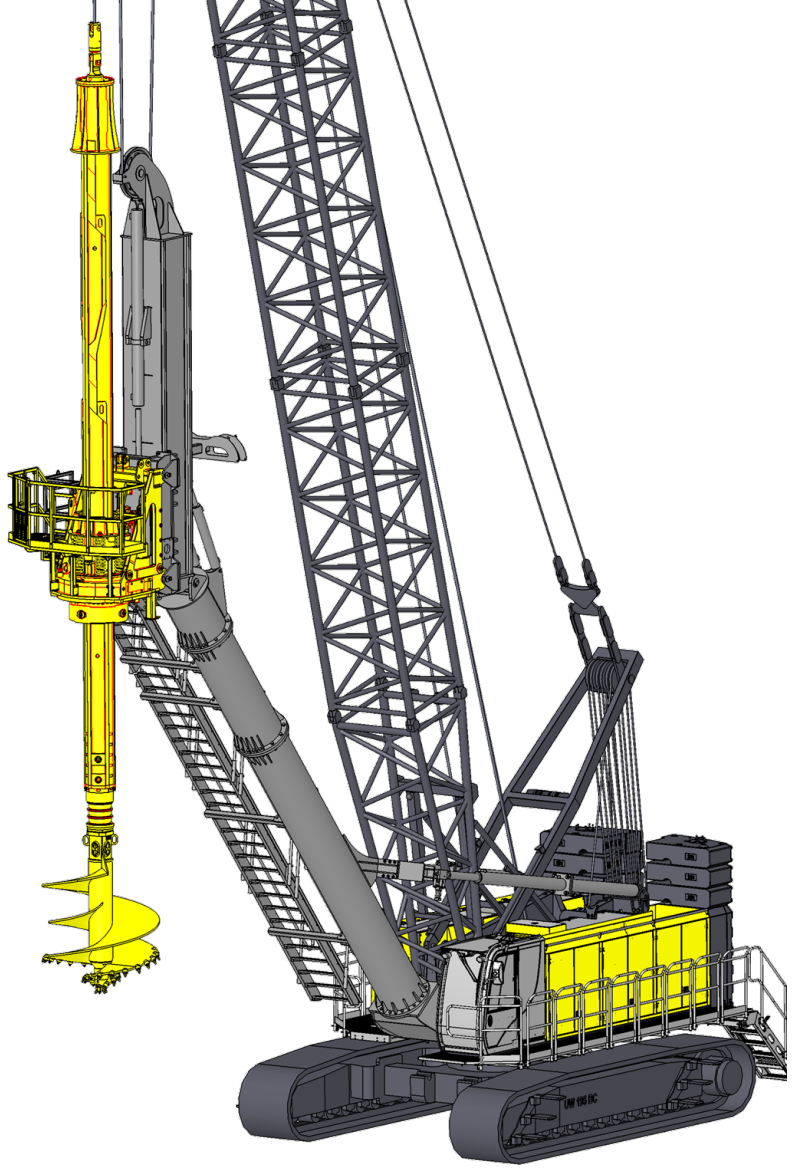


SCOPE & CONSTRUCTION SEQUENCE

PRODUCTION CYCLE

- Cannot initiate a new excavation adjacent to recently completed shaft within 24 hours
- Limited to one pile to completion at a time
- Primary – Secondary installation sequence





3 TECHNICAL CHALLENGES & DRILLING EQUIPMENT

TECHNICAL CHALLENGES

INSTALLATION FROM BARGES

- Requires detailed planning and logistics
- MOVING working platforms – stability of equipment



TECHNICAL CHALLENGES

DRILLING BARGE – MAIN WORK PLATFORM

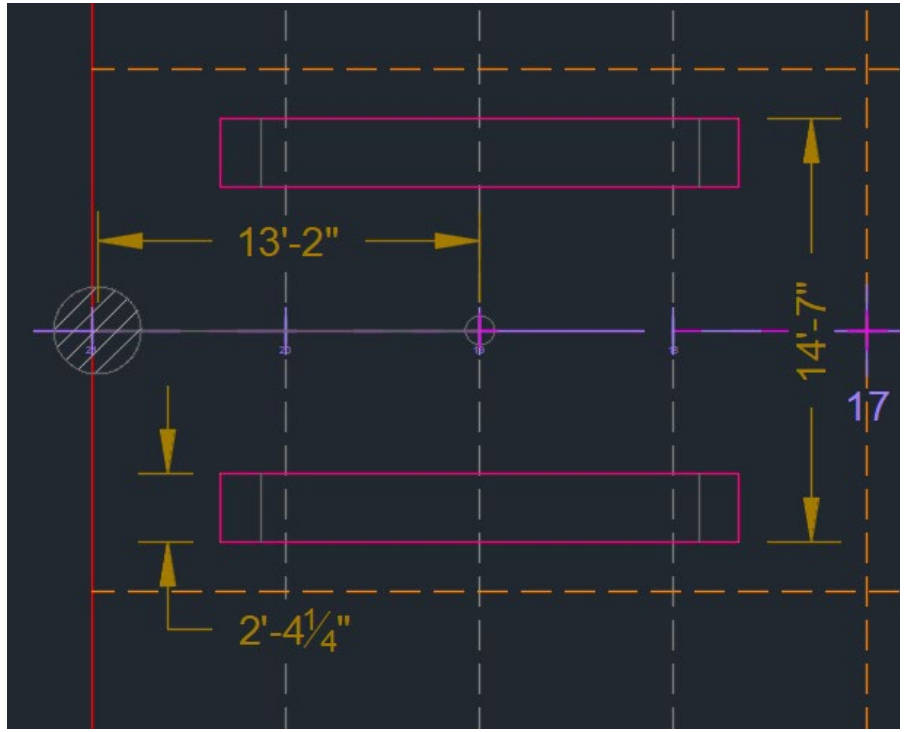
- 140 ft x 55 ft x 8 ft spud barge
  - (42.7 m x 16.8 m x 2.4 m)

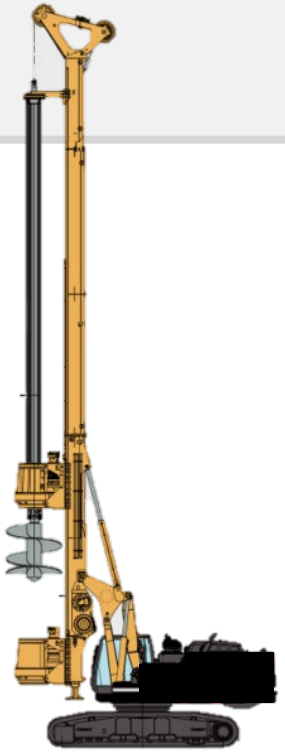


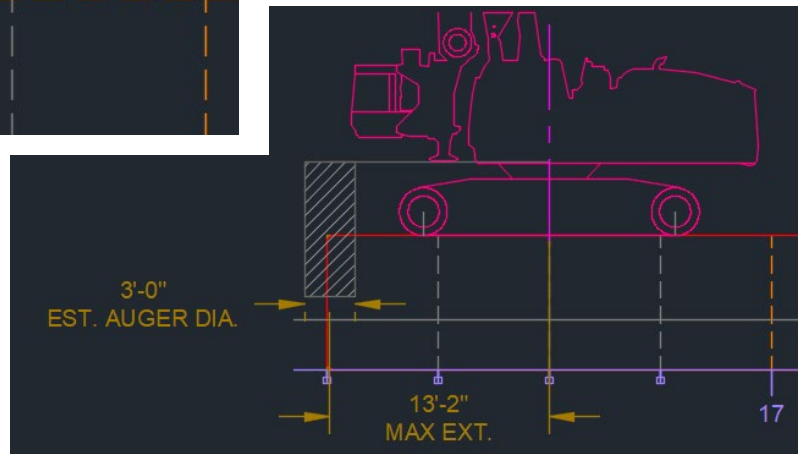
TECHNICAL CHALLENGES

EVALUATION OF STABILITY

- Traditional Drilled shaft piling rig
- Rig Location: Stability vs Pile Location
  - Not enough outreach to drill off the barge and maintain stability of machine and barge



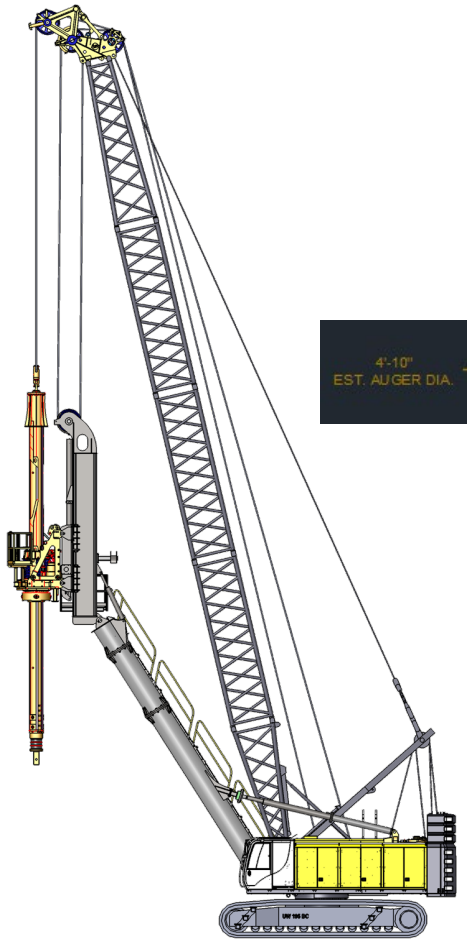


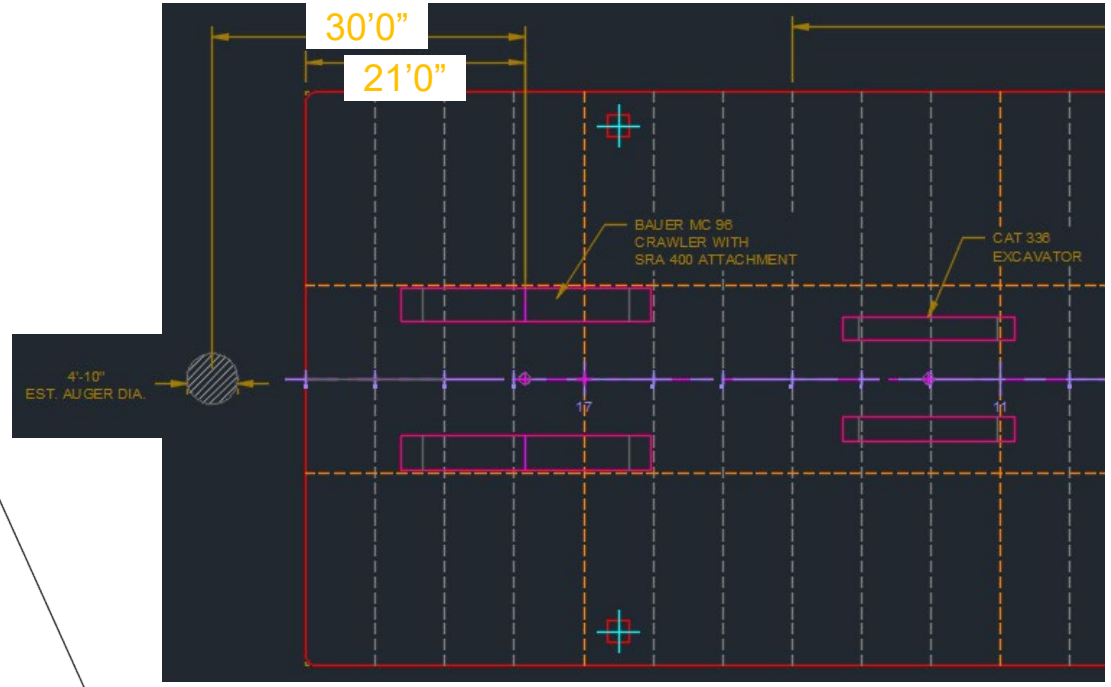


TECHNICAL CHALLENGES

IDENTIFYING ALTERNATIVES

- Super Reach Attachment (SRA)
- Integrates rotary drilling tools with flexible kinematics of a duty-cycle crane
  - Significantly extended outreach for drill axis
  - 30 ft (9.14 m) required from center pin for stability

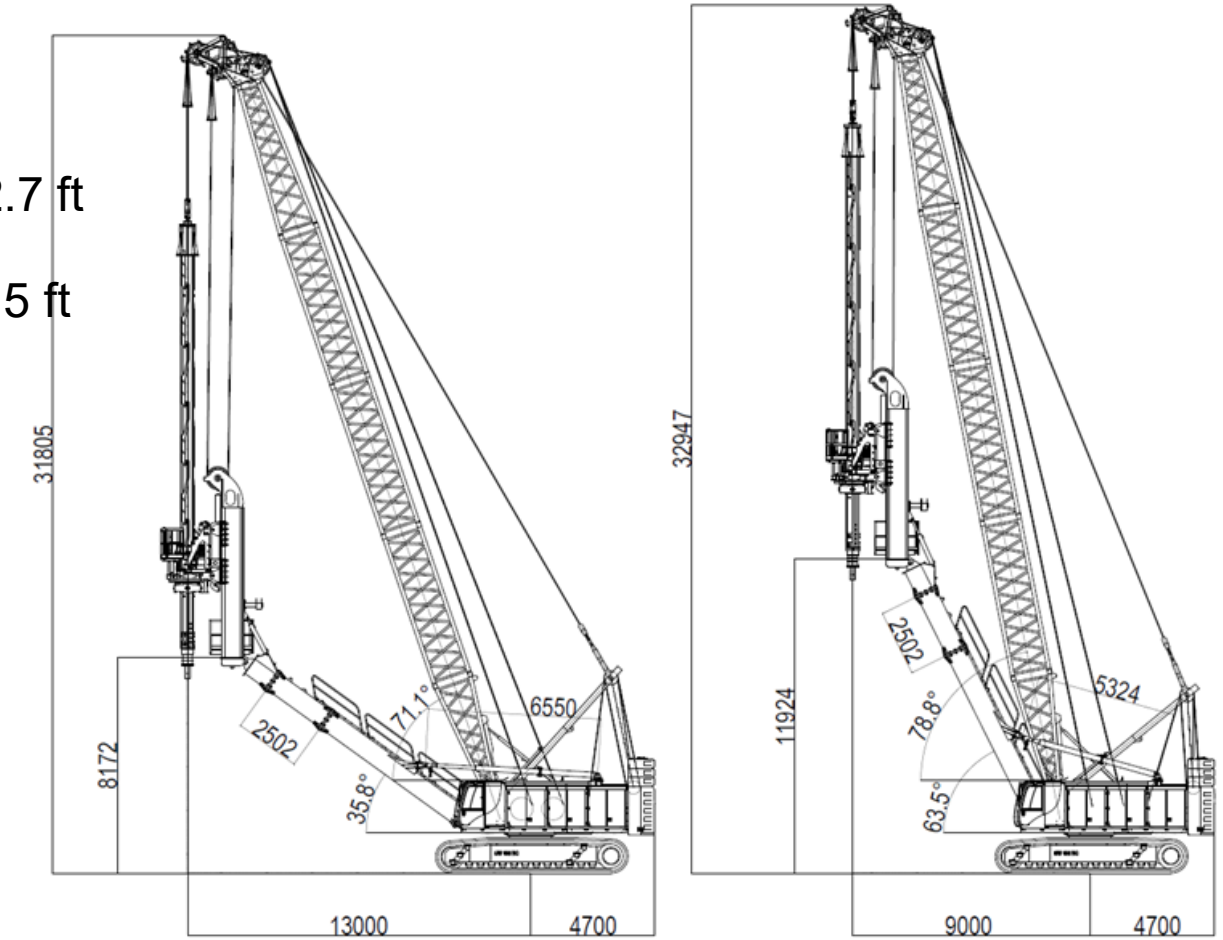




TECHNICAL CHALLENGES

THE SRA SOLUTION

- Super Reach Attachment (SRA)
  - Maximum outreach (w/ extension) 13000 mm = 42.7 ft
  - Minimum outreach (w/ extension) 9000 mm = 29.5 ft
- Mounted on MC 96 Duty- Cycle Crane



TECHNICAL CHALLENGES

THE SRA SOLUTION - ADVANTAGES

- MC 96 Crane + Super Reach Attachment (SRA)
  - Remain in stable location on barge and access all pile locations without movement of base carrier
  - Manufacturer provided support for adapting the system for site specific challenges

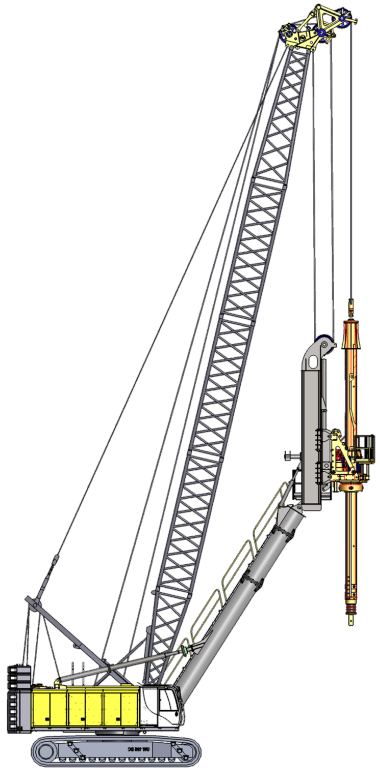


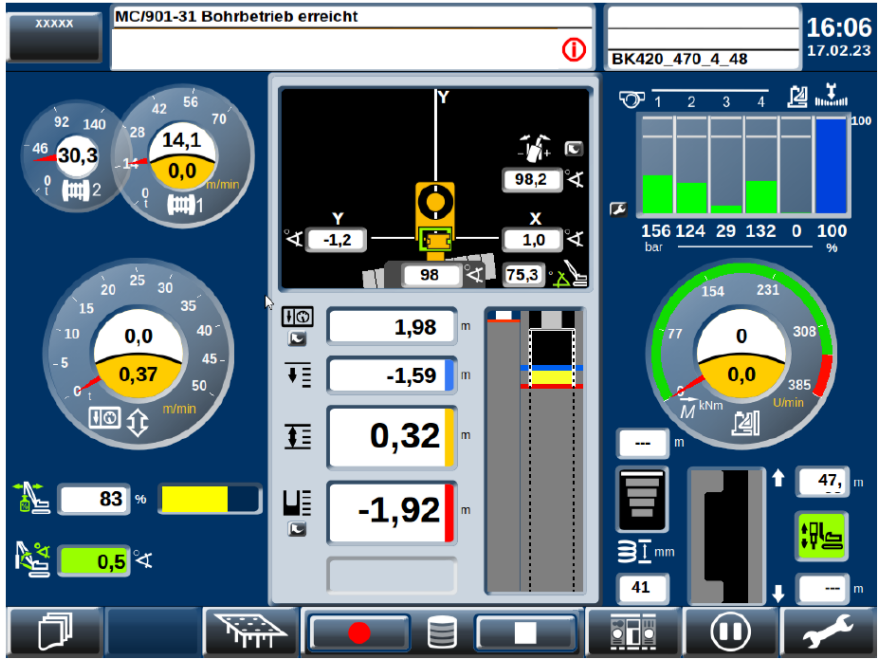


TECHNICAL CHALLENGES

THE SRA SOLUTION - ADVANTAGES

- SRA equipped with automated drilling assistant features
  - Enhance production quality
  - Reduce wear on tooling
  - Minimize mechanical stress on equipment and operator
  - Improved installation consistency and efficiency
  - Reduced maintenance needs and costs





TECHNICAL CHALLENGES

RIVER LEVELS AND FLOODING

- Flooded drilled shafts
- Production schedule impacts





TECHNICAL CHALLENGES

OBSTRUCTIONS

- Casing installation and drilling challenges due to obstructions encountered in the riverbed





4 CONCLUSIONS

CONCLUSIONS

COMPLEX ENVIRONMENTS REQUIRE…

- Innovation, flexibility, collaboration across disciplines
- Creative thinking and close coordination between
  - Designers
  - Engineers
  - Contractors
  - Equipment Manufacturers



Solving one operational challenge can sometimes lead to others.

CONCLUSION

SPECIAL THANKS TO:











PARALLEL SESSION C1
MSSP Case Studies



Helical Pile Foundation Replacement for Cross Rope Towers in the Energized 500kV Transmission Line

DANIEL RENOSTO

Disclaimer: "The opinions expressed in this presentation and on the following slides are solely those of the presenter(s) and not necessarily those of DFI. DFI does not guarantee the accuracy or reliability of the information nor endorse the recommendations offered by the presenter(s).

Summary

01. **Overview of the region**
02. **Overview of the problem**
03. **Existing foundations**
04. **Timeline**
05. **Challenge**
06. **Technical analyses**
07. **Geotechnical overview**
08. **Technical solution**
09. **Methodology, procedure**
10. **Conclusion and recommendations**



Overview of the region

Map



Geotechnical observations of the region

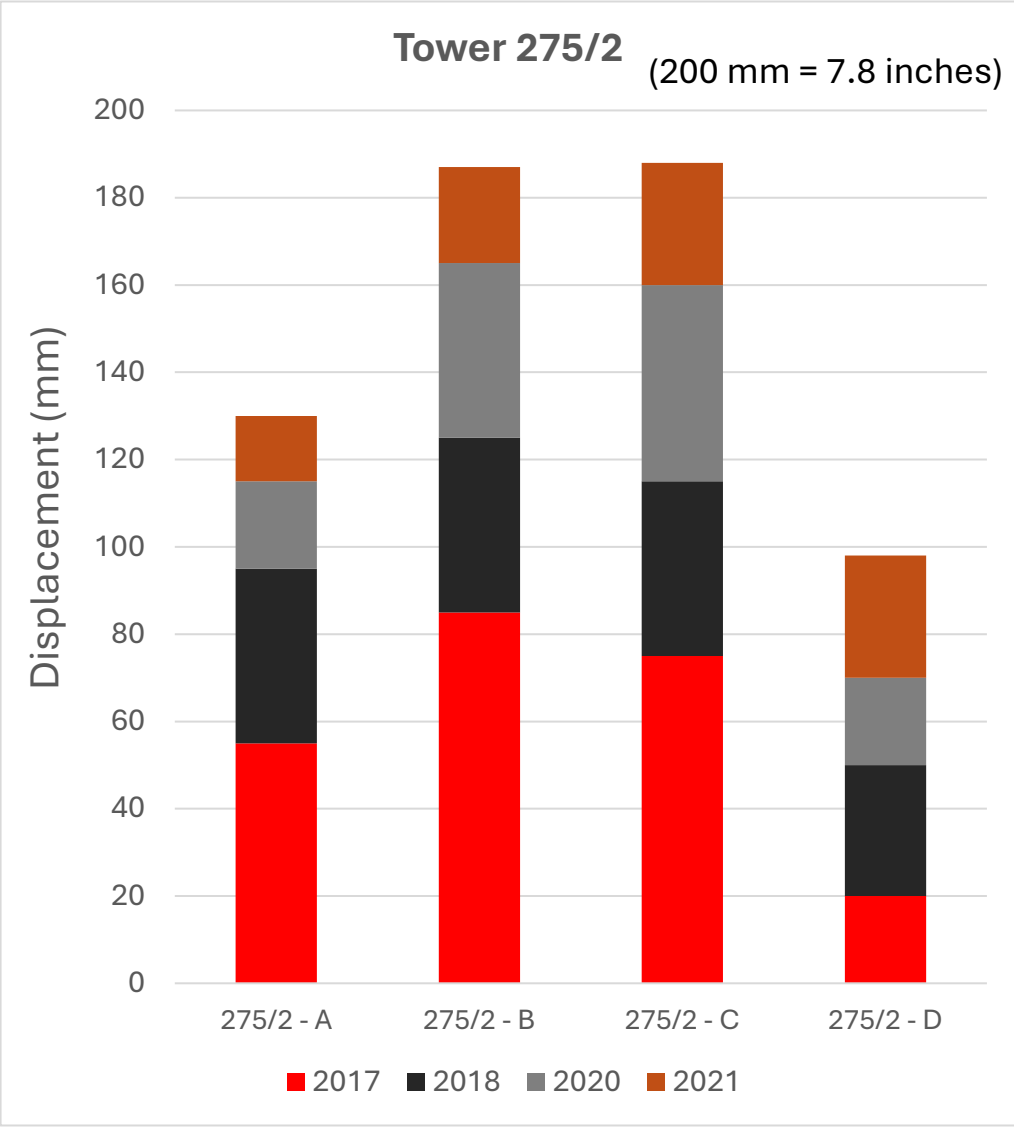
- Medium and fine-grained aeolian (wind-deposited) sands and silts
- Very dry region with little rainfall
- Soils are degraded by water erosion
- Drastic change in geotechnical properties when wet

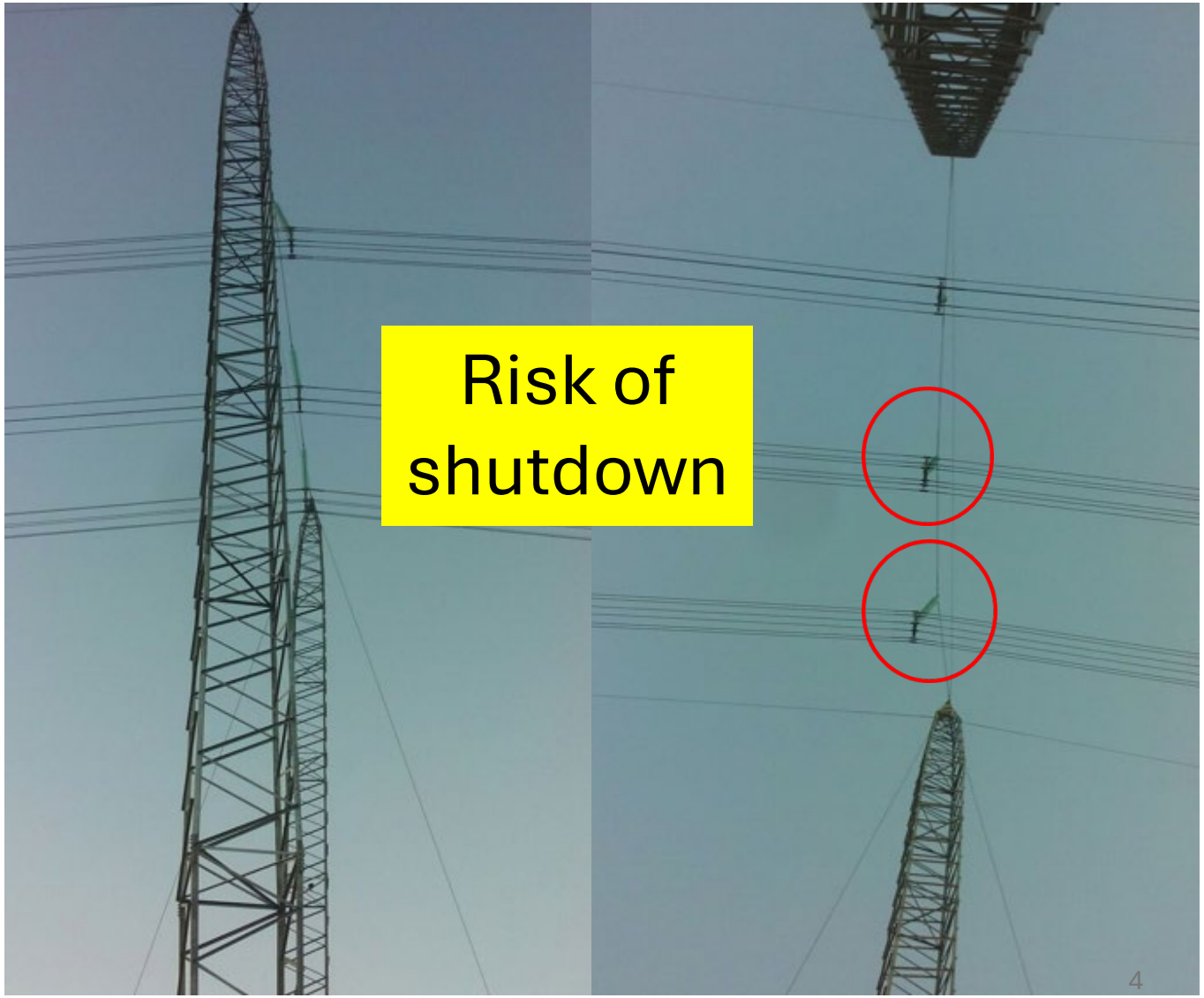




Overview of the problem

Misaligned electrical insulator due to the displacements of the piles





Existing foundation, Helical Piles

Table 1 - Transmission line details	
Total Structures	820
Extension of the Transmission line	328 km
Guyed Cross Rope Towers	697
Guyed Anchors	2788
Helical Pile Guy Anchors	414

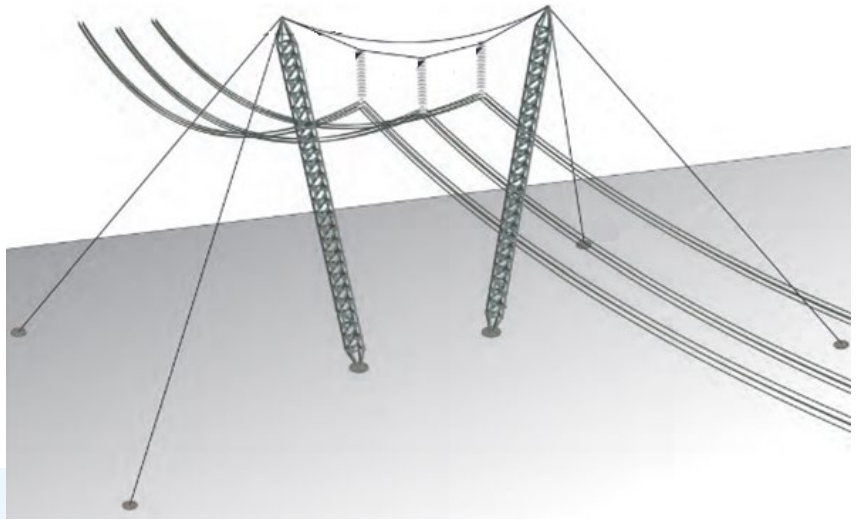


Table 2 - Helical Piles	
Average Length (m)	5,4 (18’)
Round Shaft (mm)	101,6 (4”)
Tickness (mm)	7,1 (.280”)
Helix 1 (mm)	254 (10”)
Helix 2 (mm)	304,8 (12”)
Helix 3 (mm)	355,6 (14”)
Helix 4 (mm)	355,6 (14”)
Project Torque kN.m	13,56 (10,000 ft-lb)
Tension load (kN)	342,7 (77 kips)
Angle (°)	34,664



Timeline of facts



BEGINNING OF THE FOUNDATIONS

POWER LINE OPERATION

2° RUPTURE, FAILURE

4° RUPTURE, FAILURE

CONSULTANCY

REINFORCEMENT START

FINAL REPORT

REINFORCEMENT PROJECT



Reinf. Period

September-15 November-15 January-16 March-16 May-16 July-16 September-16 November-16 January-17 March-17 May-17 July-17 September-17 November-17 January-18 March-18 May-18 July-18 September-18 November-18 January-19 March-19 May-19 July-19 September-19 November-19 January-20 March-20 May-20 July-20 September-20 November-20 January-21 March-21 May-21 July-21 September-21 November-21 January-22 March-22 May-22 July-22 September-22 November-22 January-23

FIELD TESTS

1° RUPTURE, FAILURE

3° RUPTURE, FAILURE

5° RUPTURE, FAILURE

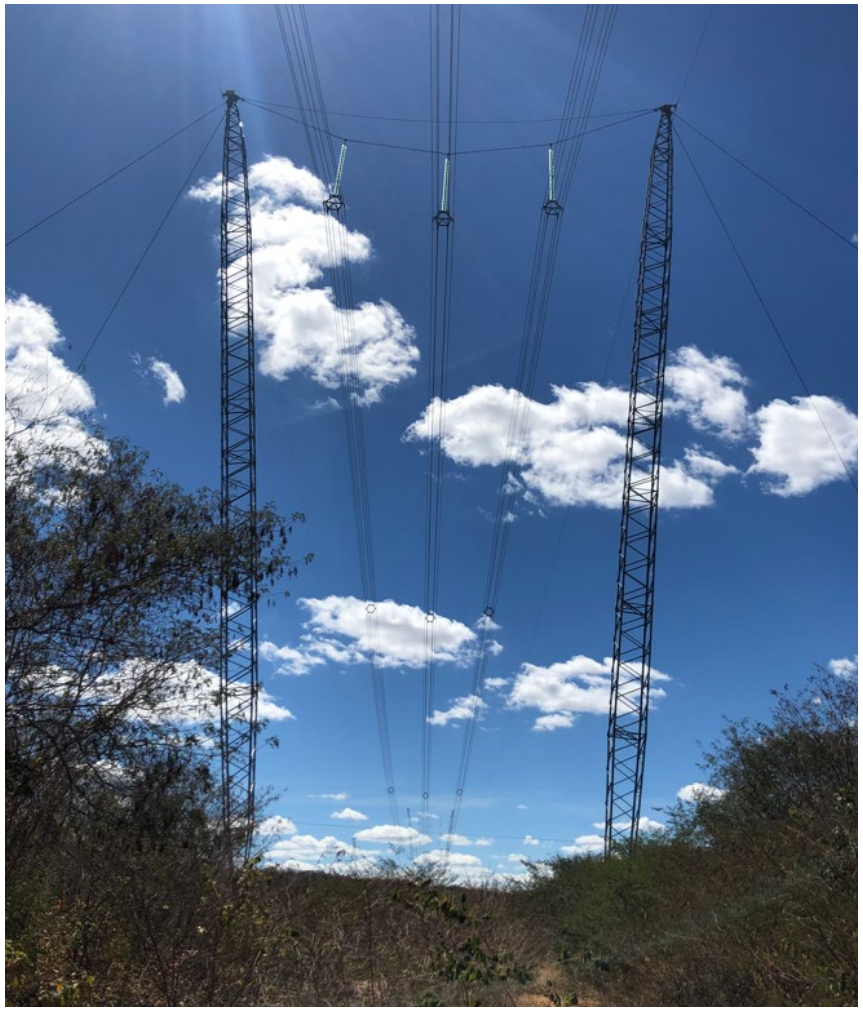
DOCUMENTS REVIEWS

TRAINING

REINFORCEMENT DONE

Challenge

How to reinforce the guy anchors of an energized transmission line?

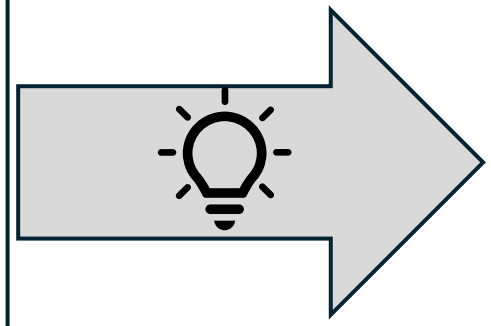








Using Helical Pile





Technical Analysis of the Problem

Existing Documentation

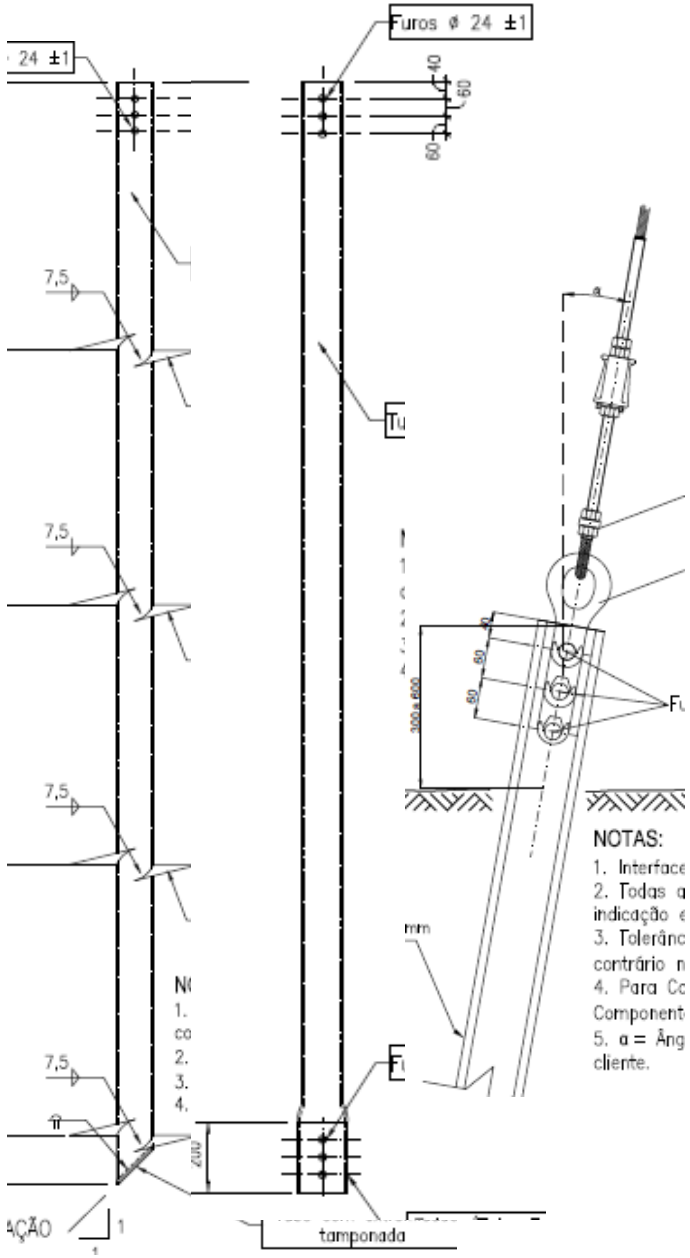
All existing project documentation (surveys, construction list, projects, installation boring logs, pull-out tests, etc.) was analyzed.
The goal was to understand all the variables we had and also look at the risks.

Structural Verification

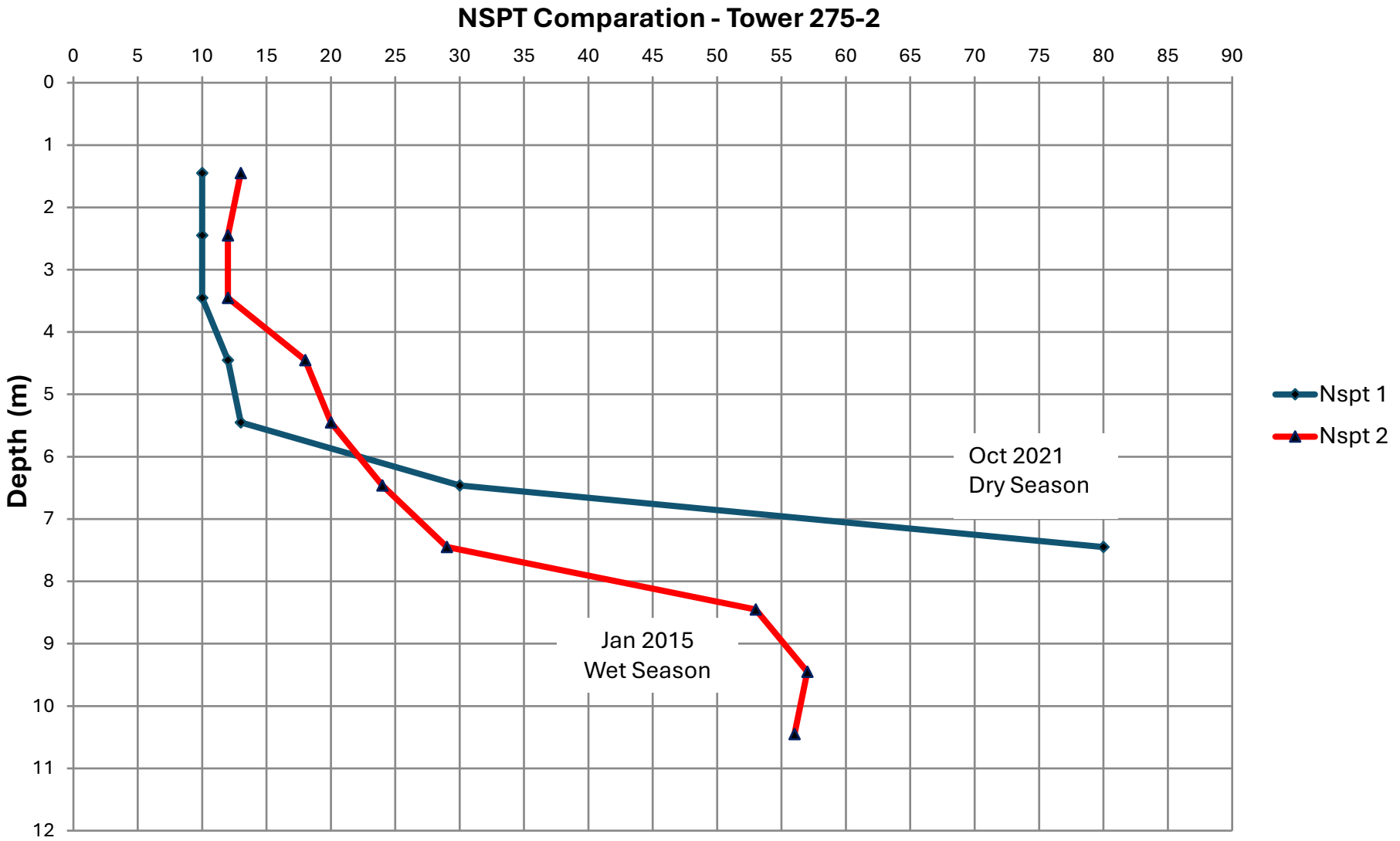
Inconsistencies were identified in the structural design of the helical pile with the Brazilian standards, including the adoption of higher structural values of yield strength (Fy) and rupture strength (Fu) of the pile, without tests that justified them.

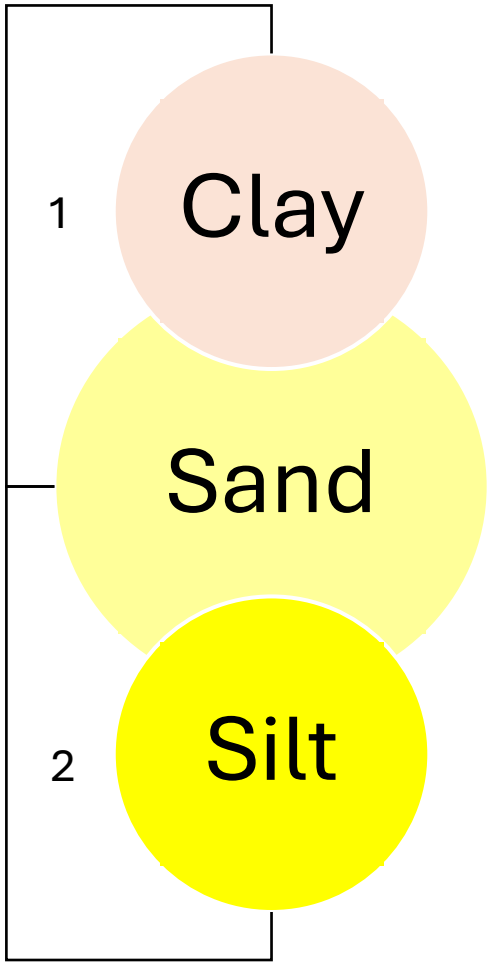
Geotechnical Verification

It was found that the designer had no geotechnical verification of the pile. The analyses concluded that, geotechnically, the pile used in the project did not have the capacity to support the loading of the tower.
S.F.= 0.36 to 0.59

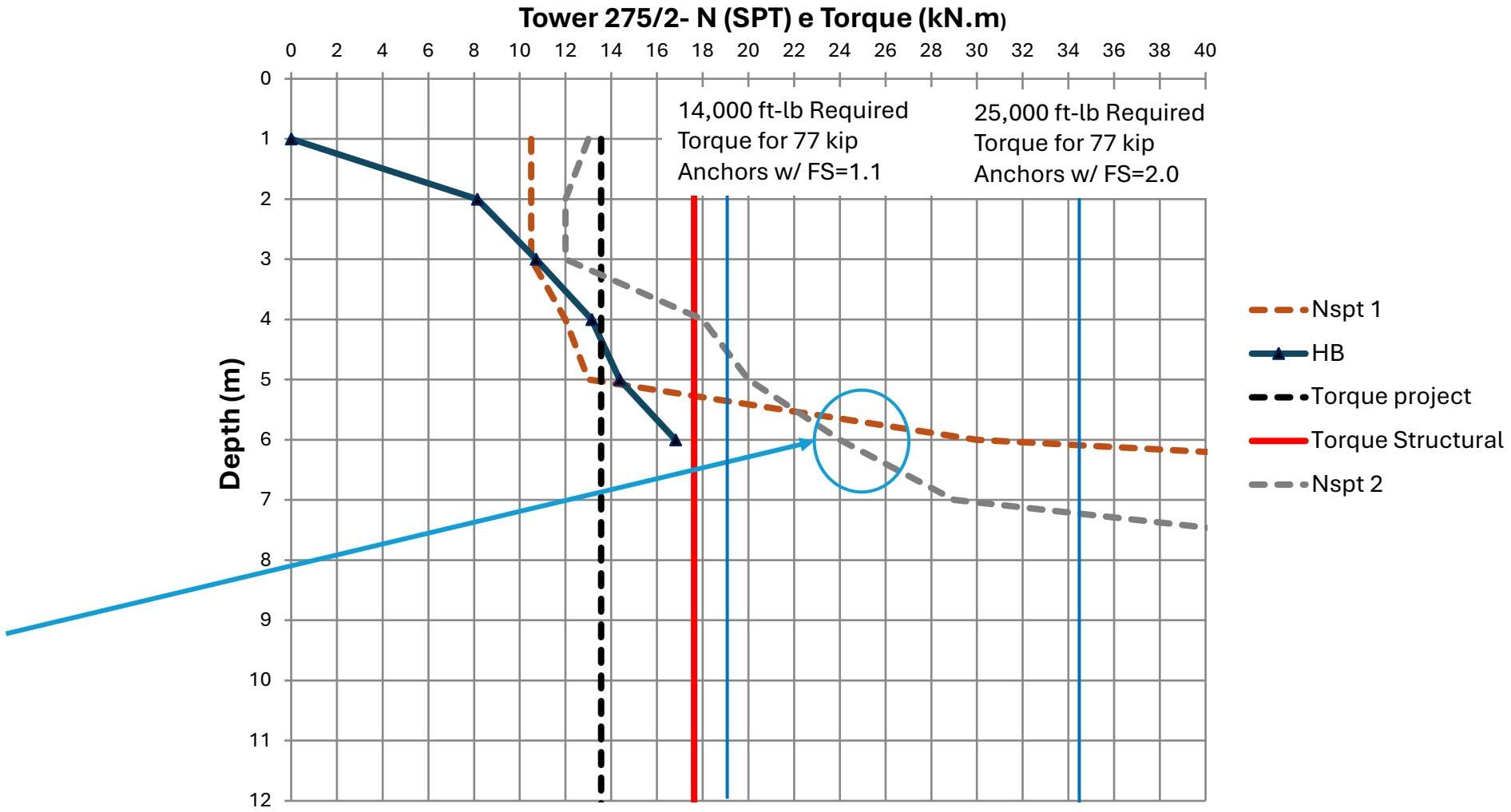


Boring logs overview





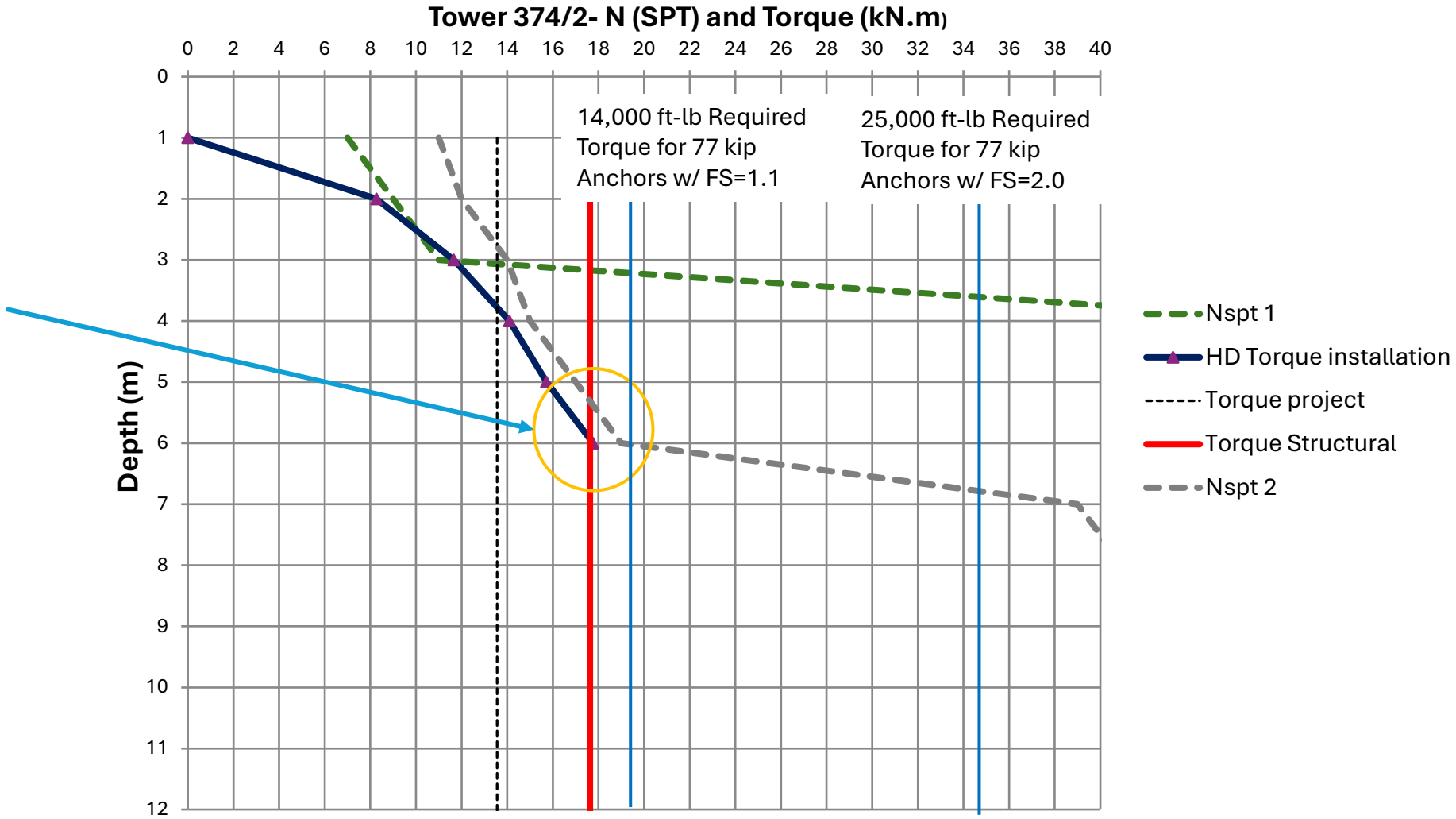
Analysis of Helical Piles Installations, Tower 275/2



It is observed that with N24 - N25 the pile is very close to the structural torque.

Analysis of Helical Piles Installations, Tower 374/2

Torque matched the Nspt 2 resistance and has reached the structural limit of the round shaft to guyed anchor HD.



Collapsible soils

To better understand the potential geotechnical problems, we performed some laboratory tests looking for the following questions:

- Triaxial compression shear tests;
- Study of the potential of collapsible soils;

Densified and non-drained Triaxial compression strenght test	Consolidation test
• Sample compaction • Saturation • Densification • Shear	• Vertical loads • Confined samples • Load cell flooding • Loading and unloading

Densification test

Vargas (1978), to identify the collapsible behavior of soils, considers that a soil is collapsible when the PC value is greater than 2%, according to Equation (01):

$$PC = \frac{\Delta e}{1 + e_0} X100\ (\%)$$

PC = collapsible potencial

Δe - variation of the void index of the specimen due to collapse;

e_0 – initial void index;

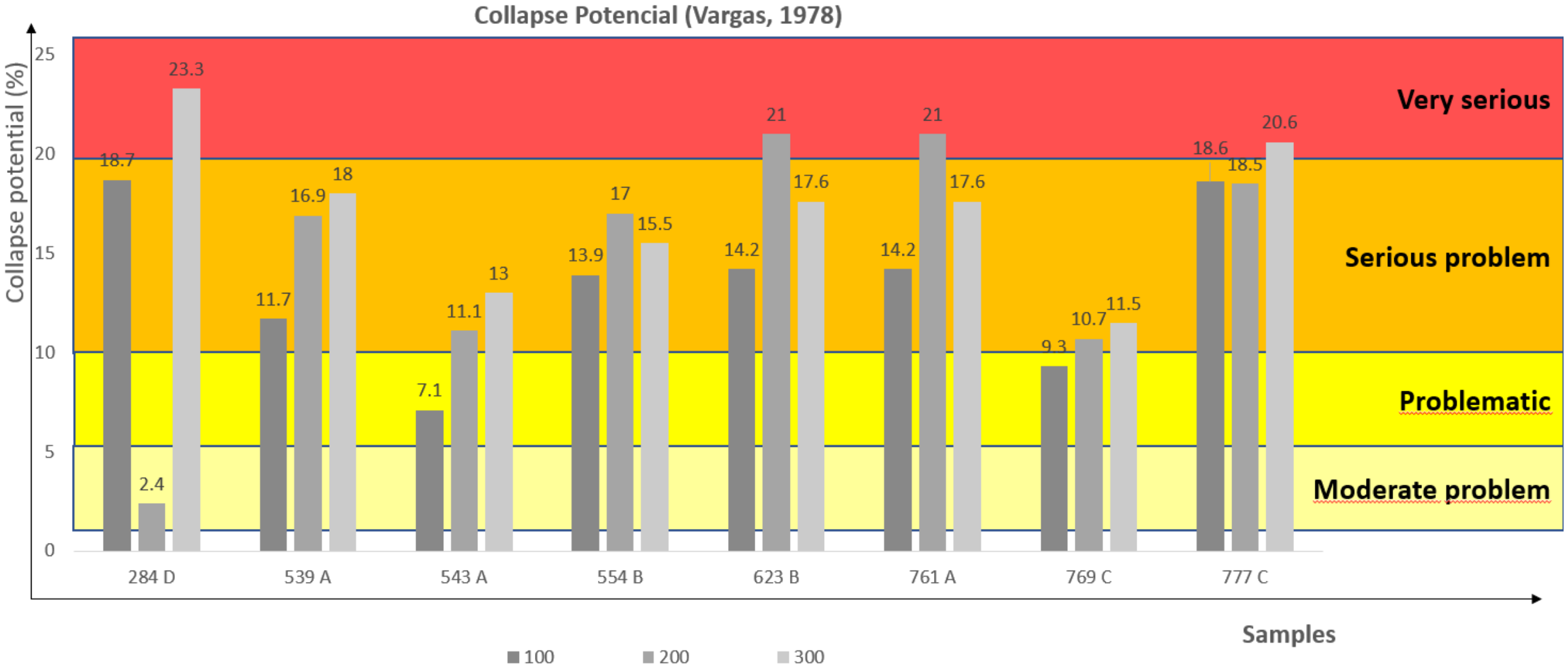
Laboratory equipment



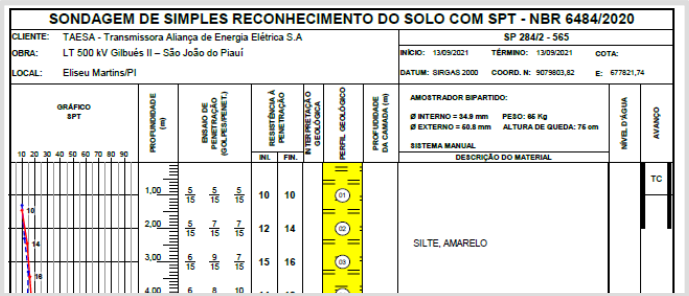
PC (%)	Problem severity
0 - 1	No problem
1 - 5	Moderate problem
5 - 10	Problematic
10 - 20	Serius problem
> 20	Very serius problem

Jennings & Knight (1975)

Coppalsible soils, Jennings & Knight (1975)



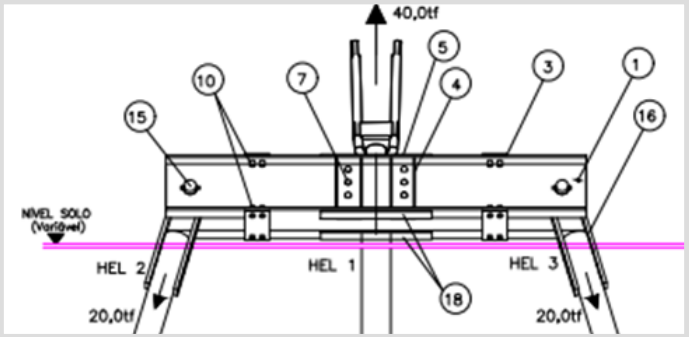
Technical Solution


SONDAGEM DE SIMPLES RECONHECIMENTO DO SOLO COM SPT - NBR 6484/2020

CLIENTE: TAESA - Transmissora Aliança de Energia Elétrica S.A
OBRA: LT 500 kV Gilbués II – São João do Piauí
LOCAL: Eliseu Martins/PI
SP 284/2 - 565
INÍCIO: 13/09/2021 TÉRMINO: 13/09/2021 COTA:
DATUM: SIRGAS 2000 COORD. N: 9079803,82 E: 677821,74

GRÁFICO SPT | PROFUNDIDADE (m) | ENSAIO DE PENETRAÇÃO (GOLPES/PENET.) | RESISTÊNCIA À PENETRAÇÃO INI. FIN. | INTERPRETAÇÃO GEOLÓGICA | PERFIL GEOLÓGICO | PROFUNDIDADE DA CAMADA (m) | AMOSTRADOR BIPARTIDO: Ø INTERNO = 34.9 mm PESO: 65 Kg Ø EXTERNO = 50.8 mm ALTURA DE QUEDA: 75 cm SISTEMA MANUAL | NÍVEL D'ÁGUA | AVANÇO

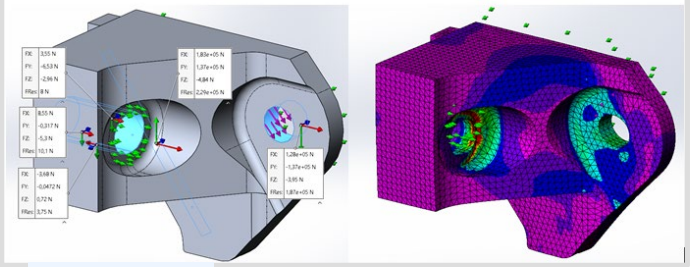
PROFUNDIDADE (m)	ENSAIO DE PENETRAÇÃO (GOLPES/PENET.)	RESISTÊNCIA À PENETRAÇÃO INI.	RESISTÊNCIA À PENETRAÇÃO FIN.	DESCRIÇÃO DO MATERIAL
1,00	5 / 5 / 5	10	10	SILTE, AMARELO
2,00	5 / 7 / 7	12	14	
3,00	6 / 9 / 7	15	16	
4,00	6 / 8 / 10			



ORDEM DE PRIORIDADE DE REFORÇO

Locação Estrutura - Torre Tipo GE2 - Estaiada

Nº (Seq.)	Nº (Proj.)	Coordenadas UTM (m) WGS84 FUNDAÇÕES DE ESTAIS EM ESTACA HELICOIDAL						
		X (Este)	Y (Norte)	A	B	C	D	PRIORIDADE REFORÇO
124	60/1	516.051,45	8.943.742,13	X	X			ESTÁVEL
125	60/2	516.341,24	8.943.881,17	X	X			ESTÁVEL
284	140/2	576.194,33	8.993.834,82	X	X	X	X	ULTRA-GRAVE
530	266/1	659.708,49	9.079.241,07	X	X	X	X	GRAVÍSSIMO
531	266/2	660.193,84	9.079.400,92	X	X	X	X	GRAVÍSSIMO
532	267/1	660.705,79	9.079.569,54	X	X	X	X	GRAVE
533	267/2	661.209,19	9.079.735,34	X	X	X	X	GRAVE
534	268/1	661.717,34	9.079.902,70	X	X	X	X	GRAVÍSSIMO
536	269/2	662.747,43	9.080.061,01	X	X	X	X	GRAVÍSSIMO
537	270/1	663.282,36	9.080.051,88	X	X	X	X	SUPER-GRAVE



Geotechnical Investigation

New boring logs were carried out for comparison with the existing ones, in addition to laboratory tests (granulometric analysis, LL, LP, compaction, expansion, shear, collapsible potential, among others) for geotechnical analysis.

Reinforcement Project

A solution was developed with two additional helical piles interconnected with the existing helical pile, distributing the tensile load on the ground. We developed a bracket to connect the piles to the tower.

Risk assessment

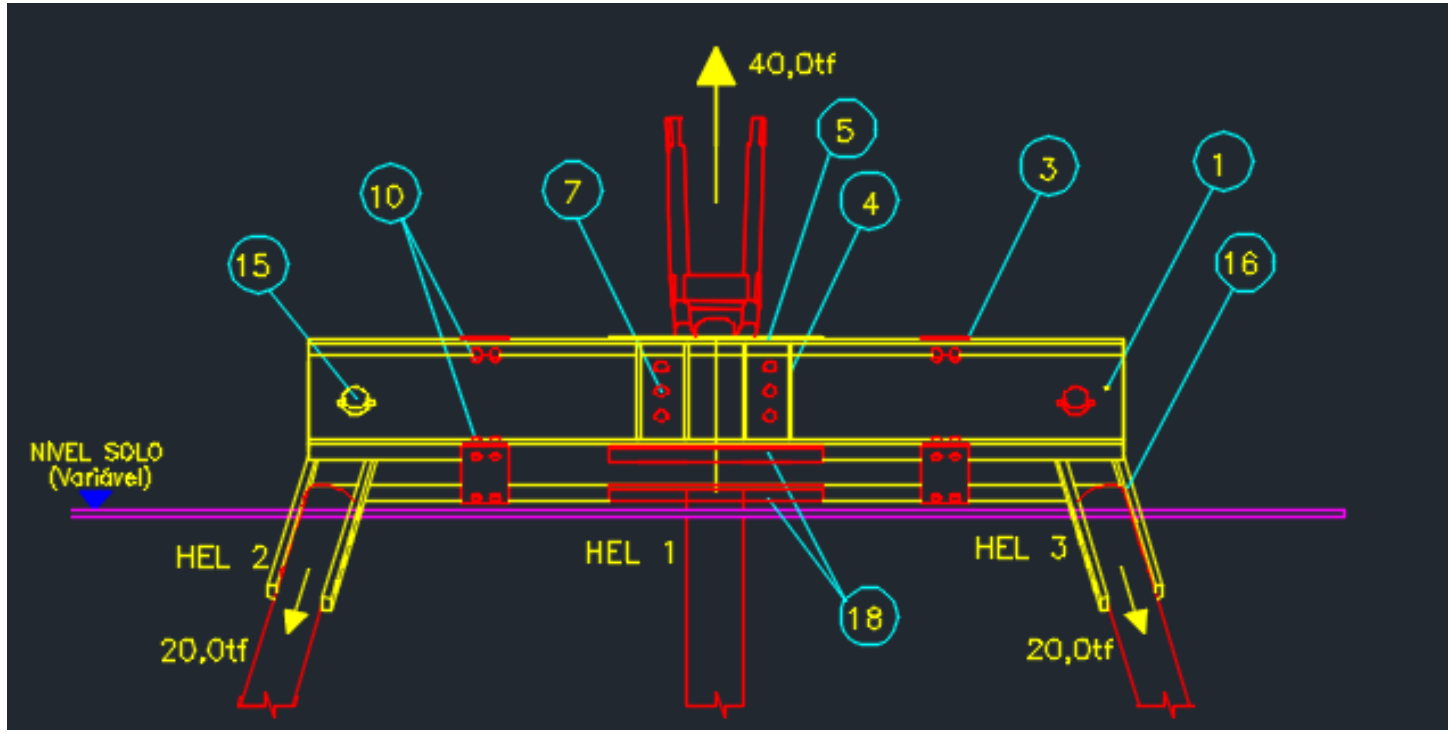
A list was developed with executive priority criteria, defining four classes: VERY SERIOUS, SERIOUS PROBLEM, PROBLEMATIC, and MODERATE PROBLEM, based on geotechnical tests and analyses.

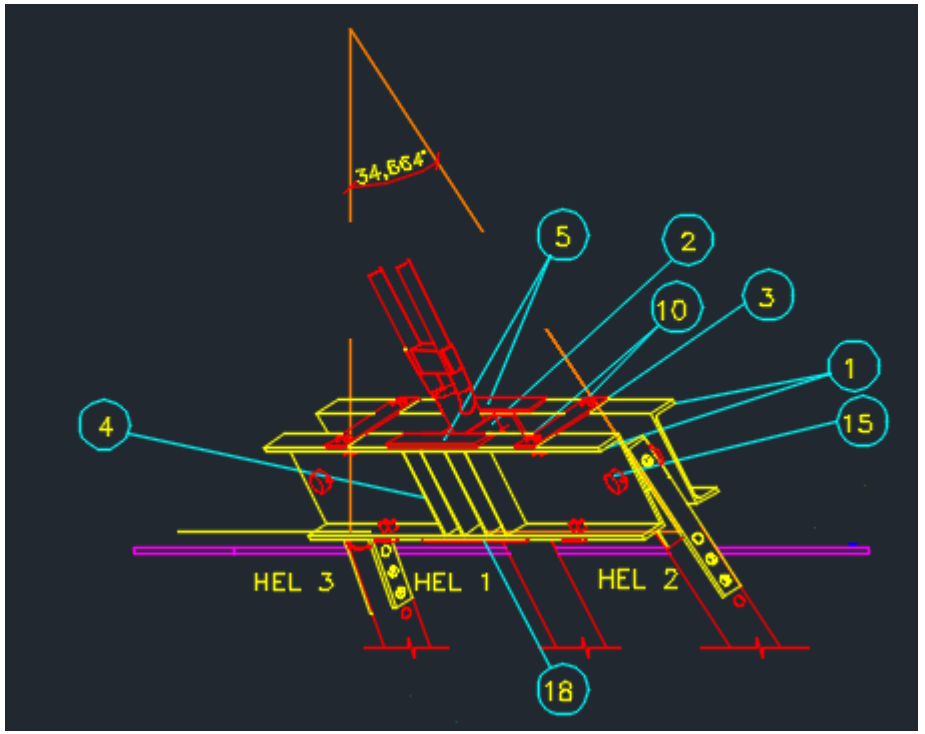
Innovative Tool

A connection device was created with the temporary foundations so that all work can be carried out safely with the transmission line energized.

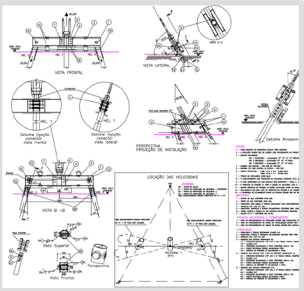
Bracket, device

- The existing pile wasn’t considered in the calculation.



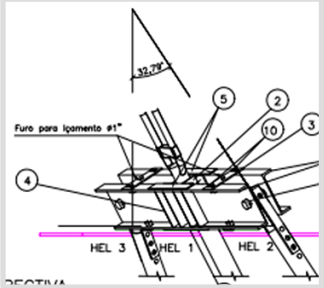


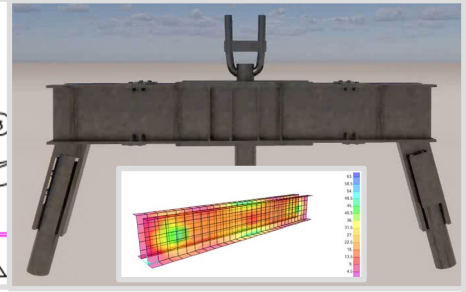
Technical Solution



Triple Helical pile Assembly Design

Installed two new helical piles to meet the necessary safety factor.
We obtained a safety factor of **1.6** for the foundation.
We run load tests to validate the solution.

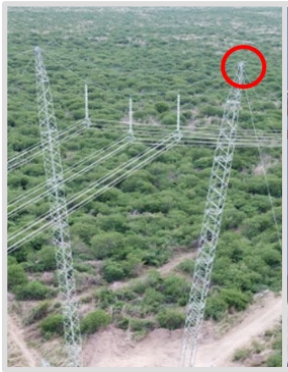


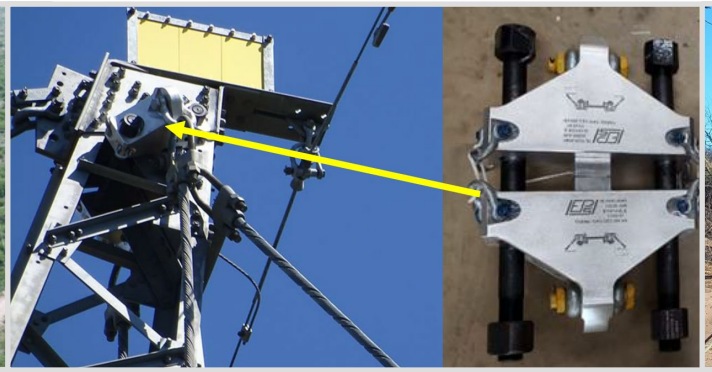




Mechanical Connection

Piles connected to the existing pile through a mechanical device (bracket).







Load Transfer

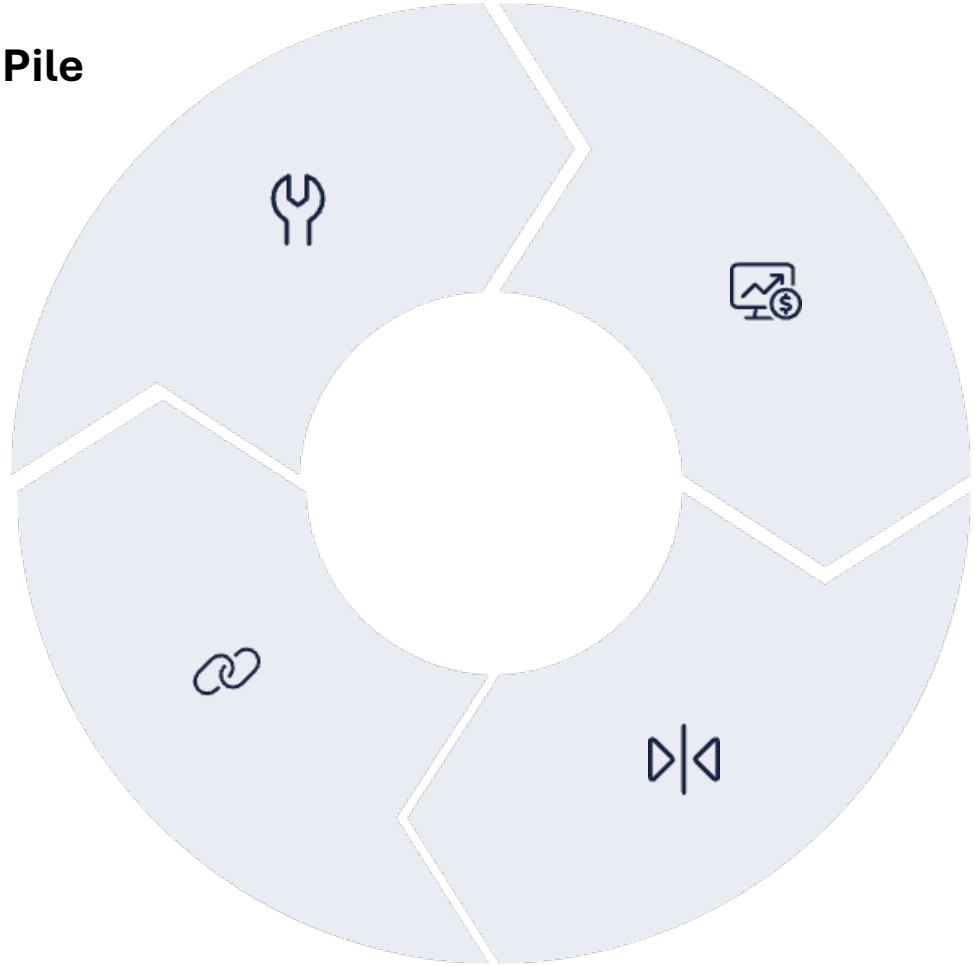
Carrying out the transfer of permanent load to the "*backup foundation*" and subsequent transfer to the definitive foundation.

This solution with this procedure was the first in the world to be carried out on an energized 500kV power transmission line, representing a significant advance in the maintenance of transmission lines without interruption of supply.

Methodology, procedure

1 - Tool Installation and Backup Pile

Installation of the yoke on the tower and the pile 15m from the original point with steel cable fixed to the tower through the yoke and carrying out a routine quick test on the pile.



2 – Load Transfer

Transfer of the load to the *backup pile*, through the new cable and accessories installed with a connection to the tower through the developed tool.

3 - Installation of the New Piles

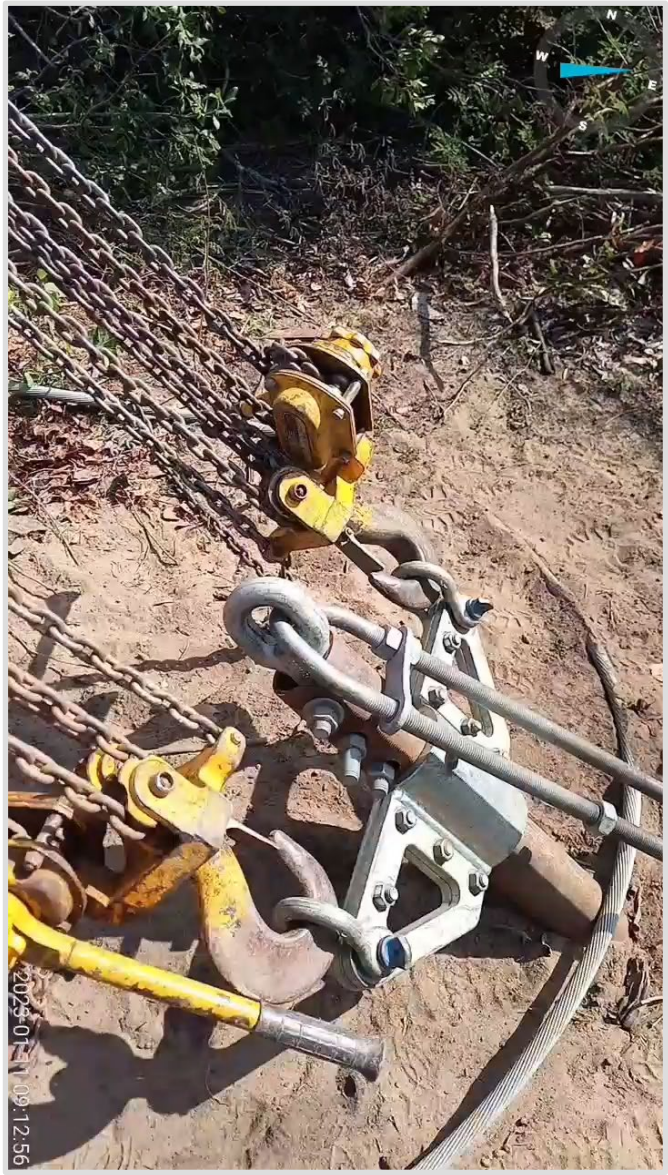
Installation of the two new piles, making any adjustments to the existing pile, and carrying out a routine test on one of the new piles.

4 – Bracket Assembly and Load Transfer

Execution of the assembly of the Steel beam and final load transfer. Disconnection of the temporary cable, accessories, and removal of the tool and backup pile to reuse.

Methodology, procedure











Innovative Challenges and Solutions

Digital Mapping

Development of a form in *Survey123* and a dashboard in *ArcGIS* to digitize the data collected by the maintenance team, facilitating the planning of interventions, real-time monitoring and optimization of time and resources.

Innovation in Security

Development of the “Tool" device, designed to enable safe load transfer between foundations, a world first in energized 500kV transmission lines, and application of reusable backup piles in 100% of new foundation assemblies.

Internal Training

40-hour training program for the inspection team, with theoretical and practical classes conducted by helical piles specialists, mitigating the lack of qualified labor in the market.

Strict Quality Control

Monitoring of our inspection in 100% of the routine tests carried out in the installed sets and backup pile of all phases by the team.

Weather Challenges

The job was carried out over 13 months, going through different weather conditions, variable loads in function of the winds, etc.

Conclusions and Recommendations

- Large-scale projects, such as transmission lines, require understanding the different geotechnical conditions in which the foundations will be installed.
- Load tests are an important tool in the design and installation phases of foundations.
- The success of any work is associated with the qualification of the team.
- Safety, quality, and production have been our principles of work.
- The use of Helical Piles in transmission lines is an excellent tool, provided they are developed, manufactured, and installed by experienced teams.
- Analyzing the foundation installation data during the construction phase could have prevented the problem.

Thank you!!!



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DATA MANAGEMENT FOR QUALITY CONTROL

Darlington New Nuclear Project

Stephen Ross
Construction Data Solutions

Located in Clarington, Ontario

Canada’s first commercial grid scale Small Modular Reactor (SMR)

The first of four planned SMRs

Owner: Ontario Power Generation (OPG)

Design: Atkins Realis

Project Delivery: OPG, Aecon, Kiewit and GE

ABOUT THE PROJECT

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PROJECT DETAILS & CHALLENGES

- Over 700 piles, 250 caissons, 8 structures, 3 secant walls
  - Pile diameters: 4 - 8 ft (1.2 - 2.4 m)
  - Pile lengths: 75 - 100 ft (23 – 30 m)
- 2 year project duration
- Significant oversight into piling operations due to:
  - First stage of construction
  - Strict schedule adherence
  - Nuclear level quality standards
- Management team comprised of 6 companies



OUR SCOPE

Subsurface Geotech (SSG) was retained for the quality verification engineering (QVE) scope

CDS was brought in to build a data management system to address these challenges

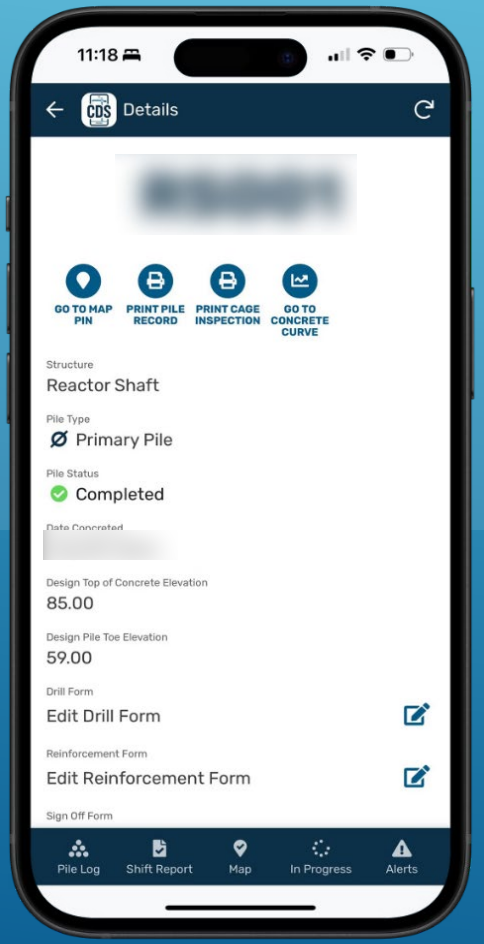


- Data capture and movement between project teams
- Intentional design and control of the flow of data
- Built-in data cleanup and validation
- Every user gets the data they need
- Accessible anywhere

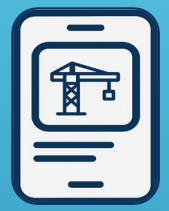
WHAT IS A DATA MANAGEMENT SYSTEM?

WHAT IS IN THE DATA MANAGEMENT SYSTEM?

Custom Mobile Application



- ✓ Collect and visualize data
- ✓ Ensure accuracy and completeness
- ✓ Provide digital tools for informed decisions
- ✓ Preliminary access to measurements
- ✓ iOS, Android and web app

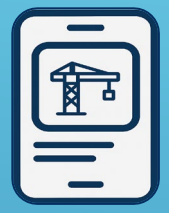


Custom Mobile Application



Database

- ✓ Single source of truth
- ✓ Scalable
- ✓ Live connection to the app
- ✓ Bulk import of design changes
- ✓ All data is available in digital format



Custom Mobile Application

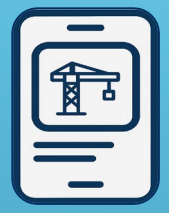


Database



Shared Project Folder

- ✓ Cloud-based file storage (SharePoint, Google Drive)
- ✓ Stores drawings, specifications, site photos, shift reports, and
- ✓ Permission-based user access
- ✓ Always audit-ready



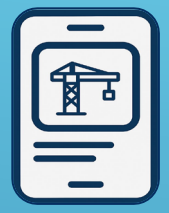


Automation





- ✓ Auto-fill PDF reports
- ✓ Organize the project folder and filing of new documents
- ✓ Send shift reports and DWRs
- ✓ Identify potential areas of concern and alert the team





Automation

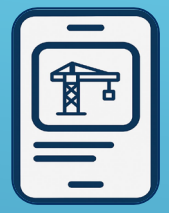




Data Analysis



- ✓ Ground conditions
- ✓ Trends in concrete quality and delays
- ✓ Drilling production
- ✓ Earned value measurement





Automation





Data Analysis



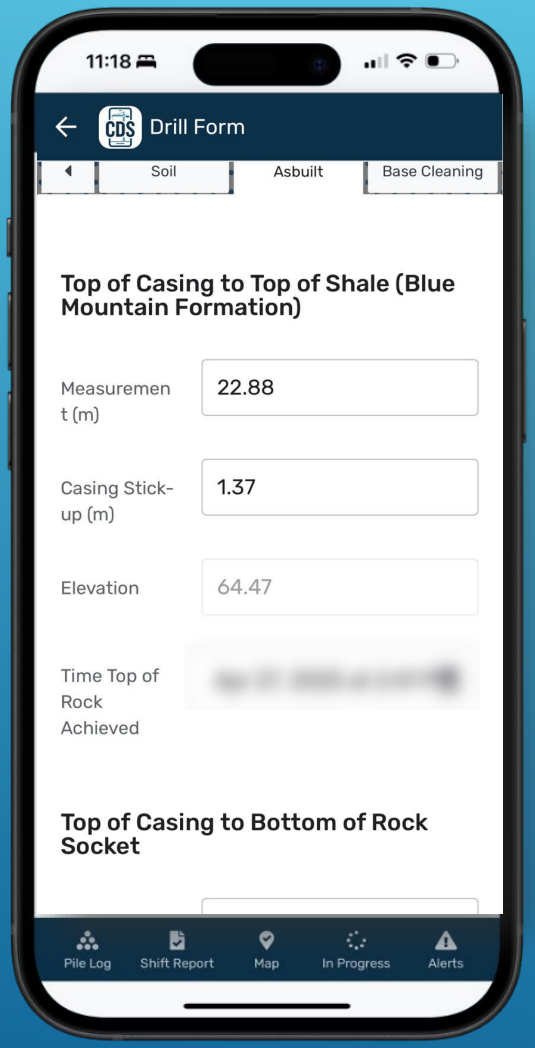


External Connections

- ✓ Client databases and shared drives
- ✓ External dashboard
- ✓ Project schedule (P6)
- ✓ 3D model
- ✓ Email distribution lists

DATA COLLECTION EXAMPLES

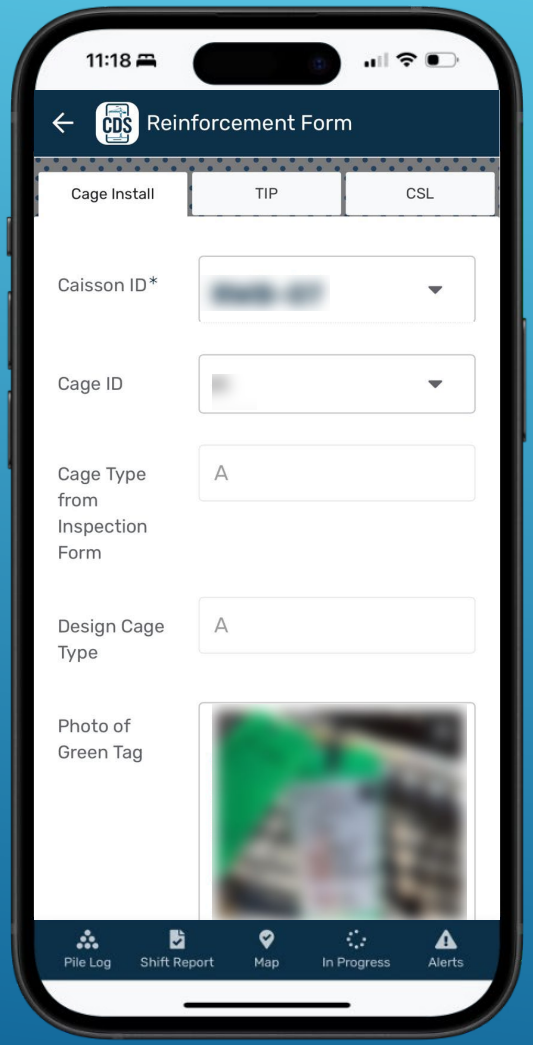
- ✓ Elevation monitoring
- ✓ Verticality Control
- ✓ Casing sequence
- ✓ Soil profile
- ✓ Polymer testing
- ✓ Base Inspection



Drilling

DATA COLLECTION EXAMPLES

- ✓ Rebar Cage Inspection
- ✓ TIP Wire QC
- ✓ CSL QC
- ✓ Cage Position Monitoring
- ✓ Cage Asbuilt Record

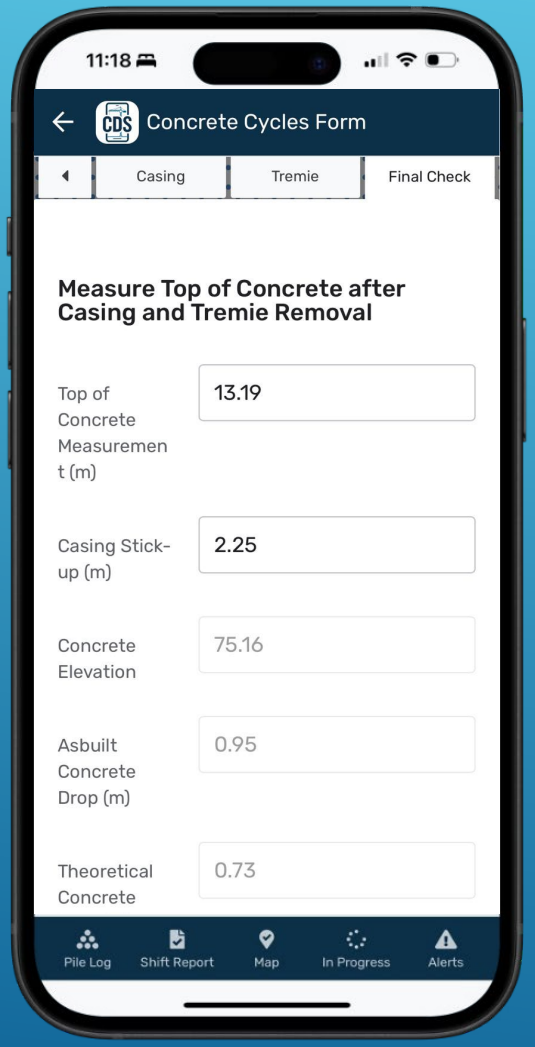


Drilling

Reinforcement

DATA COLLECTION EXAMPLES

- ✓ Concrete ticket logging
- ✓ Concrete testing
- ✓ Pour log
- ✓ Drop analysis
- ✓ Tremie records
- ✓ Consumption analysis



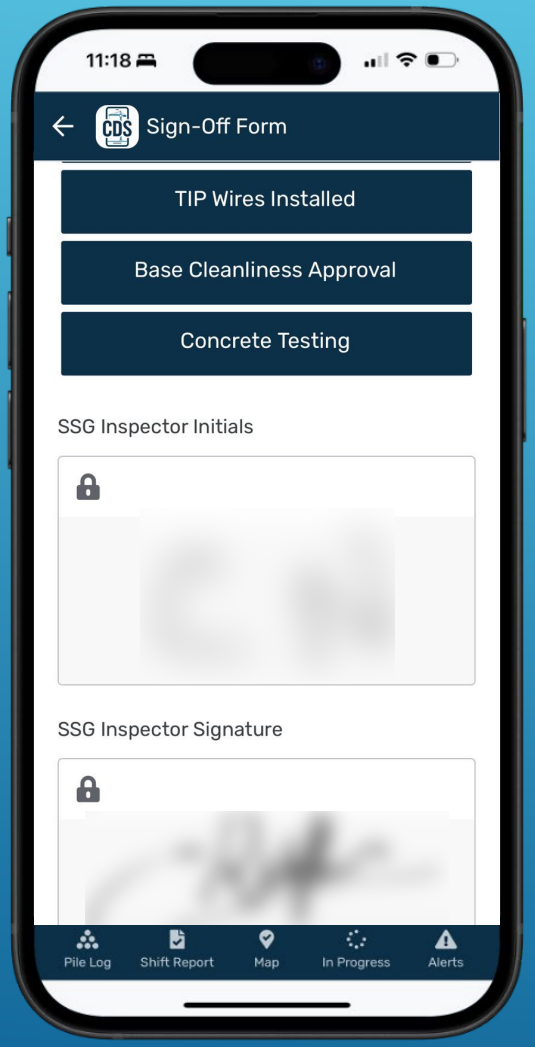
Drilling

Reinforcement

Concreting

DATA COLLECTION EXAMPLES

- ✓ Records sign-off
- ✓ Daily reporting
- ✓ Photos
- ✓ Alerts
- ✓ Timesheets



Drilling

Reinforcement

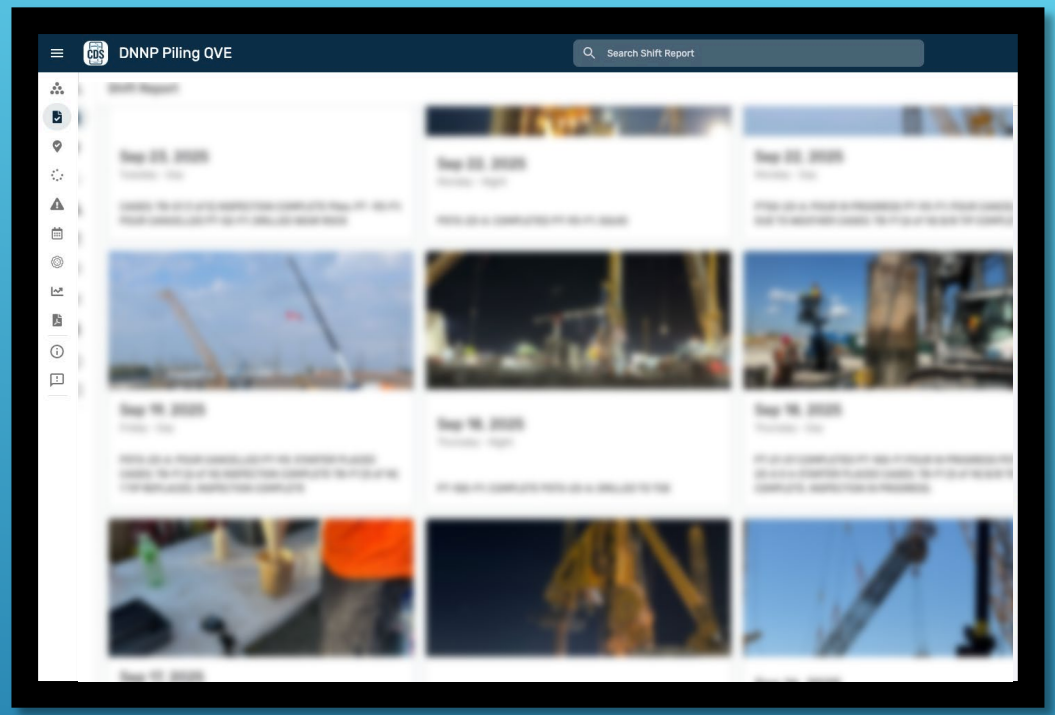
Concreting

Sign-Off

SOME OF OUR DIGITAL TOOLS

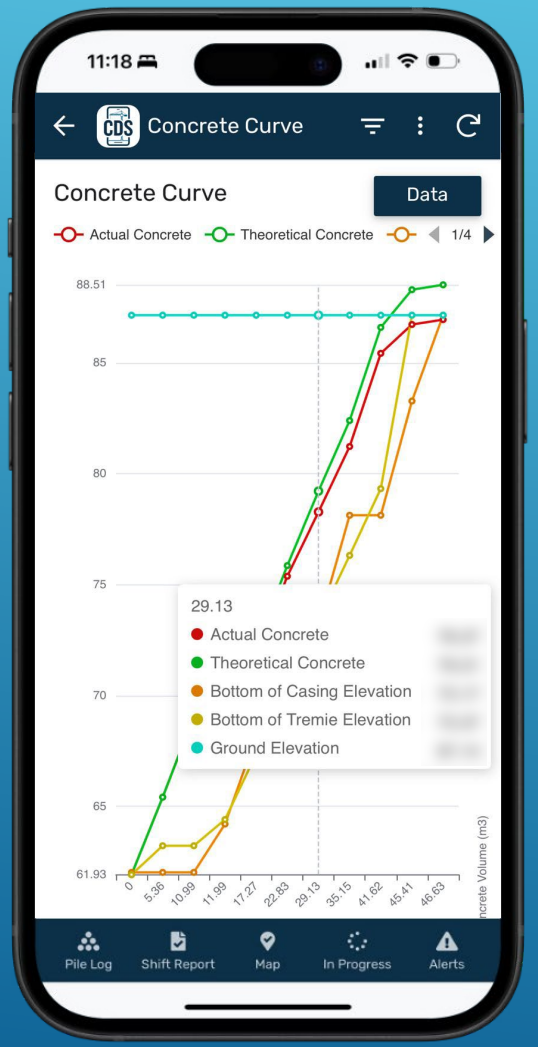


Maps integration for pile locations and status (tablet view)



Daily Reports (browser view)

SOME OF OUR DIGITAL TOOLS



Live Concrete Curve

- Elevations of concrete, tremie, casing toe
- Theoretical vs actual concrete volume
- Max casing available to be removed

IMPLEMENTATION

Initial Development

- 3 weeks to develop
- User training & system testing

Throughout the Project

- Monitor the system performance
- Changes/updates as requested
- Provide technical support

Project Closure

- Handover all data
- Decommission the system

OUTCOMES

- ✓ 80+ active users
- ✓ 3,000 pages of QC documents auto-filled
- ✓ 4,800 concrete tickets tracked
- ✓ 5,200 site photos archived
- ✓ Confidence in the quality program

THANK YOU



Stephen Ross

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www.constructiondatasolutions.com

OCTOBER 2025

INTERNALLY REINFORCED CONCRETE-FILLED STEEL COLUMNS FOR TEMPORARY SHORING OF A HERITAGE BUILDING WITH LIMITED ACCESS



PRESENTED BY

MOHAMMAD MIRMOEZI
M. ENG., P.ENG.

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CONTENTS

+ ONE
PROJECT OVERVIEW

+ THREE
TEST PROGRAM

+ TWO
DESIGN REQUIREMENTS

+ FOUR
OUTCOME SUMMARY

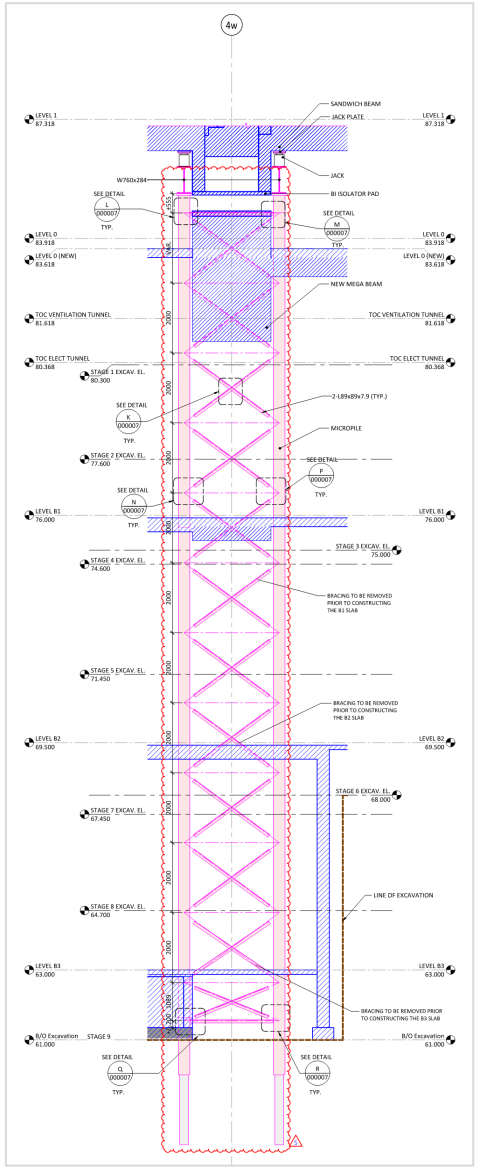


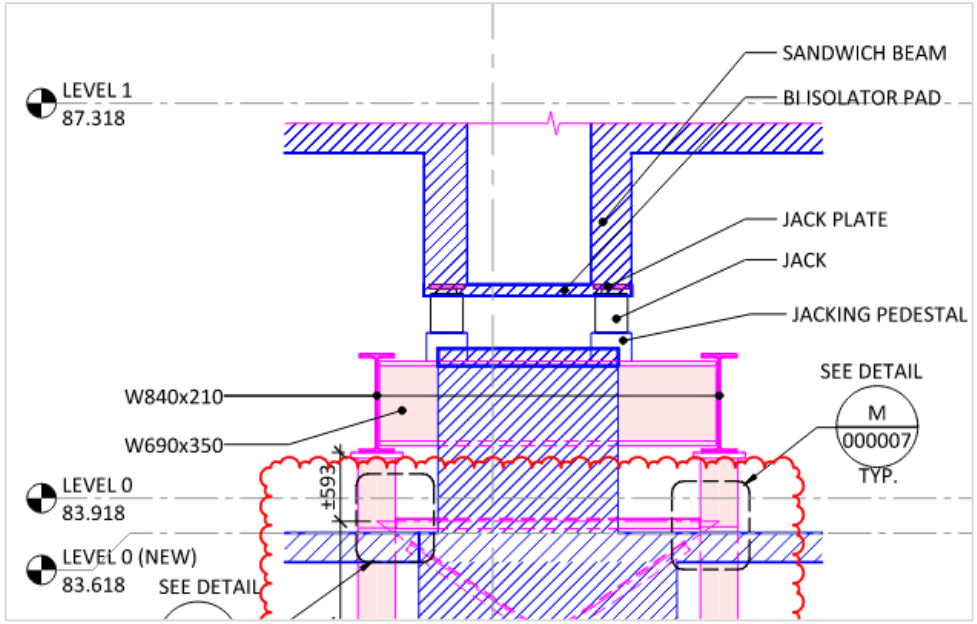
PROJECT OVERVIEW

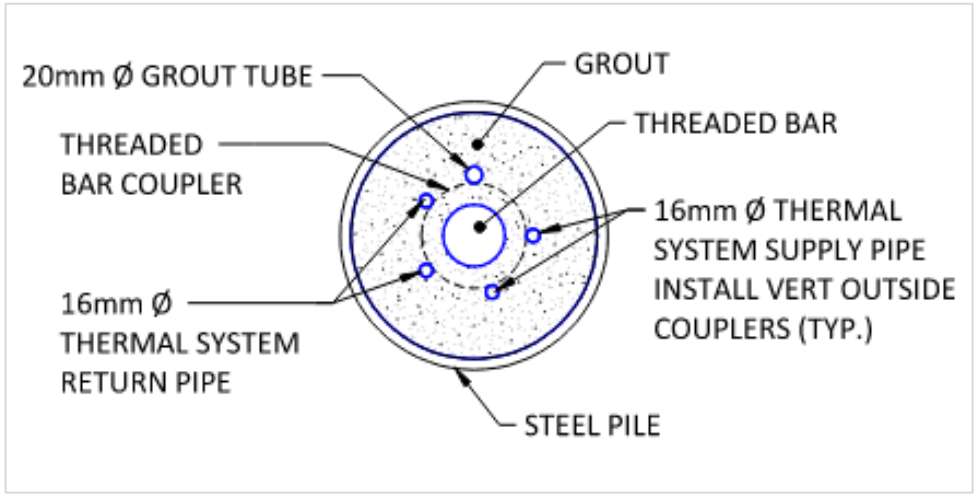
- 140 m wide by 75 m deep and is a six-storey high heritage building
- One level of basement founded on limestone bedrock
- Major seismic rehabilitation and below-grade extension
- Up to 24m of excavation in the bedrock
- Load transfer operation
- Over 900 Micropiles/Columns
- Headroom access (1.8m (~6') to 2.5m (~8')
- 1m segments threaded together

SHORING COMPONENTS

- Sandwich beams
- Hydraulic jacks
- Load transfer platform grillage
- CFSC
- Rock socket
- Thermal control system







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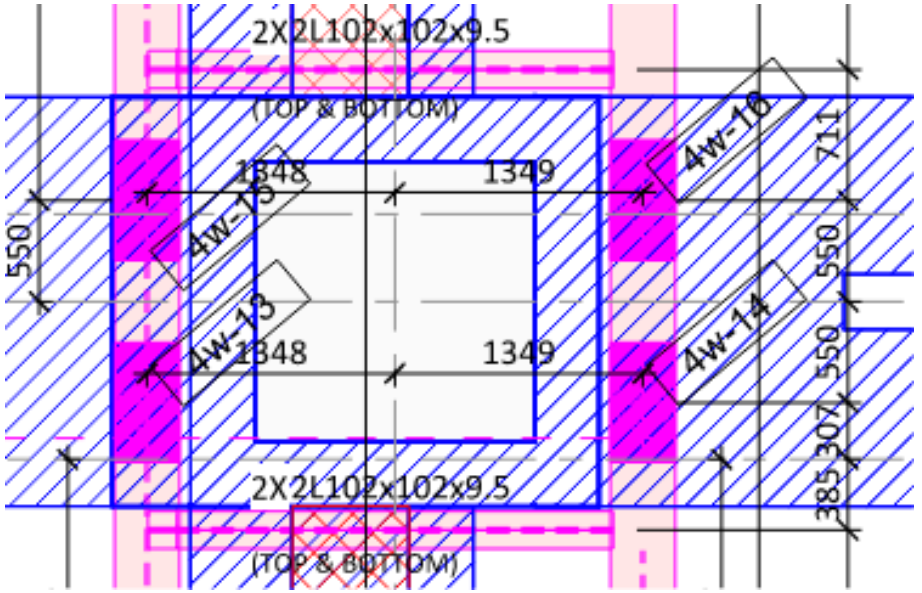
DESIGN REQUIREMENTS

- Super structure loads at each load transfer
- Pile capacities
- Pile and rock interface
- Pile inclinations
- BIT test
- Bending moment forces
- Pile damage scenario
- Thermal impact



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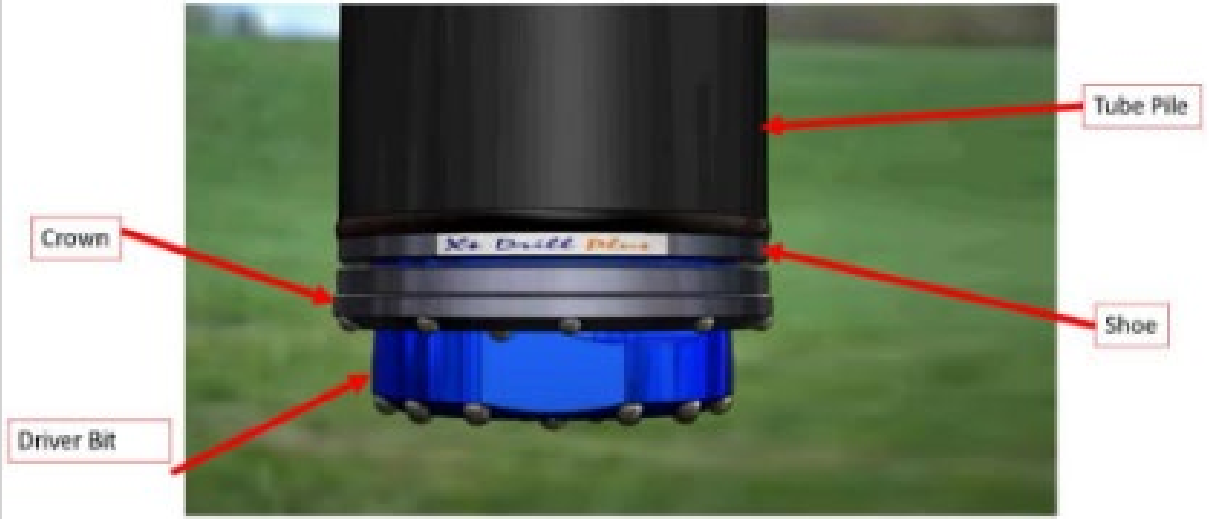

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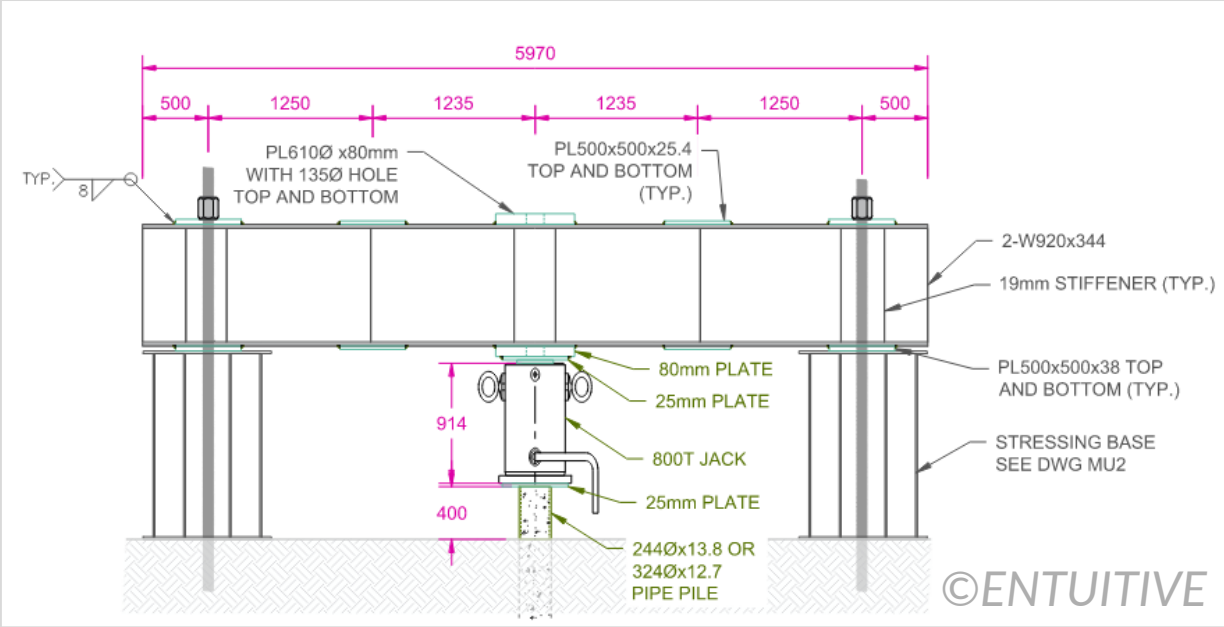
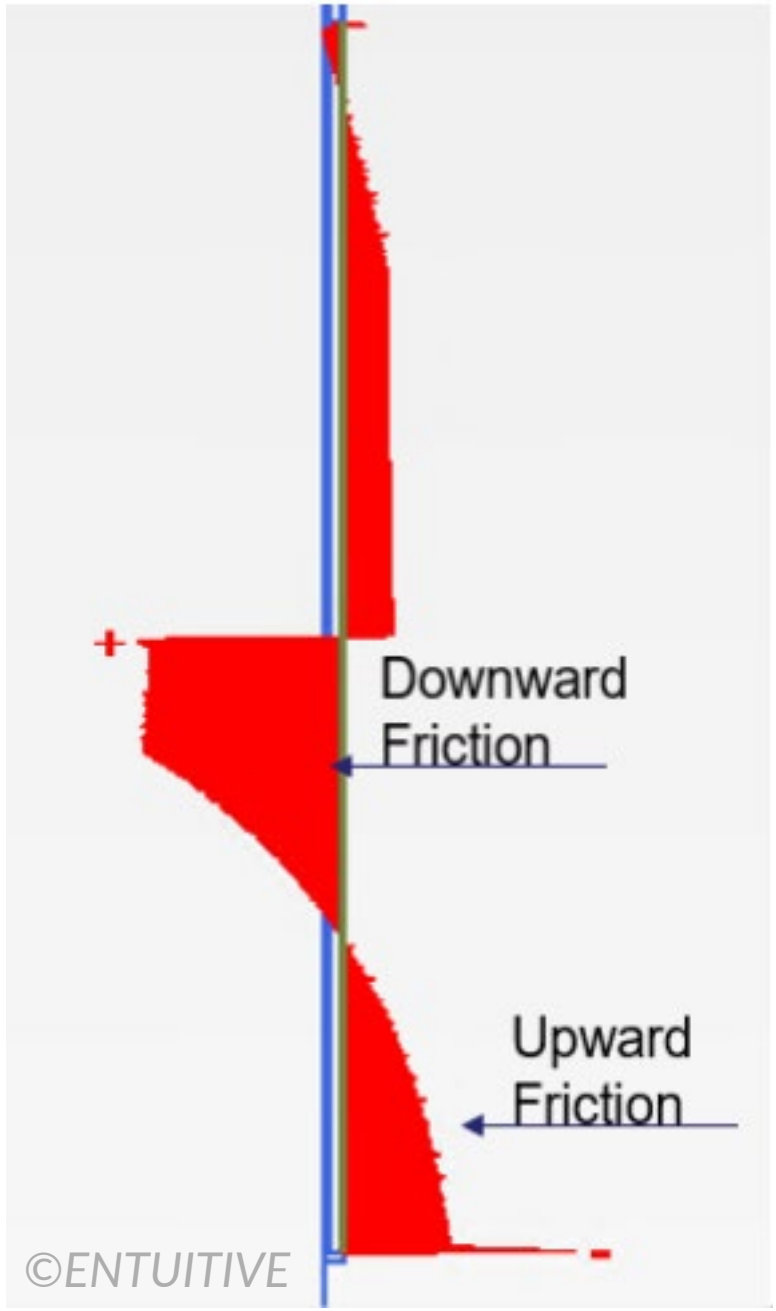
PRE-PRODUCTION
PILE TESTS

- Annular space between pile and bedrock
- Friction between pile and bedrock
- Excavation and settlements





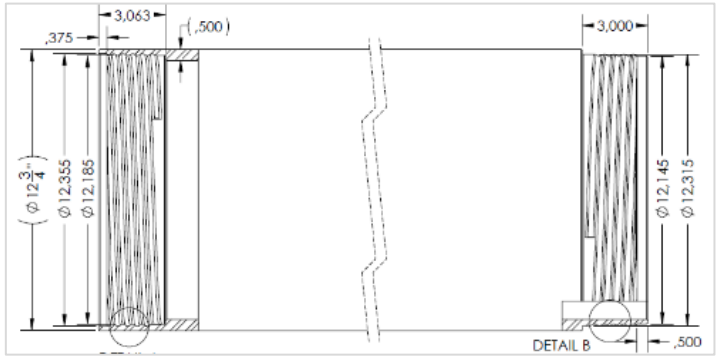
THREADED JOINTS
TESTING PROGRAM

CONCRETE FILLED STEEL COLUMNS

ROUND SHAPE THREADS

FAILURE MODES

TEST PROGRAM



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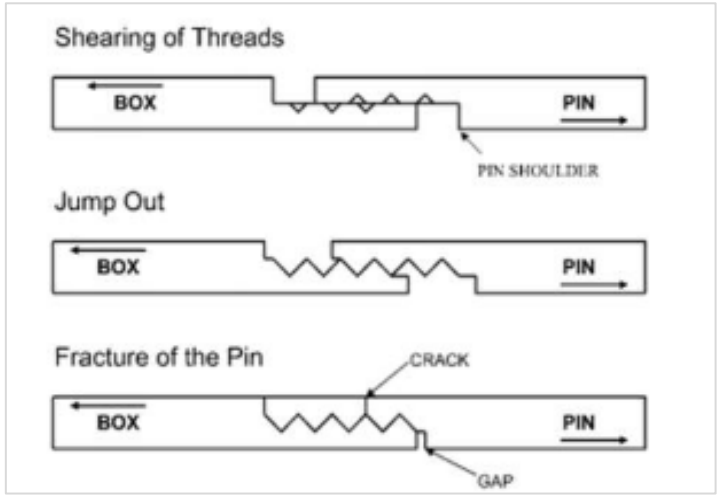

THREADED JOINTS
TESTING PROGRAM

Tests set-up



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THREADED JOINTS
TESTING PROGRAM

PULL-OUT TEST (HSS)



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THREADED JOINTS
TESTING PROGRAM

UNDER COMPRESSION (HSS)



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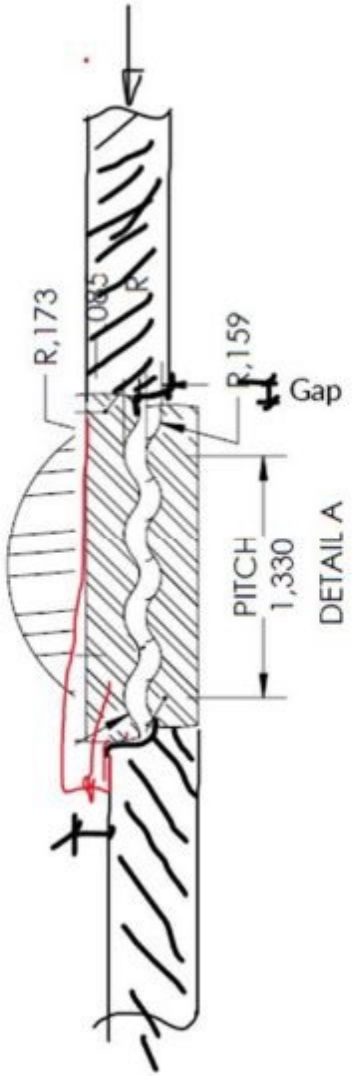
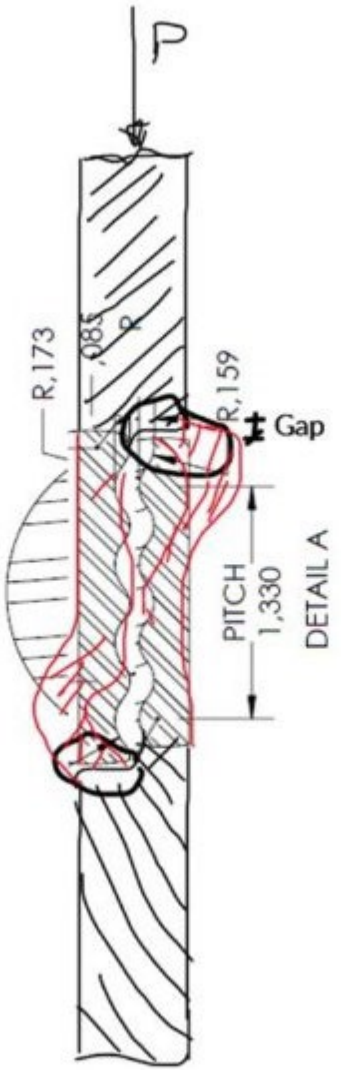


a) *Insufficient Contact Area (box slippage over pin)*



b) *Local Buckling at Threaded Joint Area*

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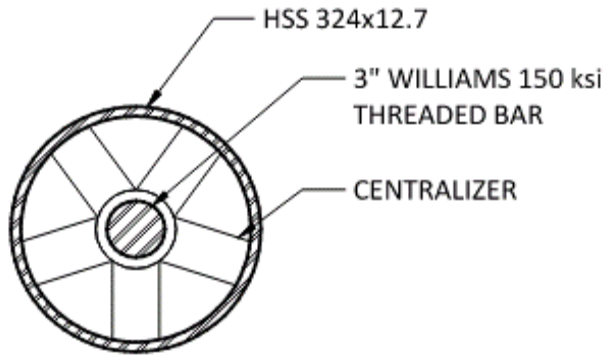
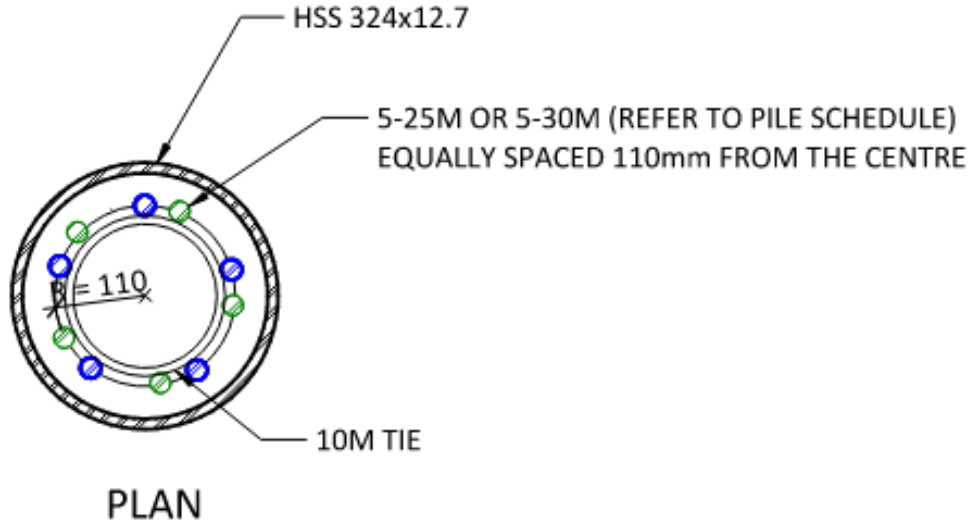
THREADED JOINTS TESTING PROGRAM:
CONCRETE FILLED REINFORCED STEEL COLUMNS



PLAN

PILE REINFORCEMENT ***TYPE 1:***

Reinforced with threaded bar



PLAN

PILE REINFORCEMENT ***TYPE 2:***

Reinforced with rebar cage

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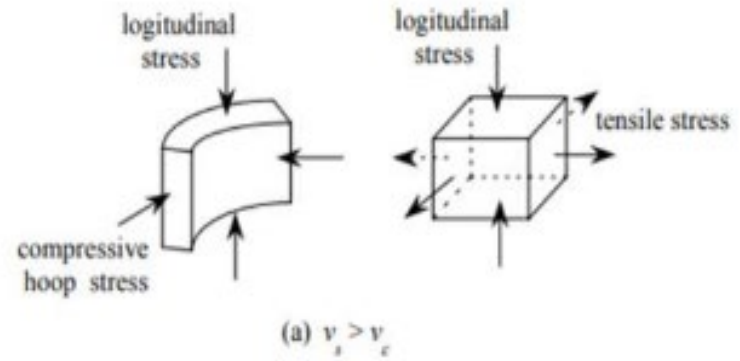
THREADED JOINTS TESTING PROGRAM:
CONCRETE FILLED STEEL COLUMNS

Stress States in Concrete and Steel



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THREADED JOINTS
TESTING PROGRAM

a) Yielding of the steel tube (P_Y) or filling the gap between the pin and box

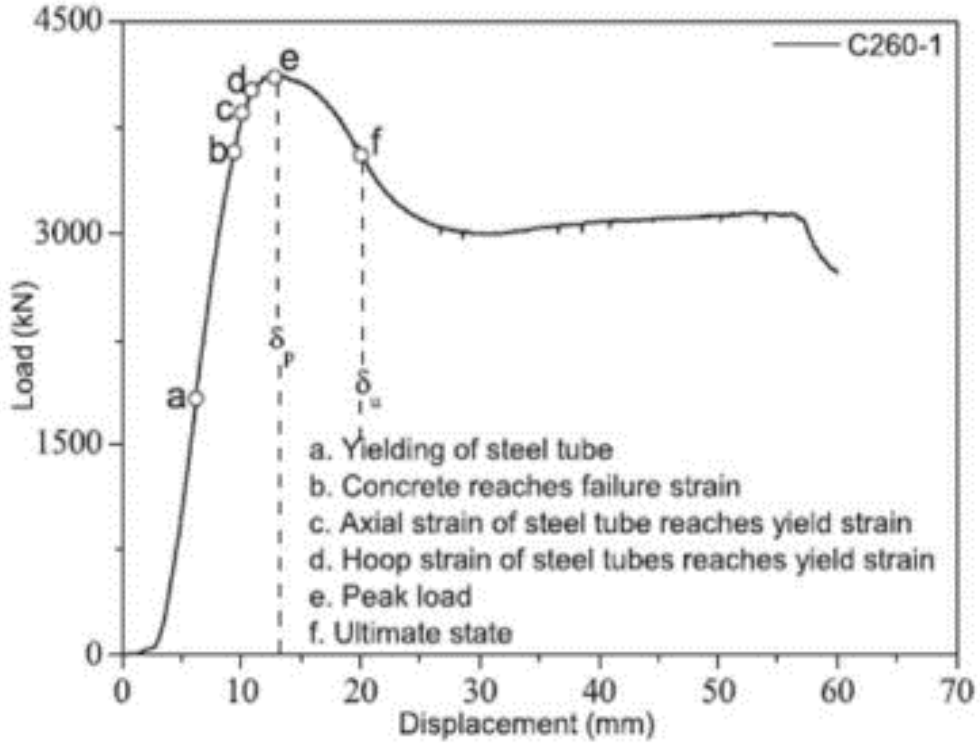
b) The concrete reaches the failure strain $\varepsilon_{cu} \approx 0.003$ under short-term loading (P_c)

c) The axial strain of the steel tube reaches the yield strain (P_{is})

d) The hoop strain of the steel tube reaches the yield strain (P_{hs})

e) Maximum bearing load (N_u)

f) Ultimate state (P_u)



Typical Axial Load-Displacement relationship of Concrete-Filled HSS

©SCIENCEDIRECT

OUTCOME
SUMMARY

a) Design evolution from HSS to ext. Reinforced CFSC

b) Considerable increase in bending capacity under compressive load

c) Threaded bar vs rebar reinforced

d) Successful reinforcement approaches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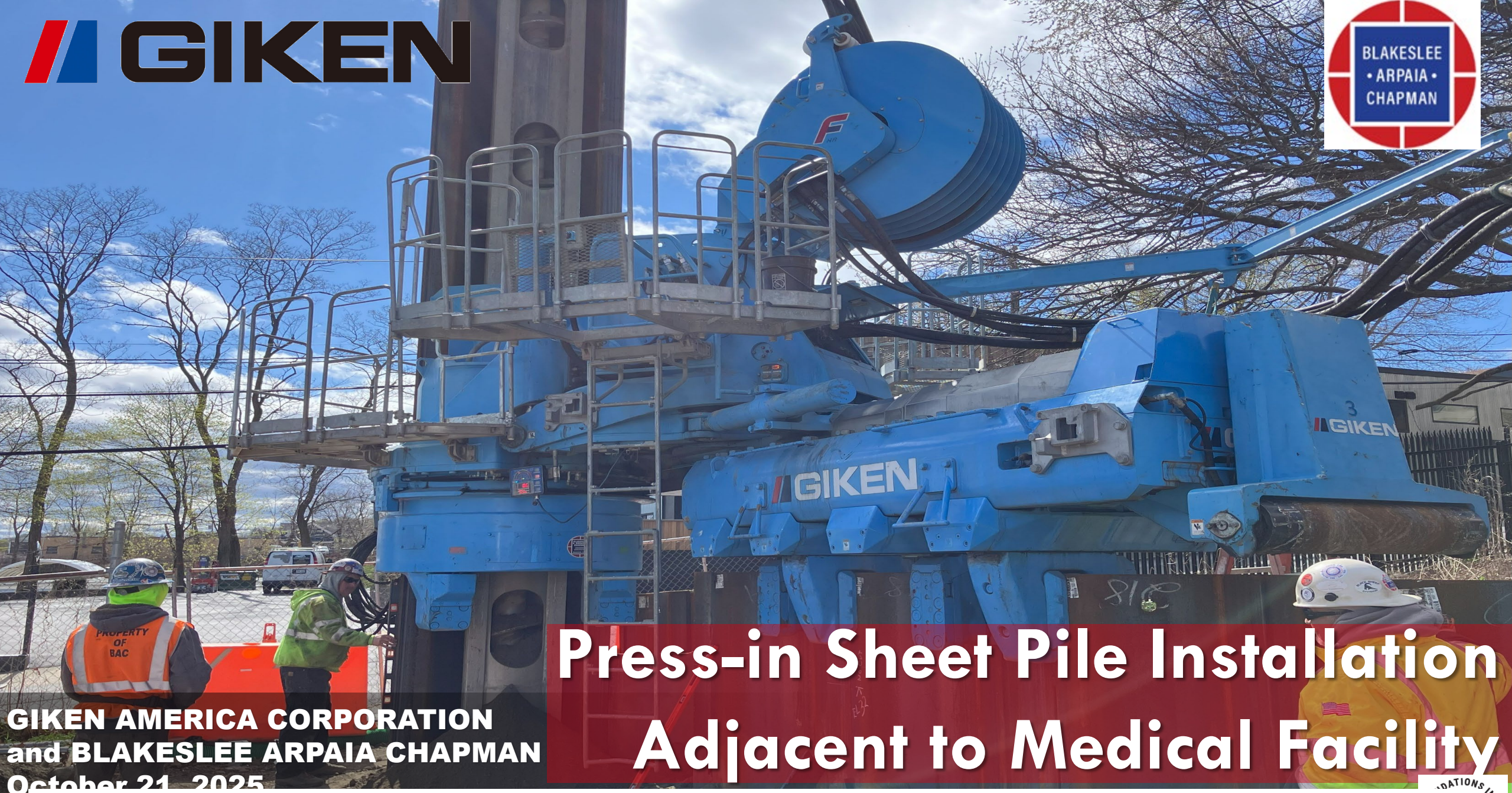

Press-in Sheet Pile Installation Adjacent to Medical Facility

GIKEN AMERICA CORPORATION
and BLAKESLEE ARPAIA CHAPMAN
October 21, 2025

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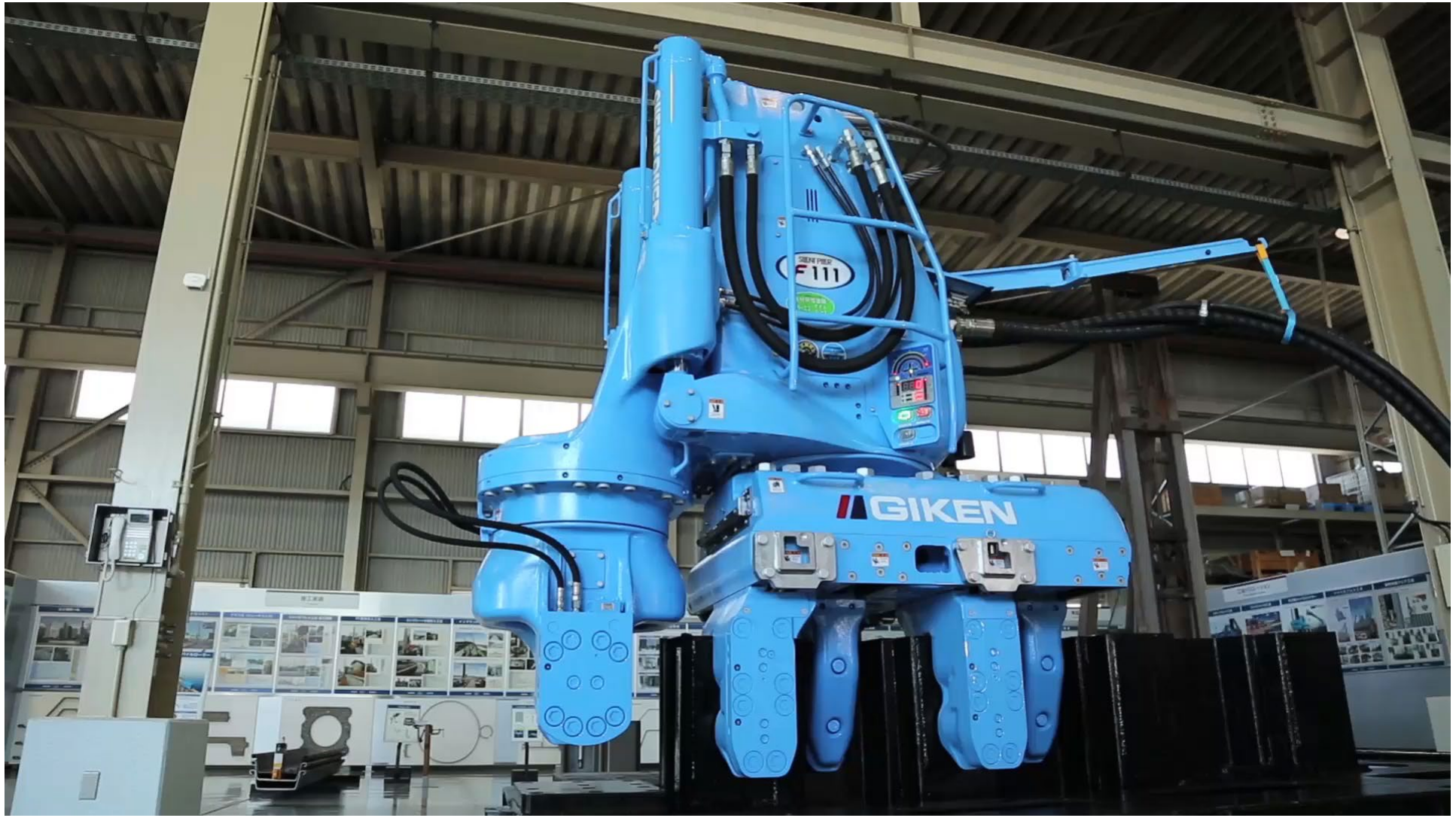
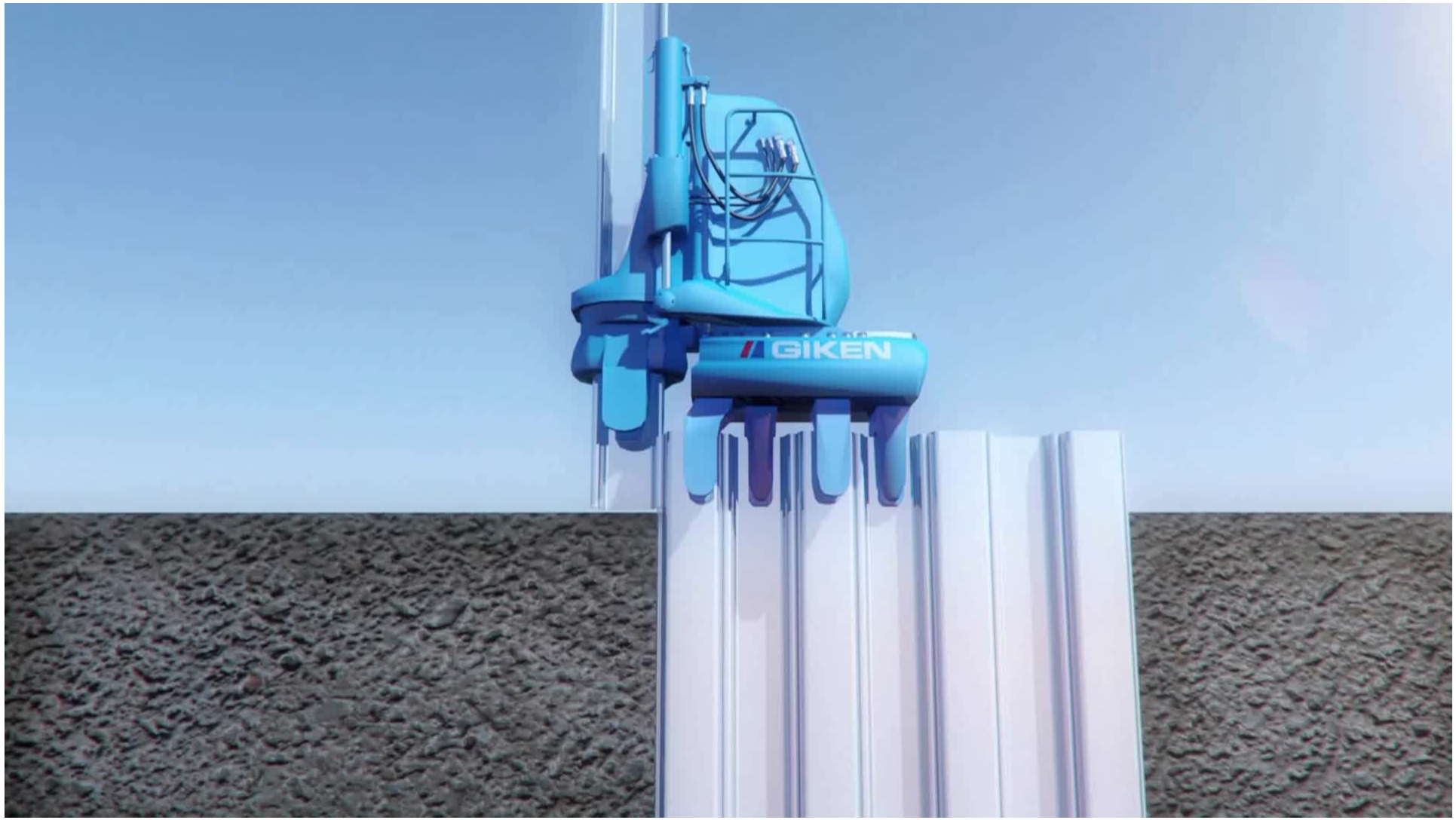
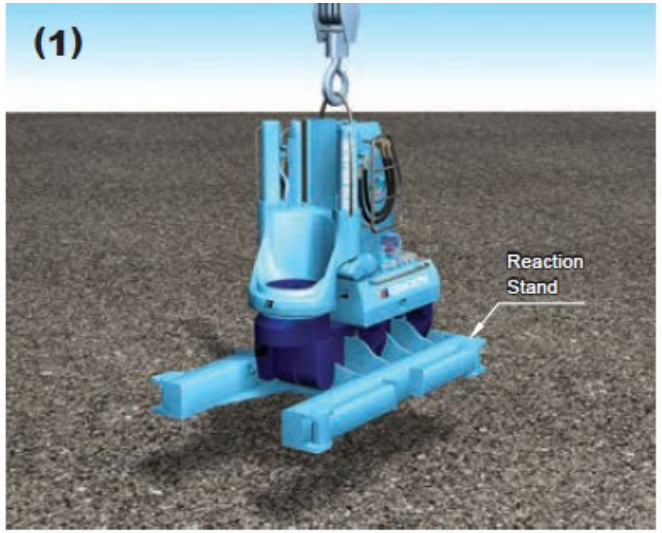
Press-in Method



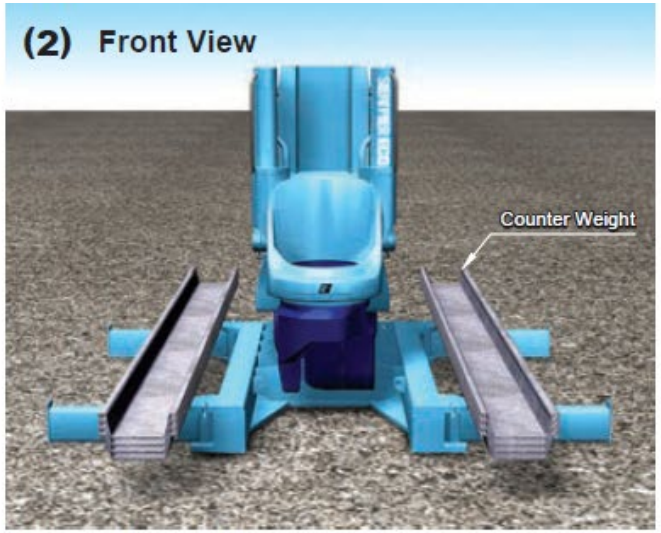
Press-in Method



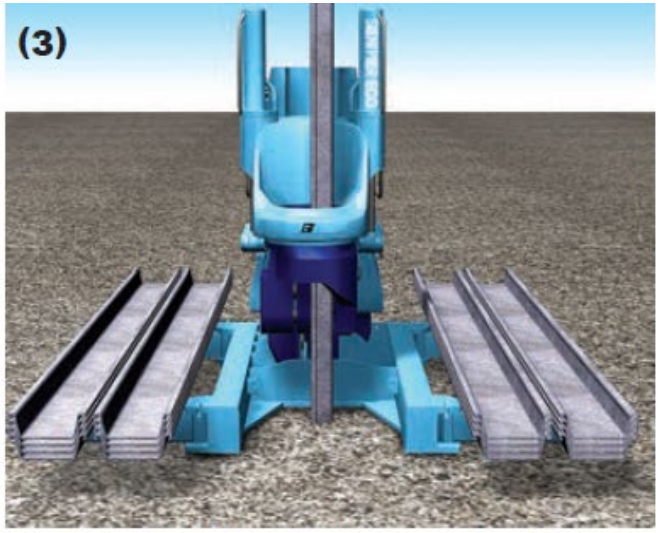
Initial Press-in Procedure



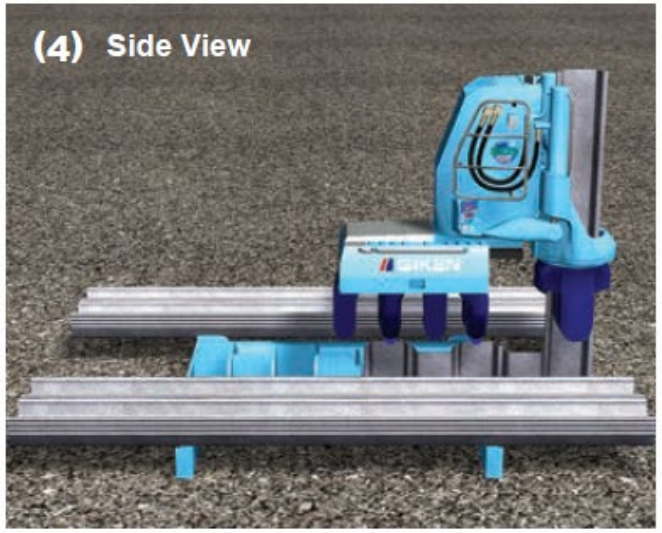
Set Press-in Machine Main Body
and Reaction Stand horizontally



Set counter weights



Pitch the first pile and start pressing-in



Install prescribed number of piles

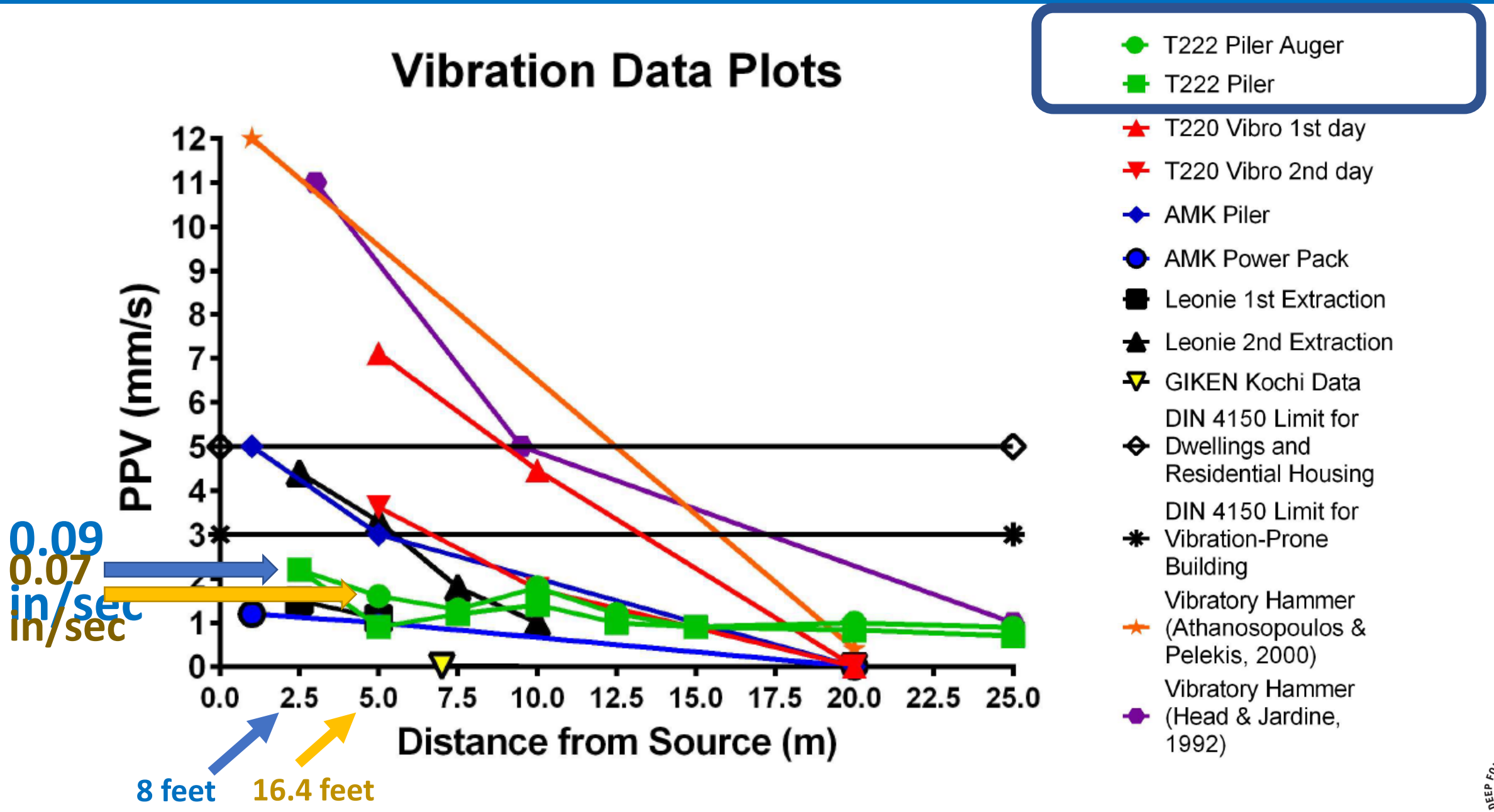


Remove counter weights



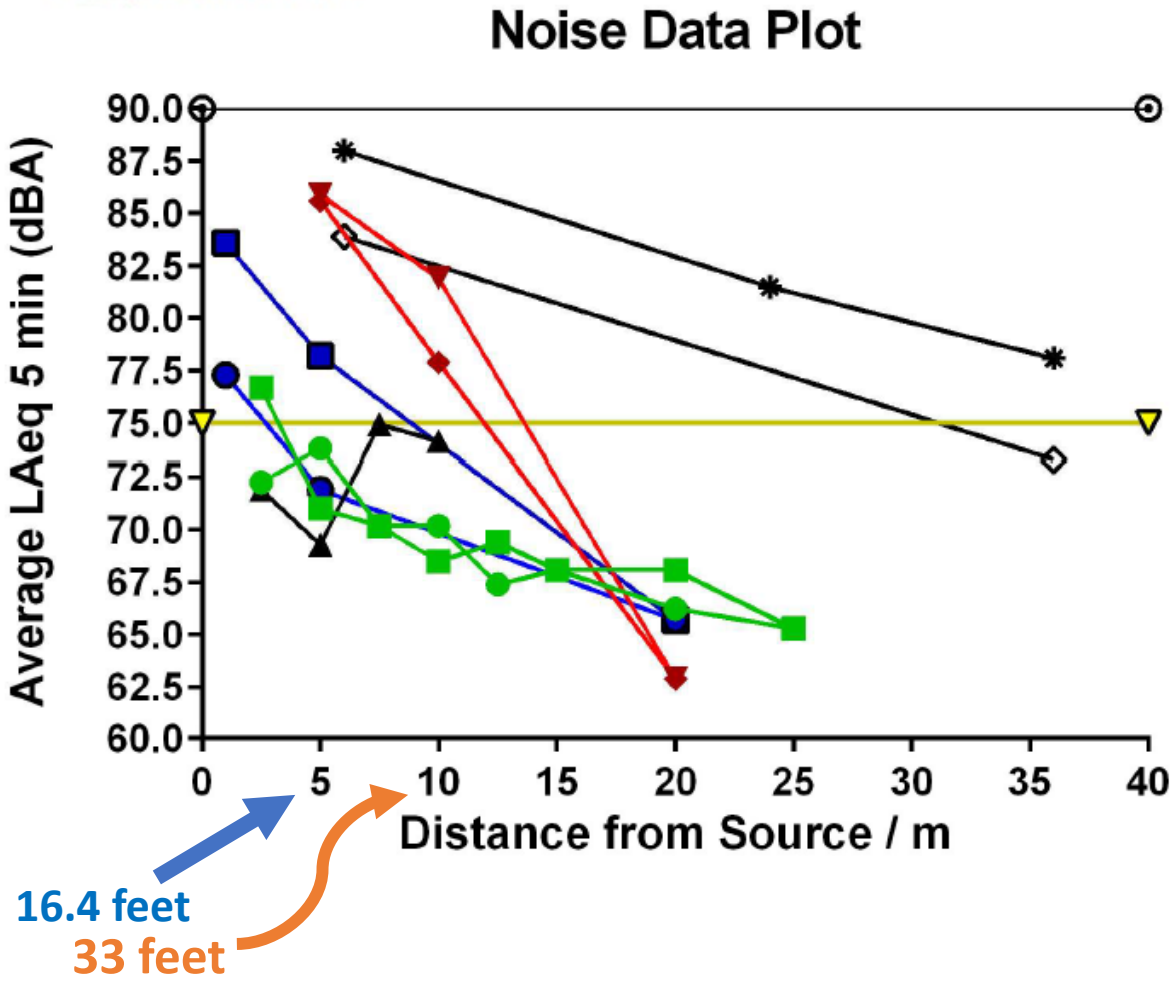
Remove Reaction Stand
and the initial piling is completed

Vibration Level Comparison



Noise Level Comparison

National Environment Agency
Safeguard · Nurture · Cherish



- T222 Auger
- T222 Piler
- Leonie 2nd Extraction
- T220 Vibro 1st day
- T220 Vibro 2nd day
- AMK Piler
- AMK Power Pack
- NEA Noise Limit on Sensitive Buildings
- NEA Noise Limit on Residential Housing
- Rockaway Park Vibratory Hammer, 60 ft Depth
- Rockaway Park Vibratory Hammer, 120 ft Depth



Power Unit

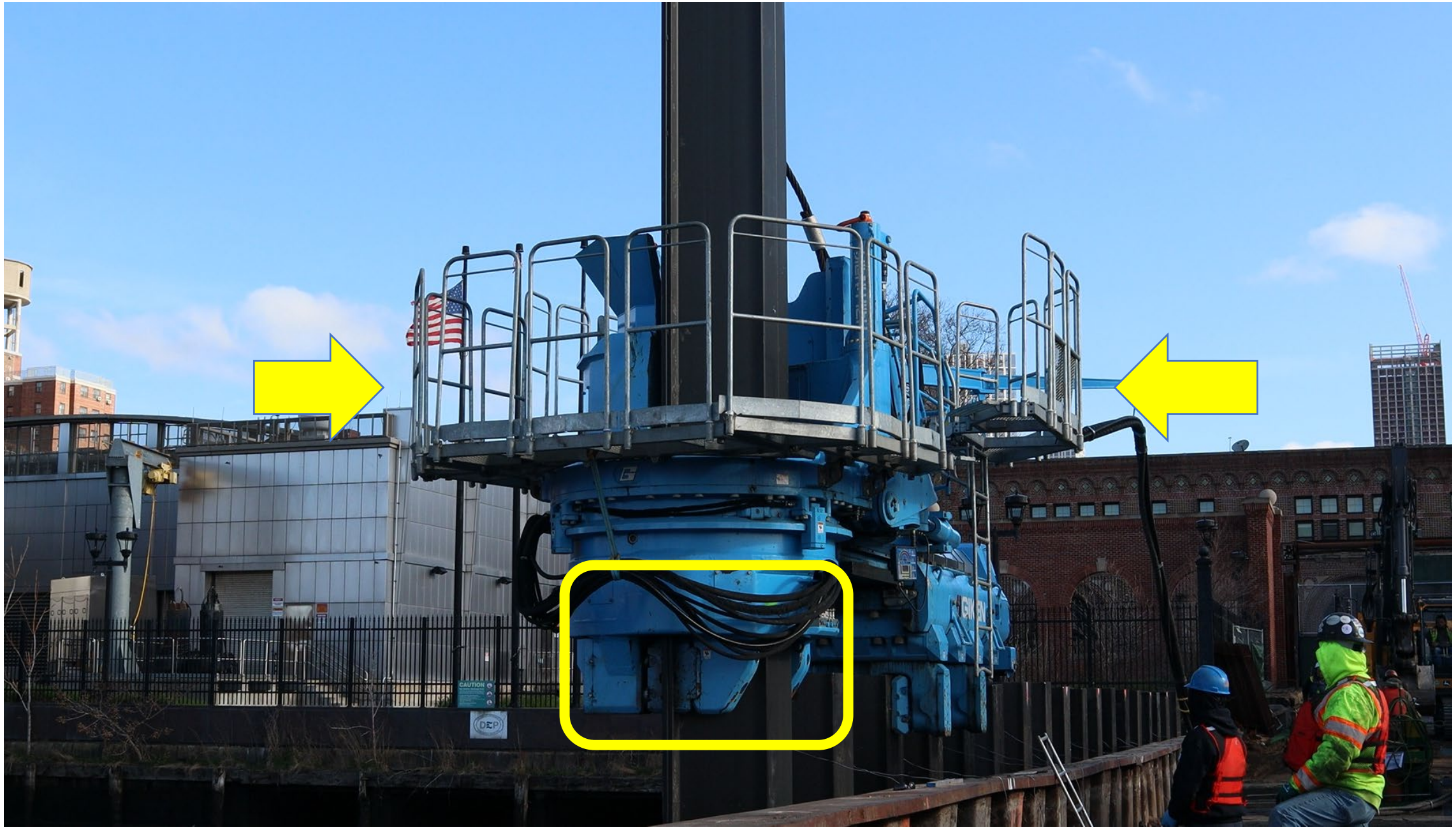


Shielded Power Unit

DEEP FOUNDATIONS INSTITUTE DFI

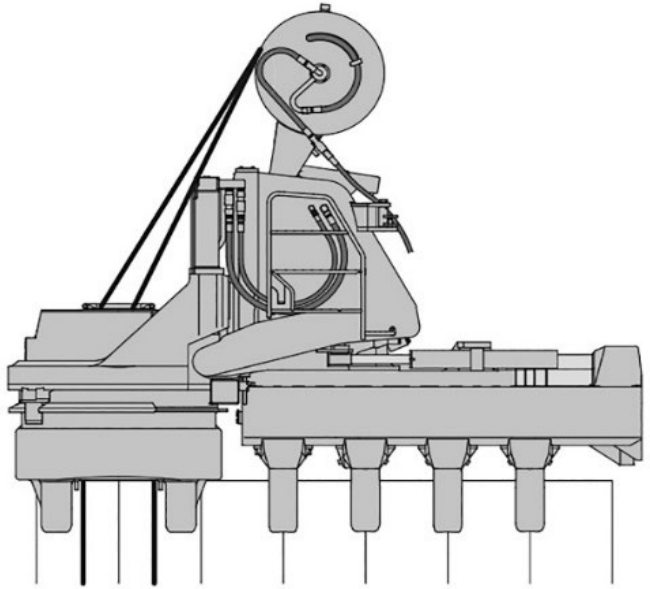

Giken
Sheet Piling
Penetration
Modes
6.25t
6.25t
SIENTPIER
GIKEN

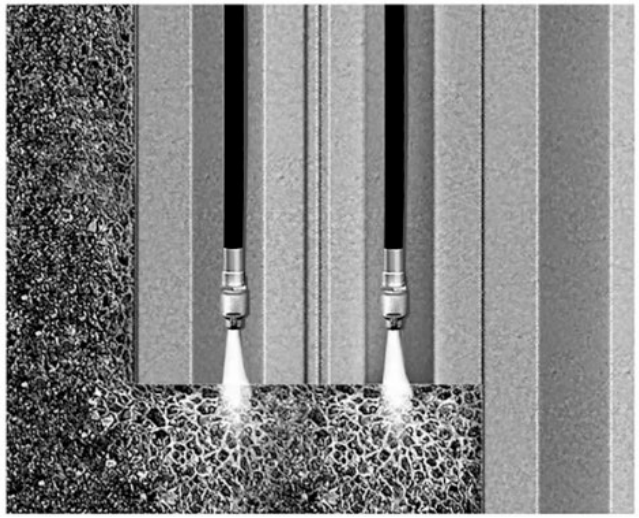
Standard Mode



Water Jetting Mode

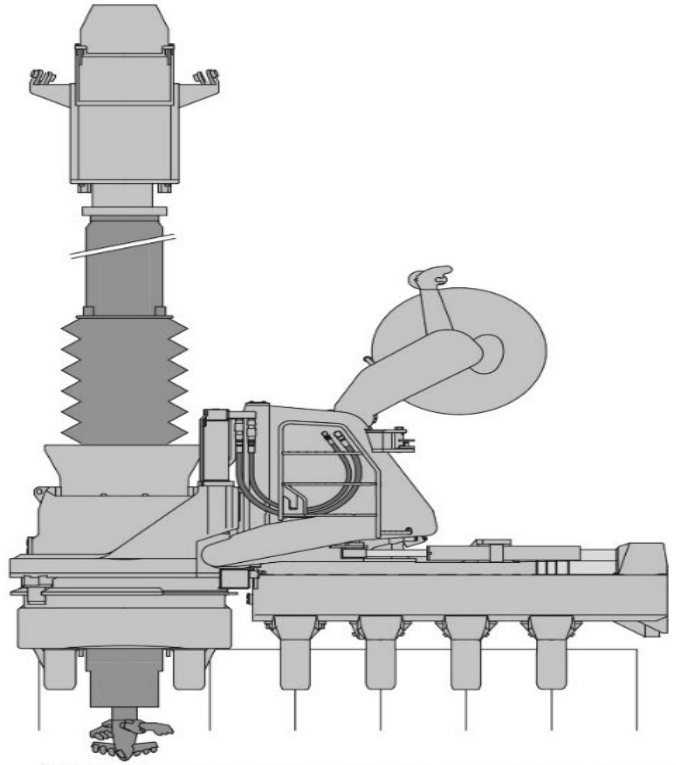


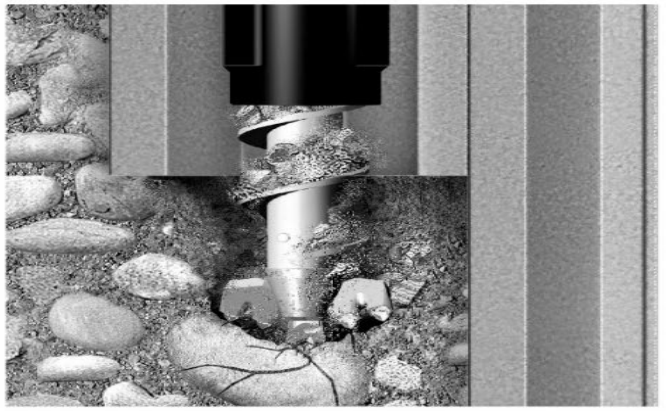




Crush Auger Mode







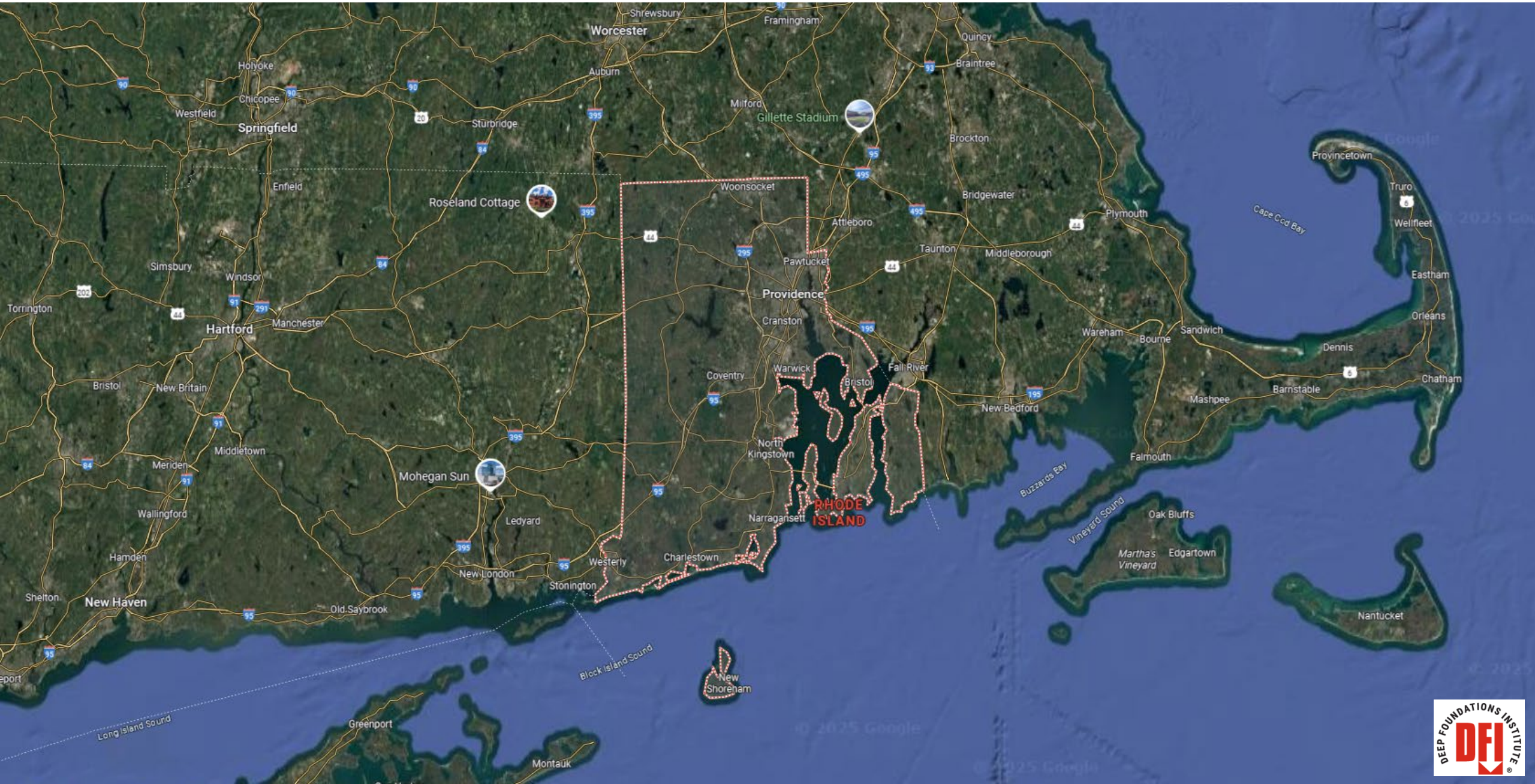


Blakeslee Arpaia Chapman's Press-in Piling Project

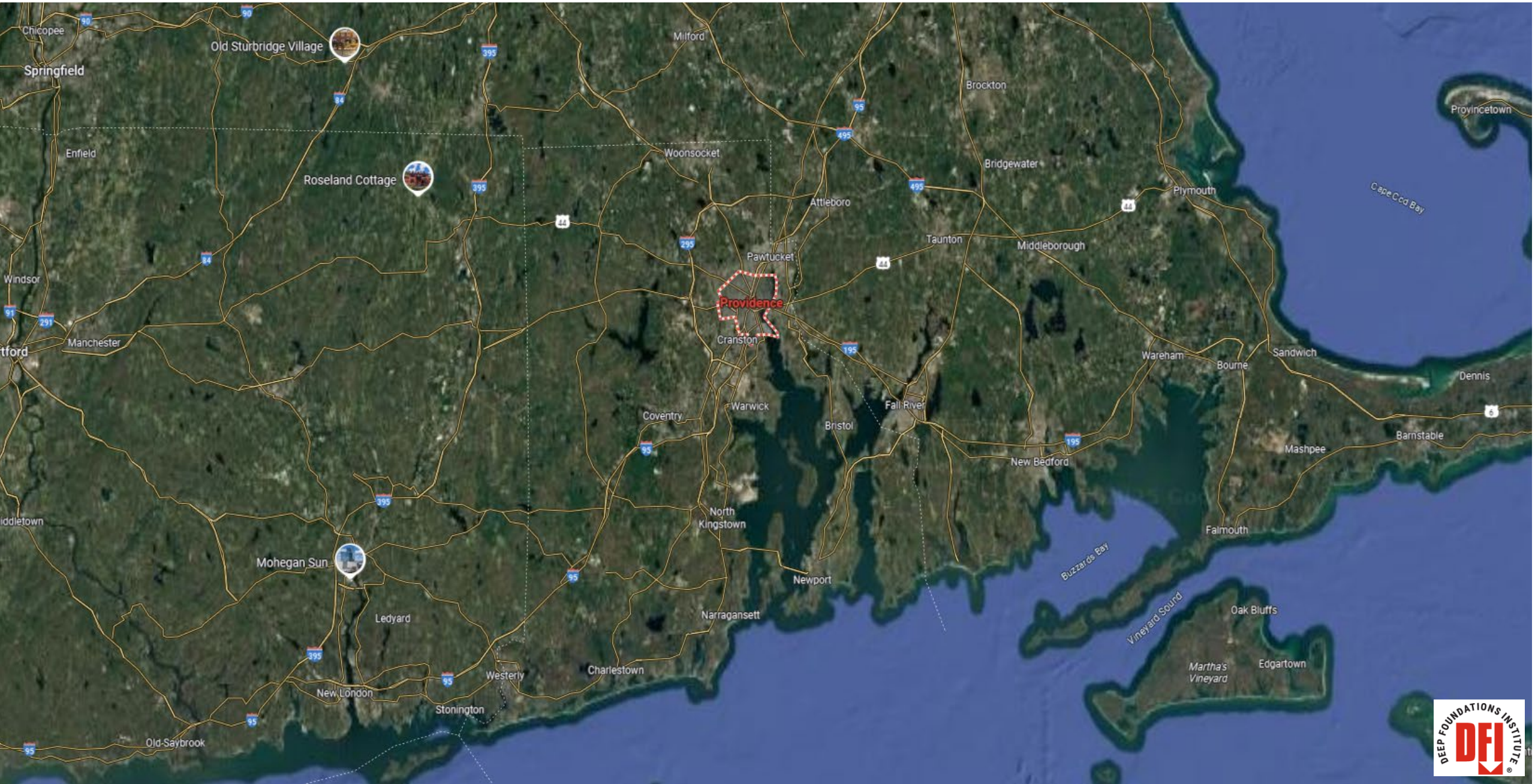
Project Location



Project Location



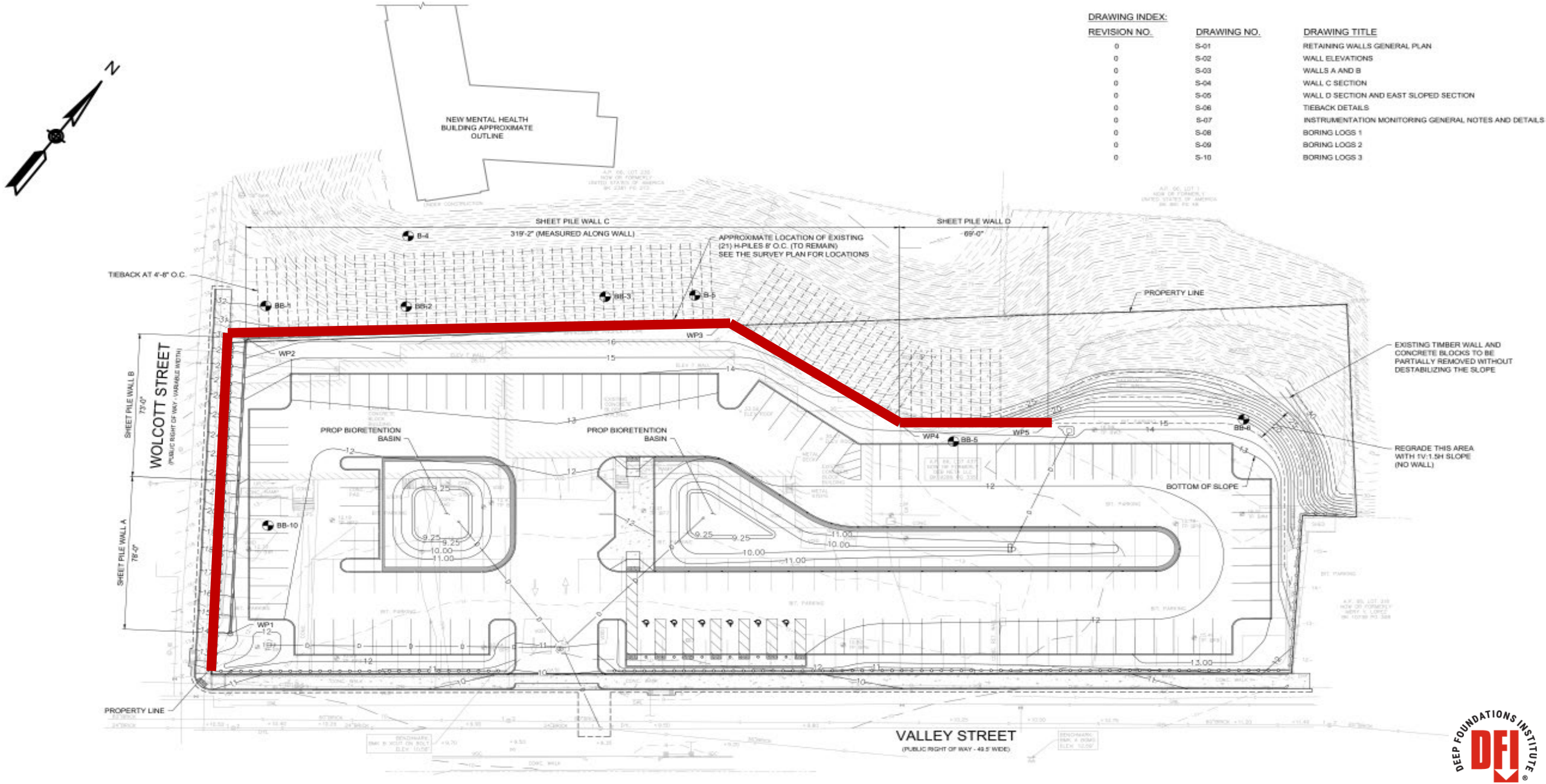
Project Location



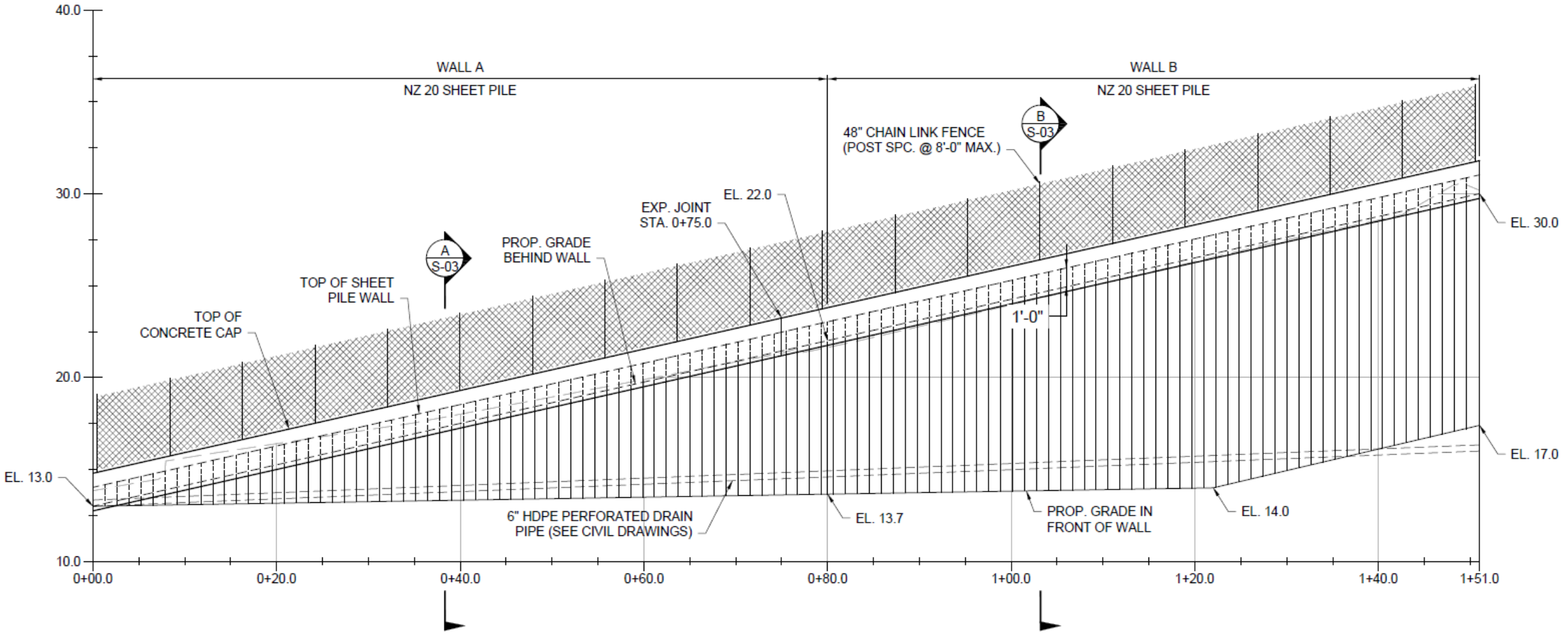
Project Details

- **Project Name**: Construction of Valley Street Parking Lot Improvements Minor Construction
- **Project Location**: Providence, Rhode Island, U.S.A.
- **Purpose of Project**: Parking Facility Construction
- **Project Start Date**: July 2024 (NTP)
- **Overall Project Cost**: $11.5 Million
- **Overall Length of Installation**: ±540 Lft
- **Type of Sheet Piles Installed**: NZ 20 (36 ft - 41ft long)

Project Layout



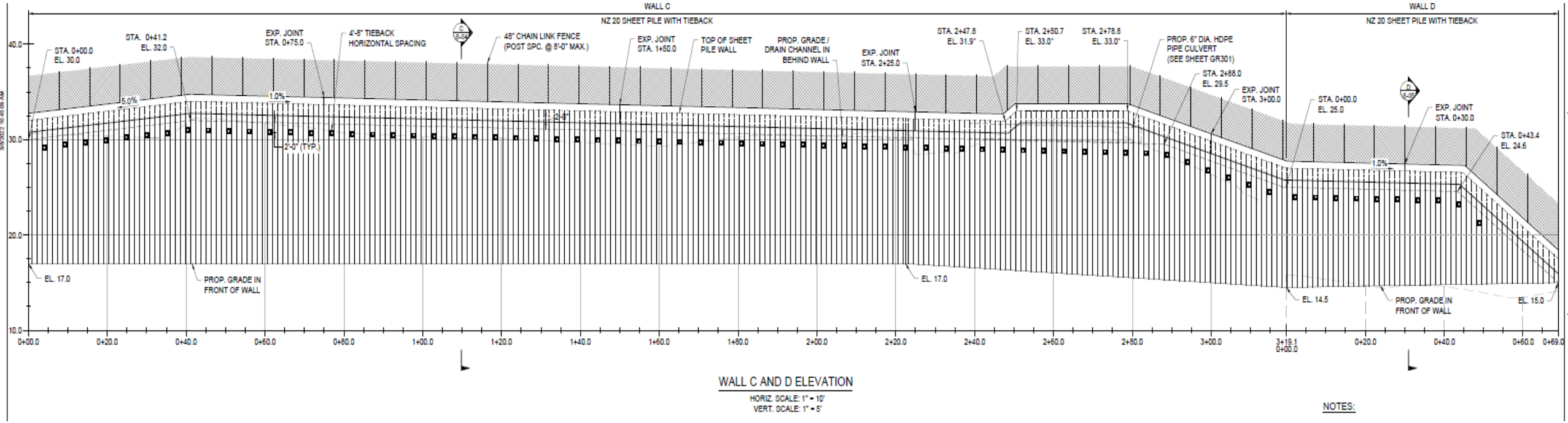
Project Layout



WALL A AND B ELEVATION

HORIZ. SCALE: 1" = 8'
VERT. SCALE: 1" = 4'

Project Layout

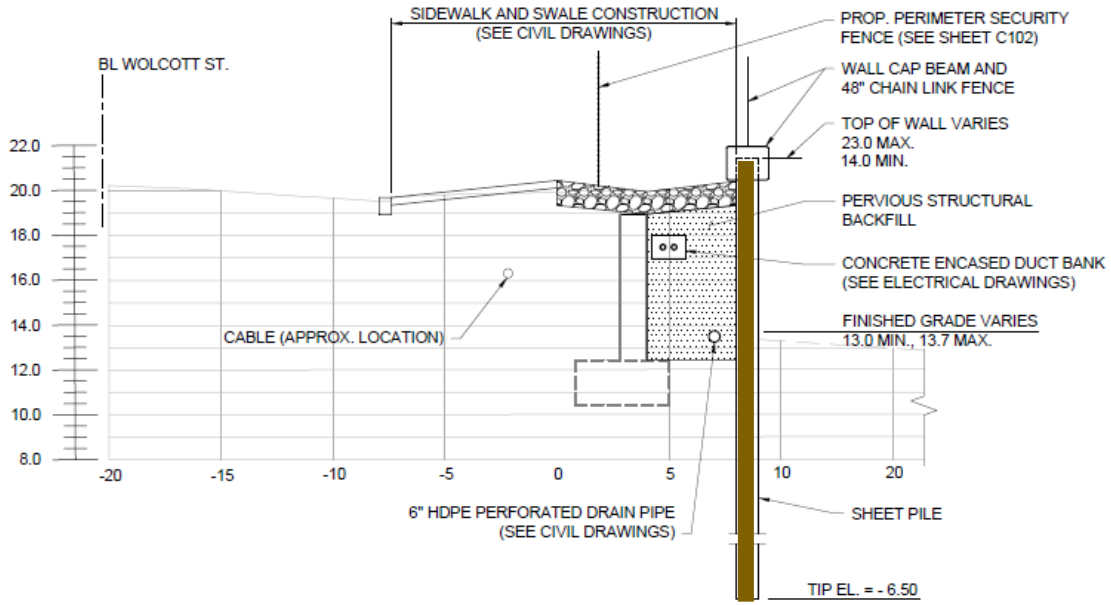


WALL C AND D ELEVATION

HORIZ. SCALE: 1" = 10'
VERT. SCALE: 1" = 5'

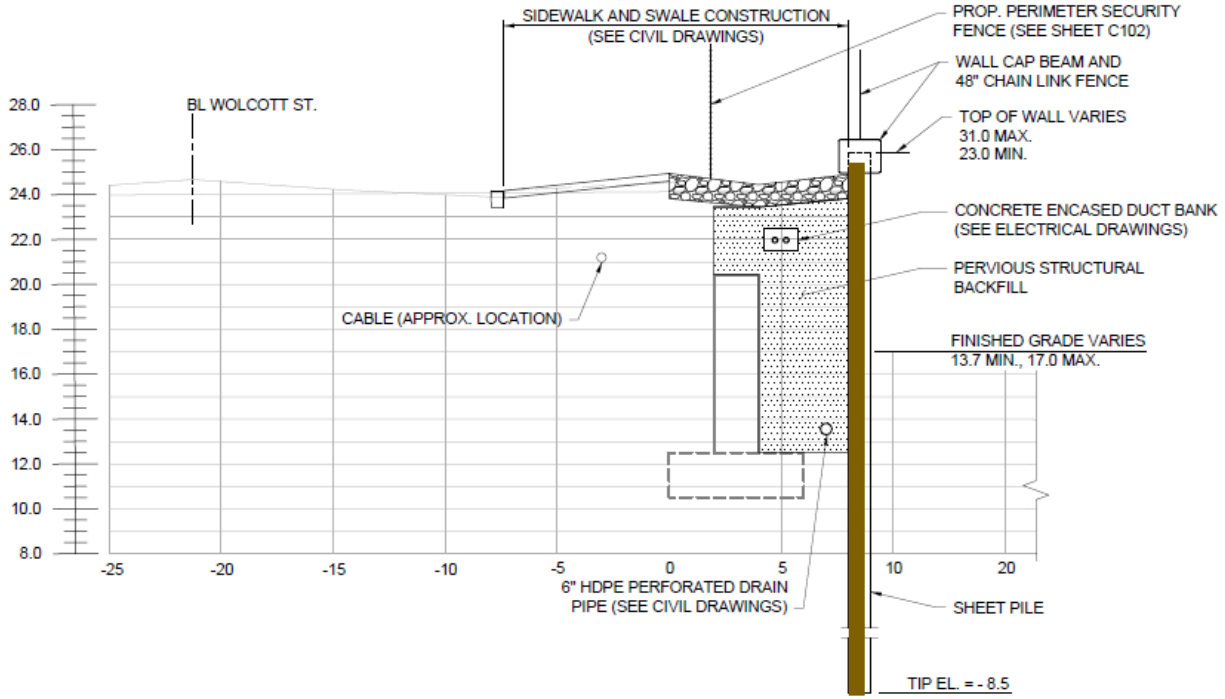
NOTES:

Project Layout



SECTION A - PROPOSED CONDITION (WALL A)

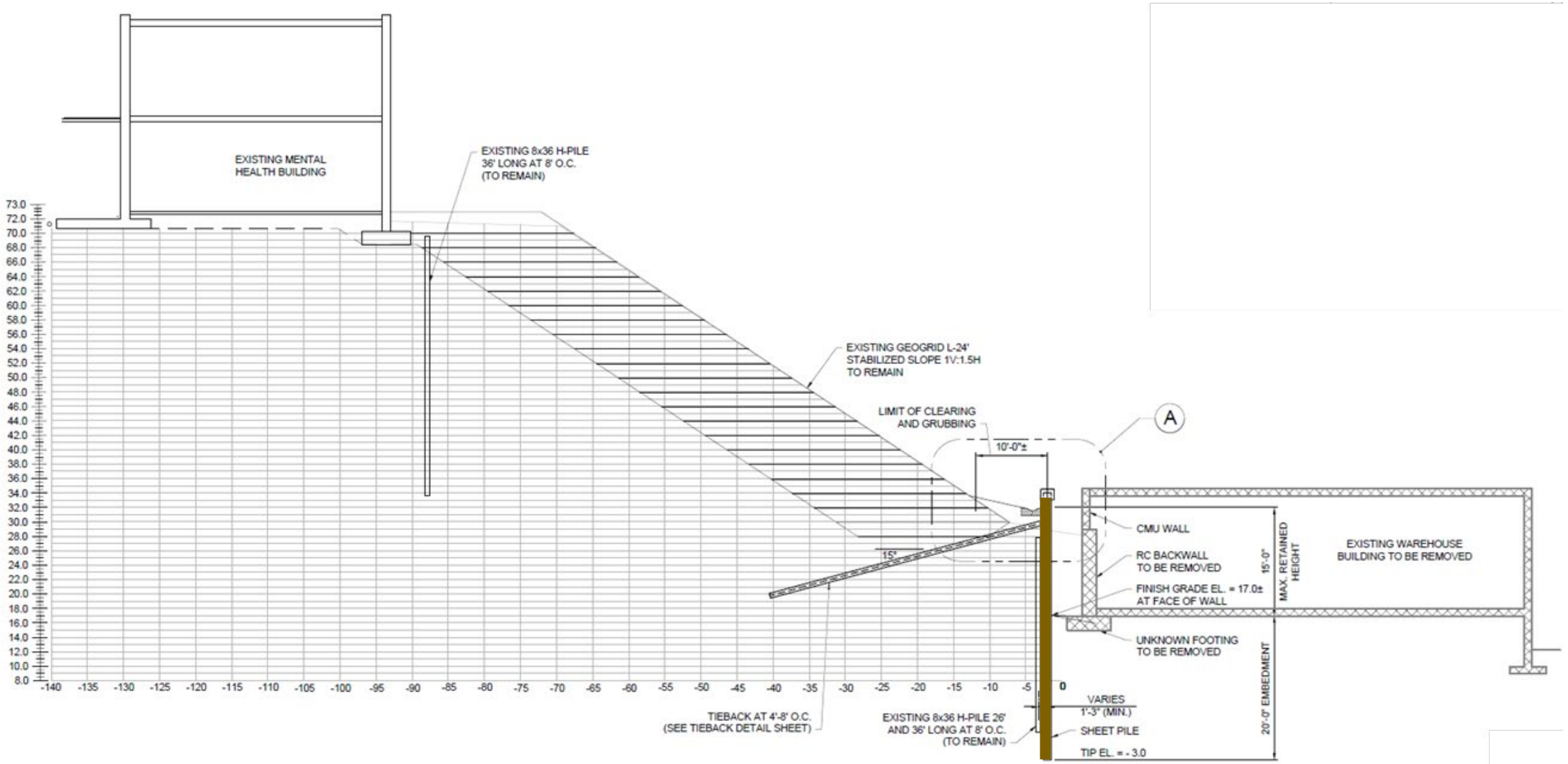
SCALE: 1" = 4'



SECTION B - PROPOSED CONDITION (WALL B)

SCALE: 1" = 4'

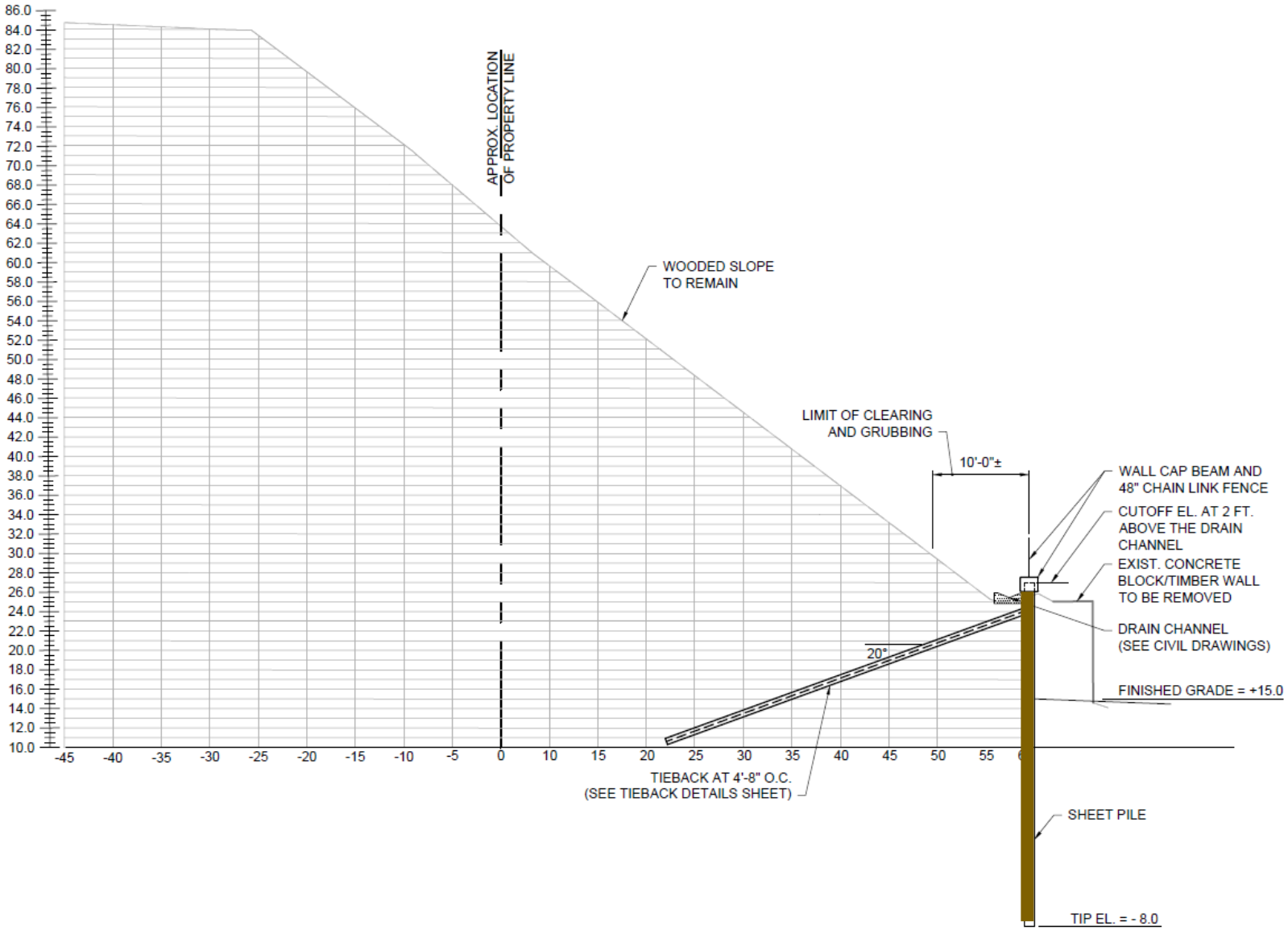
Project Layout



SECTION C - EXISTING AND PROPOSED CONDITION (WALL C)

SCALE: 1" = 8'

Project Layout



SECTION D - EXISTING AND PROPOSED CONDITION (WALL D)

SCALE: 1" = 8'

Before Demolition



Before Demolition



Project Soil Conditions

BORING BB-3
(CONTINUED)

(603) 437-1610	New England Boring Contractors P.O. Box 165 Derry, NH 03038		Fax: (603) 437-0034
Boring # BB-3	**Project:** Paul B. Aldinger VA Hospital Due Diligence		**Project #** 26974
Project Address: 500 Valley Street		**City:** Providence	**State:** RI **Zip:**
Date Start:	**Date End:**		**Elevation:** +33'

Casing: Size: 4" Hammer: **140# Automatic Hammer**	**Sampler:** S/S	Fall: 30"	**Sampler:** 1-3/8 in. I.D. 30 in.

GROUNDWATER OBSERVATION

Date:	**Depth:** 18 feet	**Elevation:** 15 ft	**Stabilization Period**

DP	S./#	DEPTH	PEN	REC	BLOWS/6"	S/C	SAMPLE DESCRIPTION
-							
-							
-							
-	S-8	34' – 36'	24"	2"	28-24-26-20		Wet, very dense, gray GRAVEL
35'0"							
-							
-							
-	S-9	39' – 41'	24"	6"	34-14-14-15		Wet, medium dense, brown fine to coarse SAND, little Gravel, trace Silt
40'0"							
-							
-							
-	S-10	44' – 46'	24"	11"	10-11-14-20		Wet, medium dense, gray fine SAND, trace medium Sand, trace Silt
45'0"							
-							
-							
-	S-11	49' – 51'	24"	12"	6-9-15-14		Wet, medium dense, gray fine SAND, trace medium Sand, trace Silt
50'0"							
-							
-							
-	S-12	24' – 56'	24"	11"	9-9-14-22		Wet, medium dense, gray fine to medium SAND, trace Silt
55'0"							
-							
-							
-	S-13	59' – 61'	24"	12"	10-14-17-100		Wet, dense, gray fine to medium SAND, trace Silt
60'0"							

Drillers. Pat Schofield	**Helper:** Don	**Inspector:** Bryan Deely
Remarks:		

S/#: Sample	**PEN:** Penetration	**REC:** Recovery	**S/C:** Strata Change

- 36ft – 41ft long NZ 20 sheet piles installed with the Silent Piler
- Soil strata consisting mainly of sand with cobbles and small boulders
- Max. N = 50 @ 35 ft bgs
  - Just above sheet pile tip elevation

Initial Piling Sequence



Initial Piling Sequence





Installation in Progress



Installation in Progress



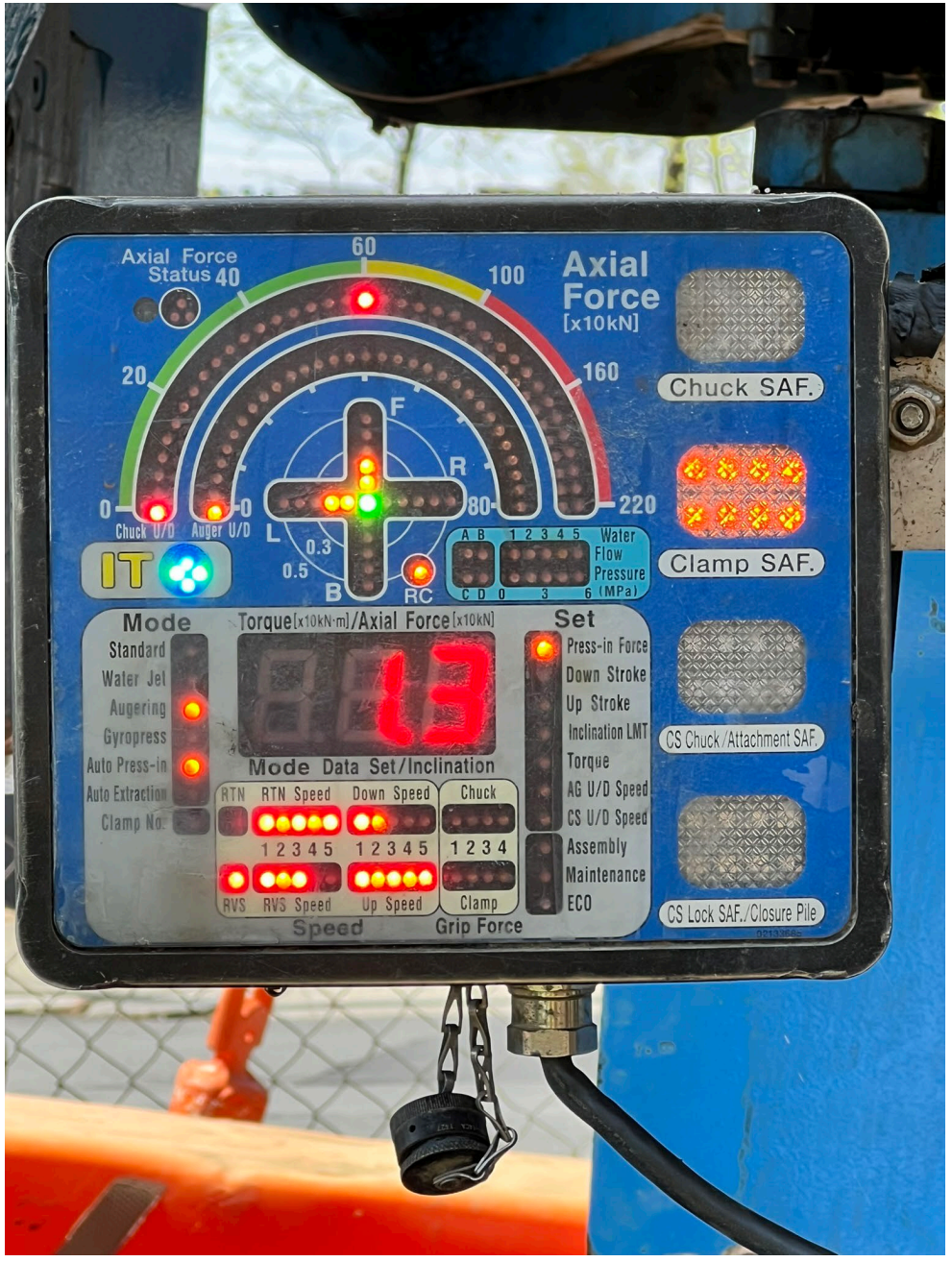


Installation in Progress





Installation in Progress



Installation in Progress





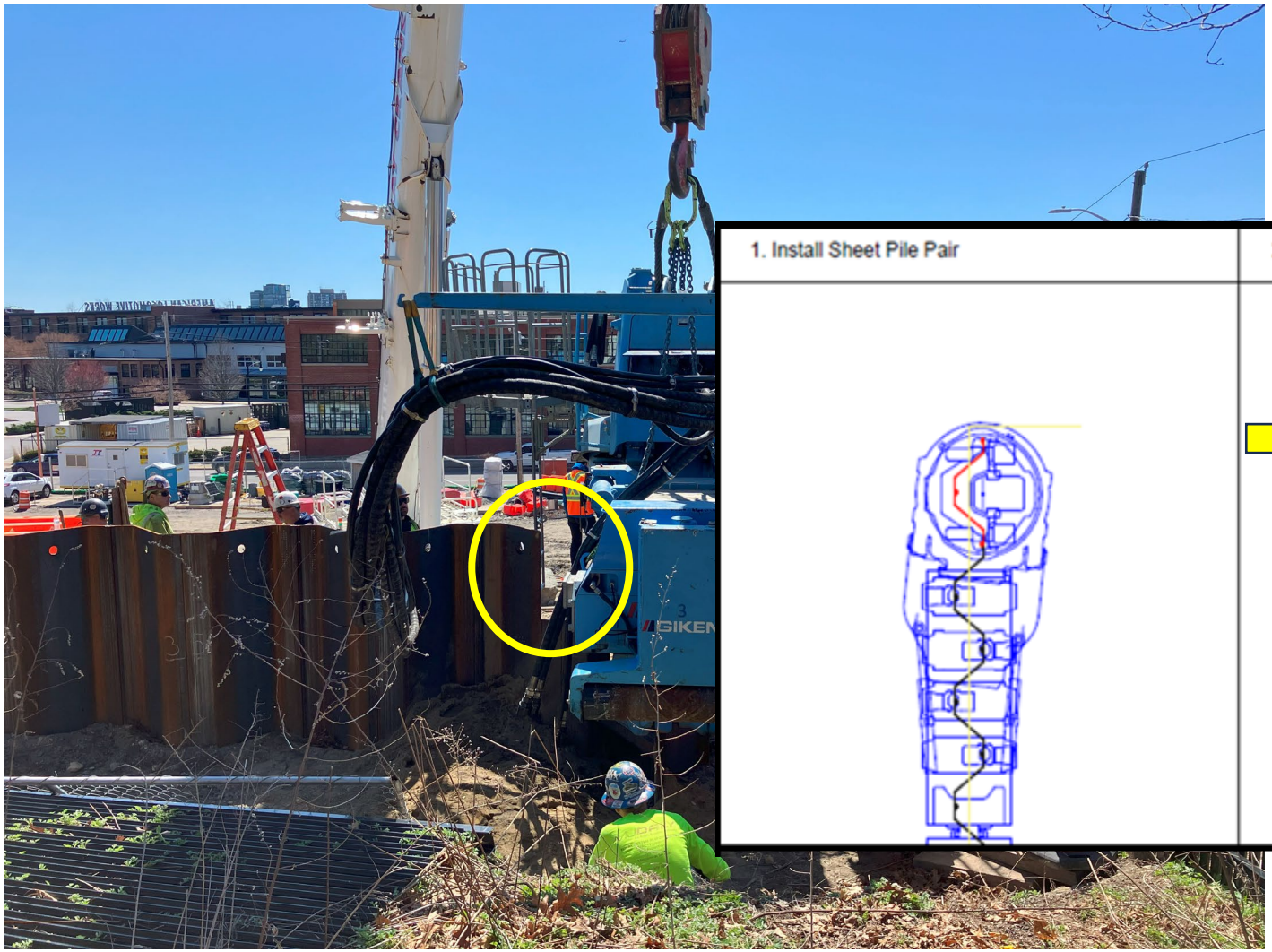
Machine Relocation

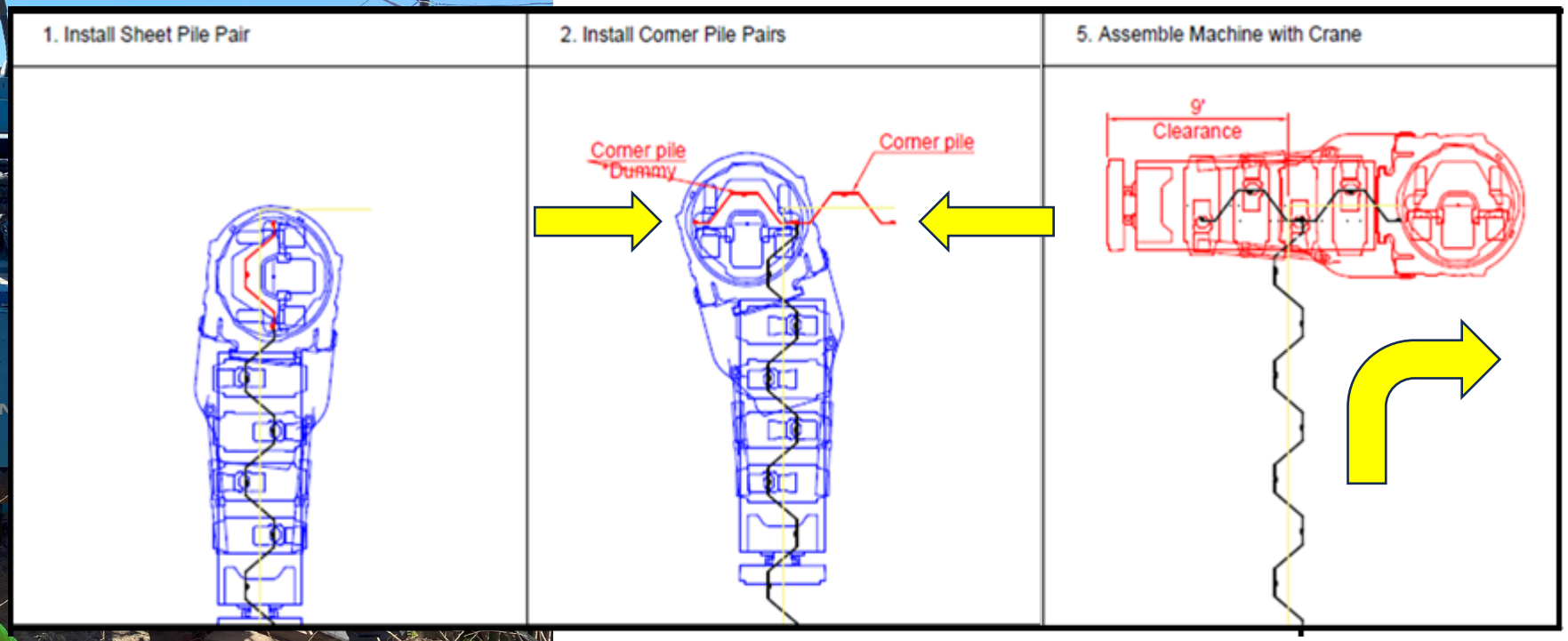


Machine Relocation



Installation in Progress





Installation in Progress



Installation in Progress



Completed Sheet Pile Installation



Conclusion

QUESTIONS?



Acknowledgements

- **Mr. Takefumi Takuma** of Giken Ltd.
- **Mr. Eric Alberghini** of Blakeslee Arpaia Chapman

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Case Study: Reducing Vibrations with a Shallow Trench

Presented by Rick Sadler - rsadler@iceusa.com
ICE®- International Construction Equipment, Inc.

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Purpose for Trench Screening Studies

To lessen ground vibration on new and existing Construction sites that can be a nuisances to the public.

Prove a technique known as trenching will lessen the outward scope from a Pile Driving, Drilling and other construction equipment on a site by stopping/breaking the ground movement /vibration.

To help facilitate the already proven safety factors around Vibratory Pile Driving by directly addressing the public perception caused by what people feel during driving.

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Equipment to Accomplish Trench Screening

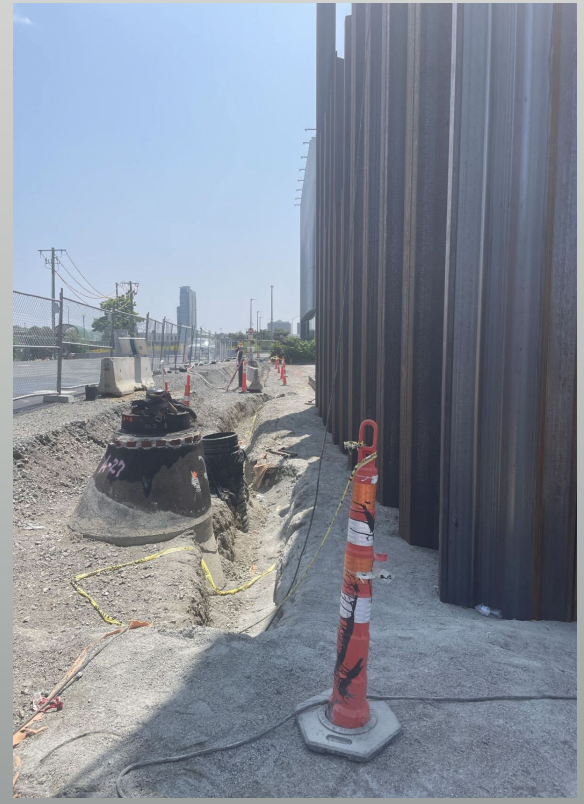
Nothing Technical



These tests used an irrigation trencher in Springfield, MA and larger excavators in other locations

Trench install approximately 3ft off the driven sheets





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Sites Utilizing Trench Screening

Selection by Availability



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Sites Utilizing Trench Screening

Location #1 – Springfield, MA

- Not Equipment Monitored -

Dug Open Trench (no support- dry soil)
- 4” wide x 18” deep
- Approx. 3 feet from the sheet pile wall

Over time, the open trench began to refill with dirt from the site making the trench less useful but even the soil disturbance as it refilled the trench provided a good outcome.

Outcome - overall perception
Every person on this site could feel less to zero vibration “at their feet” on the opposite side of this small trench.

** There was no tangible data collected during this exercise.*



Overview - Springfield, MA

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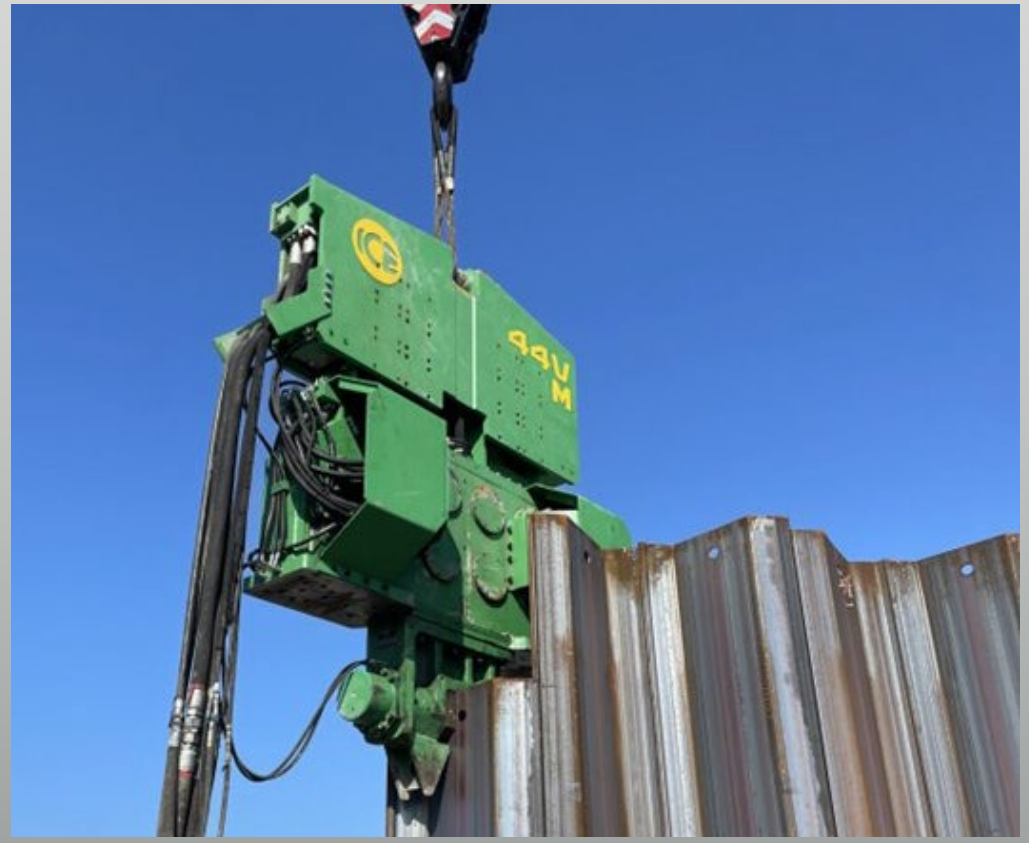
Sites Utilizing Trench Screening

Location #2 - Boston, MA

- Geophone & Seismograph Monitored -



ICE® Model 44 Variable Moment Vibratory Hammer
ICE® Model 580T - Tier 4F Power Unit

Dug Open Trench (no support- dry soil)
- 20” wide x 18” deep
- Approx. 3 feet from the sheet pile wall
- 300 feet in length (full sheet wall length)

ICE® 44 Variable Moment (VM) Vibratory Hammers installed over 1000 pairs of 13 sections of AZ-18 to AZ-42 with sheets ranging 35’-45’ in length.

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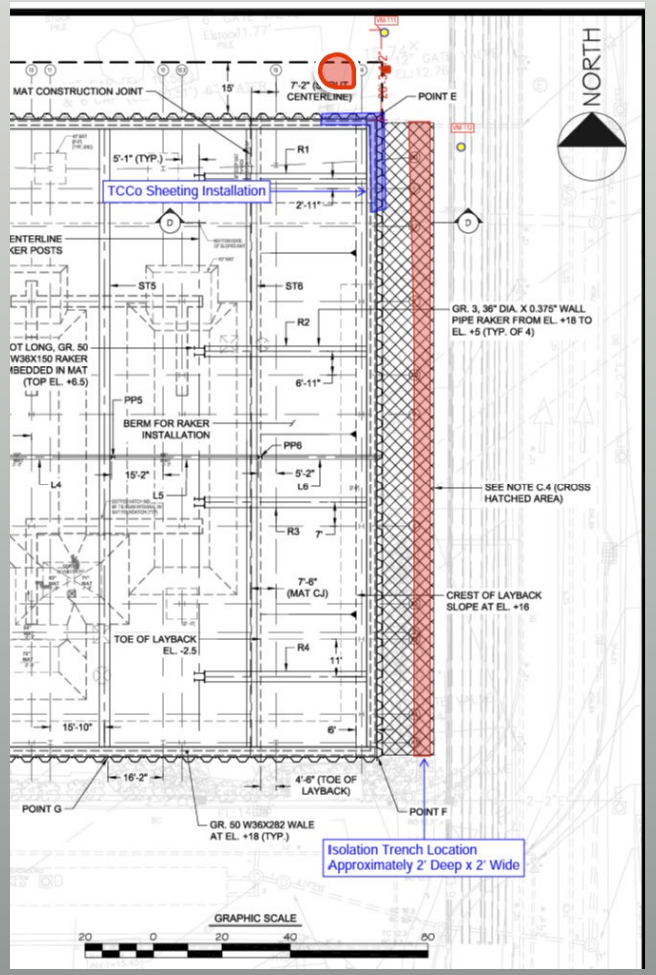
Sites Utilizing Trench Screening

Location #2 - Boston, MA

- Geophone & Seismograph Monitored -



This project was well planned and executed with monitoring program.

The ICE® 44 Variable Moment Vibratory Hammer was selected for its ability to begin the drive already in sync without extra startup or shutdown energy.

Choices were made to monitor both trenched and non-trenched driving

Seismograph Monitoring was also utilized on the Underground Utilities with both trench and non-trenched driving.

2 Methods were selected for monitoring based on the underground utilities running within 2 feet of the projects driving.

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Sites Utilizing Trench Screening

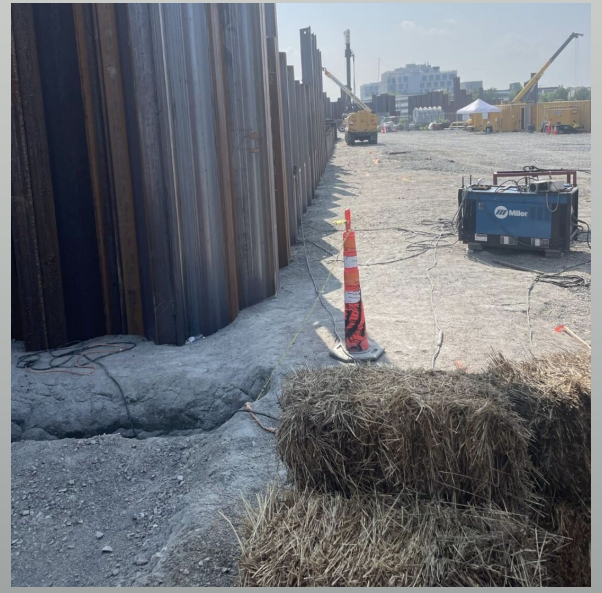
Location #2 - Boston, MA

- Geophone & Seismograph Monitored -

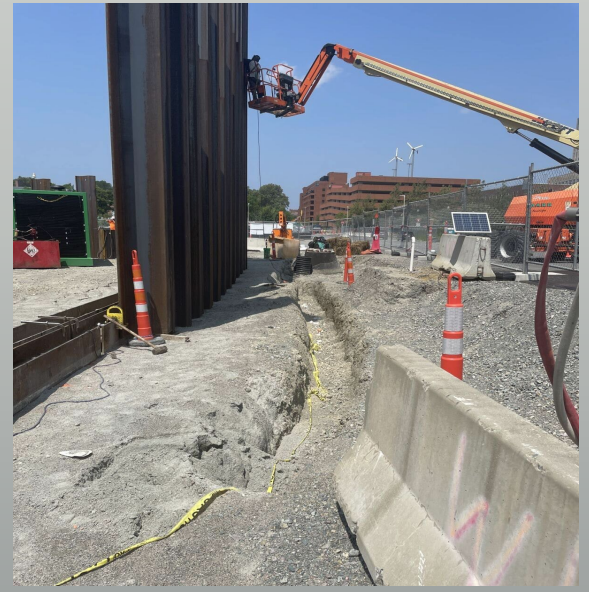




Trench Screening occurred for 300 Linear Feet and between the buried utility and sheet pile wall. The trench was cut with an onsite excavator.



Trenching ended at corner before the sheets turn 90⁰ and continue



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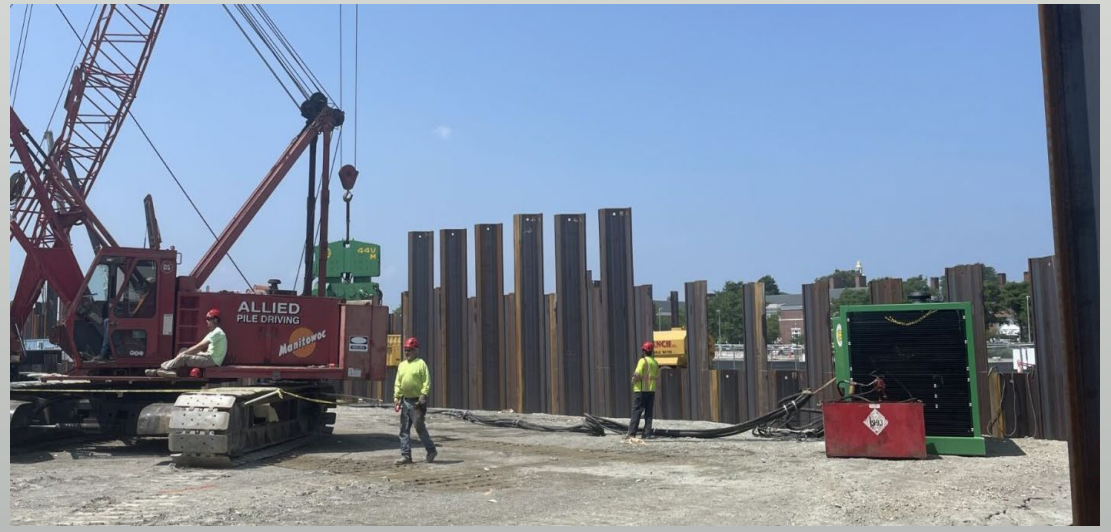
Sites Utilizing Trench Screening

Location #2 - Boston, MA

- Geophone & Seismograph Monitored -





There was zero issue with trench decomposition as the soil conditions were extreme.

Due to extreme hard soils, Contractor choose to switch from pairs to single sheets as we got close to final tip elevation.

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Sites Utilizing Trench Screening

Location #2 - Boston, MA

- Geophone & Seismograph Monitored -



Geophone - VM-111

This geophone (VM-111) is monitoring the area without the trench.

It is protected by orange cones.

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Sites Utilizing Trench Screening

Location #2 - Boston, MA

- Geophone & Seismograph Monitored -

Trench - - - - - -



Geophone - VM-112

This geophone (VM-112) is monitoring the area with the trench.

- - - - - - - - - - - - -

It is protected by orange cones.

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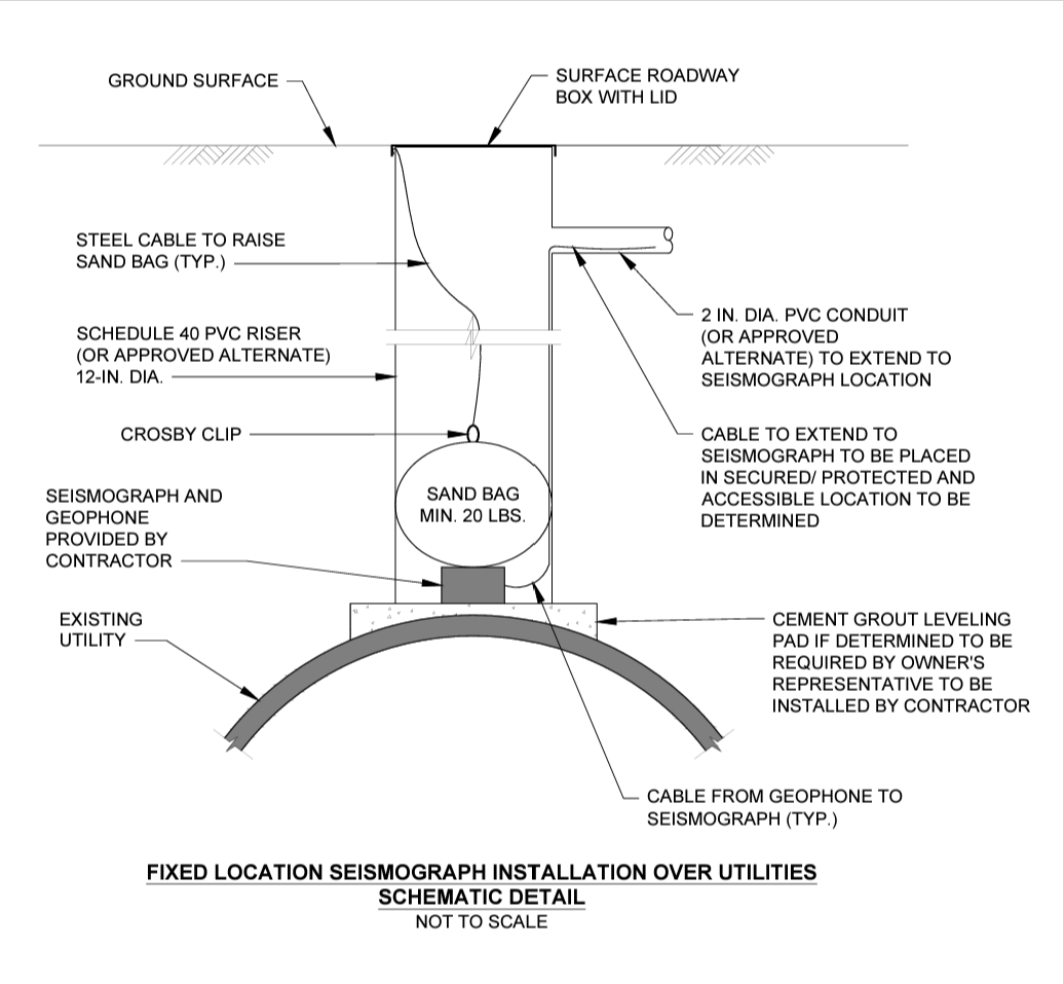
Sites Utilizing Trench Screening

Location #2 - Boston, MA

- Geophone & Seismograph Monitored -



This Seismograph was placed to monitor the public utilities.

The utilities were a main focus for the Trench Screen to protect.

Trench Screen was dug between the utility, and the sheet pile wall install.

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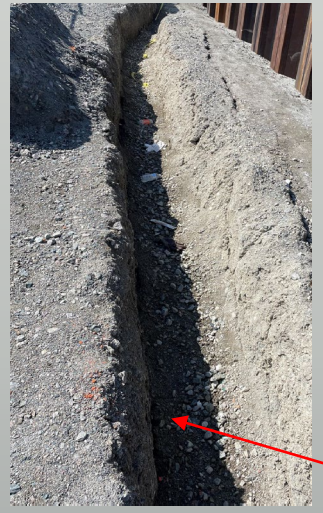
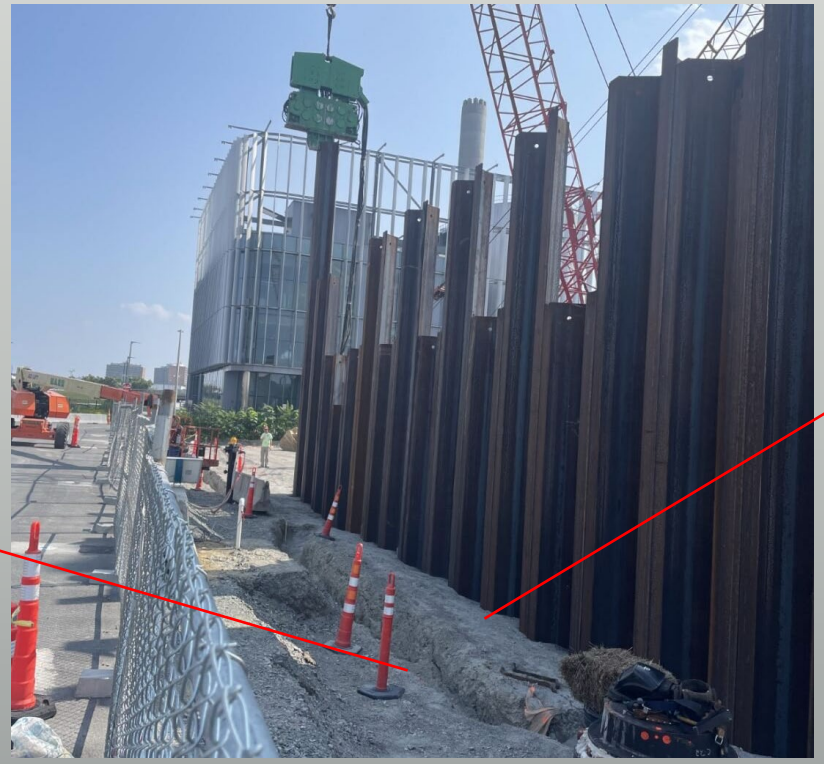
Sites Utilizing Trench Screening

Location #2 - Boston, MA

- Visual Results & Human Factors -

Trench side away from driving.





Trench side next to driving.



No physical signs of vibration
Nothing felt by the onsite team on this side of trench

Physical surface signs of vibration/ earth penetration
Did not cross over the 18” deep trench
Did not show effects past few inches on surface

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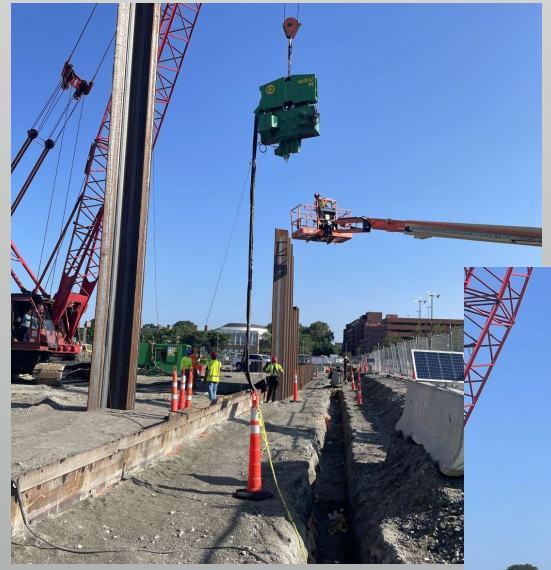
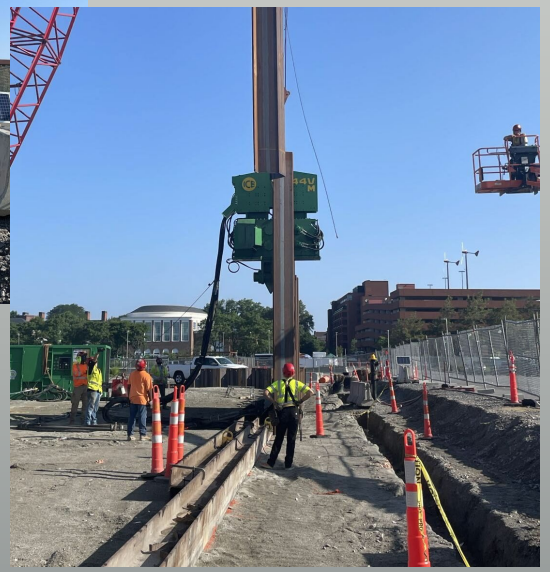
Sites Utilizing Trench Screening

Location #2 - Boston, MA

- Visual Results & Human Factors -





As the drive penetrated deeper, we saw progressively less disruption to the top layers of soil where there was trenching.



In all areas without trenching, we did see surface disruption but nothing more then 3” of superficial disruption – most likely from hard soil

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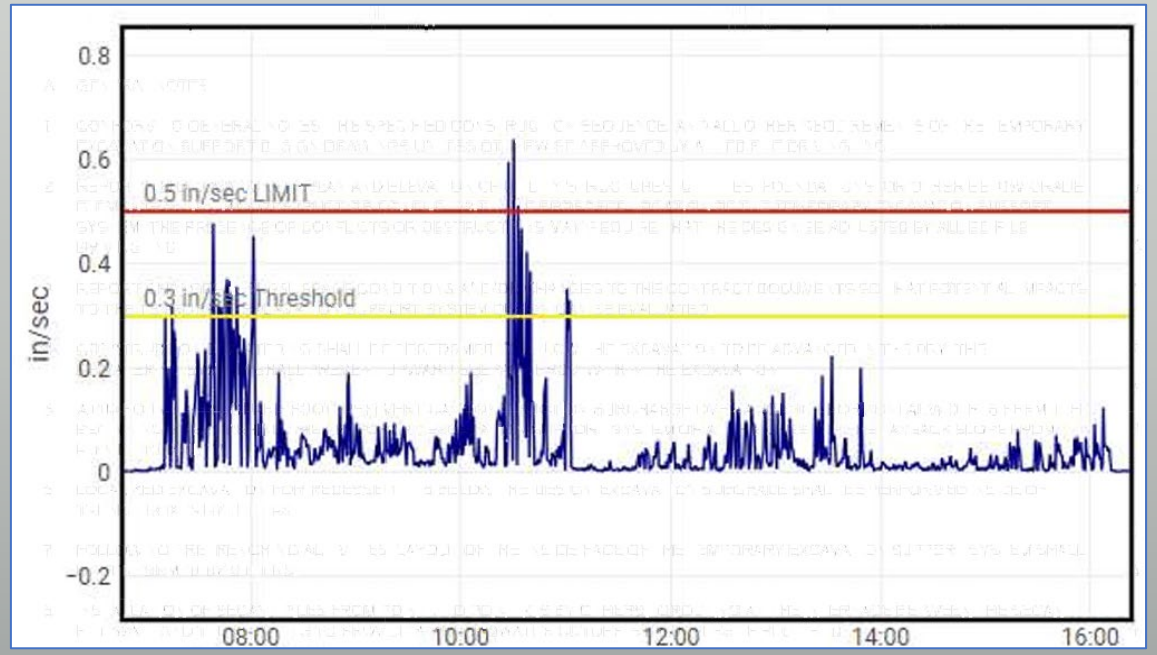
Sites Utilizing Trench Screening

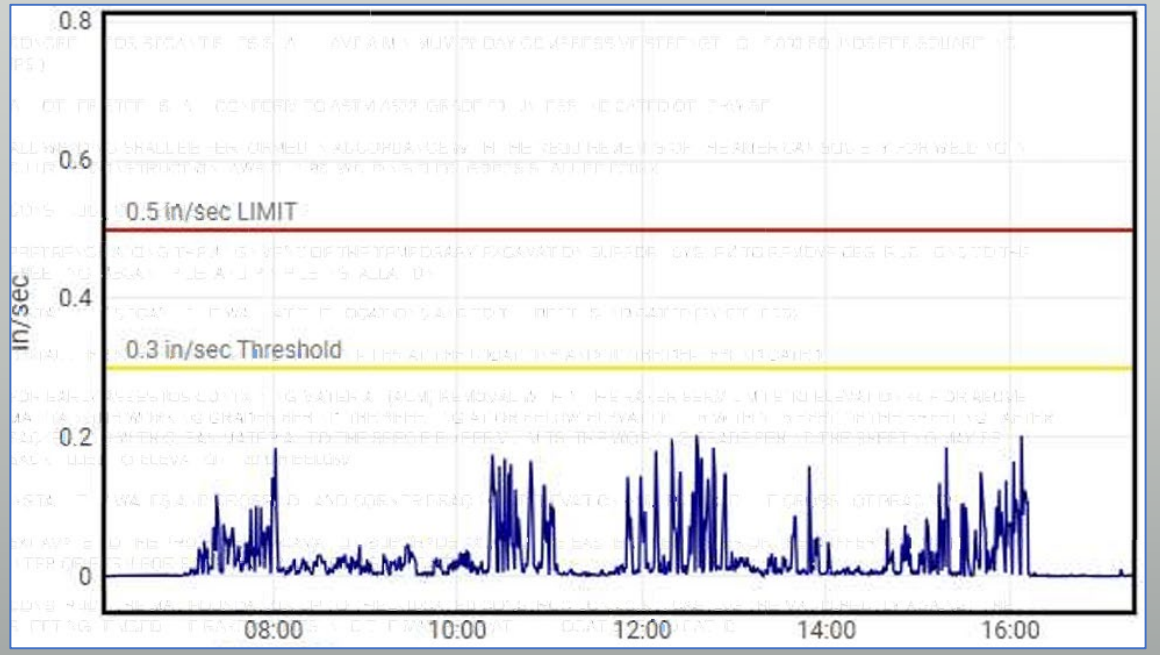
Location #2 - Boston, MA

- Geophone Results -

Geophone VM-111 – NO TRENCH



Geophone VM-112 – TRENCH SCREEN



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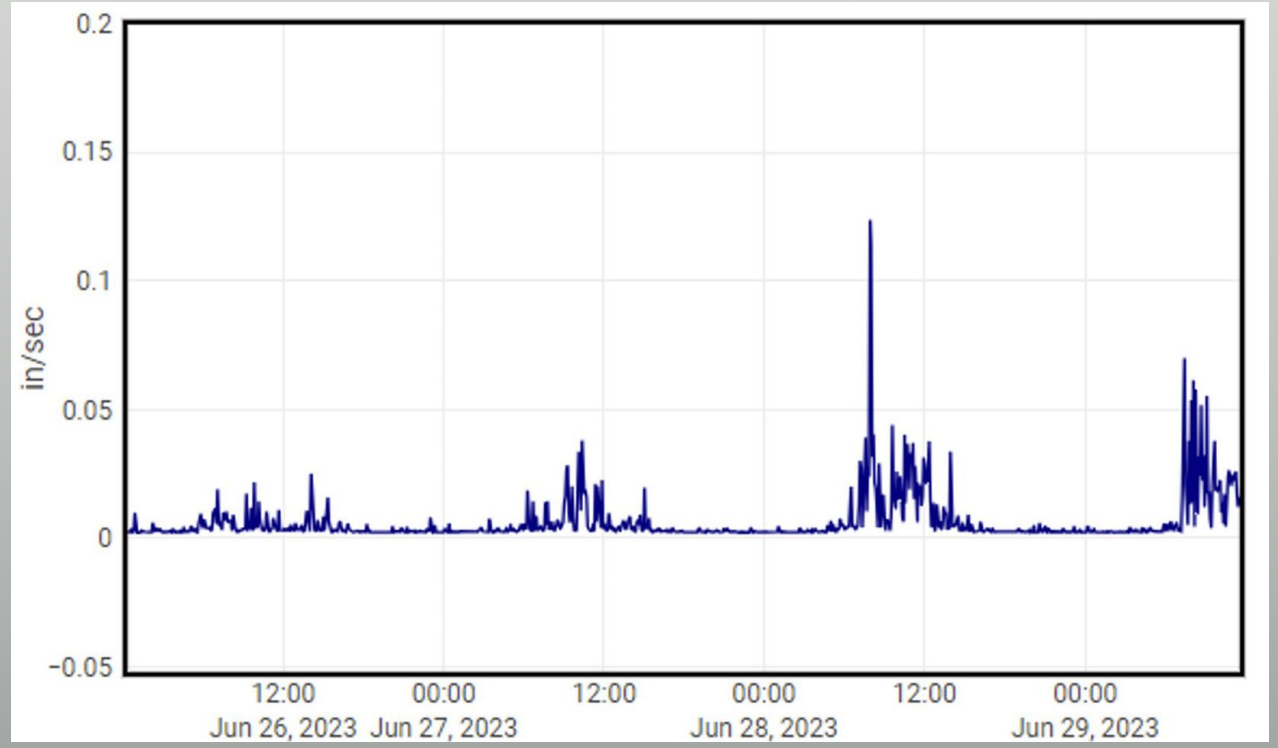
Sites Utilizing Trench Screening

Location #2 - Boston, MA

- Seismograph Results -



Seismograph placed on underground utility

Thresholds of .5 never reached…

Trench screen readings never past 0.122

Job owner and onsite crews thrilled with what a simple trench produced for safety and community…

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TEST BORING REPORT

HALEY ALDRICH

Boring No. TB21-HB(OW)

Project	HARVARD ENTERPRISE RESEARCH CAMPUS (ERC)-PHASE A, ALLSTON, MA
Client	TISHMAN SPEYER ERC DEVELOPER, LLC
Contractor	NORTHERN DRILL SERVICE, INC.

	Casing	Sampler	Barrel
Type	HW	S	
Inside Diameter (in.)	4	1.4	
Hammer Weight (lb)	140	140	-
Hammer Fall (in.)	30	30	-

Drilling Equipment and Procedure
Rig Make & Model: Mobile Drill B-59
Bit Type: Roller Bit
Drill Mud: None
Casing: Wash and Drive
Hoist/Hammer: Winch / Automatic hammer
PID Make & Model: Tiger 10.6 eV

File No. 134043-003
Sheet No. 1 of 2
Start August 6, 2021
Finish August 6, 2021
Driller J. Stevens
H&A Rep. C. Romero
Elevation 16.7
Datum Boston City Base
Location See Plan
N 2957491.7049
E 765718.4914

Depth (ft)	Sampler Blows per 6 in.	Sample No. & Recv. (in.)	Sample Depth (ft)	PID Readings (ppm)	USCS Symbol	Stratum Change Elev/Depth (ft)	VISUAL-MANUAL IDENTIFICATION AND DESCRIPTION (Color, GROUP NAME, max. particle size†, structure, odor, moisture, optional descriptions) GEOLOGIC INTERPRETATION	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	Sand % Fines	Field Test Dilatancy	Field Test Toughness	Field Test Plasticity	Field Test Strength
0	15 13 9 10	S1 7	0.0 2.0	0.4	SM		Medium dense gray silty SAND with gravel (SM), no odor, dry	20	25	15	15	10					
	14 15 16 13	S2 16	2.0 4.0	0.0	SM		Medium dense gray to brown silty SAND with gravel (SM), no odor, moist -FILL-	15	10	15	25	20	15				
5	7 6 4 4	S3 11	4.0 6.0	0.4	SM		Loose gray to dark brown silty SAND with gravel (SM), no odor, wet, brick and ash fragments	15	25	15	20	10	15				
	5 7 6 2	S4 15	6.0 8.0	0.7	SM		Medium dense brown to dark brown silty SAND with gravel (SM), no odor, wet	10	20	15	15	25	20				
	WOH WOH 1 2	S5 23	8.0 10.0	9.1	OL/OH	7.7 8.8	Very soft dark brown to black ORGANIC SOIL (OL/OH), no odor, moist - ORGANIC DEPOSITS -						100				
10	WOH WOH WOH 5	S6 21	10.0 12.0	8.7	SW-SM	8.7 10.0	Very loose brown to dark brown well-graded SAND with silt (SW-SM), chemical-like odor, moist	20	15	20	20	15	10				
	13 17 21 24	S7 17	12.0 14.0	11.8	SW		Dense gray well-graded SAND with gravel (SW), no odor, moist	10	10	20	25	20	15				
	16 32 31 35	S8 11	14.0 16.0	0.7	SW-GM		Very dense gray well-graded GRAVEL with silt and sand (SW-GM), no odor, moist	10	15	15	20	20	15				
15							Note: Laboratory grain size analysis performed.	29	19	11	21	11	10				
	35 30 26 31	S9 23	16.0 18.0	0.0	SW		Very dense gray well-graded SAND with gravel (SW), no odor, moist - FLUVIAL DEPOSITS -	10	15	20	25	20	10				
	12 13 15 15	S10 10	18.0 20.0	0.7	SW		Medium dense gray to light-brown well-graded SAND with gravel (SW), no odor, wet	15	25	15	20	15	10				
20																	

Water Level Data

Date	Time	Elapsed Time (hr.)	Depth (ft) to: Bottom of Casing	Bottom of Hole	Water
8/6/2021	11:54	8.0	20.0	20.0	3.8

Sample ID
O - Open End Rod
T - Thin Wall Tube
U - Undisturbed Sample
S - Split-spoon Sample
G - Geoprobe

Well Diagram
Riser Pipe
Screen
Filter Sand
Cuttings
Grout
Concrete
Bentonite Seal

Summary
Overburden (ft) 30.0
Rock Cored (ft) -
Samples S10
Boring No. TB21-HB(OW)

Field Tests: Dilatancy: R - Rapid S - Slow N - None
Toughness: L - Low M - Medium H - High
Plasticity: N - Nonplastic L - Low M - Medium H - High
Dry Strength: N - None L - Low M - Medium H - High V - Very High

†Note: Maximum particle size is determined by direct observation within the limitations of sampler size.

Note: Soil identification based on visual-manual methods of the USCS as employed by Haley & Aldrich, Inc.

HALEY ALDRICH

TEST BORING REPORT

Boring No. TB21-HB(OW)
File No. 134043-003
Sheet No. 2 of 2

Depth (ft)	Sampler Blows per 6 in.	Sample No. & Recv. (in.)	Sample Depth (ft)	PID Readings (ppm)	USCS Symbol	Stratum Change Elev/Depth (ft)	VISUAL-MANUAL IDENTIFICATION AND DESCRIPTION (Color, GROUP NAME, max. particle size†, structure, odor, moisture, optional descriptions) GEOLOGIC INTERPRETATION	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	Sand % Fines	Field Test Dilatancy	Field Test Toughness	Field Test Plasticity	Field Test Strength
20	12 20 20 25	S11 12	20.0 22.0	0.0	SW		Dense gray well-graded SAND with gravel (SW), no odor, wet	15	25	20	15	15	15				
	24 21 19 17	S12 18	22.0 24.0	0.0	SW		Dense gray to orange well-graded SAND with gravel (SW), no odor, wet	15	25	15	20	15	10				
25	5 12 14 15	S13 9	24.0 26.0	0.0	SW		Medium dense gray well-graded SAND with gravel (SW), no odor, wet	15	20	15	25	15	15				
	10 14 17 19	S14 33	26.0 28.0	0.0	SW		Dense gray well-graded SAND with gravel (SW), no odor, wet - FLUVIAL DEPOSITS -	15	20	15	25	15	10				
	10 13 9 10	S15 7	28.0 30.0	0.0	SW	-12.3 28.8	Medium dense gray well-graded SAND with gravel (SW), no odor, wet	15	25	15	25	15	10				
30						-14.3 30.8	BOTTOM OF EXPLORATION 30.0 FT										

NOTE: Soil identification based on visual-manual methods of the USCS as employed by Haley & Aldrich, Inc.

Boring No. TB21-HB(OW)

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Sites Utilizing Trench Screening

Location #3 – Keene, NH

- Not Equipment Monitored -



Overview

- Extreme necessarily for a Trench Screen
- House is located only 5 feet, 6 inches from steel sheet wall and water's edge
- 50' (Ft) PZ-27 are necessary to be installed to support earth excavations for new bridge and save house in the process
- A 12" wide 36" deep trench was hand dug with shovels
- 2" thick foam insulation boards used to support trench walls.
- Trench placed between the sheet pile and the house, approximately 12" from the sheet pile wall.

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Sites Utilizing Trench Screening

Location #3 – Keene, NH

- Not Equipment Monitored -

This was so effective that everyone standing on site was surprised by the noticeable reduction in the vibrations felt.



Aerial image #1



Aerial image #2

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Sites Utilizing Trench Screening

Location #3 – Keene, NH

- Not Equipment Monitored -

Going the extra mile or in this case 12” is the right move for those involved.

Contractor and Homeowner both explained:

You don’t need technology to tell us that this trench is effective.

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Theories & Next Site:

Most vibration is at the time of soil penetration to about 36” into the soil.

Vibration wave shape is best managed when the Trench that is V shaped even though any trench is better than none for the human sensitivity factor.

What is the ideal depth…?

Testing at next site on 36” and 72”, with press, vibratory and impacts all running.

Want to be a part of the next test?

Testing and film crew's pre-academy – ask ICE® how you can get involved…

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- Hands-on equipment experience
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- Participate with a full load test
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Contact - Pollyanna Cunningham
pcunningham@iceusa.com
Cell - 717-360-0935
Contact - Diane Fischer
dfischer@pile.com



Let’s debunk the next construction noise and vibration complaint together!

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PARALLEL SESSION A2
Gold Innovations II



PILING SYSTEMS
FAST. SIMPLE. SAFE.

THE FOUNDATIONS OF THE RAILWAY BRIDGE OVER THE B25 IN DONAUWÖRTH (GERMANY)

MSc.-Ing. Freddy Lopez (Tiroler Rohre GmbH, AUSTRIA)
Ing. Dipl.-Bw. Erich Steinlechner MBA (Tiroler Rohre GmbH, AUSTRIA)

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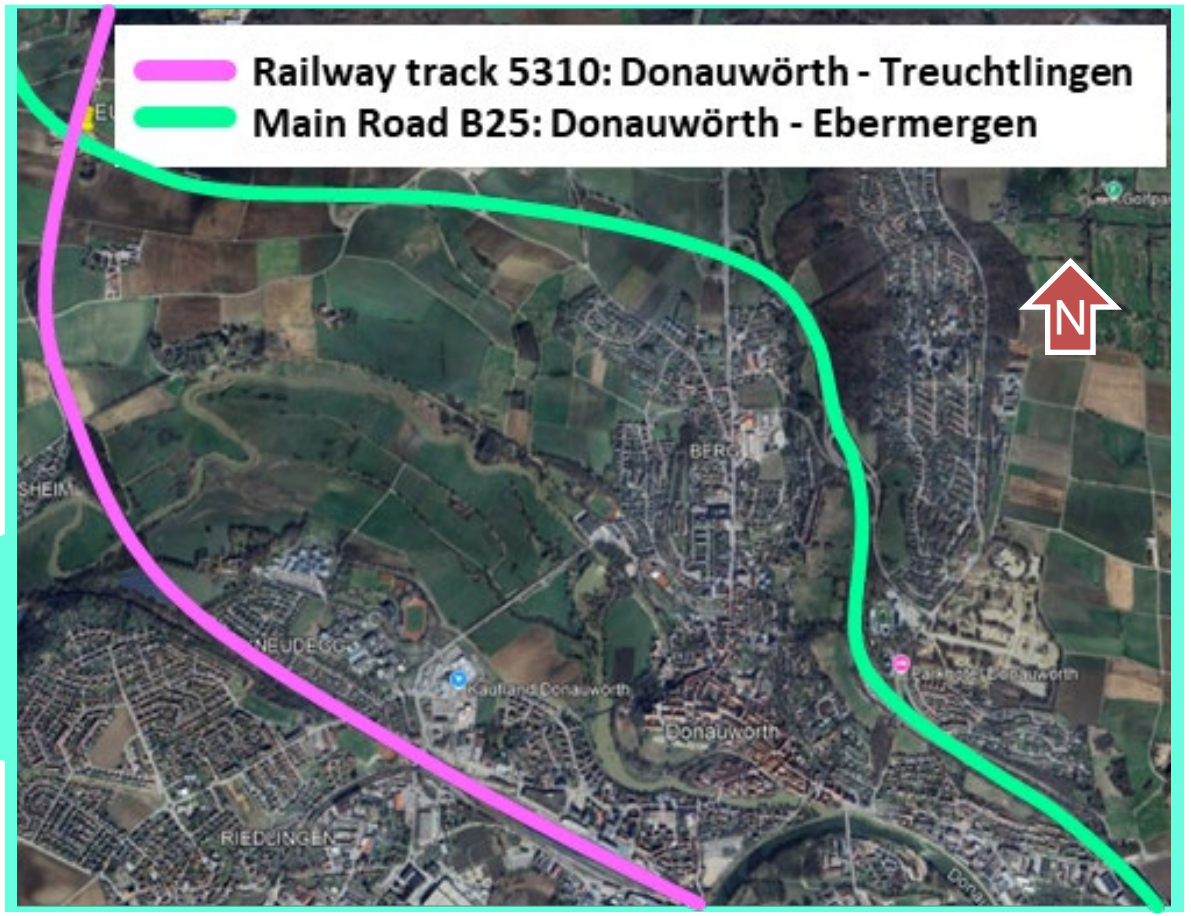
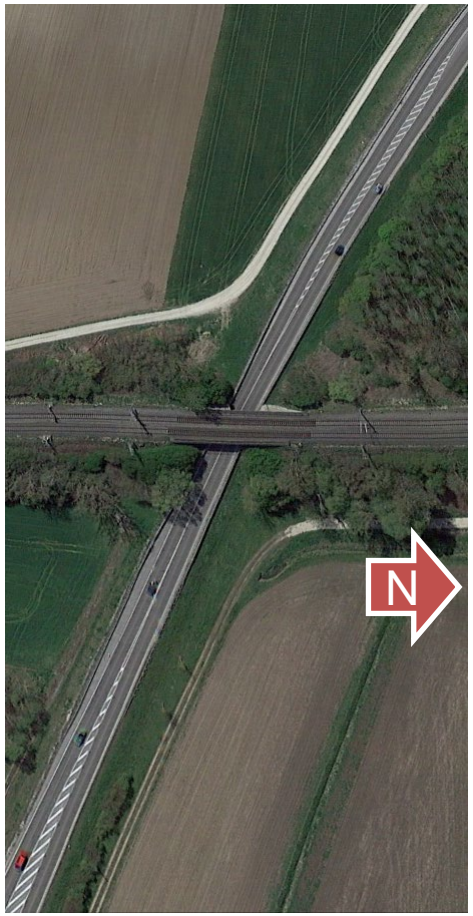
PROJECT DESCRIPTION

THE GERMAN RAILWAYS (***DEUTSCHE BAHN***) EXECUTED AN INTERVENTION TO MODERNIZE A RAILWAY OVERPASS ALONG THE **RAILWAY TRACKS 5310 (DONAUWÖRTH - TREUCHTLINGEN)** OVER THE **MAIN ROAD B25**, LOCATED AT THE OUTSKIRTS OF **DONAUWÖRTH** (SOUTHERN GERMANY).







SOURCE: © GOOGLE MAPS (2024) AND © GOOGLE EARTH (2021)

PROJECT DESCRIPTION

THE GERMAN RAILWAYS (***DEUTSCHE BAHN***) EXECUTED AN INTERVENTION TO MODERNIZE A RAILWAY OVERPASS ALONG THE **RAILWAY TRACKS 5310 (DONAUWÖRTH - TREUCHTLINGEN)** OVER THE **MAIN ROAD B25**, LOCATED AT THE OUTSKIRTS OF **DONAUWÖRTH** (SOUTHERN GERMANY).



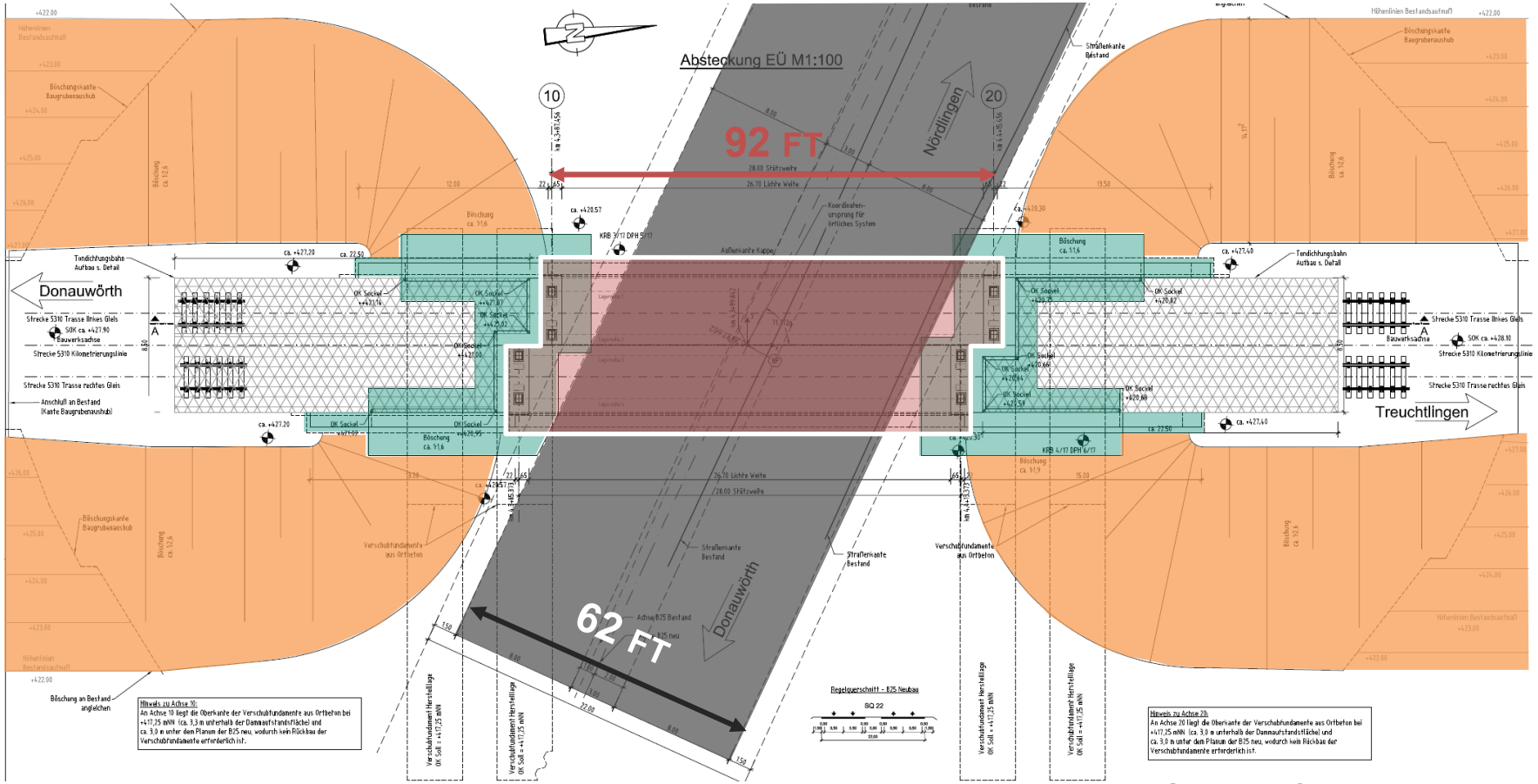


- REINF. CONCRETE STRUCTURE
- LENGTH/WIDTH = 40 FT / 36 FT
- BUILT IN 1931
- EXTENSIVE RENOVATION OF THE SUPERSTRUCTURE IN 1977
- EXTENSIVE REPAIRS OF THE ABUTMENTS PLANNED FOR 2022 (DAMAGE DUE TO SUBSIDENCE)

SOURCE: © GOOGLE MAPS (AUGUST 2023)

CONSTRUCTION OF THE NEW OVERPASS

IN COORDINATION WITH THE BAVARIAN STATE BUILDING DIRECTORATE (AUGSBURG), THE GERMAN RAILWAYS USED THE OPPORTUNITY TO MODERNIZE THE ENTIRE BRIDGE AND TO RESOLVE TRAFFIC FLOW PROBLEMS ALONG THE MAIN ROAD B25 IN THE LONG TERM.

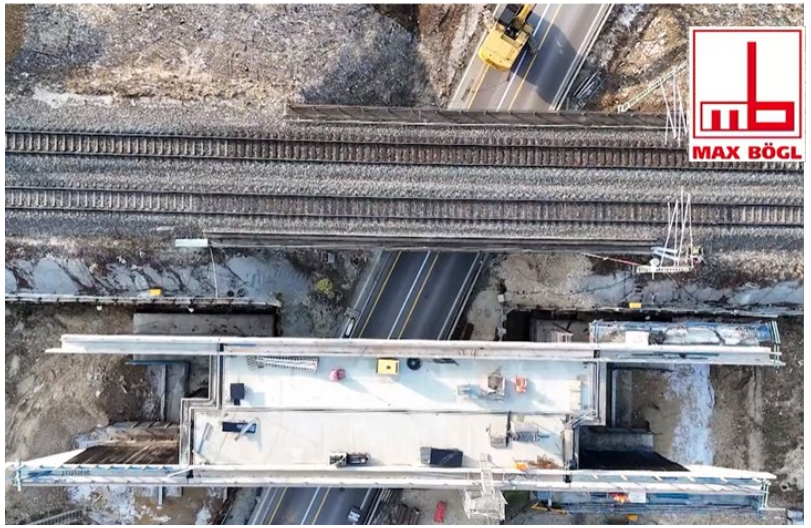


SOURCE: © ZPP INGENIEURE AG (DECEMBER 2023)

CONSTRUCTION OF THE NEW OVERPASS

LATERAL BRIDGE SLIDE (LBS) PROCEDURE: DUE TO TIGHT SCHEDULE ➔ 10 DAYS OF TRACK POSSESSION: TO DEMOLISH THE EXISTING BRIDGE, MOVE THE NEW BRIDGE AND REESTABLISH TRACK OPERATIONS







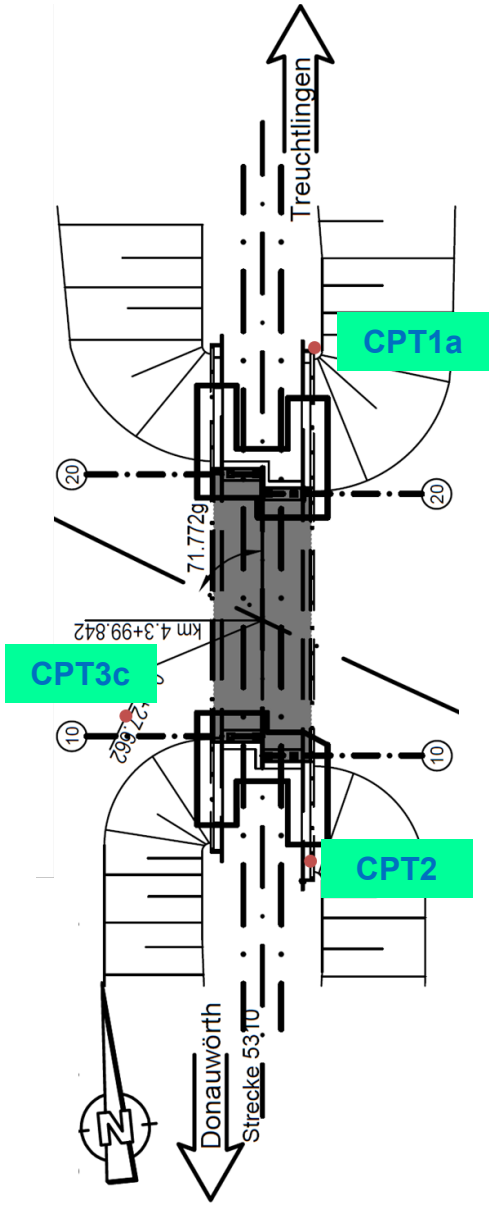


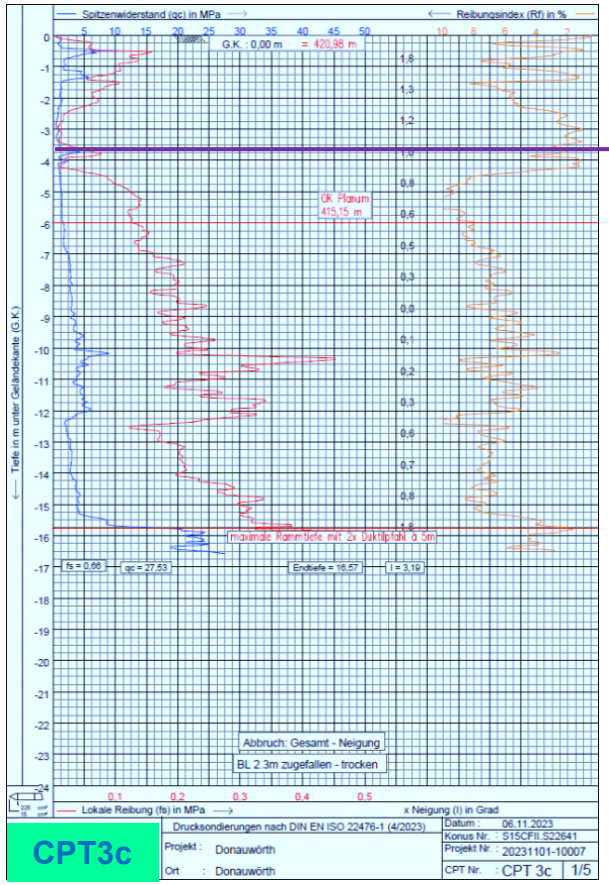




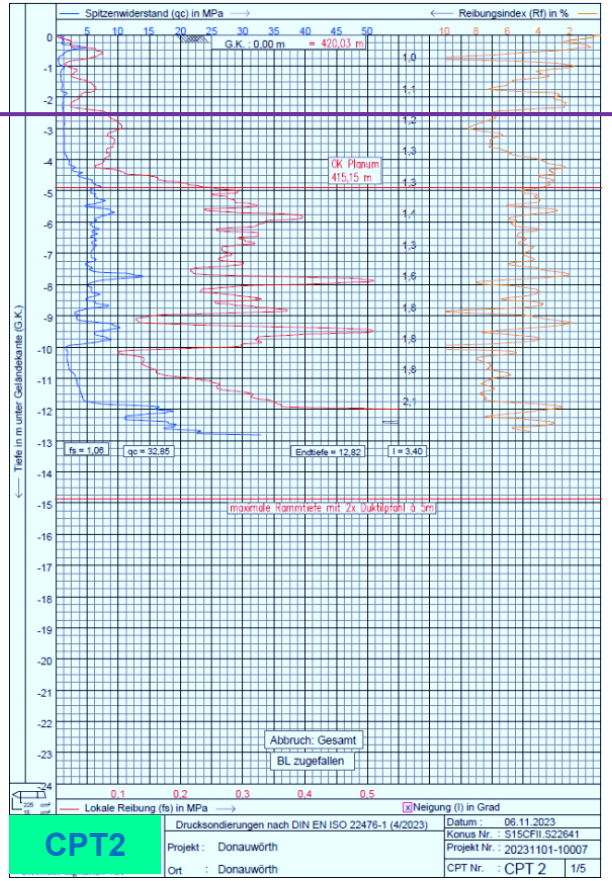
SOURCE: © RATISBONA MEDIA DOCUMENTATION SERVICES (2024)

GROUND CONDITIONS



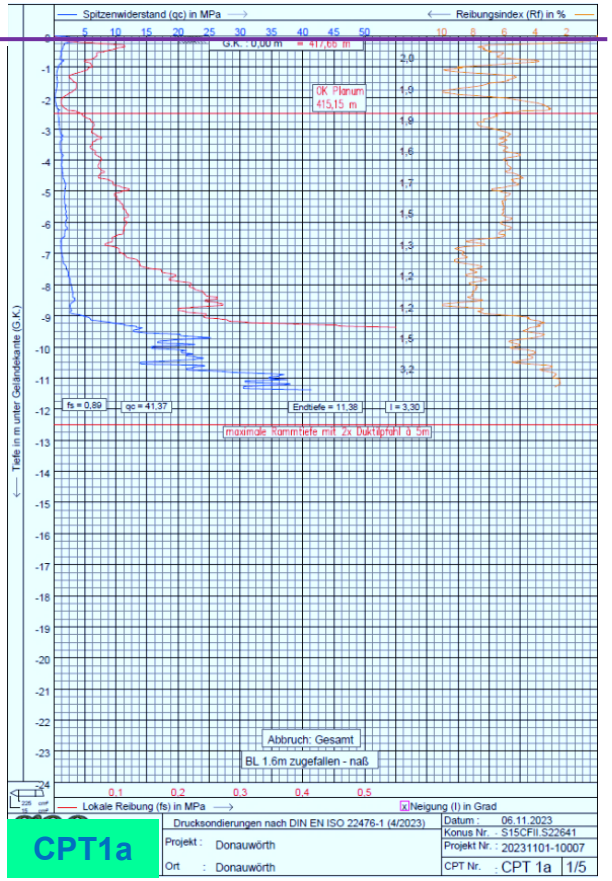


CPT3c



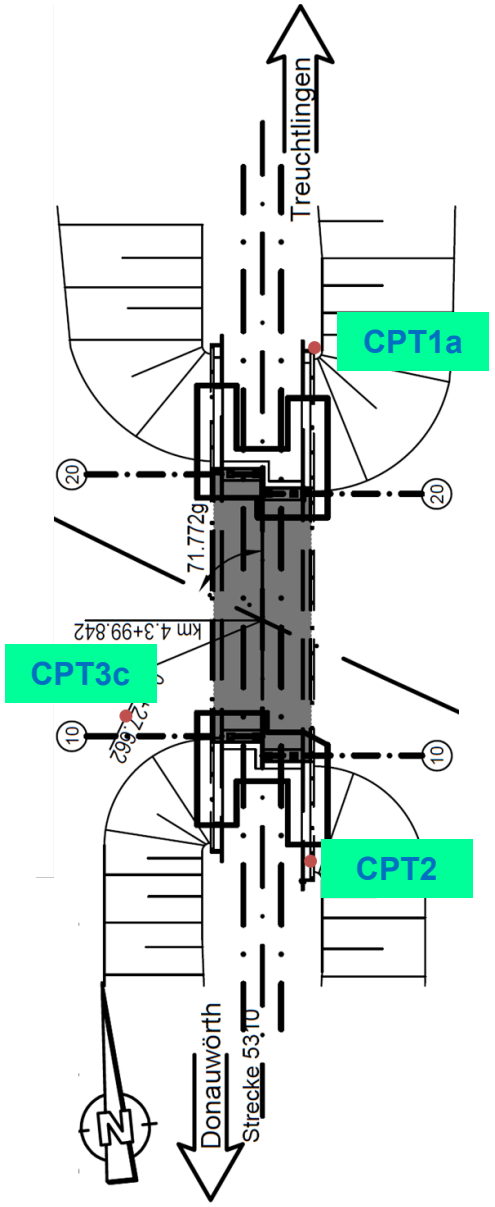
CPT2

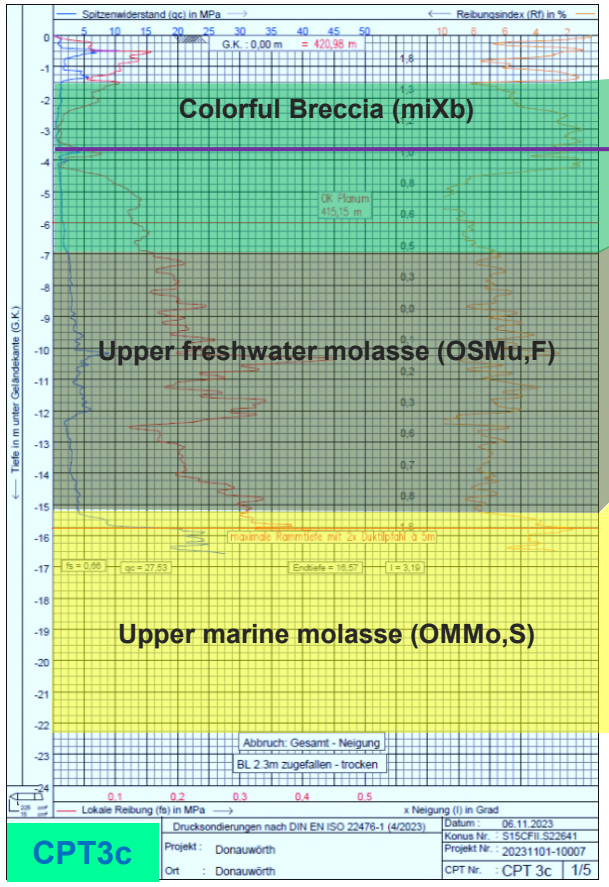
Underside abutment foundation +417.6m

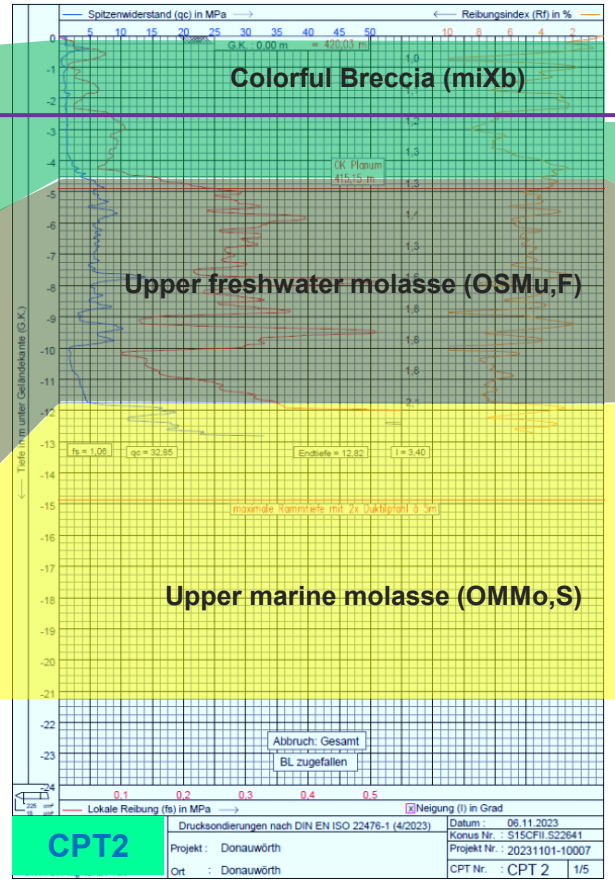


CPT1a

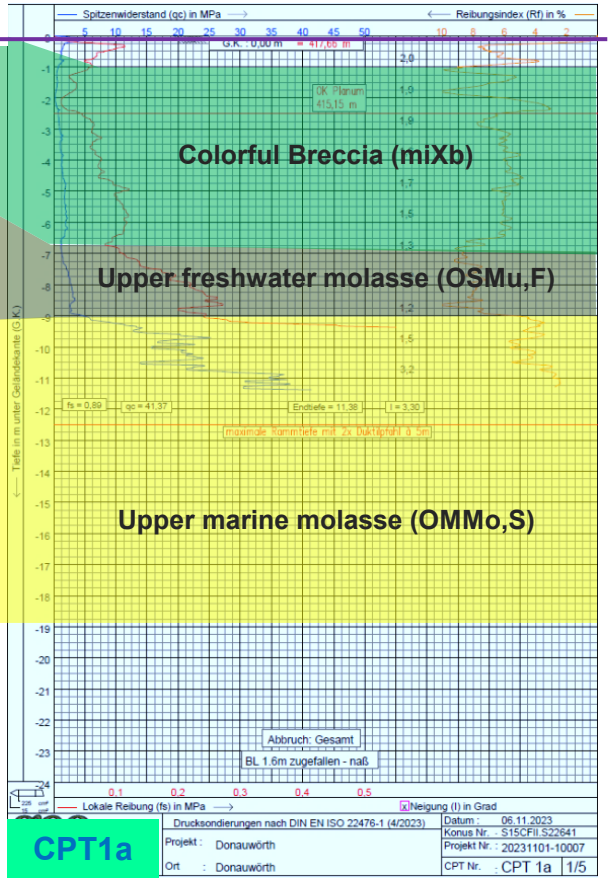
GROUND CONDITIONS







Underside abutment foundation +417.6m



COLORFUL BRECCIA: HIGHLY HETEROGENEOUS SILTY CLAYS, PARTIALLY SENSITIVE TO WATER AND FROST, SOFT TO STIFF CONSISTENCY

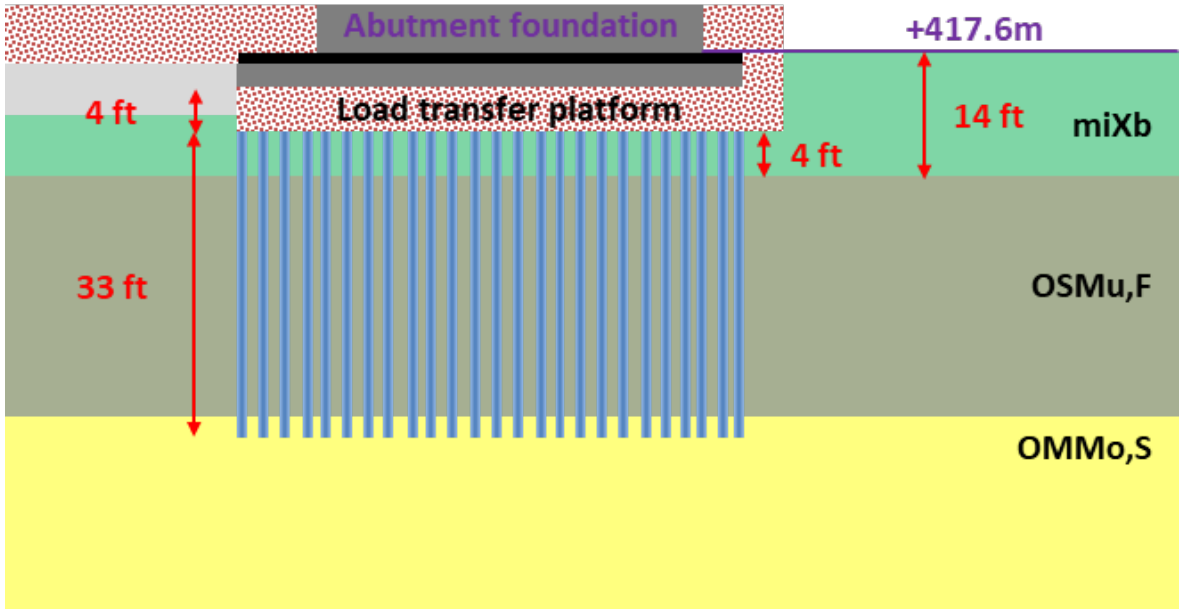
UPPER FRESH WATER MOLASSES: STIFF TO SEMI-SOLID SILTY CLAYS

UPPER MARINE MOLASSES: DENSE TO VERY DENSE SILTY FINE SANDS

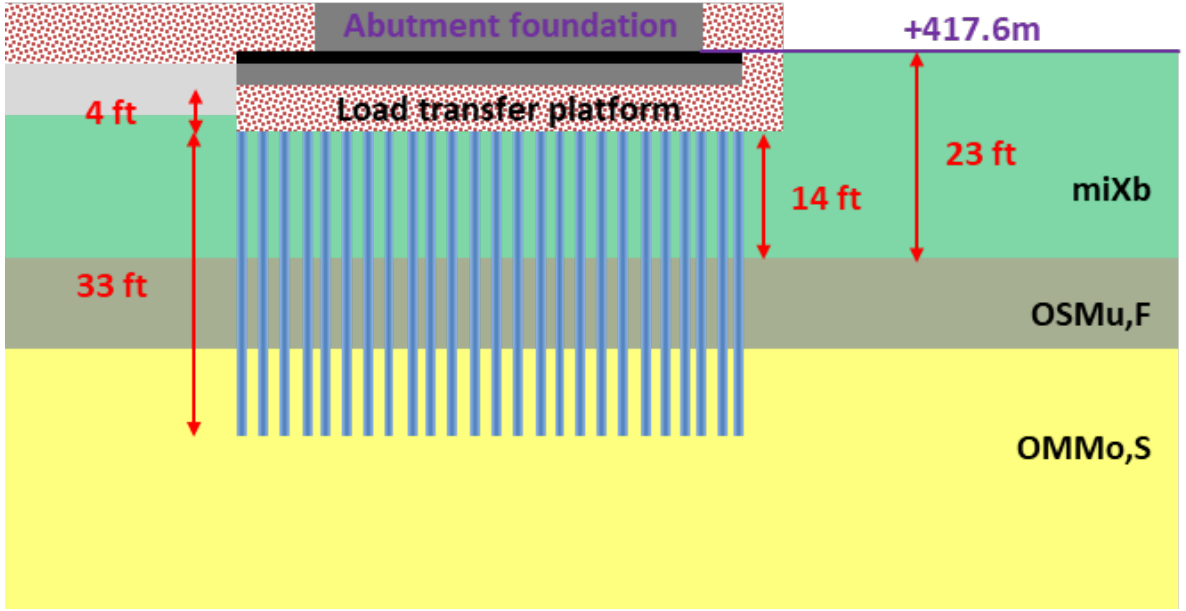
THE FOUNDATIONS OF THE NEW OVERPASS: GROUND IMPROVEMENT ➔ RIGID INCLUSIONS

THE ORIGINAL SOLUTION CONSIDERED A **SHALLOW FOUNDATION + SUBSOIL REPLACEMENT** (COLORFUL BRECCIA)

DUE TO THE HETEROGENEOUS GROUND CONDITIONS AND THE UNEXPECTED LARGE EXTENSION OF THE COLORFUL BRECCIA (23 FT AT THE NORTH ABUTMENT), IT WAS DECIDED TO CARRY OUT A **GROUND REINFORCEMENT** WITH **RIGID INCLUSIONS (RIS)** TO TRANSFER THE LOADS TO THE DEEPER LAYER OF **MARINE MOLASSES (OMMO,S),** TO MINIMIZE THE DIFFERENTIAL SETTLEMENTS DURING THE TEMPORARY LBS-PROCEDURE (**≤ 0.25 IN**) AND TO REDUCE THE SETTLEMENTS OF THE ABUTMENTS (**≤ 0.5 IN**).



SOUTH ABUTMENT (AXIS 10)



NORTH ABUTMENT (AXIS 20)

THE FOUNDATIONS OF THE NEW OVERPASS: GROUND IMPROVEMENT ➔ RIGID INCLUSIONS

DUE TO THE VERY TIGHT TIME SCHEDULE (**10 DAYS OF TRACK POSSESSION**), THE **RIS** NEEDED TO BE INSTALLED WITHIN 60 HOURS. UNDER THESE CONDITIONS, THE MOST SUITABLE, FAST AND COST-EFFECTIVE SOLUTION WAS PROVIDED BY **NON-GROUTED DUCTILE IRON PILES (DIPS)**.



- LOW VIBRATION, FULL-DISPLACEMENT DRIVEN PILES
- **COST-EFFECTIVE** AND **SUSTAINABLE** PILING SYSTEM (LOW CARBON MATERIAL)
- NON-GROUTED **DIPS** CAN BE **FULLY LOADED IMMEDIATELY** AFTER COMPLETION OF PILE DRIVING WORKS
- **TRM 170/10.6** PILES WERE CONSIDERED FOR THE PROJECT

SOURCE: © TIROLER ROHRE GMBH (TRM)

DESIGN CONSIDERATIONS:

TEMPORARY: LATERAL SLIDING PROCESS

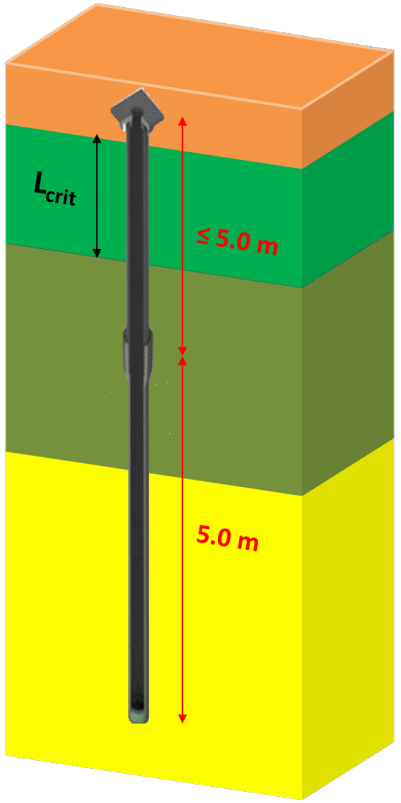
SELF-WEIGHT OF THE NEW BRIDGE STRUCTURE (**4,000 METRIC TONS**),
DISTRIBUTED OVER FOUR REINFORCED CONCRETE THRUST BEDS

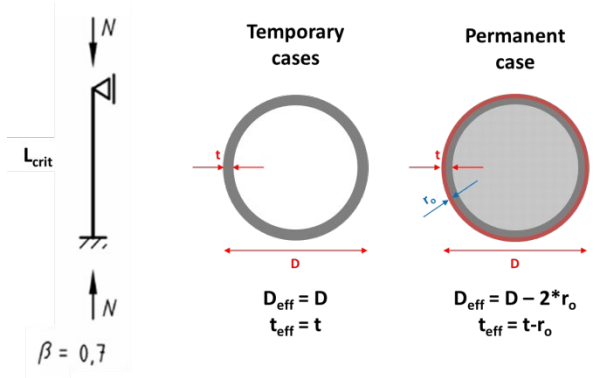
TEMPORARY: CONSTRUCTION OF THE NEW OVERPASS

SELF-WEIGHT OF THE NEW BRIDGE STRUCTURE (**4,000 METRIC TONS**),
INCLUDING THE BACKFILLING OF THE ABUTMENTS AS WELL AS THE NEW TRACK (BALLAST, SLEEPERS AND RAILS)
DYNAMIC EFFECTS FROM COMPACTION

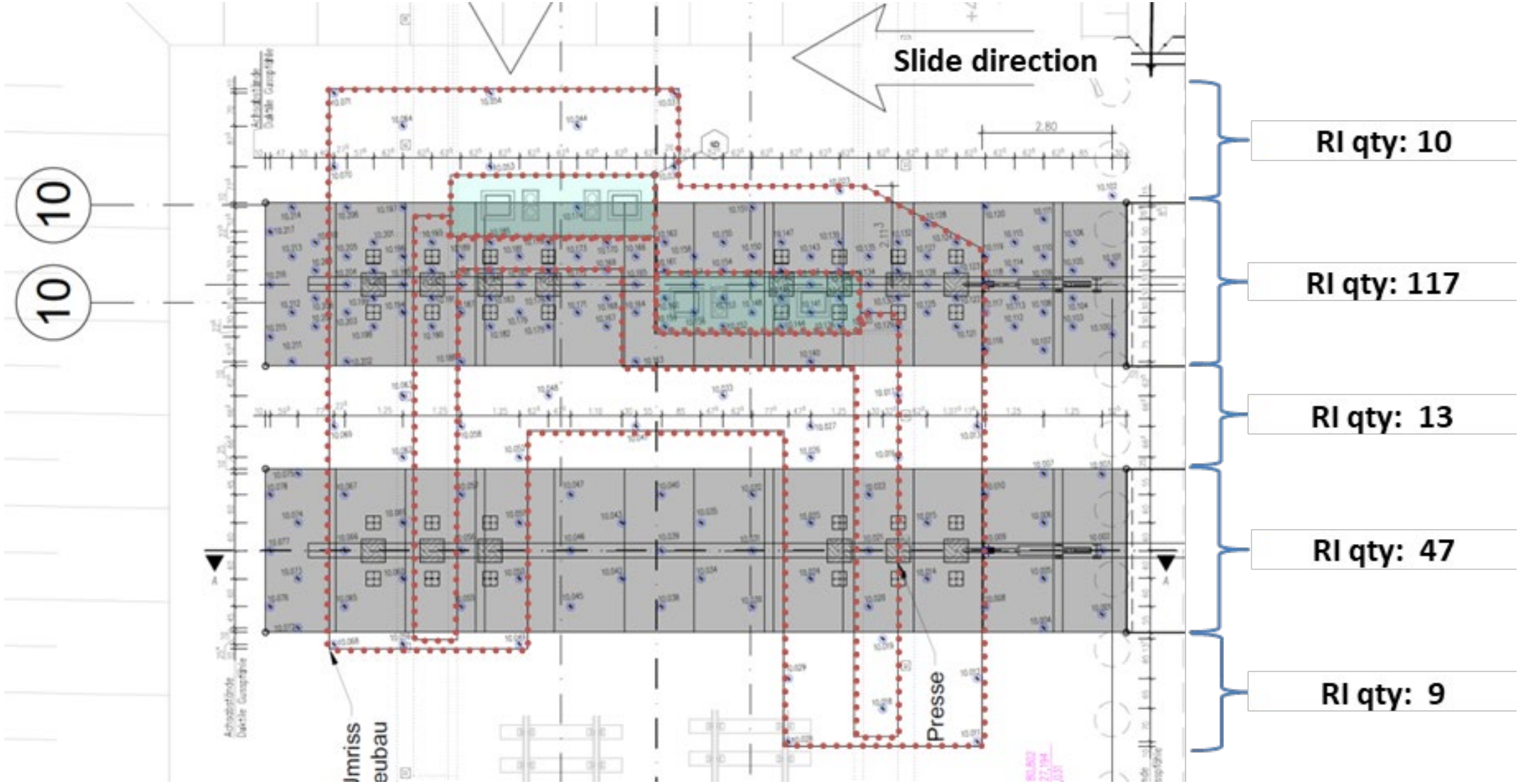
PERMANENT: SERVICE LIFE OF THE BRIDGE (100 YEARS)

SELF-WEIGHT OF THE NEW BRIDGE STRUCTURE (**4,000 METRIC TONS**),
INCLUDING THE BACKFILLING OF THE ABUTMENTS AS WELL AS THE NEW TRACK (BALLAST, SLEEPERS AND RAILS)
LONGITUDINAL VERTICAL TRAFFIC LOADS FOR A SPEED OF **118 MILE/HOUR**
LONGITUDINAL HORIZONTAL TRAFFIC LOADS (START-UP AND BREAKING LOADS)
TRANSVERSAL HORIZONTAL TRAFFIC LOADS (LATERAL IMPACT)

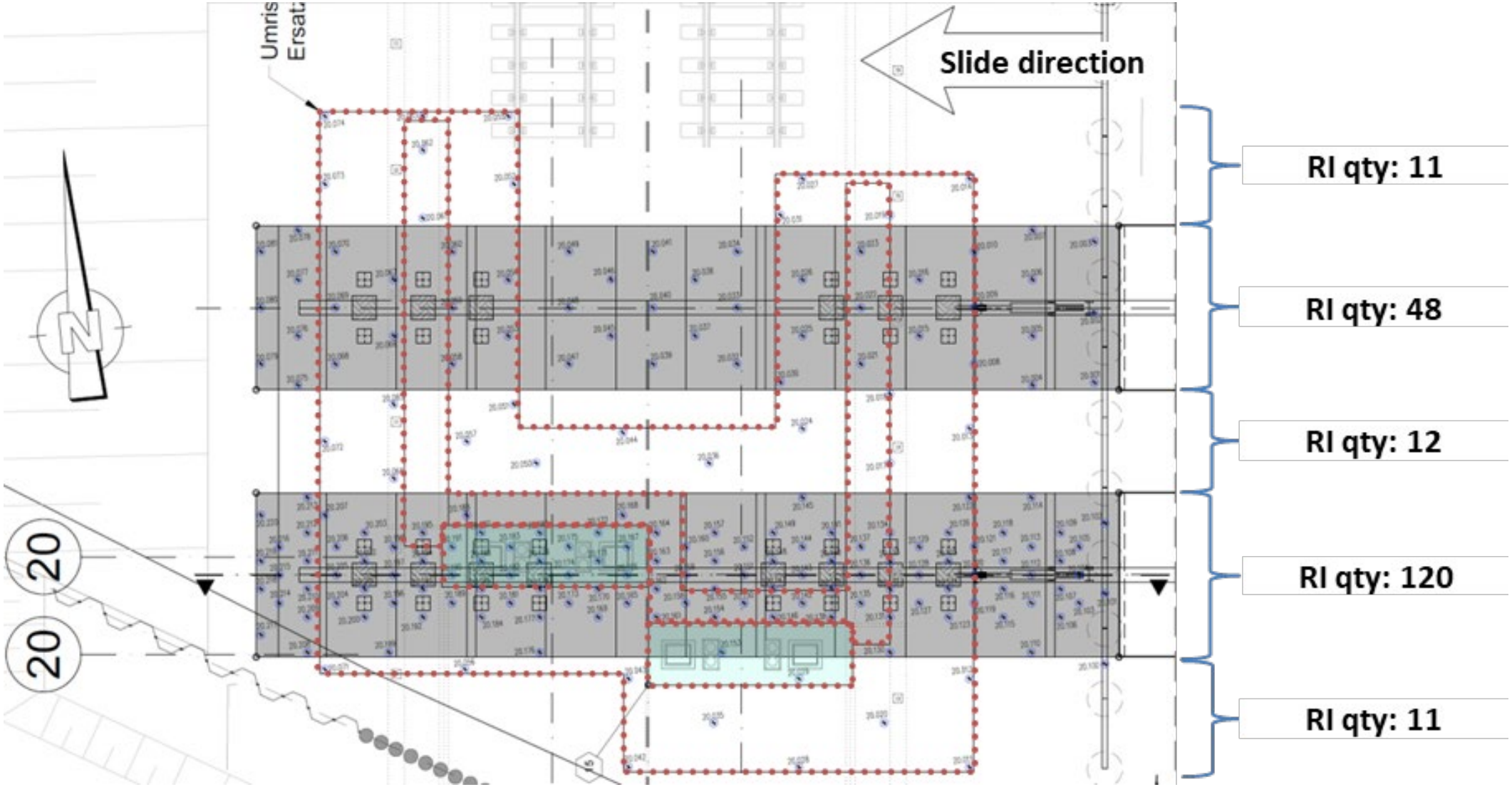




PILE LAYOUT - SOUTH ABUTMENT (AXIS 10):



PILE LAYOUT - NORTH ABUTMENT (AXIS 20):



THE FOUNDATIONS OF THE NEW OVERPASS: EXECUTION

THE PROJECT WAS CARRIED OUT BY **MAX BÖGL BAUSERVICE GMBH & CO. KG** (GERMANY) AS GENERAL CONTRACTOR

398 X DIPS WERE INSTALLED BY **CARL HEUCHEL GMBH & CO. KG** (GERMANY)

- JANUARY 25 – 27, 2024 (**60 HOURS**)
- **4 X CREWS** INSTALLED THE PILES (+ 2 ADDITIONAL MACHINES IN STAND-BY)

THE **LBS-PROCEDURE** TOOK PLACE ON JANUARY 28, 2024 (**3 HOURS**)

THE BACKFILLING OF THE ABUTMENTS, THE CONSTRUCTION OF THE NEW TRACKS AND ANCILLARY WORKS WERE FINISHED JANUARY 28 – 31, 2024

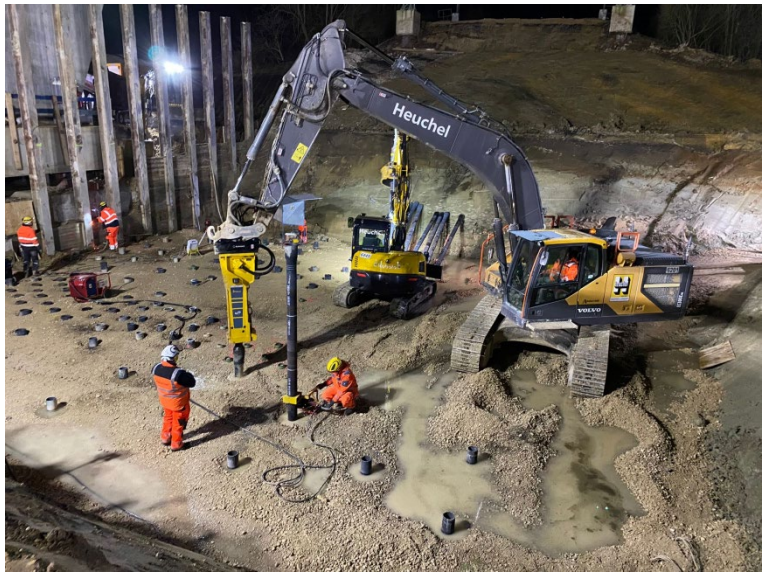
THE NEW RAILWAY OVERPASS OVER THE MAIN ROAD B25 WAS REOPENED TO TRAFFIC ON FEBRUARY 1, 2024

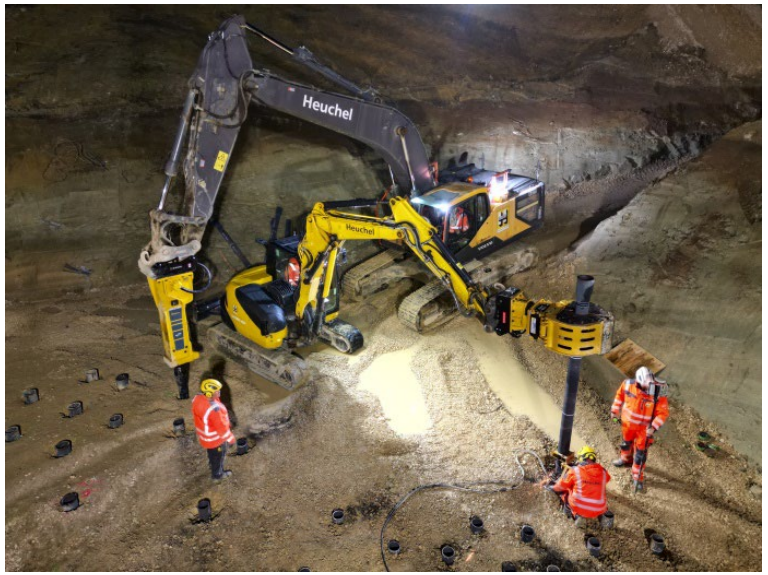
THE MAIN ROAD B25 WAS REOPENED TO TRAFFIC ON FEBRUARY 9, 2024

WITH AN APPROX. INVESTMENT OF 13,400,000.- €, THE INTERVENTION PROJECT (MODERNIZATION OF THE OVERPASS AND UPGRADE OF THE MAIN ROAD B25) WAS FINISHED IN DECEMBER, 2024

THE FOUNDATIONS OF THE NEW OVERPASS: EXECUTION













SOURCE: © CARL HEUCHEL GMBH & CO. KG (2024)

THE FOUNDATIONS OF THE NEW OVERPASS: EXECUTION



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MUSIC: GOOD RIDDANCE (TIME OF YOUR LIFE) – GREEN DAY, (NIMROD, REPRISE RECORDS, 1997)

Acknowledgments

Lidan Jin

(DB Netz AG – Regionalbereich Süd, Germany)

Patrick Mayer

(Max Bögl Bauservice GmbH & Co. KG, Germany)

Simon Hählein

(Carl Heuchel GmbH & Co. KG, Germany)

Ratisbona Media (RMB) – Dokumentationsdienst

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West Seattle Link Extension Project: Drilled Shaft Foundations and Bi-Directional Test Program

Matthew Preedy - Sound Transit

Yang Jiang - HNTB Corporation

Annual Conference on Deep Foundations
Nashville, Tennessee
October 20 -23, 2025





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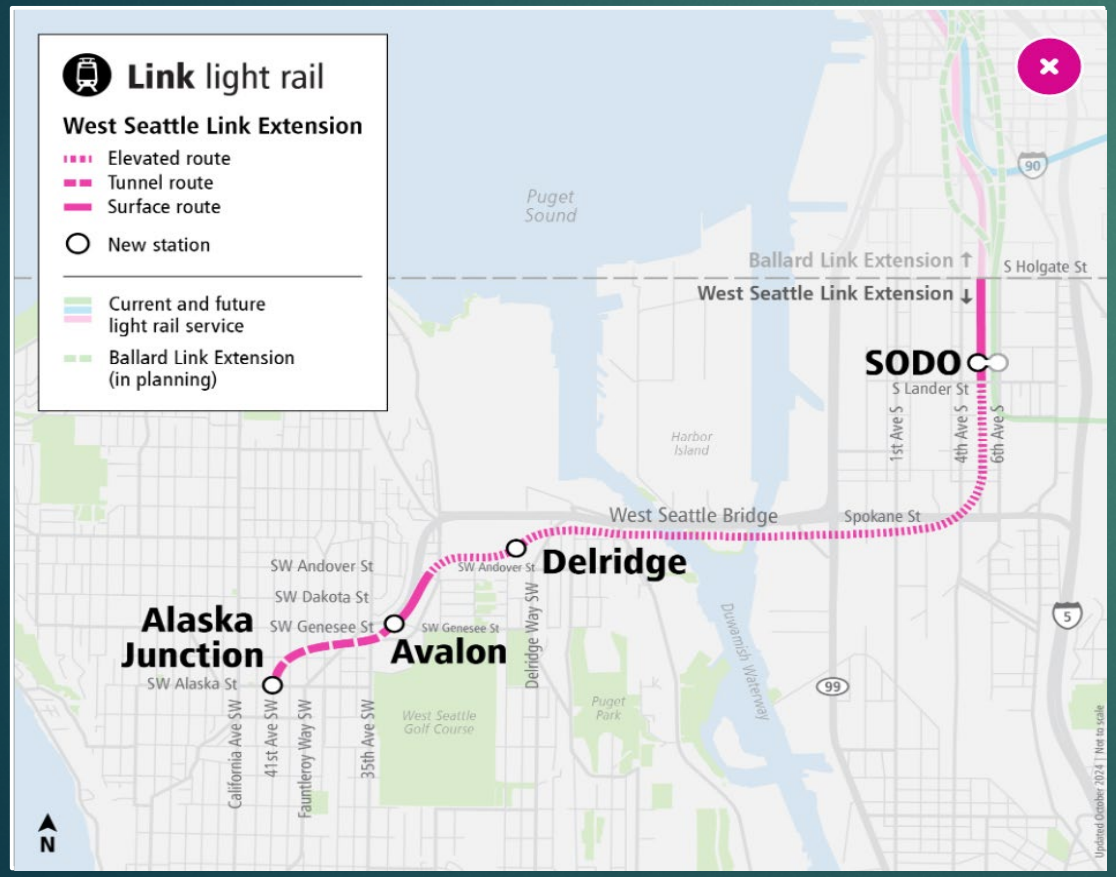
Outline

- Project Overview & Timeline
- Elevated Guideway Alignment and Structure Types
- Geotechnical Profile and Seismic Hazards
- Choice of Foundation Types
- Current Foundation Design and Possible Enhancement
- B-Directional Tests

West Seattle Link Extension Project Overview



Project Overview

The West Seattle Link Extension (WSLE) extends light rail by 4.1 miles from SODO to Alaska Junction, featuring 2.1 miles of elevated guideway.

Stations

Four stations: one at-grade, one elevated, one in lidded retained cut, and one underground.

Cable-Stayed Bridge Design

Features the longest cable-stayed bridge for light rail in the US, engineered to endure a 2,500-year seismic event.

Geotechnical and Seismic Challenges

Complex soil conditions and seismic hazards require robust foundation systems.

Load Testing and Validation

Two bi-directional load tests will validate design assumptions and enhance constructability of bridge foundations.

Project Timeline



Record of Decision and PE Completed

Preliminary Engineering (PE) completed, and a Record of Decision (ROD) issued April 29, 2025

Advanced PE

An advanced PE and design validation contract for the transition to final design is underway.

Final Design

Final Design will take place from 2025 to 2027.

Delivery Method TBD

Delivery methods include a combination of DBB and GCCM.

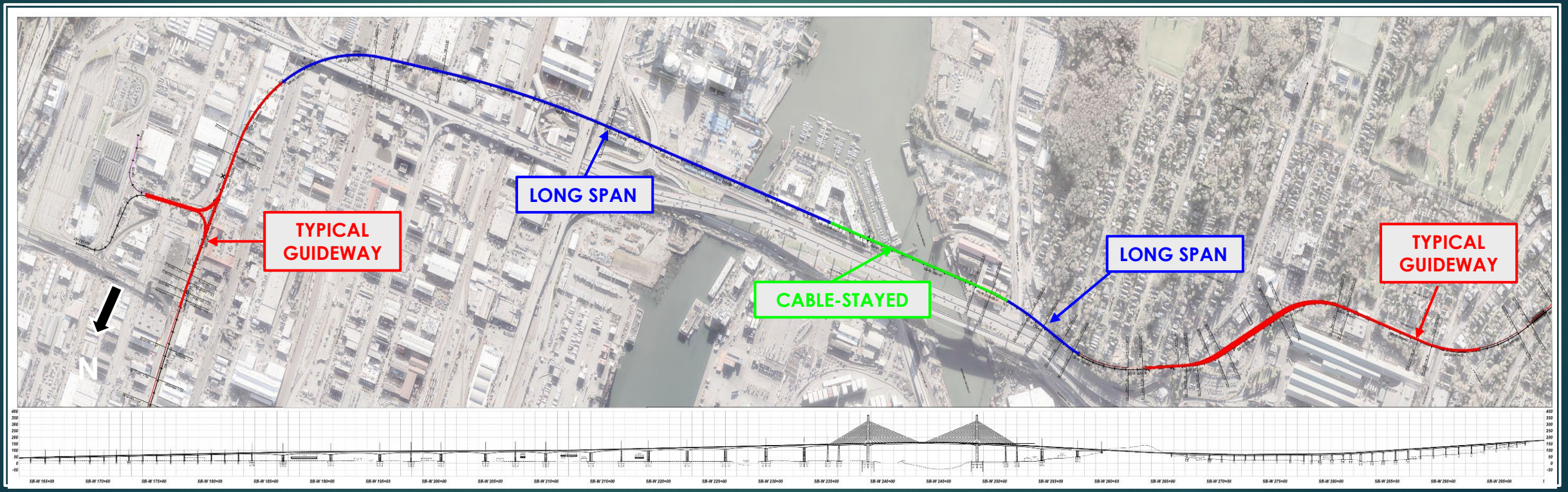
Start of Service

2032

Alignment and Structural Types

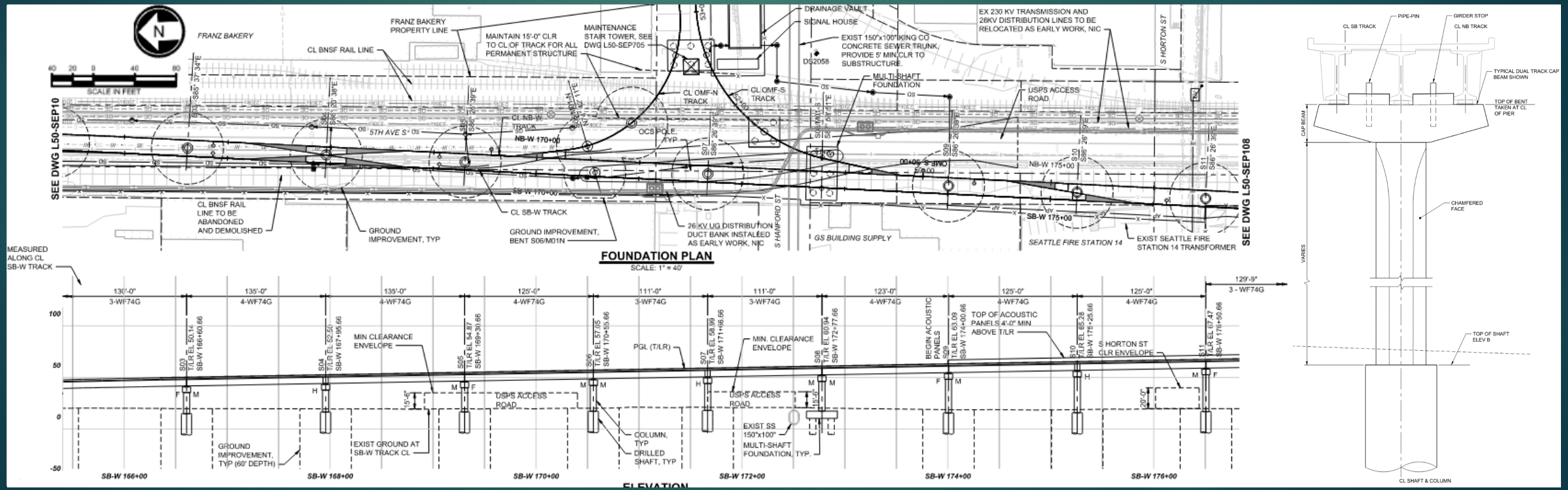
Three Primary Structural Types:

- Typical Prestressed Concrete Girders (5130')
- Long Span Segmental Balanced Cantilever Box Girders (5900')
- Cable-Stayed Duwamish Crossing (1690')
- At-grade (620')



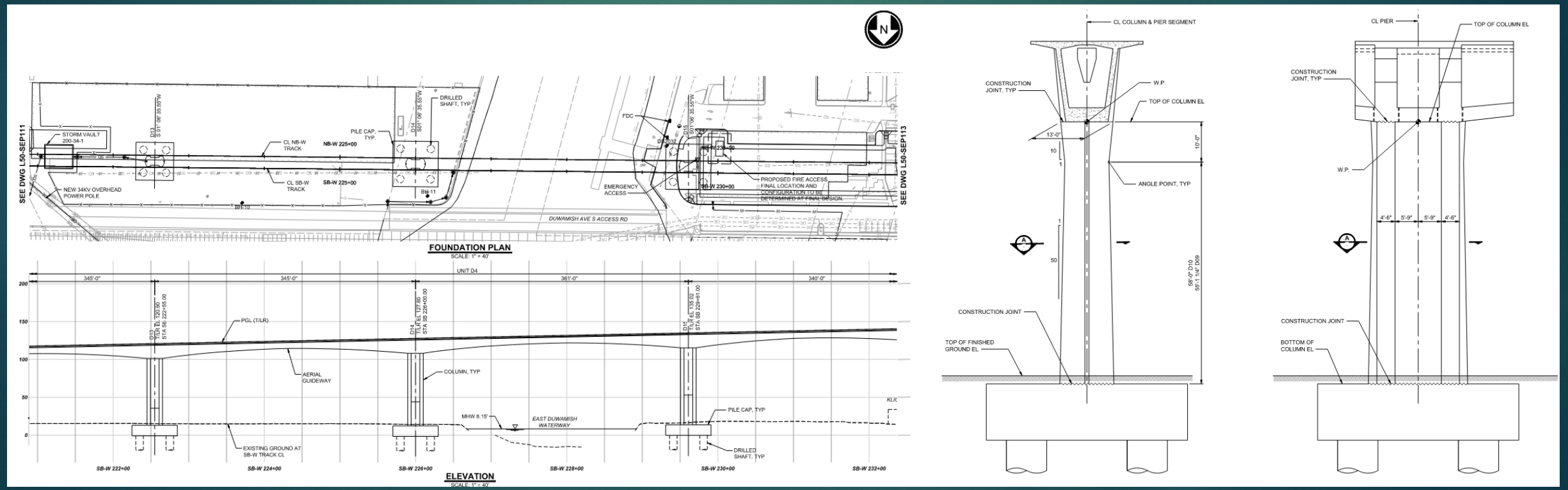
Typical P/S Girder Guideway

- Where the guideway is ~60’ or less above existing grade
- 3 lines of prestressed wide flange girders
- Typical span length 110’ to 135’ simple spans, total 5130’
- Single column
- **Single shaft foundations – shafts up to 220’ long**
- **Most with ground improvement**



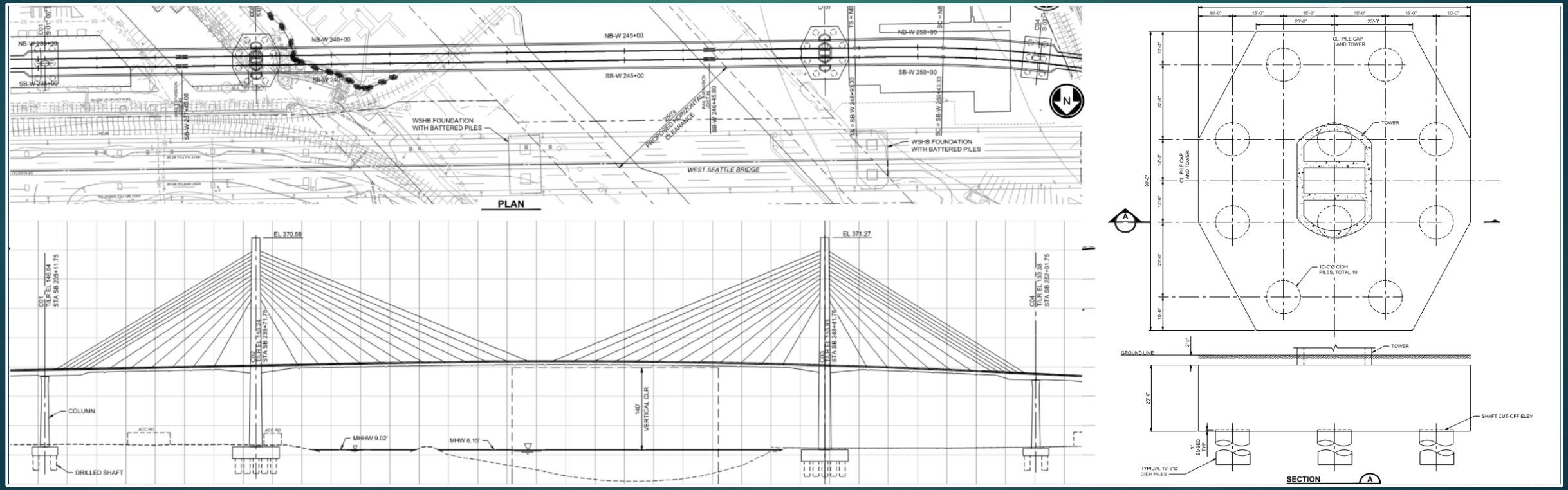
Long-Span Guideway

- Structure height ranging from 60’-130’ above existing grade
- Segmental box girders with balanced cantilever construction, either cast-in-place or precast
- 5 units – 3-6 spans each, 280’ to 435’ long spans, 5900’ total
- **4- 10’ shaft foundations with shaft caps**
- **Shaft length varying from 75’ to 295’**

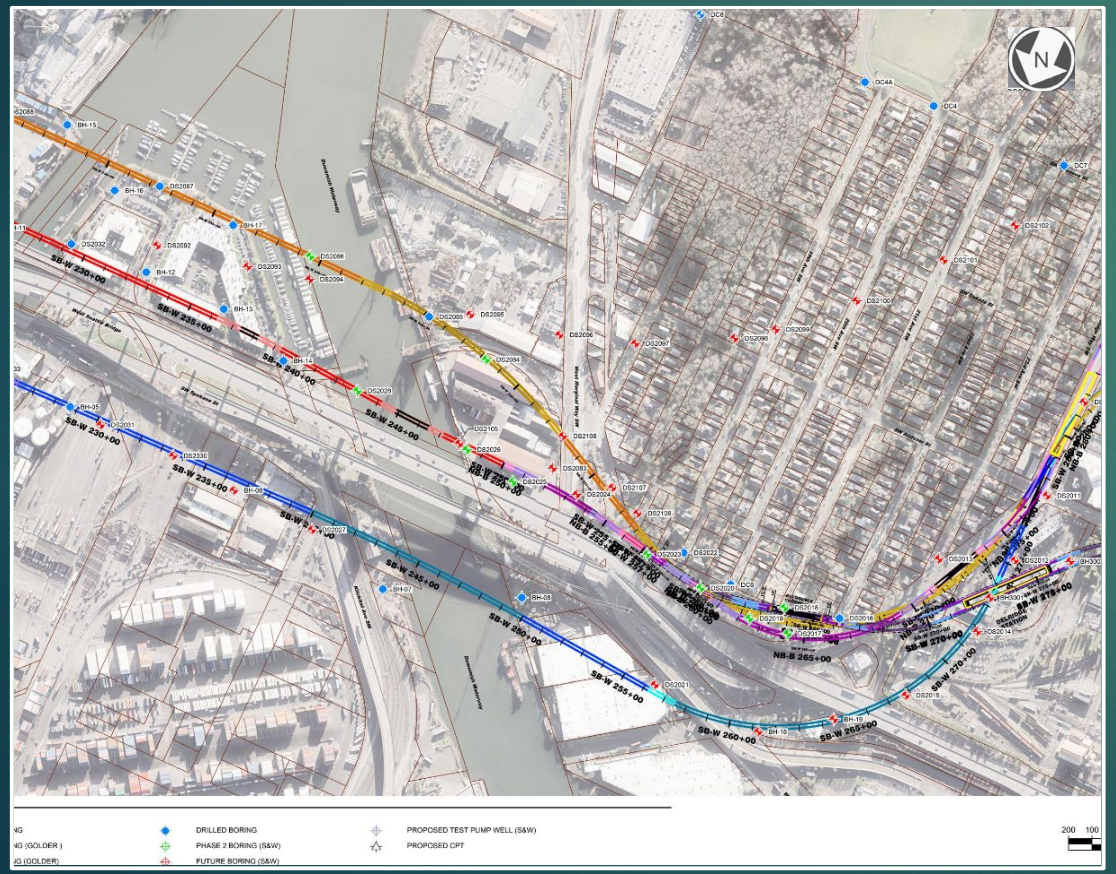


Cable-Stayed Duwamish Crossing

- Cable-stayed with concrete open deck and diamond pylons
- 360’-970’-360’ span arrangement, 1690’ total
- 140’ vertical clearance and no in-water piers
- **Main pylons supported on 10- 10’ shafts with shaft caps**
- **End piers supported on 4- 10’ shafts with shaft caps**
- **Shaft length varying from 95’ to 260’**



Field Exploration and Subsurface Conditions



Soil Characteristics

20 to 25 feet of fill consisting of sand to gravelly sand

20 to 270 feet of normally consolidated soils

Glacially overridden deposits are as deep as 270 feet below grade

Soil Classification

Soils were classified based on ASTM D2487, and ASTM D2488-17

Shear Wave Testing

Seismic velocity measurements were obtained using a suspension PS logging system

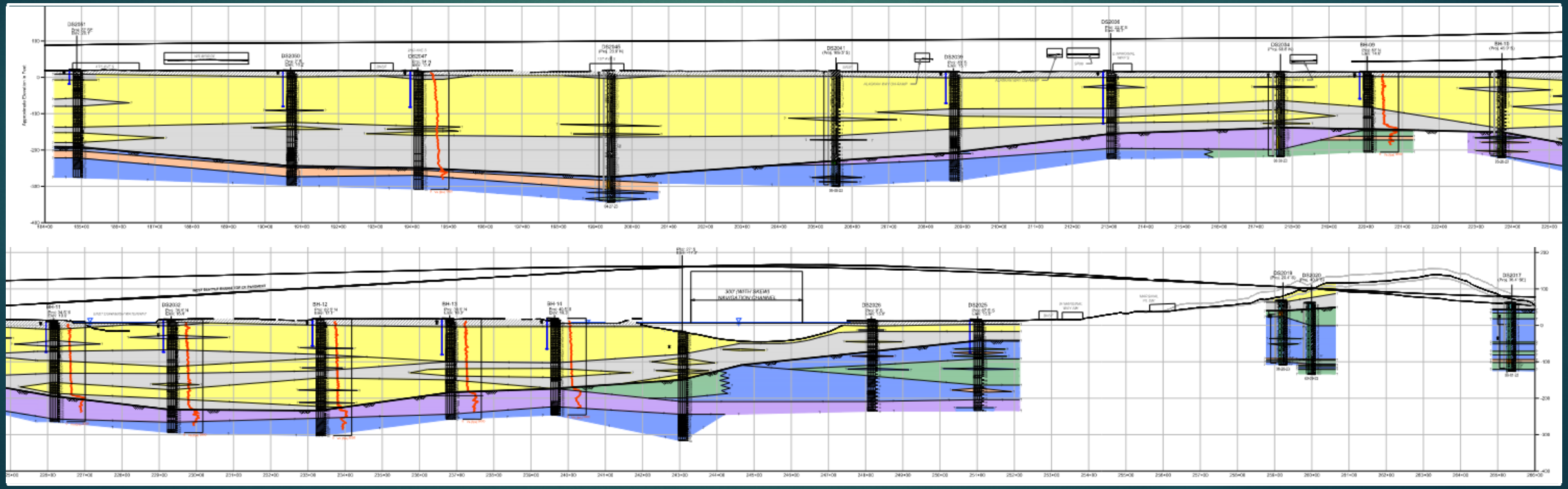
Soil and Site Characterization

Field investigations including SPT, Shelby tube sampling, and downhole logging to assess soil properties and seismic site class

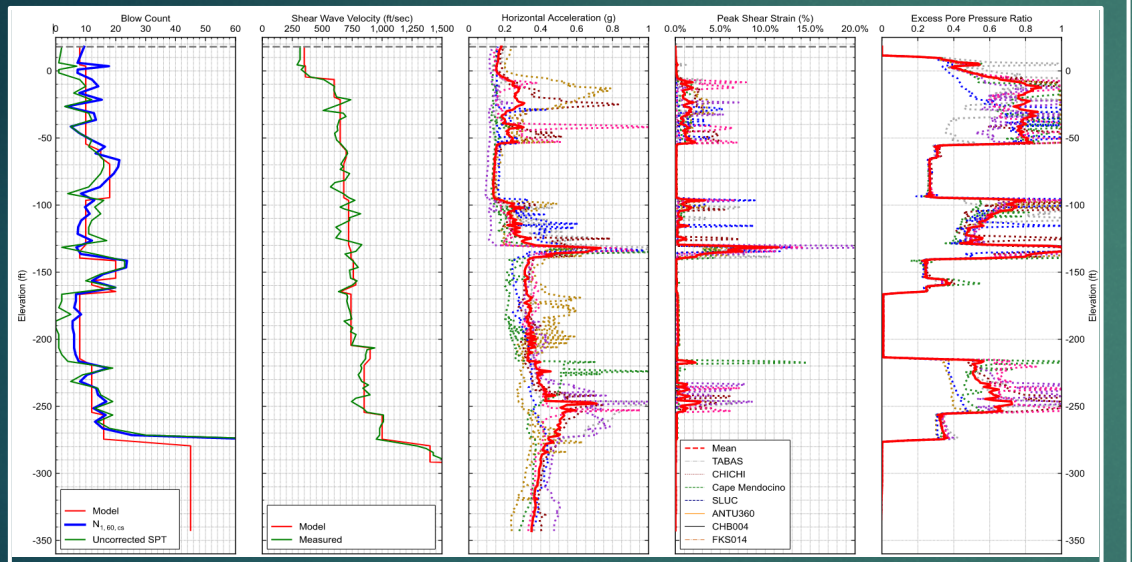
Geotechnical Profile

ESU No.	
1	ENGINEERED AND NON-ENGINEERED FILL (ENF): Includes granular and cohesive FILL
2	RECENT GRANULAR DEPOSITS (RGD): Loose to dense SILT and SAND with gravel; includes normally consolidated alluvium, beach deposits, reworked glacial deposits, and recessional ice-contact deposits.
3	RECENT CLAY AND SILT (RCS): Soft to very stiff CLAY and SILT with fine sand beds; includes normally consolidated estuarine deposits, and recessional lacustrine deposits.
4	TILL DEPOSITS (T or TL): Dense to very dense, silty SAND and GRAVEL, and hard, silty CLAY with sand and gravel; cobbles and boulders are common in these deposits; includes glacially overconsolidated till, glaciomarine drift, and till-like deposits.

ESU No.	
5	COHESIONLESS SAND AND GRAVEL (CSG): Very dense SAND and GRAVEL to SAND with variable silt; cobbles can be found in these deposits; includes glacially overconsolidated fluvial and glacial outwash deposits.
6	COHESIONLESS SILT AND FINE SAND (CSF): Very dense SILT, silty fine SAND, and fine sandy SILT with trace of clay; predominantly cohesionless; includes glacially overconsoidated lacustrine deposits.
7	COHESIVE CLAY AND SILT (CCS): Very stiff to hard, silty CLAY and clayey SILT with trace of sand and gravel; scattered cobbles and boulders can be found in these deposits; includes glacially overconsolidated lacustrine, peat, and paleosol deposits.
	PEAT OR WOOD



Seismic Hazards and Site Response



1D Site Response Analysis Boring DS2045
MDE Ground Motion

Seismic Risk and Regional Context

WSLE is in a seismically active area influenced by the Cascadia Subduction Zone with possible earthquakes up to magnitude 7.5.

Two-Level Seismic Design

ODE – 150-year return earthquake – no significant damage – repair during normal operating hours

MDE – 2500-year return earthquake – collapse prevention

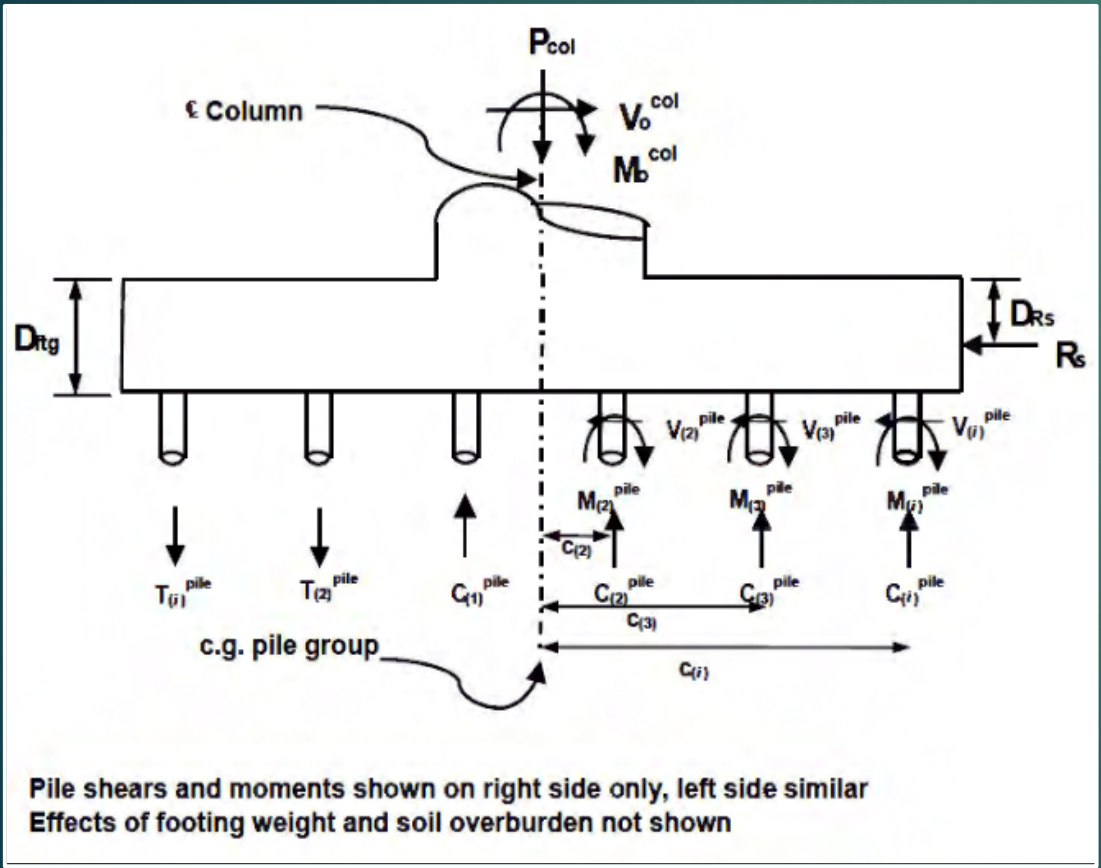
Site-Specific Ground Response

The 2018 USGS NSHMP was used for ground response

Site Response Analysis Results

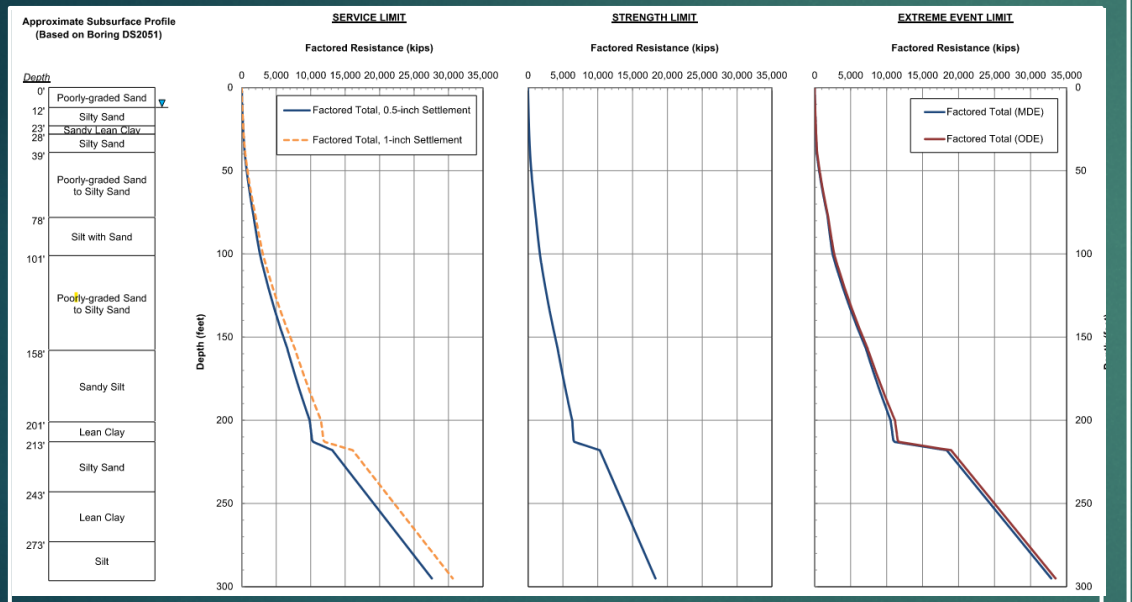
Include Excess Pore Pressure Ratio as a function of depth to indicate liquefication potentials

Choice of Foundation Types



- Liquefiable soils to significant depth
- The glacially overridden deposits are deep
- Structures are sensitive to settlements
- Need to provide significant lateral resistance under seismic event
- Lateral loads include inertial loads as well as lateral spreading loads at east and west Duwamish waterways
- Strong desire to minimize noise and vibration during construction
- Frame action from shafts and shaft cap can provide much needed lateral resistance
- **Large-diameter drilled shafts were identified and selected as the preferred foundation type for PE**

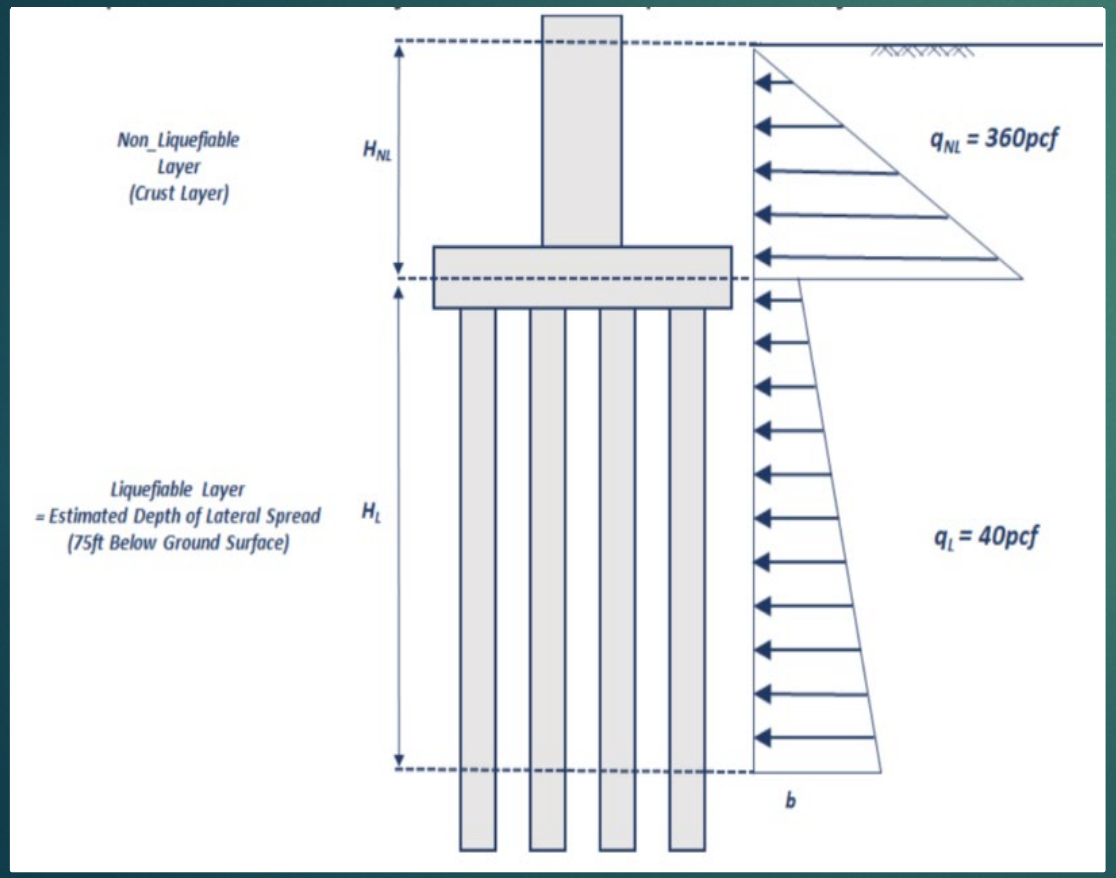
Drilled Shaft Foundation Design



Estimate Axial Resistance 10-Foot-Diameter Drilled Shaft without Casing

- Design follows AASHTO BDS and WSDOT GDM using LRFD methodology
- Axial resistance depends on subsurface data, SPT N-values, and lab tests
- Soil interaction with drilled shafts provides lateral resistance
- Analyzed lateral loads using LPILE and GROUP software with seismic adjustments for soil properties
- Considered group effects
- Analyzed downdrag for downdrag loads and settlement using the neutral plane method (Fellenius, 2004)

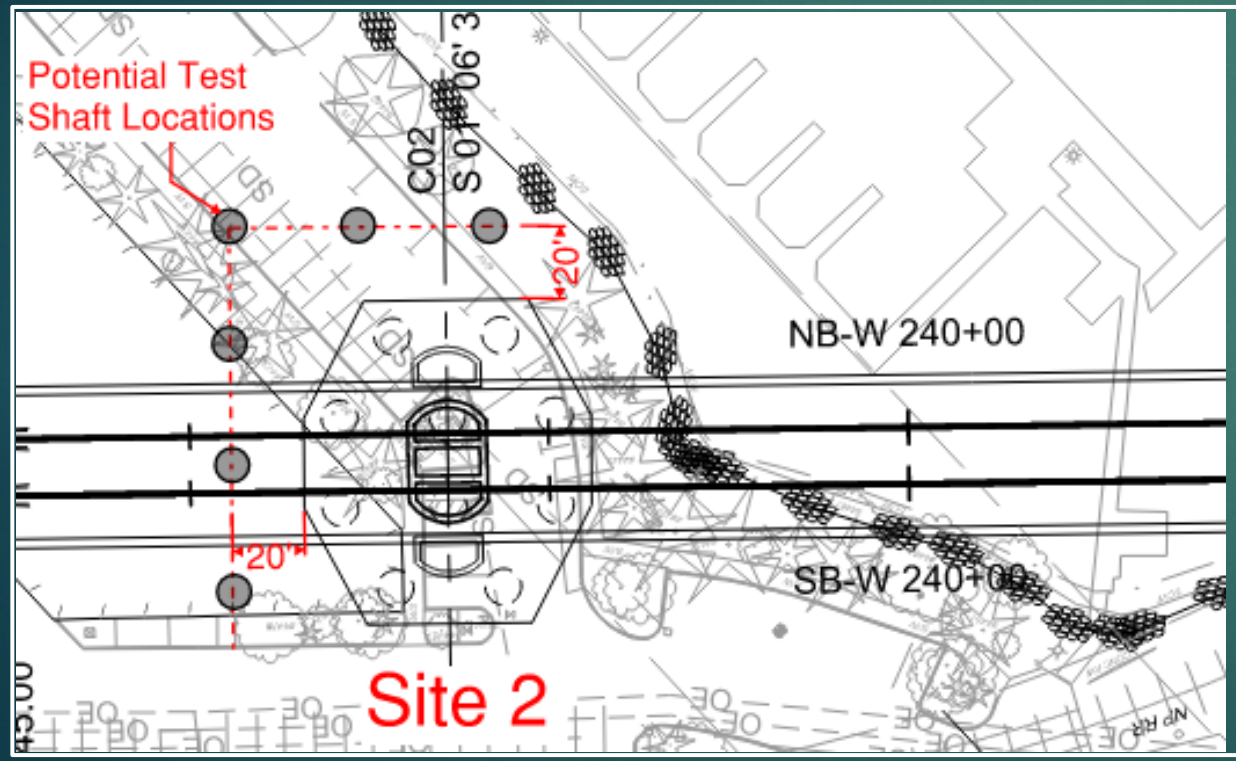
Potential Foundation Design Enhancement

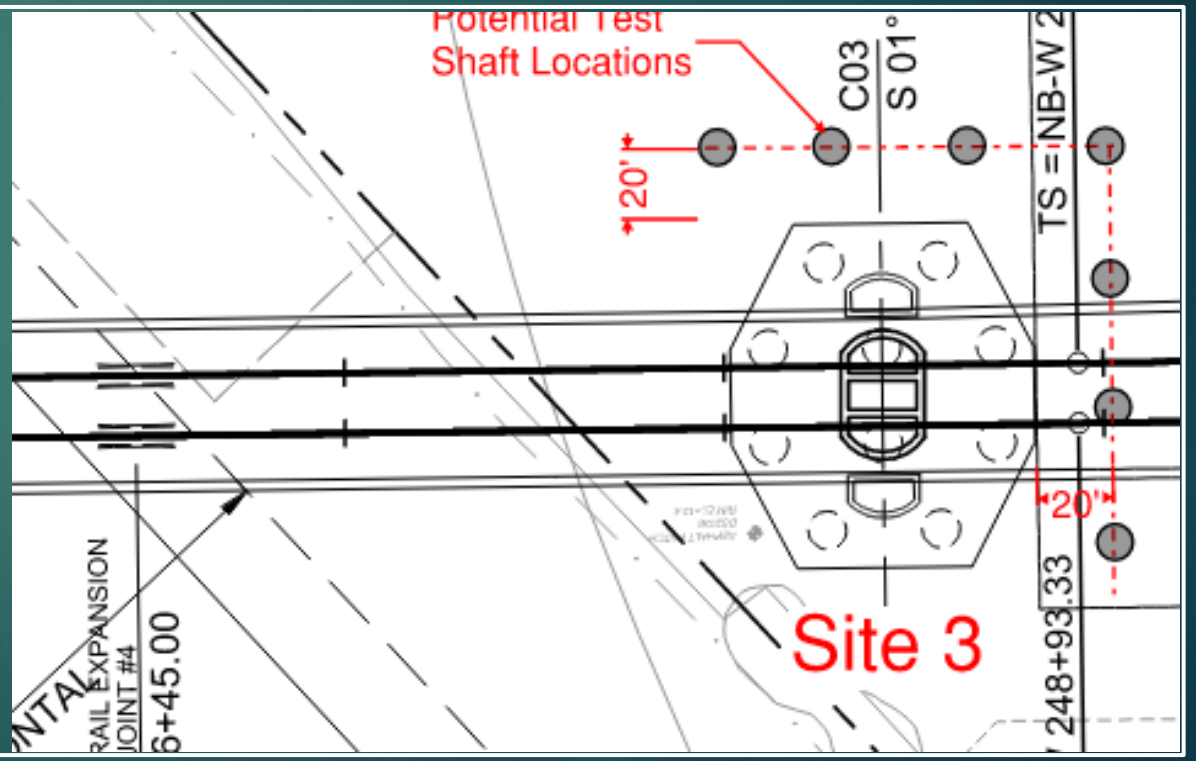


- Consider load reduction when combining inertial load with lateral spreading load
- Allowing in-ground hinging under MDE lateral spreading load

Bi-Directional Load Tests

- Two load tests are being planned – Separate DBB contracts
- The load test program is designed to represent the sizes, depths, geologic and construction conditions
- The tests will be performed on designated, sacrificial test shafts that will not be incorporated into the final foundation footprint





Benefits of Bi-Directional Load Tests

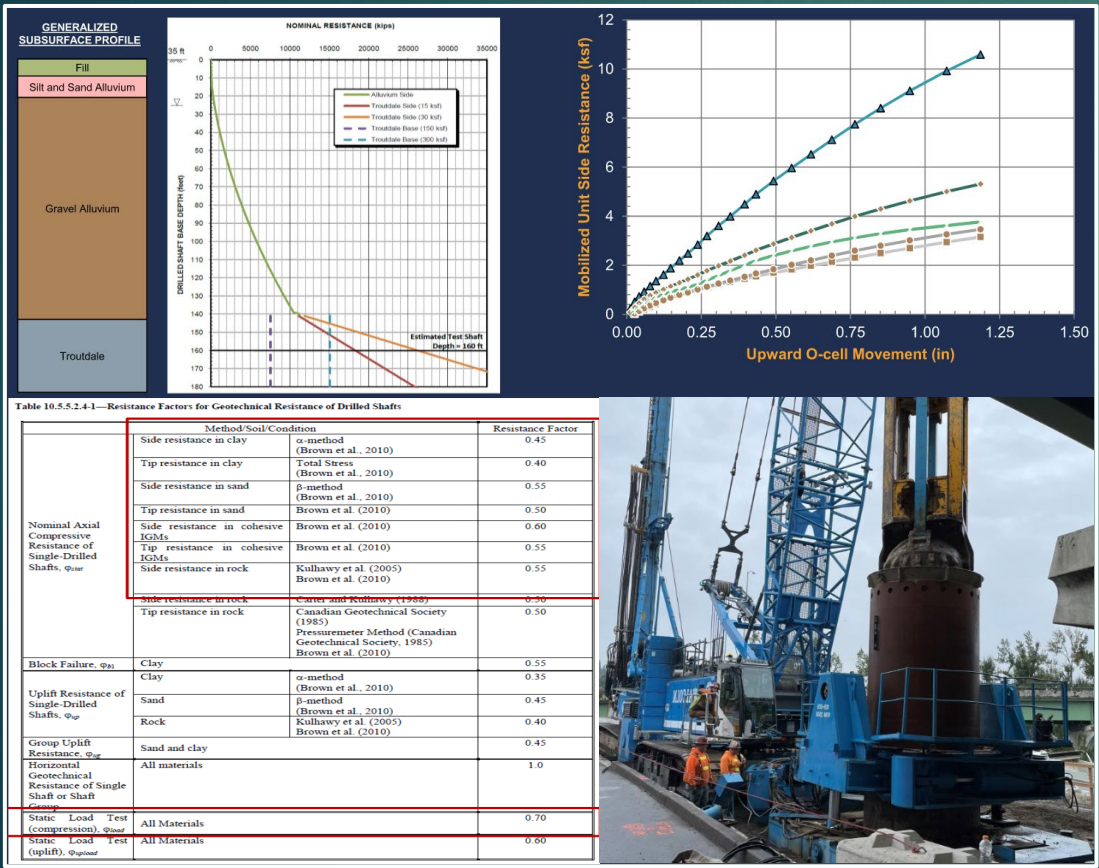


Table 10.5.5.2.4-1—Resistance Factors for Geotechnical Resistance of Drilled Shafts

	Method/Soil/Condition		Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts, φ_{stat}	Side resistance in clay	α-method (Brown et al., 2010)	0.45
	Tip resistance in clay	Total Stress (Brown et al., 2010)	0.40
	Side resistance in sand	β-method (Brown et al., 2010)	0.55
	Tip resistance in sand	Brown et al. (2010)	0.50
	Side resistance in cohesive IGMs	Brown et al. (2010)	0.60
	Tip resistance in cohesive IGMs	Brown et al. (2010)	0.55
	Side resistance in rock	Kulhawy et al. (2005) Brown et al. (2010)	0.55
	Side resistance in rock	Carter and Kulhawy (1988)	0.50
	Tip resistance in rock	Canadian Geotechnical Society (1985) Pressuremeter Method (Canadian Geotechnical Society, 1985) Brown et al. (2010)	0.50
Block Failure, φ_{bl}	Clay		0.55
Uplift Resistance of Single-Drilled Shafts, φ_{up}	Clay	α-method (Brown et al., 2010)	0.35
	Sand	β-method (Brown et al., 2010)	0.45
	Rock	Kulhawy et al. (2005) Brown et al. (2010)	0.40
Group Uplift Resistance, φ_{ug}	Sand and clay		0.45
Horizontal Geotechnical Resistance of Single Shaft or Shaft Group	All materials		1.0
Static Load Test (compression), φ_{load}	All Materials		0.70
Static Load Test (uplift), φ_{upload}	All Materials		0.60

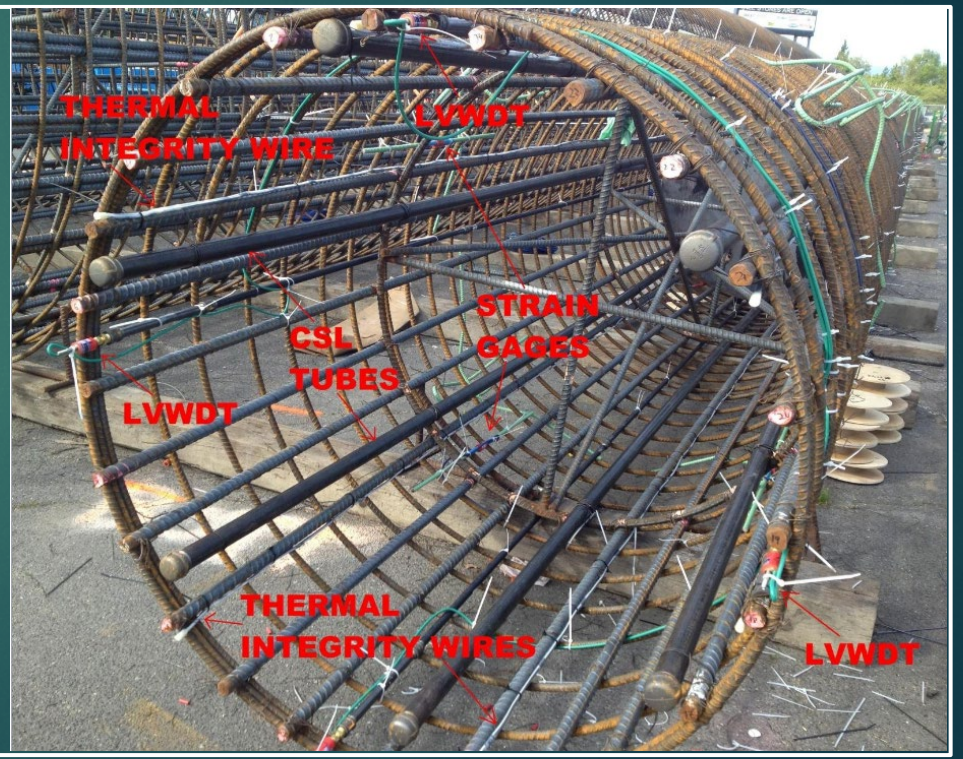
- Assess nominal side and base resistances
- Enable use of increased LRFD resistance factors
- Calibrate load-deformation response
- Validates design assumption and increase confidence
- Provide insights into constructability, including excavation techniques, clean-out methods, concrete placement, and the potential use of permanent casing

Bi-Directional Load Tests - Instrumentation

- Typical shaft construction
- Embedded hydraulic jacks
- Instrumentation incorporated into rebar cage
- Located along shaft length at soil contacts
- Enable differentiation of side and base resistances
- Redundant for reliability







Photos are courtesy of Shannon & Wilson, Inc.

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BIOTECHNICAL EROSION CONTROL – CURRENT PRACTICES AND CONSIDERATIONS FOR THE FUTURE

Rebecca Jones, Rose-Hulman Institute of Technology

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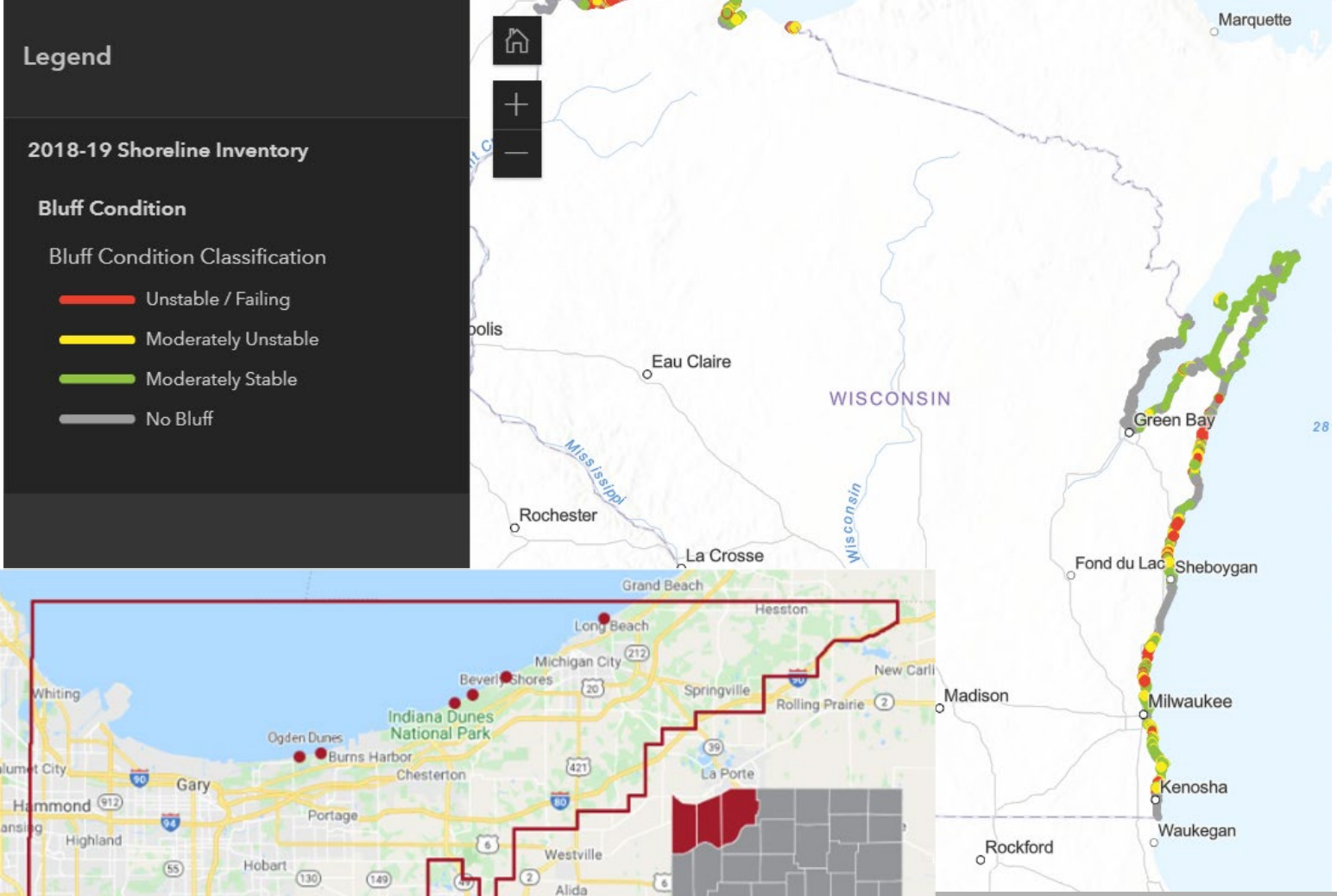
LEADING QUESTIONS

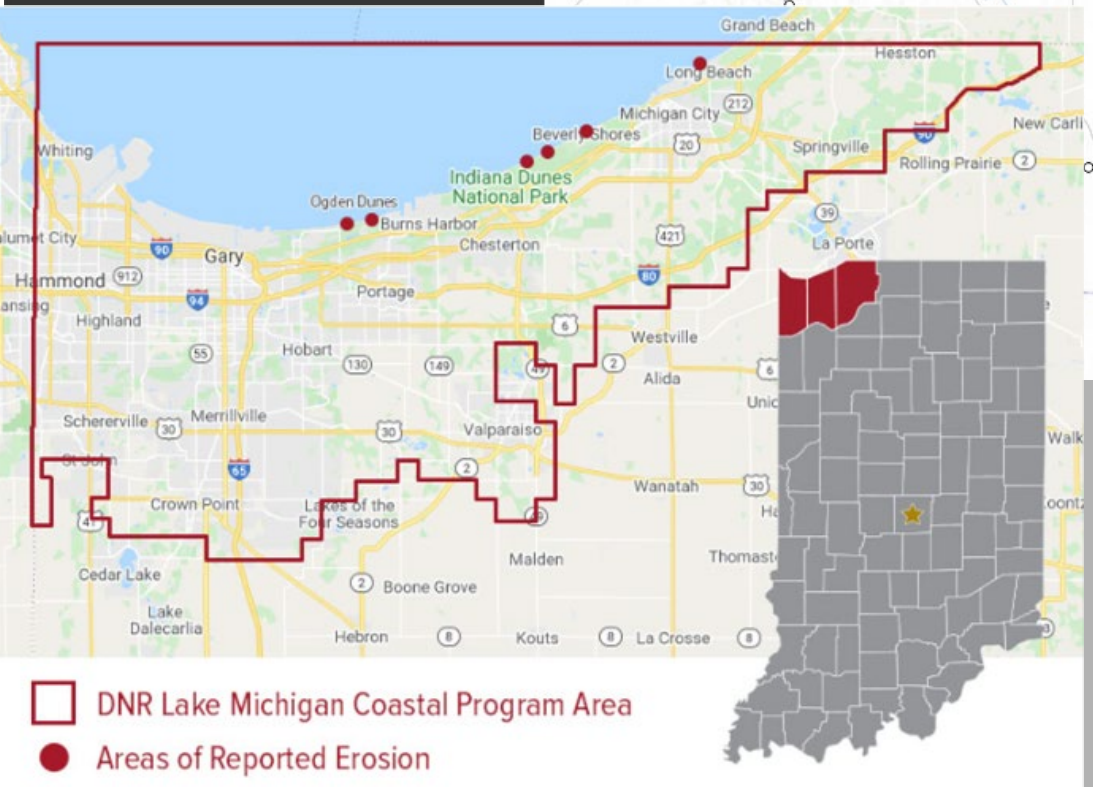
- What is the current state of practice of Lake Michigan shoreline erosion control?
- What methods of shoreline erosion control are the most sustainable and resilient?
- How well will erosion control methods adapt to a changing climate change?
- What are the current limitations and needs for additional research?

LAKE MICHIGAN

- Geographic Location
- Economic Value
- 1,638 Miles
  - 70-80% privately owned
- High Risk Erosion



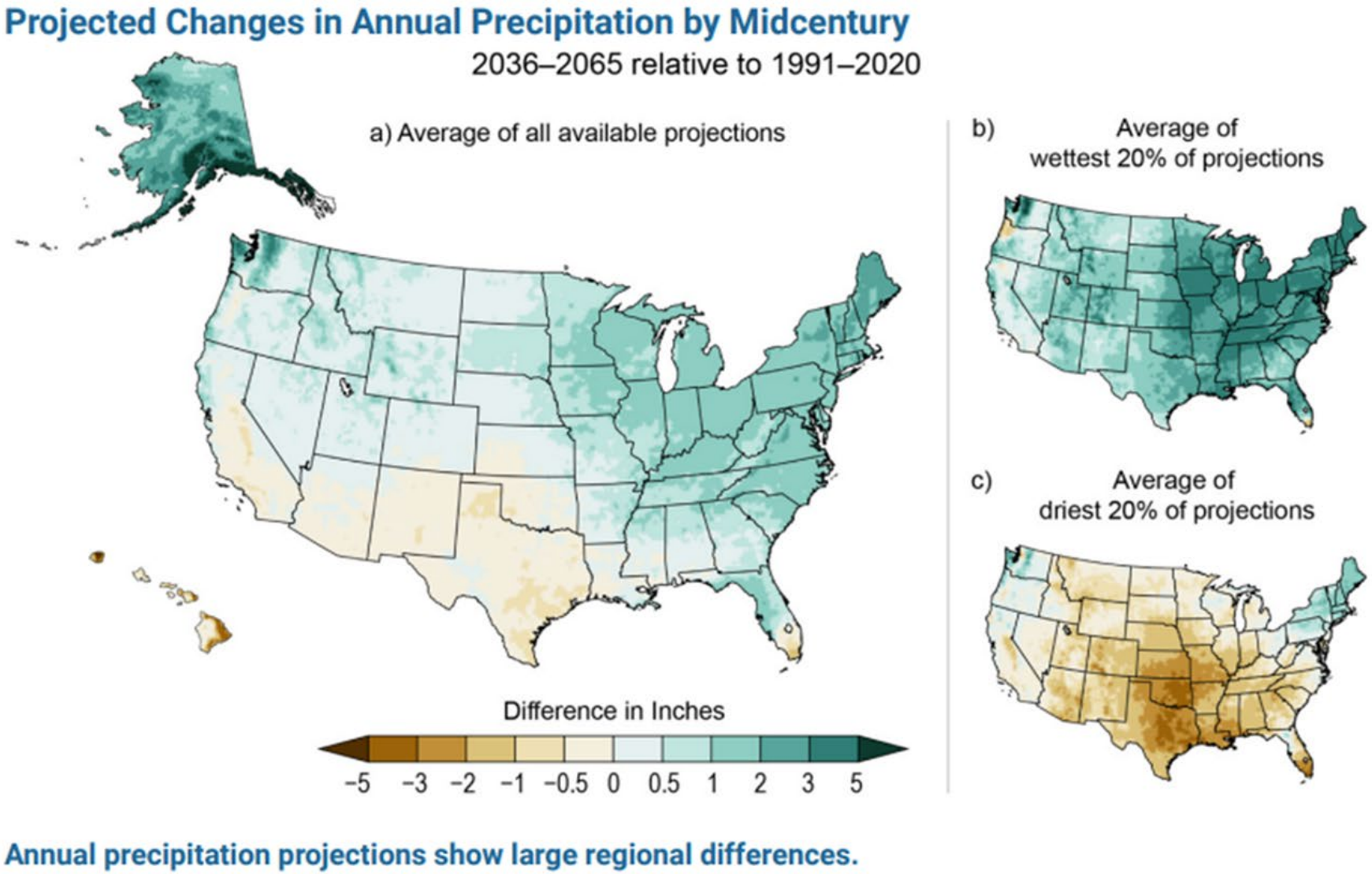




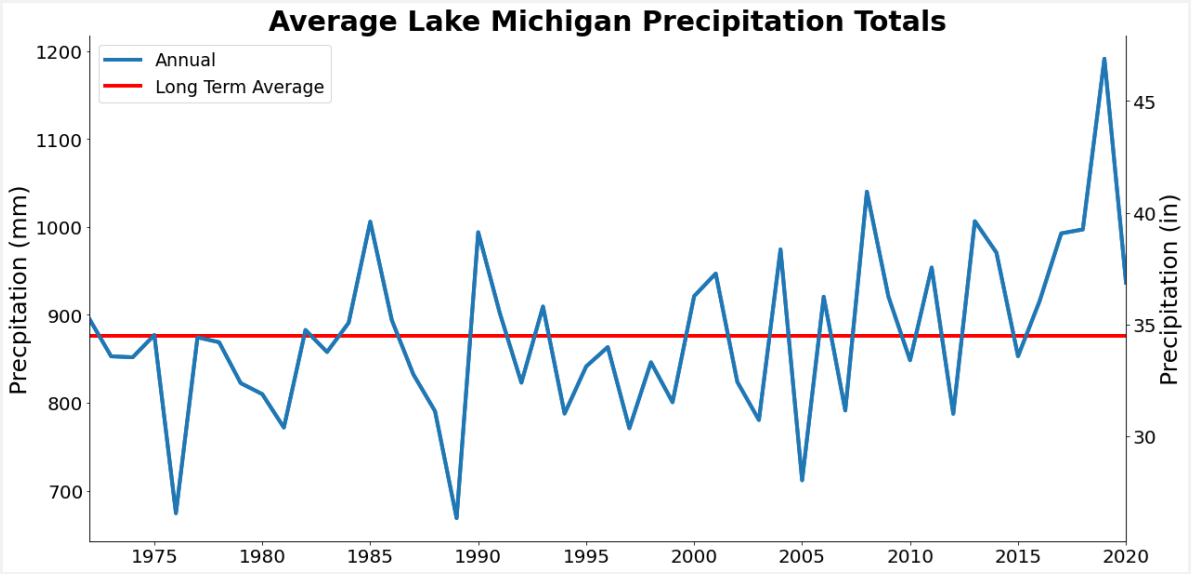


EROSION MAPS

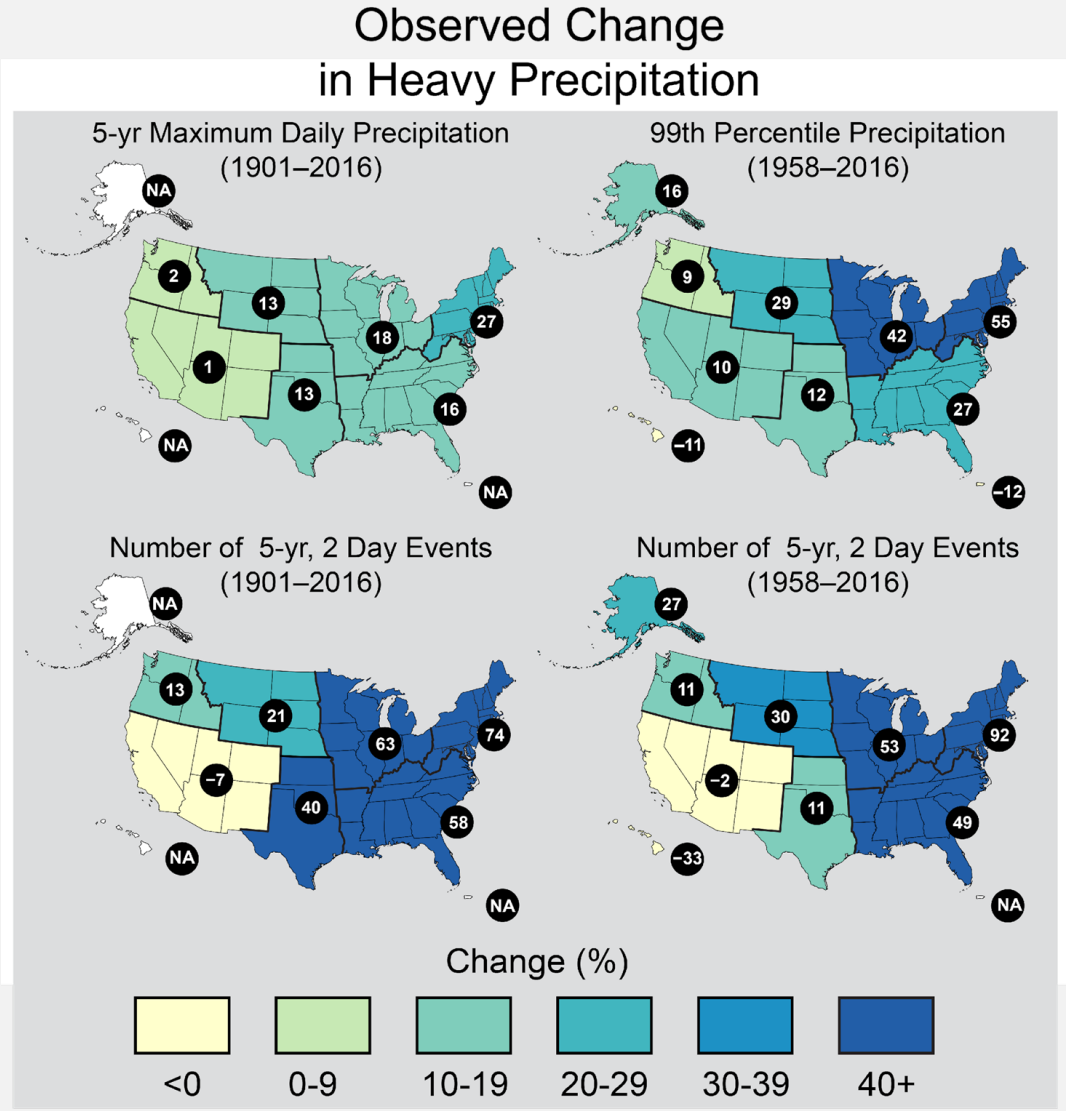
EFFECTS OF CLIMATE CHANGE ON LAKE MICHIGAN SHORELINE



LAKE MICHIGAN PRECIPITATION

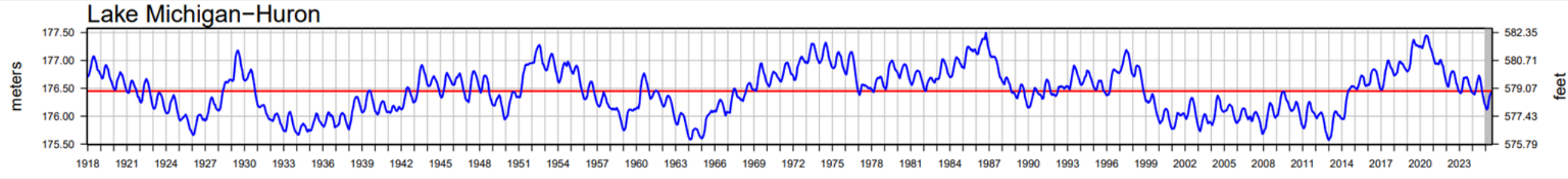


(NOAA, 2023)



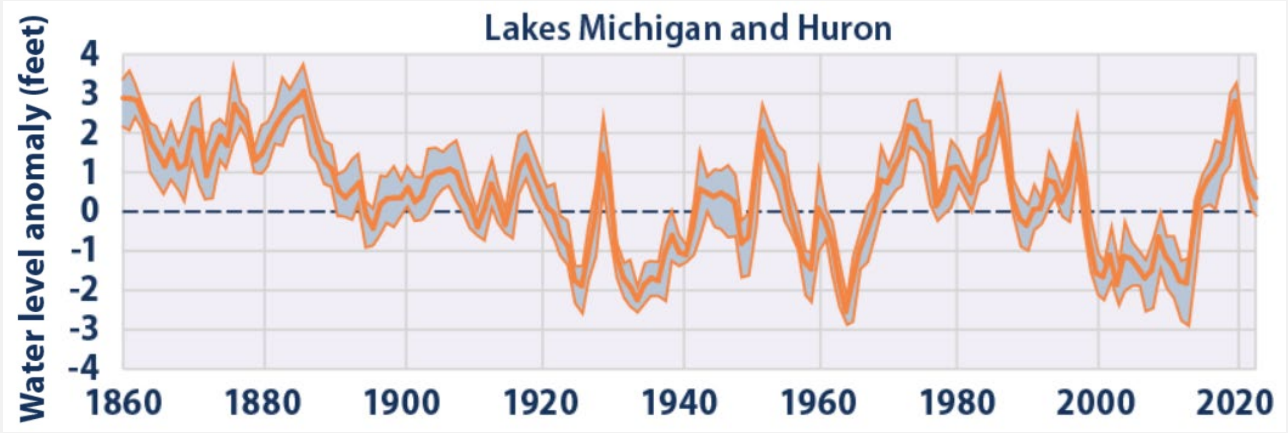
WATER LEVEL FLUCTUATIONS

Monthly Average Fluctuations



(USACE, 2025)

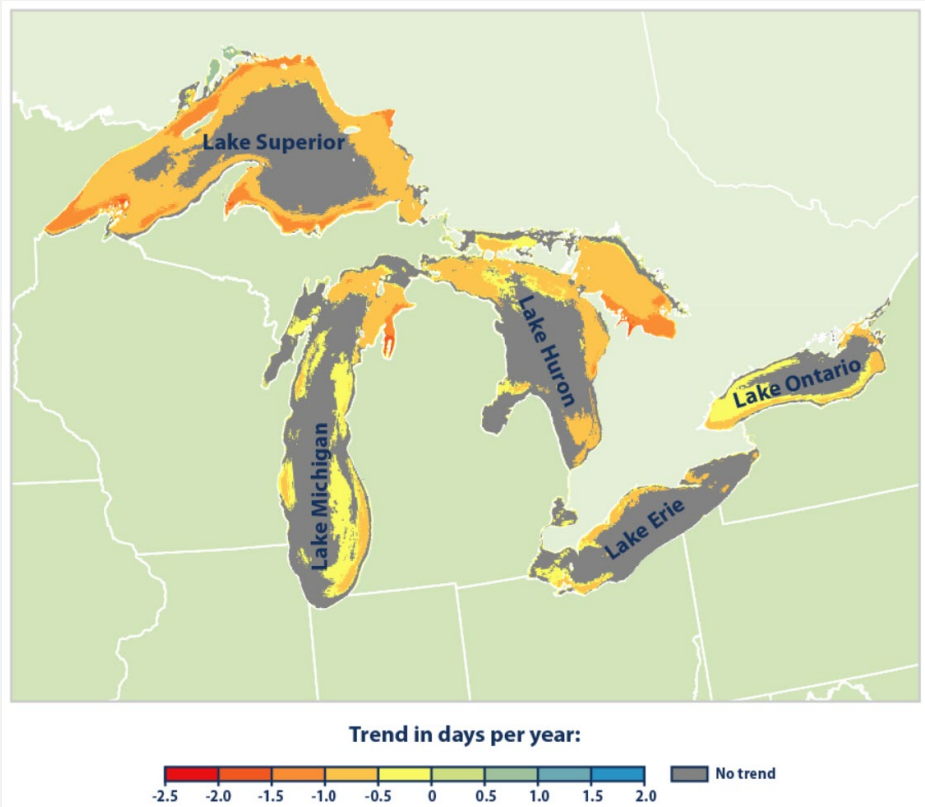
Annual Average Fluctuations



Gray band shows range of monthly high/low

(USEPA, 2024)

ICE COVER FLUCTUATIONS



Change in Ice Cover Duration in the Great Lakes, 1973-2019
(USEPA, 2024)



Effects of high water level and less ice cover

METHODS OF EROSION CONTROL







Erosion Control Material	Embodied Energy MJ/kg of material	Embodied Carbon $kgCO_2$/kg of material
Straw	0.24	0.01
Wood	7.50	0.46
Lightly Reinforced Concrete	1.10	0.127
Stone Riprap	1.00	0.057
Polypropylene	99.20	2.7

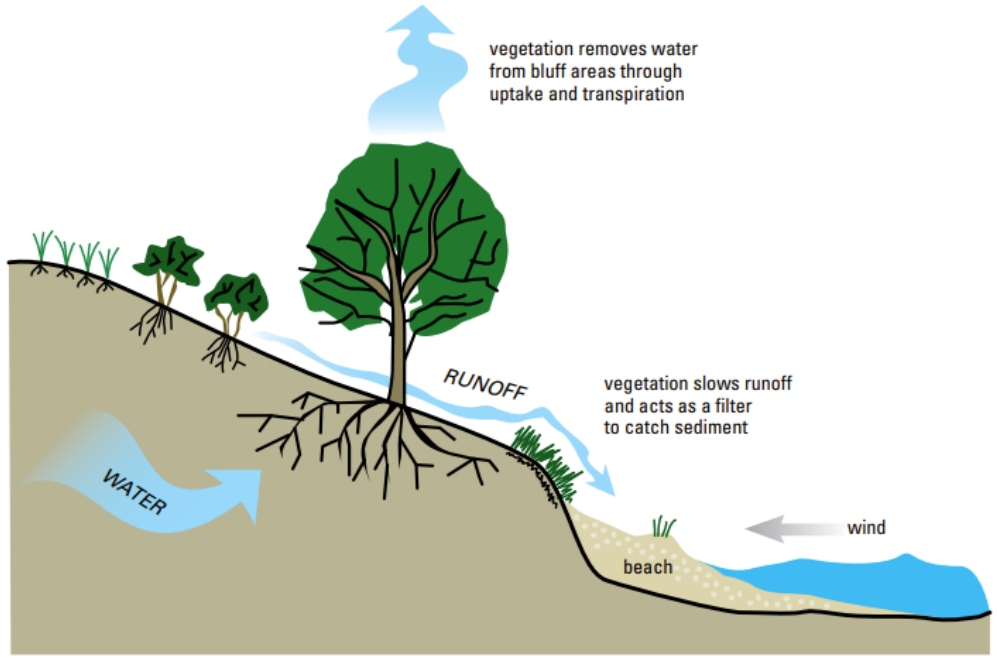
METHODS OF EROSION CONTROL

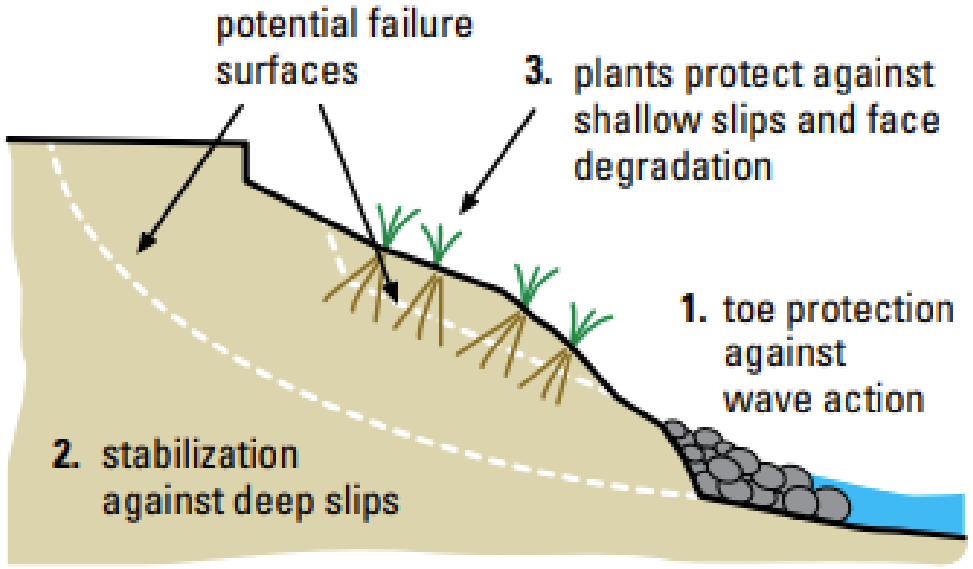






BIOTECHNICAL EROSION CONTROL





BIOTECHNICAL EROSION CONTROL

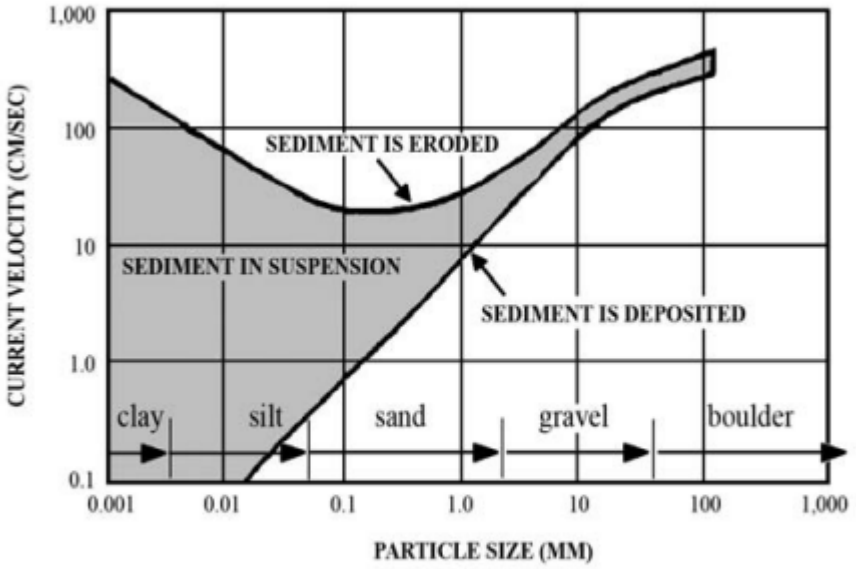


Figure 1: *Erodibility of different shoreline materials by streamflow.*



BELOW THE WATER LINE

Plant Name	Category	Adaptive Feature
Peltandra virginica	FORBS	Massive root system; Forms clumps; Breaks up wave action
Sparganium eurycarpum	Grasses/Sedges/Rushes	Provides a nice wave buffer; Has potential to outcompete cattails; Spreads readily from rhizomes; Can be opportunistic
Cephalanthus occidentalis	Trees/Shrubs	Deep spreading roots: Commonly used as live stakes

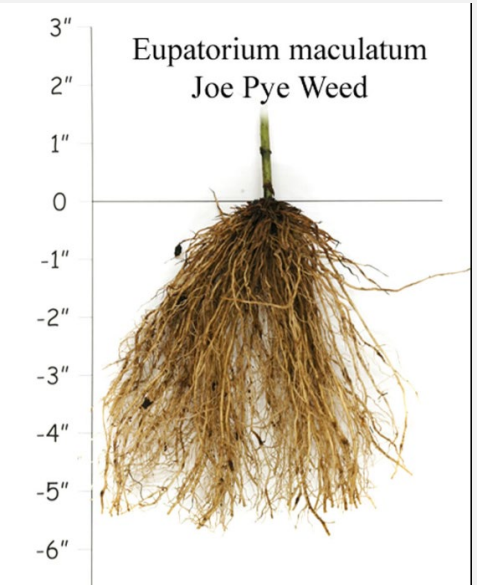






BETWEEN WATER LEVEL AND ORDINARY HIGH WATER MARK

Plant Name	Category	Adaptive Feature
Eupatorium maculatum	FORBS	Shallow fibrous roots
Juncus balticus	Grasses/Sedges/Rushes	Spreads opportunistically by rhizomes, forming clumps, prefers sandy shores
Cornus sericea	Trees/Shrubs	Fibrous roots, can be opportunistic: commonly used as live stakes







ABOVE THE ORDINARY HIGH WATER MARK

Plant Name	Category	Adaptive Feature
Chelone glabra	FORBS	Deep fibrous roots
Cinna arundinacea	Grasses/Sedges/Rushes	Roots are fibrous and rhizamatous; usually deer resistant
Quercus rubra	Trees/Shrubs	Fast growing

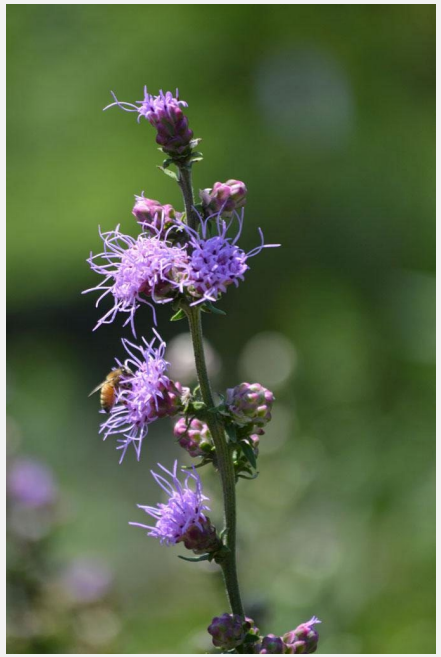


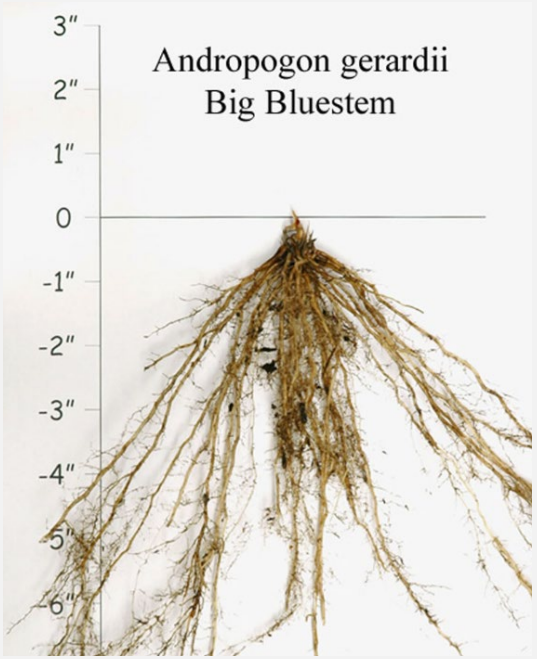




UPLAND

Plant Name	Category	Adaptive Feature
Liatris aspera	FORBS	Drought tolerant; Attracts butterflies; Blooms late
Andropogon gerardii	Grasses/Sedges/Rushes	Grass; Erosion control use, beneficial to birds; Can be opportunistic
Ceanothus americanus	Trees/Shrubs	Shrub; Taprooted; Drought tolerant

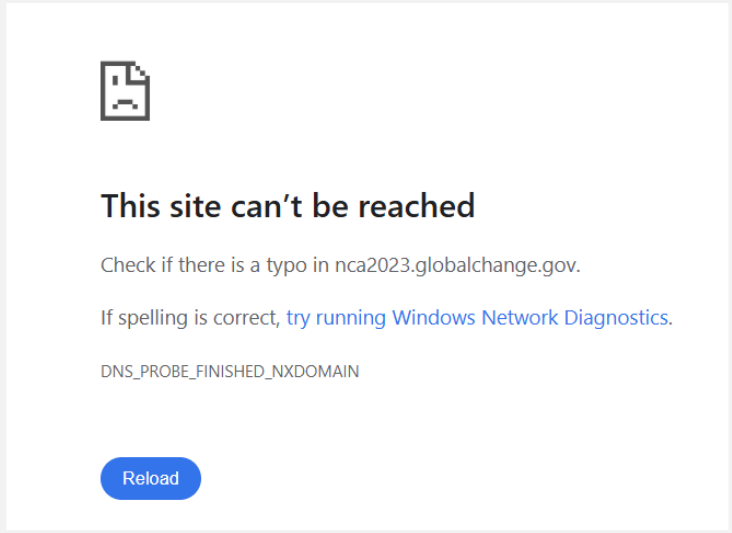


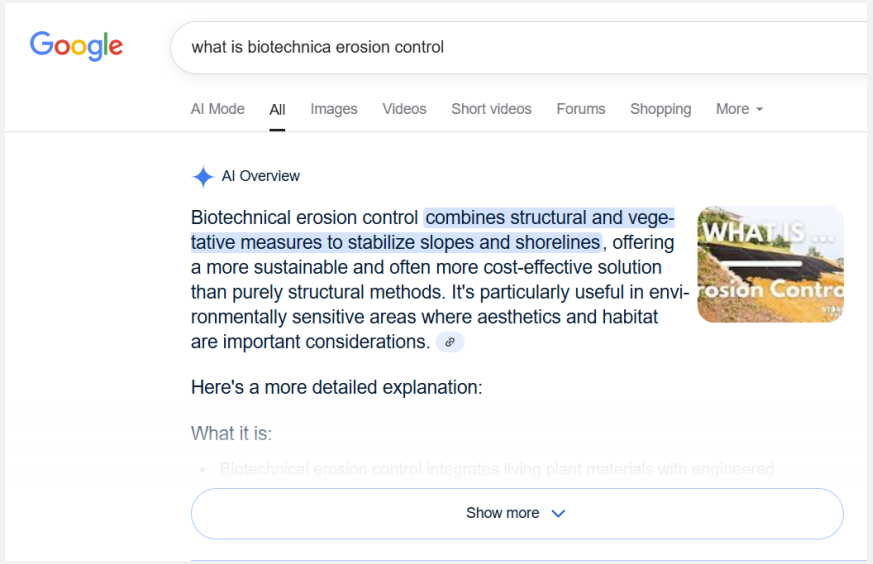


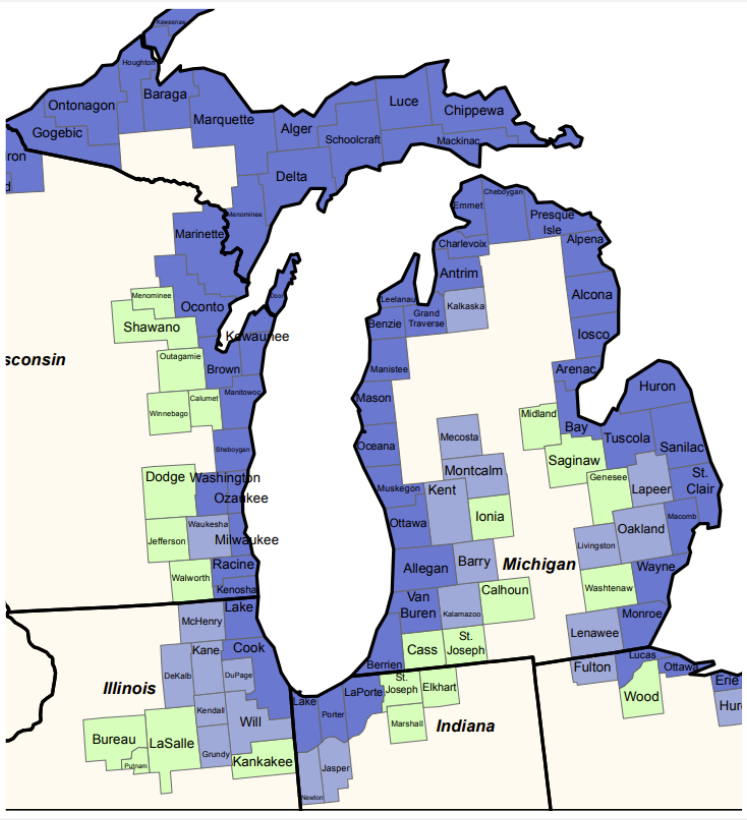


CONCERNS FOR THE FUTURE OF BIOTECHNICAL EROSION CONTROL

- Quantity and quality of data
- Differences in state legislation and ordinances
- General Public
- Accessibility
- Artificial Intelligence







RESEARCH NEEDS

- Rose – Hulman Institute of Technology
- Scale model
- Fibrous root structures in sandy soil
- Wave energy







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DFI50 - Presentation

North East Link

Lower Plenty Road

Diaphragm Walls



Thadu Khin

21/10/2025

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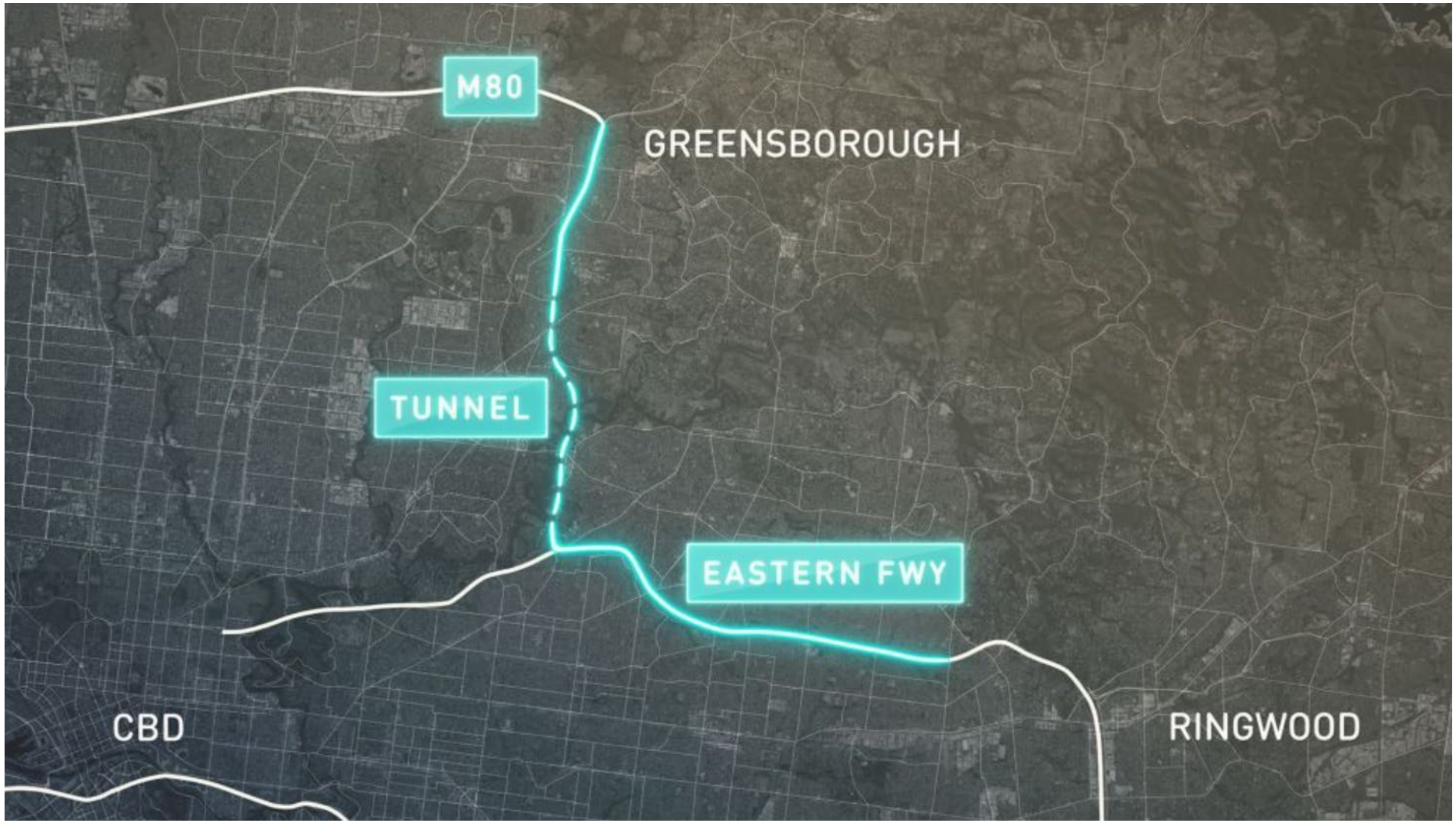
WAGSTAFF PILING

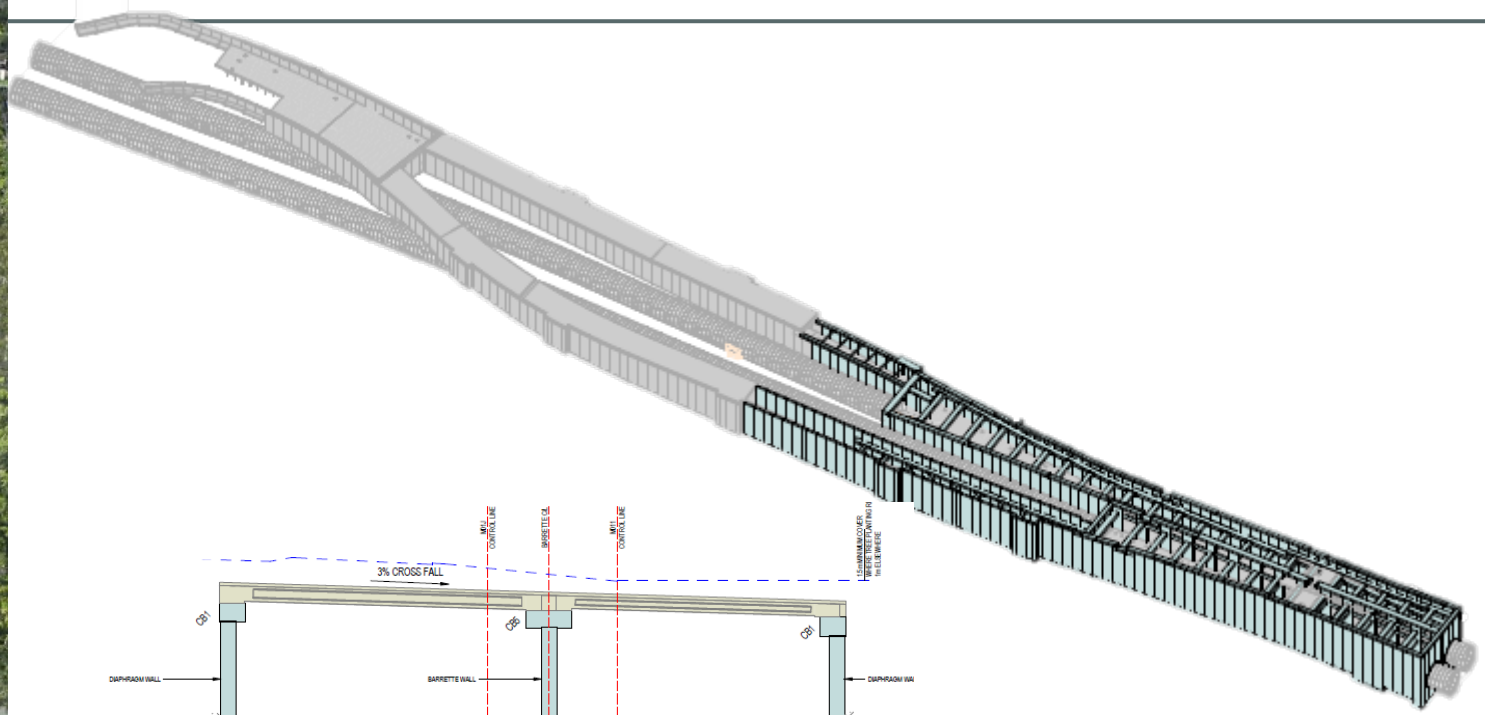
40 YEARS 1981-2021 ANNIVERSARY

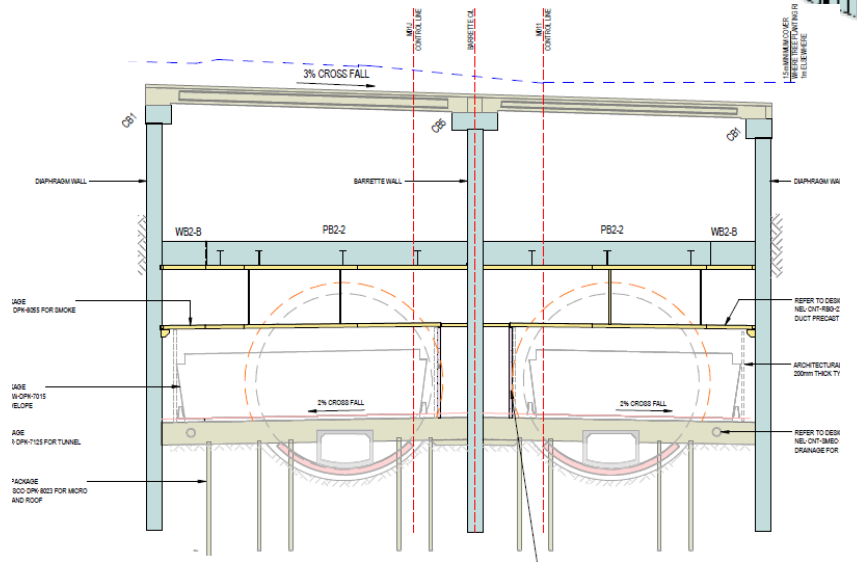
North East Link Project – Lower Plenty Interchange



North East Link Project – Lower Plenty Interchange







Project Summary – Lower Plenty Road

- Trevi Australia / Wagstaff Piling Joint Venture (TAWP JV)
- 471 No. Panels x 1200mm wide from 2.8m to 6.8m upto 43m deep
- 35,000 Tons of Steel
- 89,090 M3 of Excavation Volume
- 79,000 M3 of Concrete
- 4 panels per week / 2.5 year installation program



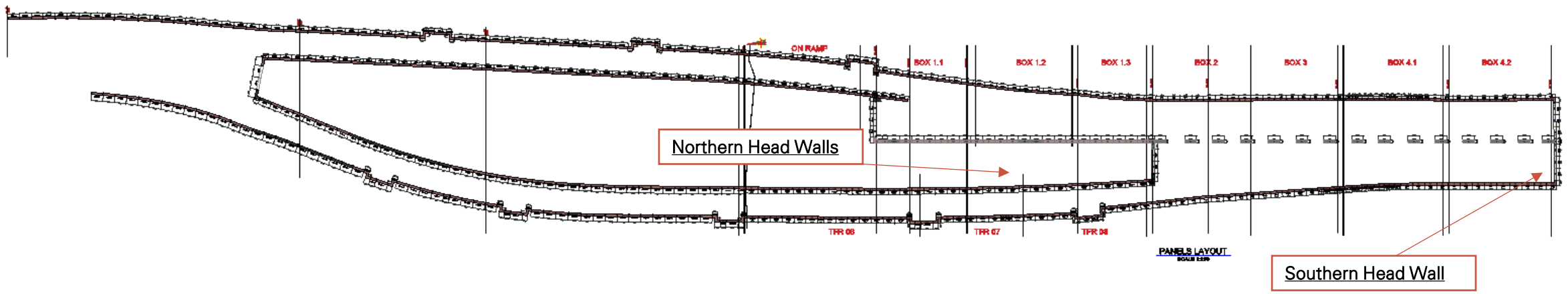
D-Wall Structure





D-Wall:
Permanent support from concrete walers and struts which are secured to the barrettes. Top down construction, wall has no anchors.

Diaphragm Wall – Ramps + Box





Plant and Equipment

Equipment utilised on project:

- Soilmec SC120HD foundation crane
- Lierbherr HS8130 foundation crane
- Soilmec SC130HD Tiger Plus (Crane base for cutter) x2
- Soilmec SC135HD x2
- Soilmec SH40 Trench cutter x4
- Stein K1210HD Rope grab x2
- MPS D500.500 Desanding Unit x2
- Sotres D450 Desanding Unit x2
- Lodos 30/SGM45/SMD100-160





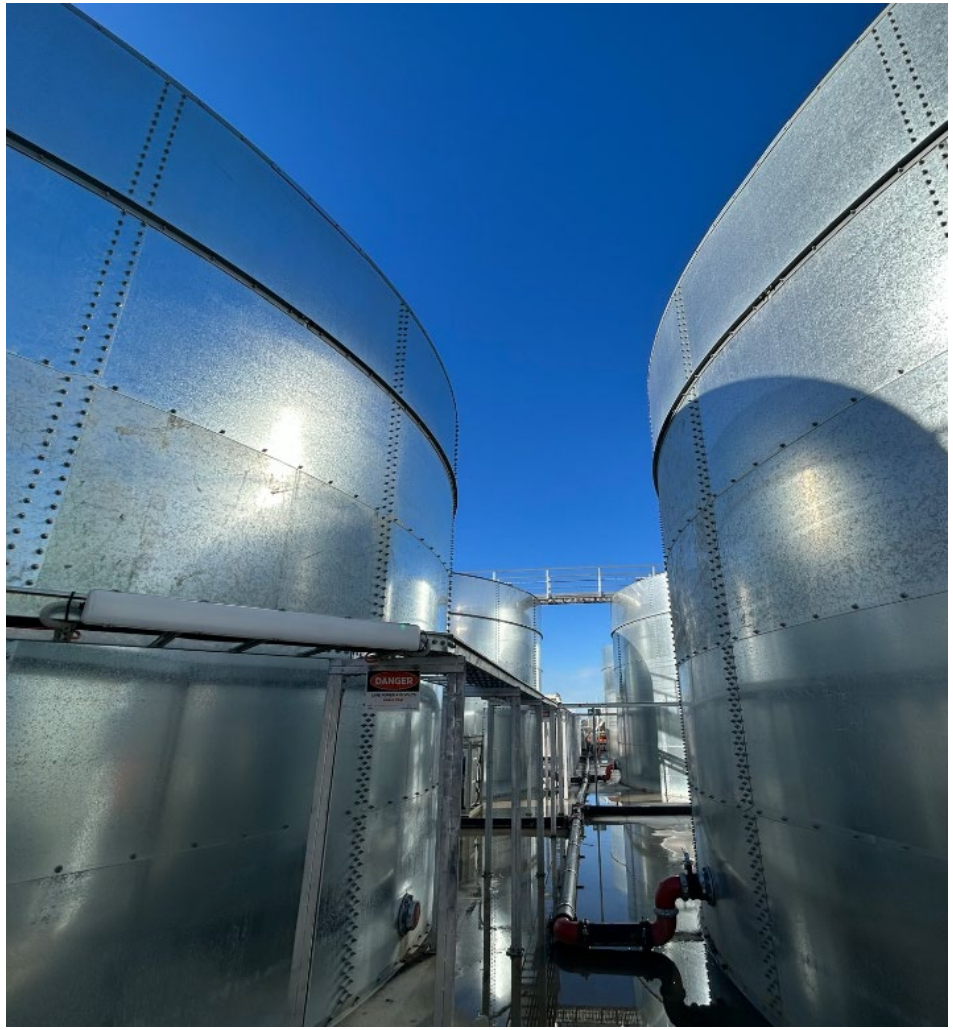
Plant and Equipment



Bentonite Plant

7x 600 cubic metre (158,000 gallon) tanks, 5.8m high and 12m in diameter

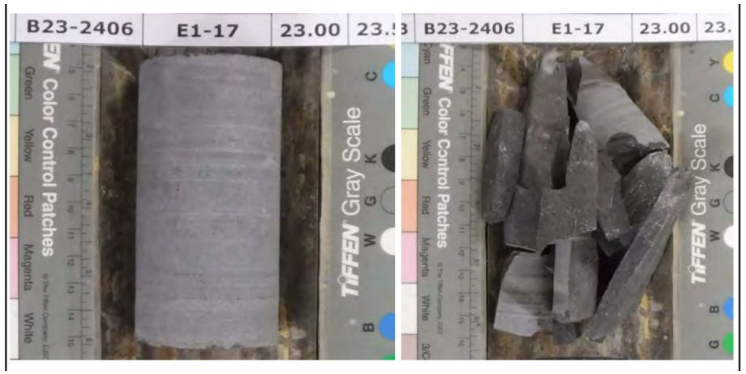




GEOTECHNICAL INFORMATION

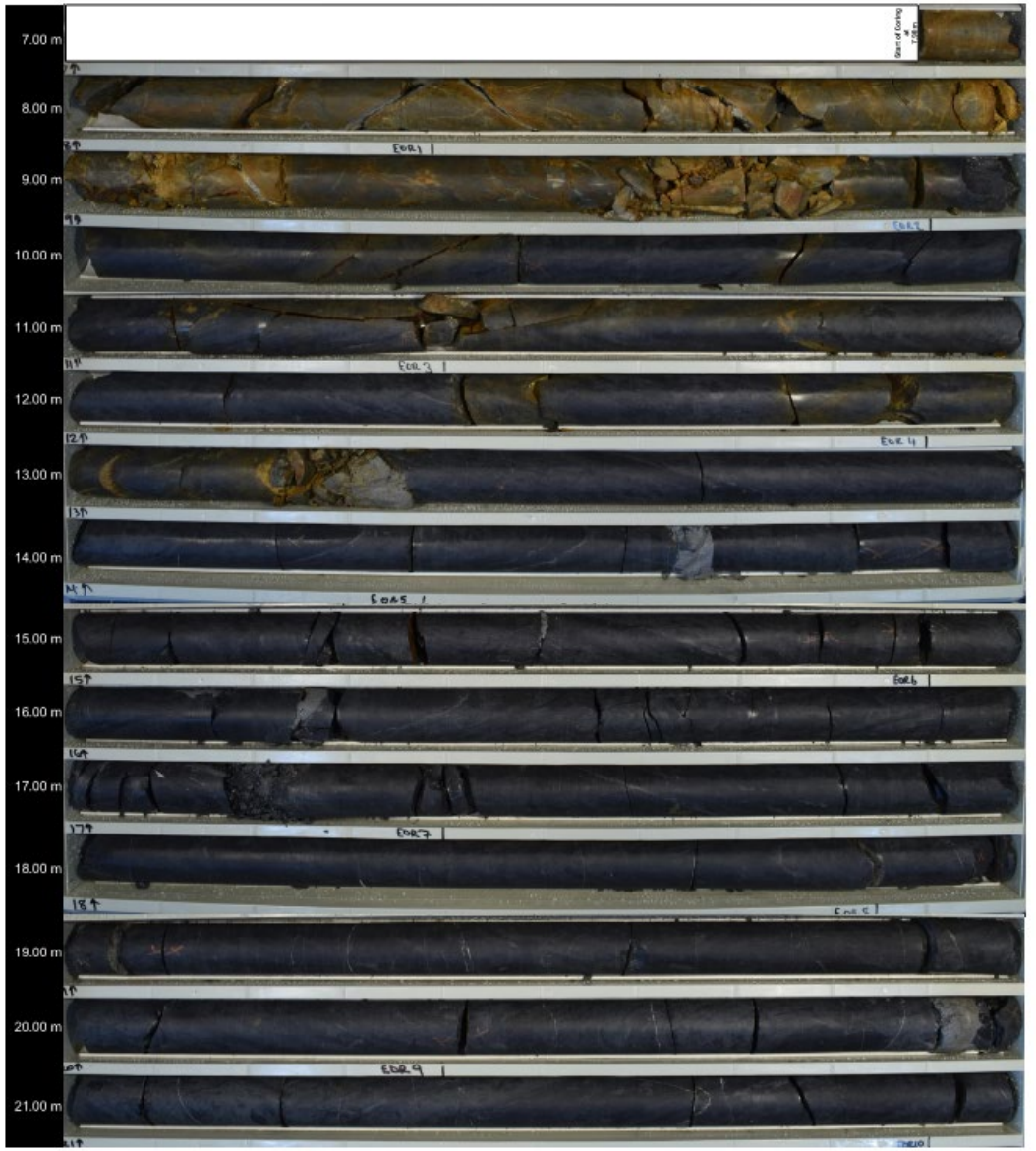
<u>Eastern Side</u> – Typically 8-10m of Clay, 4m M/H MW Siltstone with <u>clay seams</u> followed by High Strength Fresh Siltstone

Hard rock & Good RQD = SLOW CUTTING (AVG 3m^3/hr. ~1m/hr)



UCS 240MPa!!! (Intrusion??)





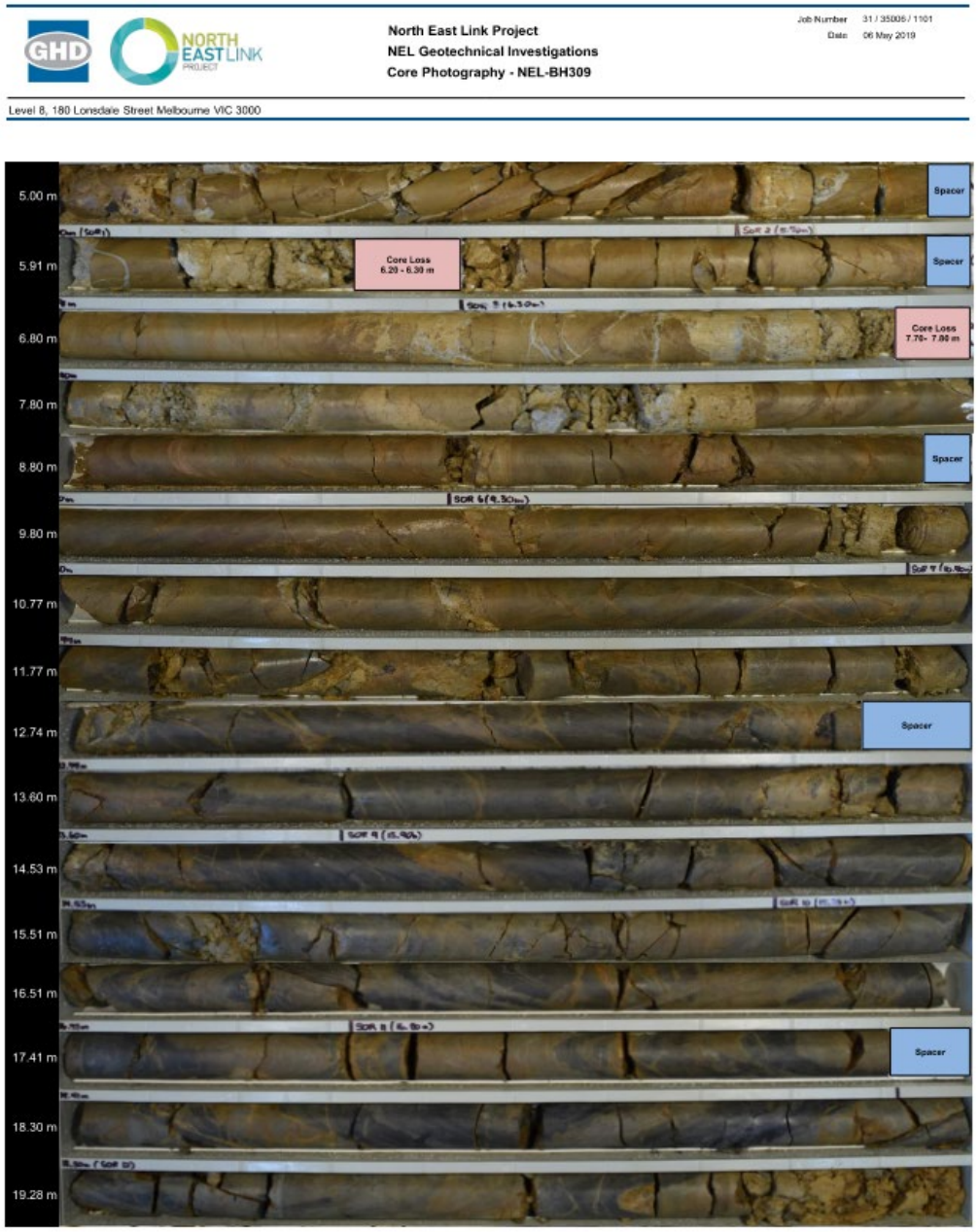
GEOTECHNICAL INFORMATION

<u>Western Side</u> – Typically 3-4m of Clay, 20m L/M MW Siltstone with <u>clay seams</u> followed by M/H FR/SW Siltstone

Weaker rock + Average RQD's = BETTER CUTTING (AVG. 10m^3/hr. ~3m/hr)







GROUNDWATER LEVEL

Very important for trench stability

Western Side – Typically >2m bgl. OK

Eastern Side – Typically >2m bgl. Except in one area, about <1m bgl. BAD





SUPPORT FLUID

Bentonite gets very heavy, SG >1.4 grams/cm3 (>0.81 oz/in3), due to the cutting process & build up of silts. Heavy fluid is harder to pump and also out of spec for re-use.

To reduce SG:

#1 – Desilter Screens.

#2 – Centrifuge.

#3 - DFA 68 (thinner/flocculant) added in plant to drop fines.

Property to be measured	Units	Test method and apparatus	Compliance values measured at 20°C		
			Freshly mixed	Ready for re-use	Sample from excavation prior to concreting
Density	g/ml	Mud balance	<1.10	<1.25	<1.15
Fluid loss (30-minute test)	ml	Low-temperature test fluid loss	<30	<50	N/A
Filter cake thickness	mm	Low-temperature test fluid loss	<3	<6	N/A
Viscosity	s	Marsh cone	32 to 50	32 to 60	32 to 50
Sand content – refer note 1	%	Sand screen set	N/A	N/A	<4
pH	N/A	Electrical pH meter; range pH 7 to 14	7 to 11	7 to 12	N/A

SUPPORT FLUID

- Type: Bentonite (GEL AUS 4) is used for support fluid.
- Mix: ~4-5% by weight to water, example 45kg of powder per 1000L of water.
- Polymer: Sometimes added for viscosity in small amounts.
- During Excavation, due to extremely high fines content in Siltstone bedrock, the specific gravity of the working bentonite inside the panel remained around SG = 1.4.
- The sand content of this working bentonite was around SC = 0.25% indicating that the fines dissolved in the bentonite were very fine and unable to screened out.
- The desilters on the MPS desanders could filter out particles to 145 microns, so particles smaller than this would remain in the bentonite



SUPPORT FLUID

- Diaphragm wall best practice is to desand the bentonite and exchange it for “casting slurry” which is maintained as light and clean as possible and usually isolated in separate tanks. This usually happens at completion of excavation/removal of stop ends, and prior to installation of reinforcement cages and concrete placement.
- This fluid is typically maintained close to fresh bentonite parameters around an SG of 1.05.
- The sand contents, viscosity, filtration and rheological properties will behave similarly to freshly mixed bentonite inside the panel.

TRENCH STABILITY

Designed in accordance with DIN4126:2013-09. In-house spreadsheet.

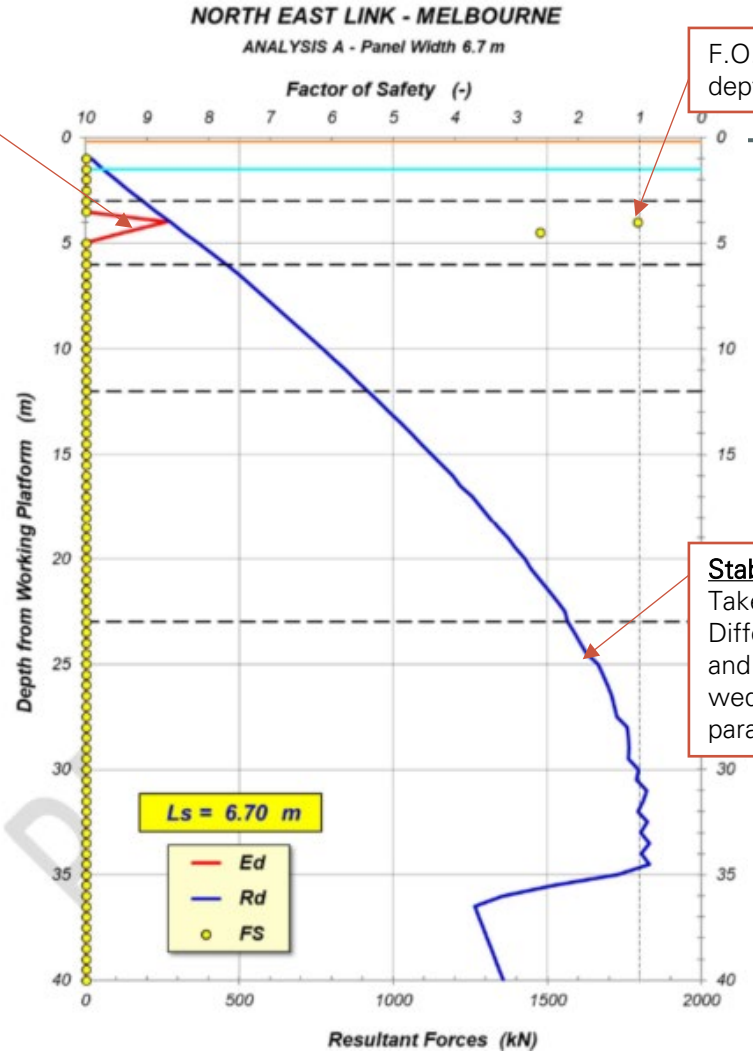
Takes into account: panel geometry, wedge geometry, loadings, support fluid level and fluid/soil parameters.

SOIL STRATIGRAPHY & GEOTECHNICAL PARAMETERS — Number of Layers: 5

Layer (-)	Soil Type	Z,str (m)	γ_nat (kN/m³)	γ_sat (kN/m³)	Gs (kN/m³)	C_K (kPa)	φ'_K (°)	K_0 (-)	D_{10} (mm)	f_{s0} (kN/m³)
1	Cohesive	3,00	16,0	17,0	26,5	25,0	0,0	0,600	0,0010	20000
2	Cohesive	6,00	18,0	19,0	26,5	70,0	0,0	0,700	0,0010	20000
3	Cohesive	12,00	23,0	23,0	26,5	40,0	35,0	1,000	0,2000	100
4	Cohesive	23,00	24,0	24,0	26,5	75,0	41,0	1,000	0,2000	100
5	Cohesive	45,00	26,0	26,0	26,5	200,0	46,0	1,000	0,2000	100

De-Stabilizing force:
Takes into account soil parameters, panel size and loadings

F.O.S vs depth



Stabilizing force:
Takes into account Difference between fluid and water pressure, wedge geometry and soil parameters

Particle size @ 10% passing. Need PSD for this or make an estimate

Pressure Gradient = 2 x Bentonite shear strength / D10 . Bentonite shear strength changes with % & time. Typically values 10 – 15

TRENCH COLLAPSE

- Multiple Panels Collapsing Immediately after Clean and Exchange of Bentonite Support Fluid.
- Working Bentonite with SG = 1.4 replaced with Casting Bentonite with SG = 1.03-1.05 (~0.59 oz/in3).
- First panel was 35m deep excavation. Collapsed from 12m at one end to 19m at the other





TRENCH COLLAPSE

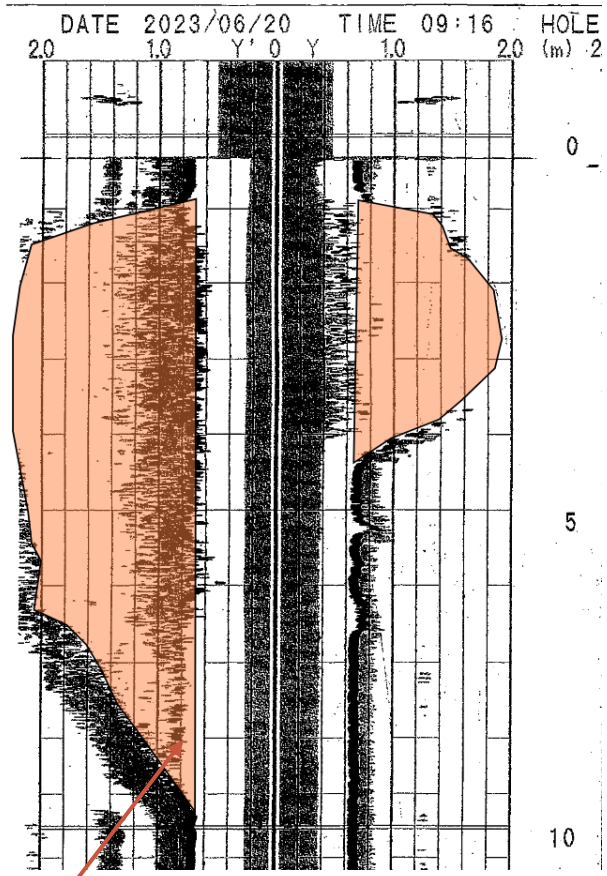
Major trench collapses:

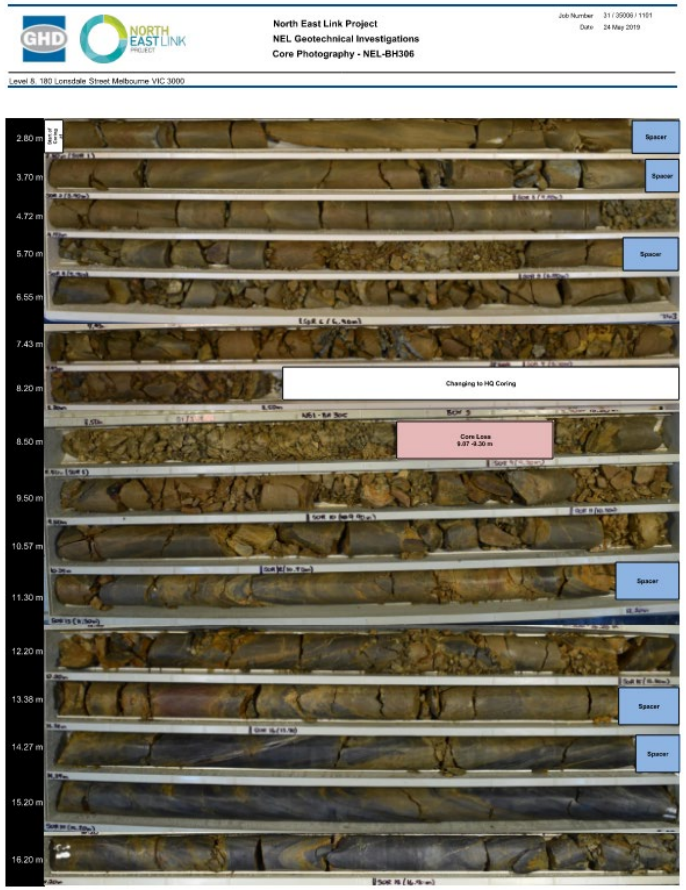
#1 – West (W1-2) – minor collapse. Collapsed material re-excavated. Panel caged and poured.

#2 - West (W2-2) – major collapse of NF. Panel backfilled with lean mix concrete.

#3 – East (E1-3) – minor collapse of CB. Panel backfilled with lean mix concrete.

#4 – West (W3-12) – major collapse of NF & FF. Panel backfilled with lean mix concrete. (Koden survey to the right and borehole)





70° bedding plane

TRENCH COLLAPSE

Area History:

Area subject to tectonic activity and folding. The folding has changed horizontal bedding to become very steep. Lots of additional joints and shear zones associated with folding/formation of a hinge in portion of the box.

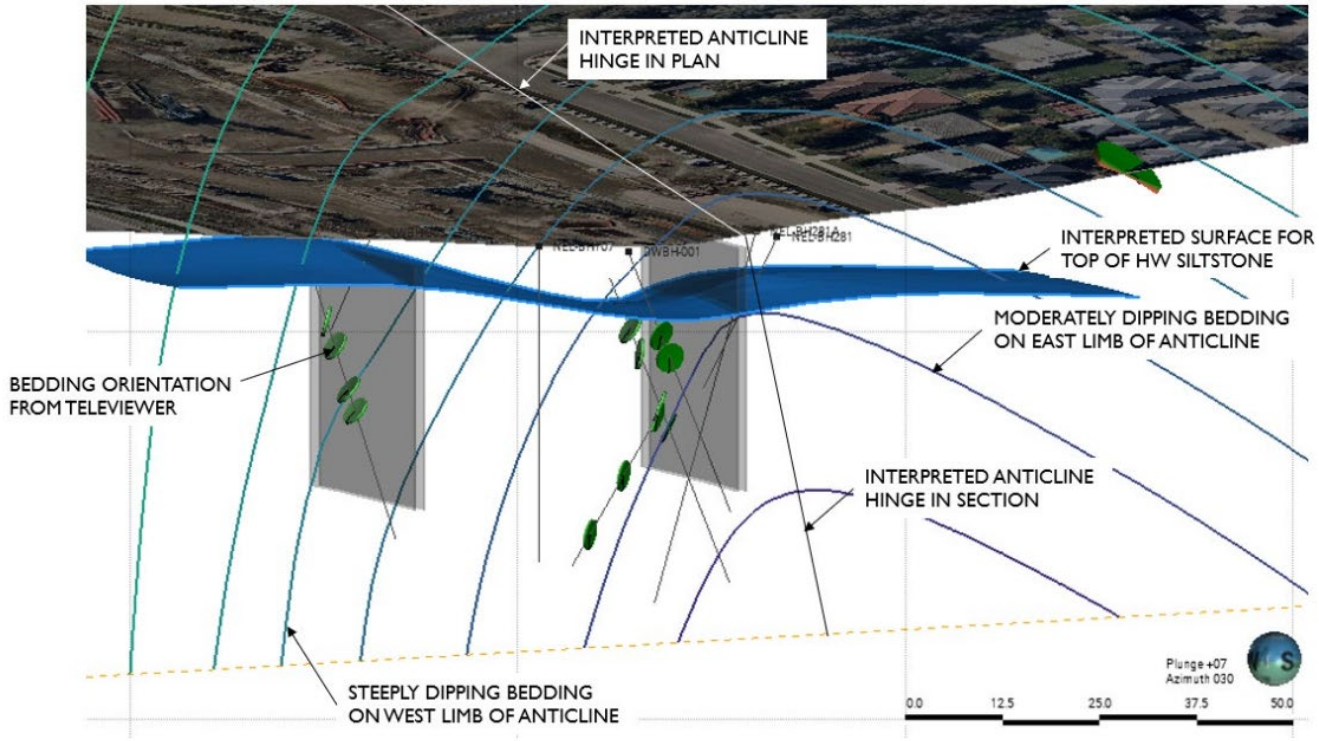
Why did the collapses happen:

#1 – Highly fractured zones

#2 – Big panels

#3 – Panels open for long time

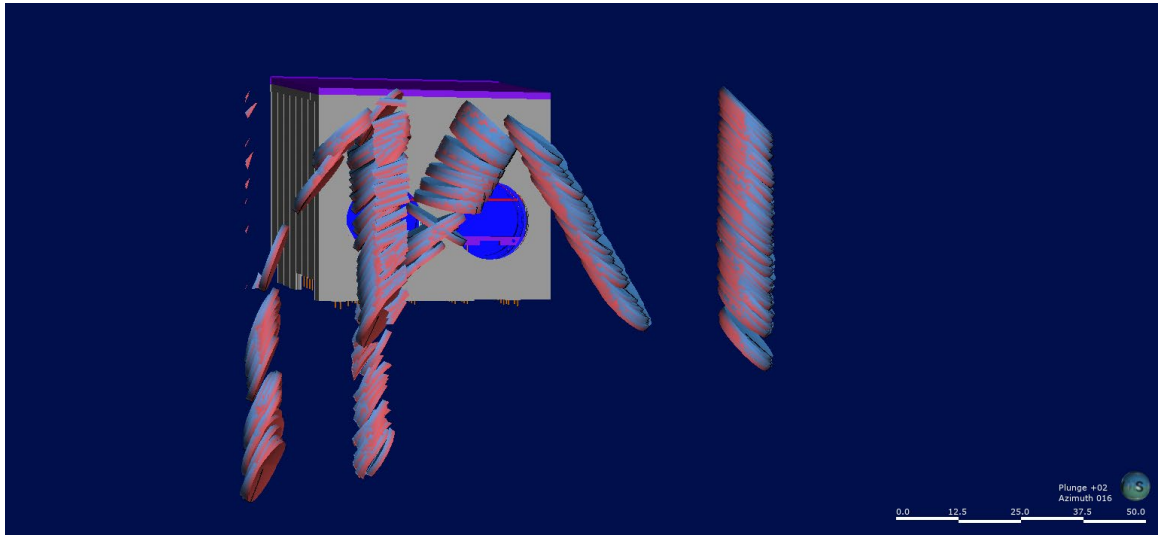
All of the above most likely lead to the collapses

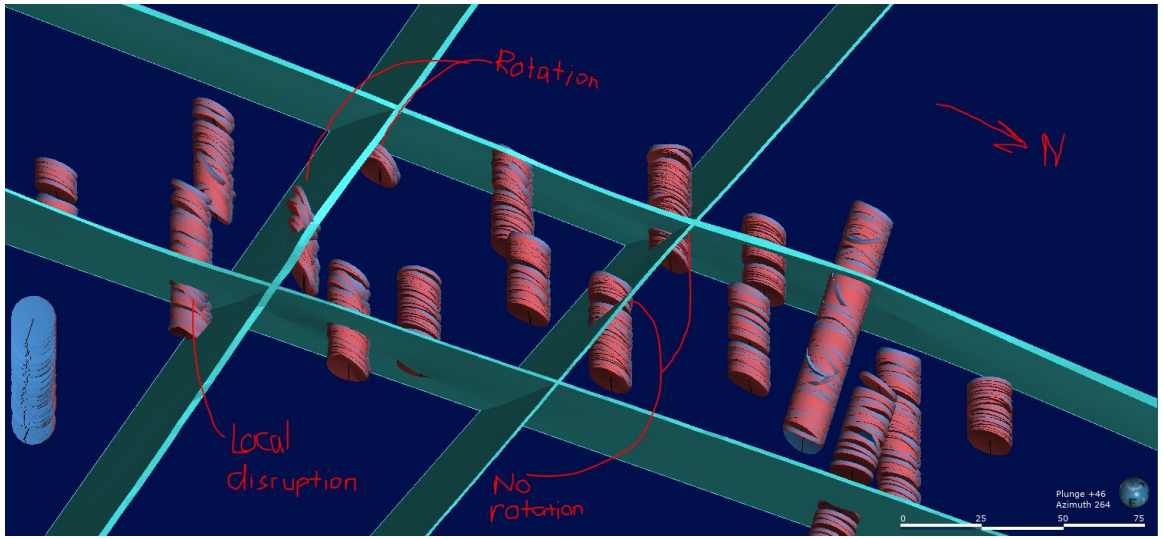


TRENCH COLLAPSE

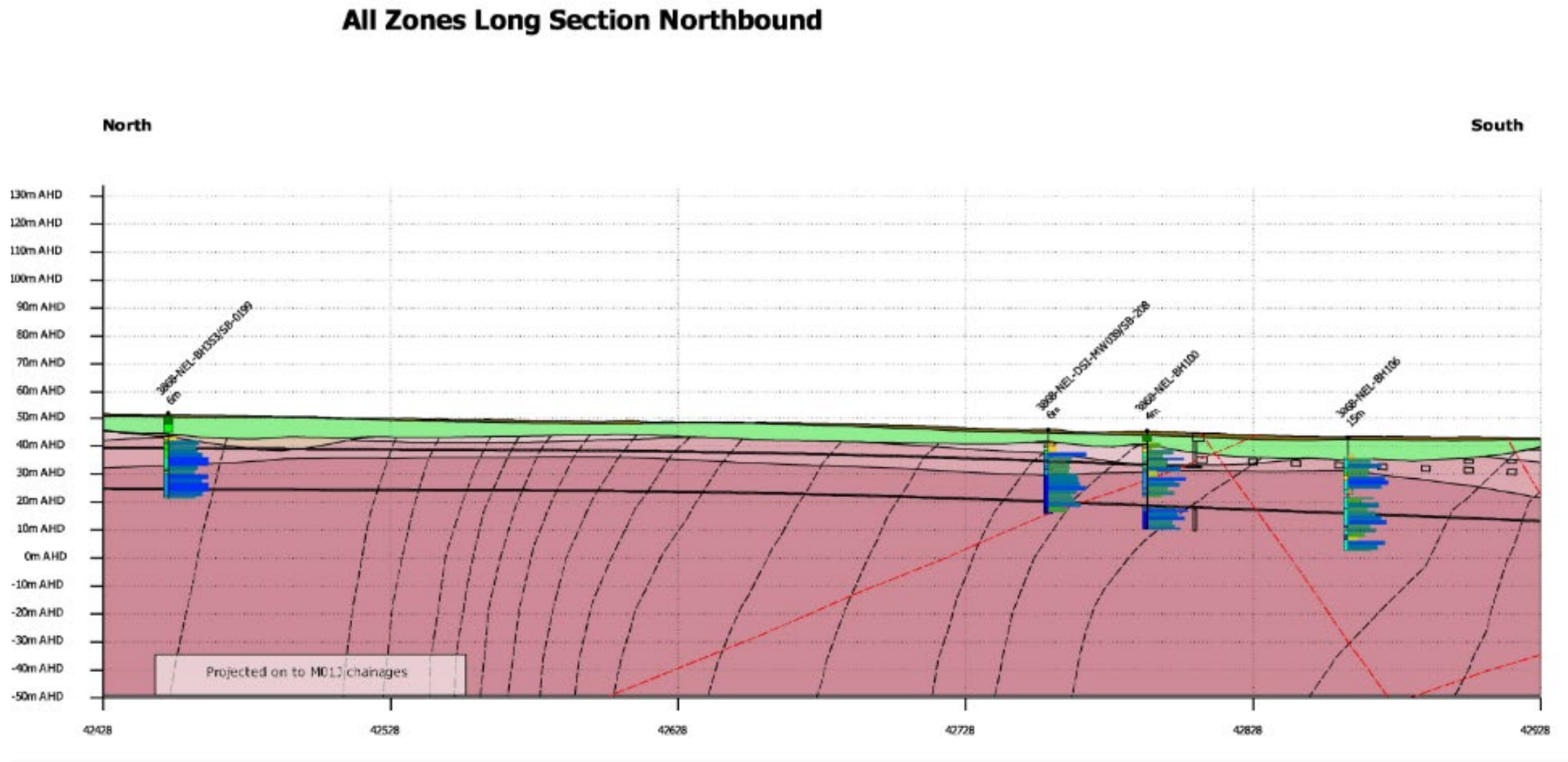
Analysis of bedding planes:

- Televiewer assessment indicates increase in relative strike in bedding plane of approximately 10-15 degrees more than elsewhere in the structure and 5-15 degrees increase in dip angle.
- Rock wedges failed along planes supported by Koden profiling of the D-Wall panels.

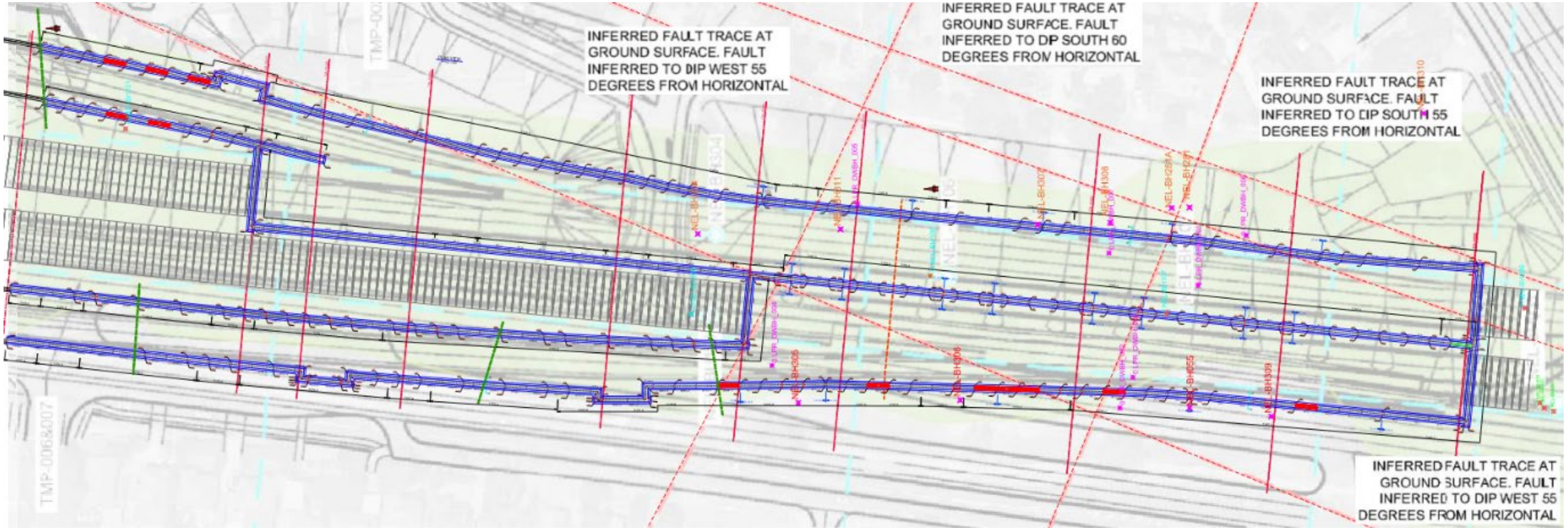




TRENCH COLLAPSE



TRENCH COLLAPSE



TRENCH COLLAPSE / STABILITY

After collapses the trench stability was re-assessed.

Used defect parameters for the rock with double panels & also single panels.

Single bite panel OK with C' = 0 and phi = 23 degrees.

Competent rock

SOIL STRATIGRAPHY & GEOTECHNICAL PARAMETERS								Number of Layers		5
Layer (-)	Soil Type	Z,str (m)	γ_nat (kN/m³)	γ_sat (kN/m³)	Gs (kN/m³)	C_K (kPa)	φ'_K (°)	K_0 (-)	D_{10} (mm)	f_{s0} (kN/m³)
1	Cohesive	3,00	16,0	17,0	26,5	25,0	0,0	0,600	0,0010	20000
2	Cohesive	6,00	18,0	19,0	26,5	70,0	0,0	0,700	0,0010	20000
3	Cohesive	12,00	23,0	23,0	26,5	40,0	35,0	1,000	0,2000	100
4	Cohesive	23,00	24,0	24,0	26,5	75,0	41,0	1,000	0,2000	100
5	Cohesive	45,00	26,0	26,0	26,5	200,0	46,0	1,000	0,2000	100

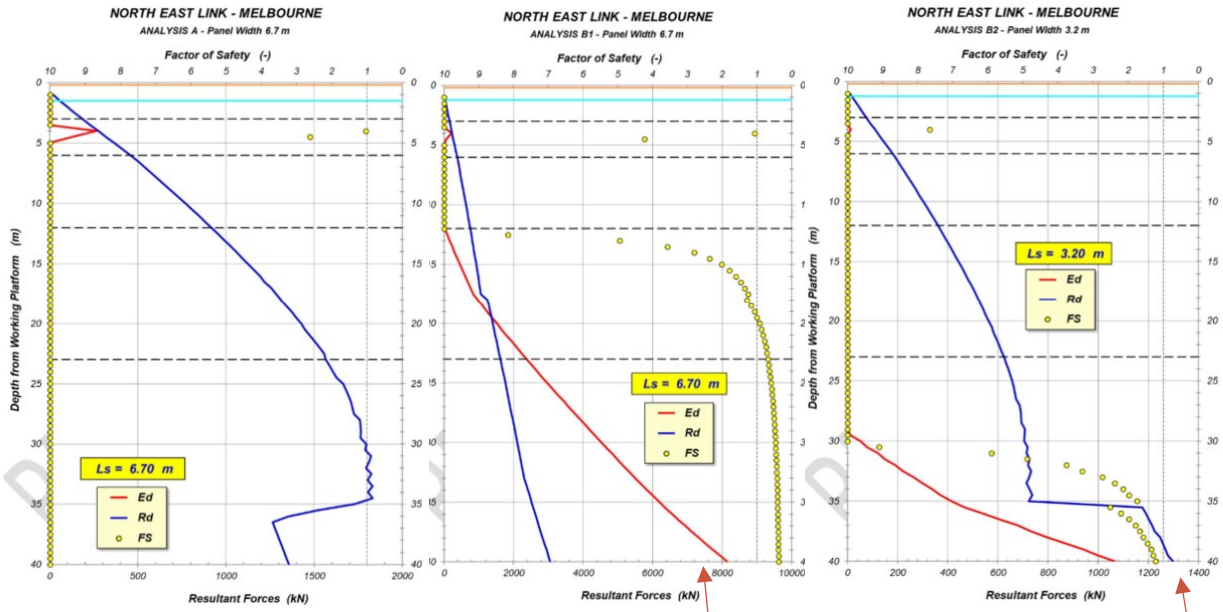
Defect rock

SOIL STRATIGRAPHY & GEOTECHNICAL PARAMETERS								Number of Layers		5
Layer (-)	Soil Type	Z,str (m)	γ_nat (kN/m³)	γ_sat (kN/m³)	Gs (kN/m³)	C_K (kPa)	φ'_K (°)	K_0 (-)	D_{10} (mm)	f_{s0} (kN/m³)
1	Cohesive	3.00	16.0	17.0	26.5	25.0	0.0	0.600	0.0010	20000
2	Cohesive	6.00	18.0	19.0	26.5	70.0	0.0	0.700	0.0010	20000
3	Cohesionless	12.00	21.0	21.0	26.5	0.0	23.0	0.609	0.2000	100
4	Cohesionless	23.00	21.0	21.0	26.5	0.0	23.0	0.609	0.2000	100
5	Cohesionless	45.00	21.0	21.0	26.5	0.0	23.0	0.609	0.2000	100

Double panel & competent rock parameters

Double panel & defect rock parameters

Single panel & defect rock parameters



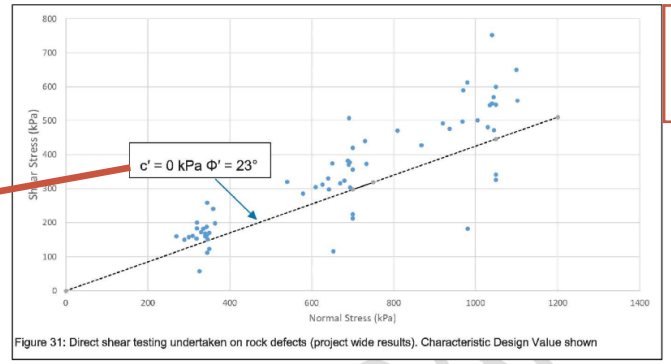


Figure 31: Direct shear testing undertaken on rock defects (project wide results). Characteristic Design Value shown

De-Stabilizing > Stabilizing (Double panel). FOS <1 @ 19m.

De-Stabilizing < Stabilizing (Single panel) OK

TRENCH COLLAPSE

Why did the collapses happen:

#1 – Highly fractured zones

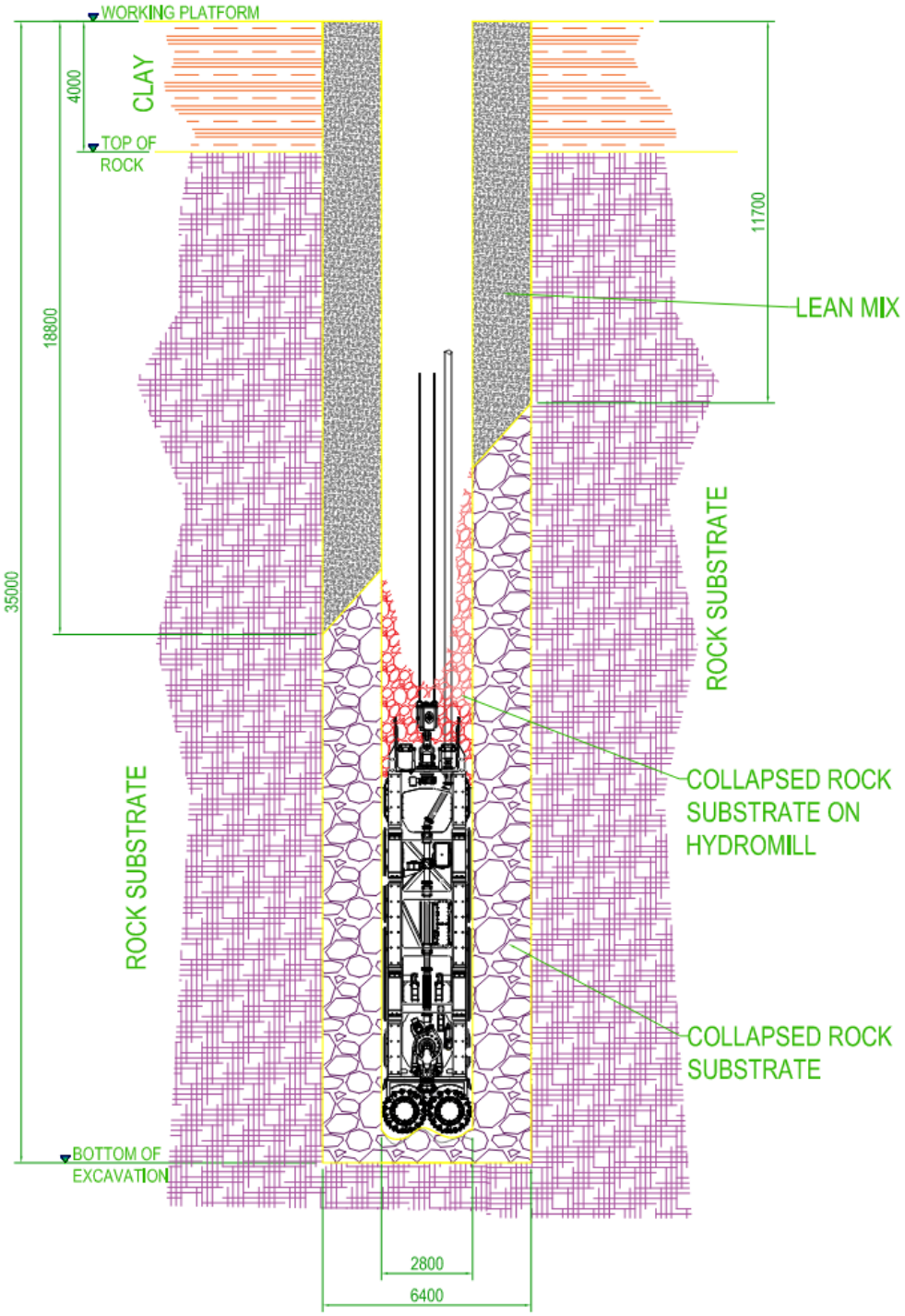
#2 – Big panels

#3 – Panels open for long time

#4 – Sudden change in specific gravity

#5 – Higher than anticipated groundwater levels / GWL too close to Guide Wall

#6 – Sudden drops in bentonite fluid level due to dynamic adjustment of fluid levels for module extraction/insertion



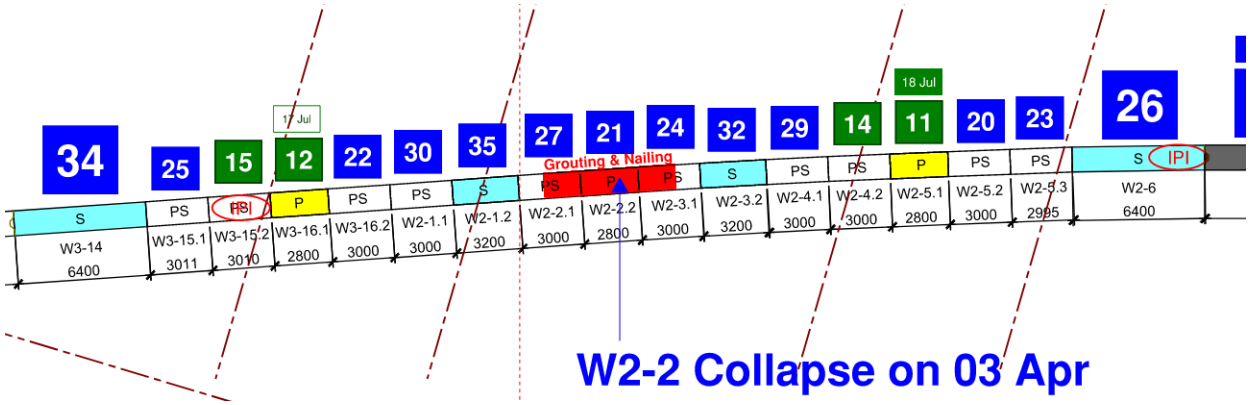
TRENCH COLLAPSE

Changes to methodology:

#1 – Panel size decreased to ~3m in lieu of 6.4m in areas with boreholes showing lots of shear zones. (Example to the right)

#2 – Panels in this area sequenced to try avoid extended periods of being open

#3 - Bentonite casting fluid kept as heavy as possible i.e. SG ~1.14. Provides more lateral support to side walls (extra 14kPa/m then water)



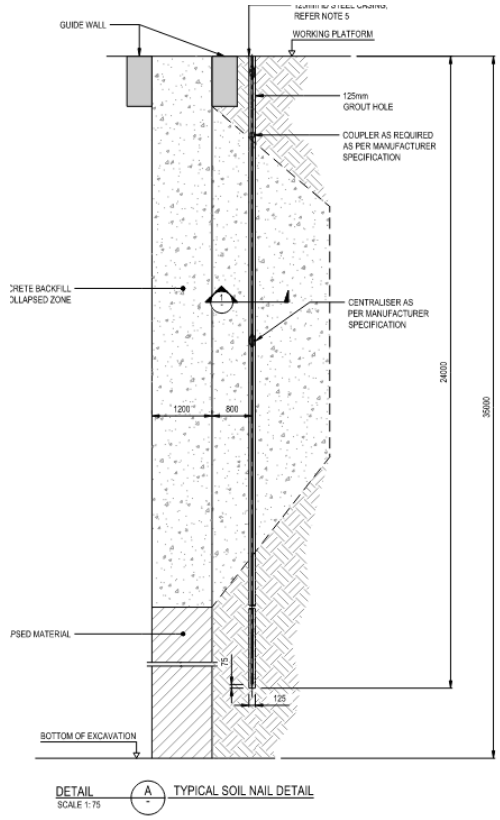
TRENCH COLLAPSE

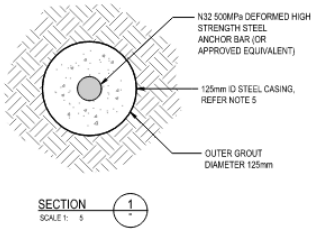
Rectification of collapsed panels

#1 – Collapsed zone in bottom of panel grouted to avoid further collapse when re-excavated (completed by SPARK)

#2 – Wedge of lean mix concrete held in place with anchors (completed by SPARK)

It was very difficult cutting the lean mix concrete as it will negatively affect the bentonite. Tons & tons of bi-carb as well as DFA68 will be used to reduce the pH level and stop bentonite getting too thick.





TRENCH COLLAPSE







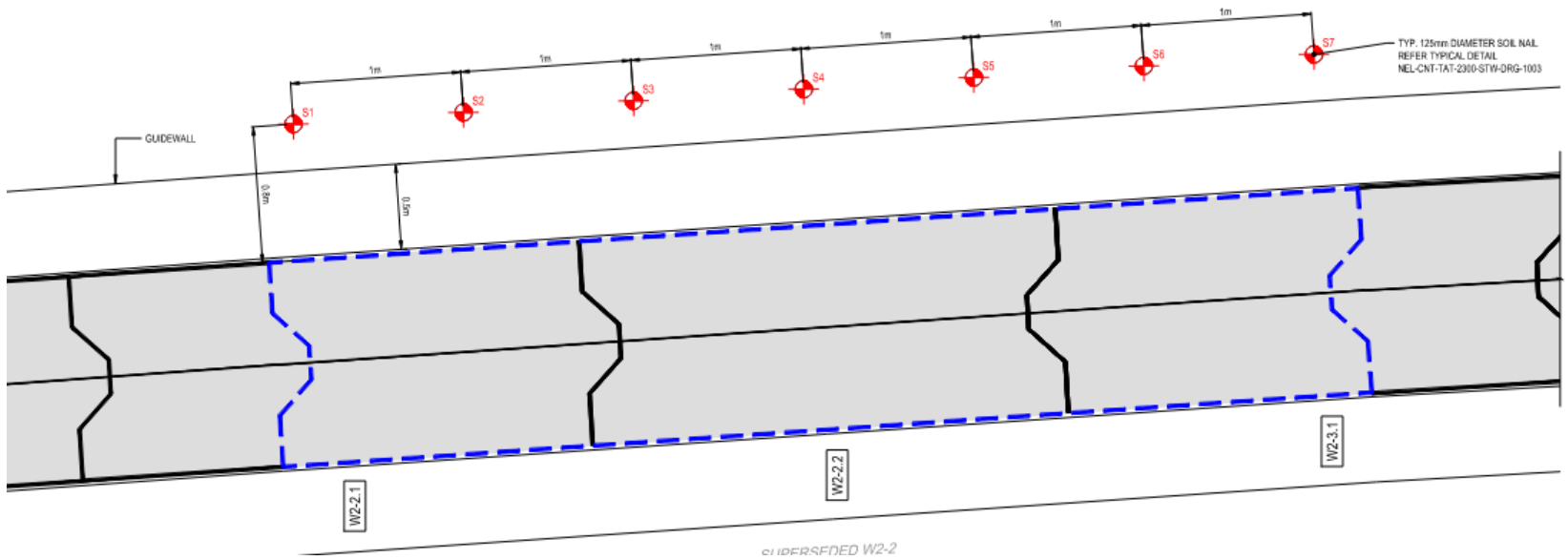




TRENCH COLLAPSE

Installation of Anchors and Grouting Problematic (by Others)

- Difficulties by contractor to drill holes through the backfilled concrete wedge without collapsing.
- Self drilling hollow bar anchors used.
- Reinforcement anchors ran into the excavation, ~4.7% verticality as entered panel 17m down and was off-set 850mm away from edge of diaphragm wall.



TRENCH COLLAPSE

Re-Excavation of Concreted Panels leading to cement contamination of Bentonite Slurry



TRENCH COLLAPSE – High Water Table

- Despite high SG's, 4 other panels collapsed in zone with faulted rock due to GWL <1.0m from platform level. Module below ground for 1hr+ waiting for client inspectors, collapsed.





Trench Collapse – Single Bite Panel in Fault Zone

One single bite panel in particular collapsed due to errors in construction methodology – very time consuming and costly fix!

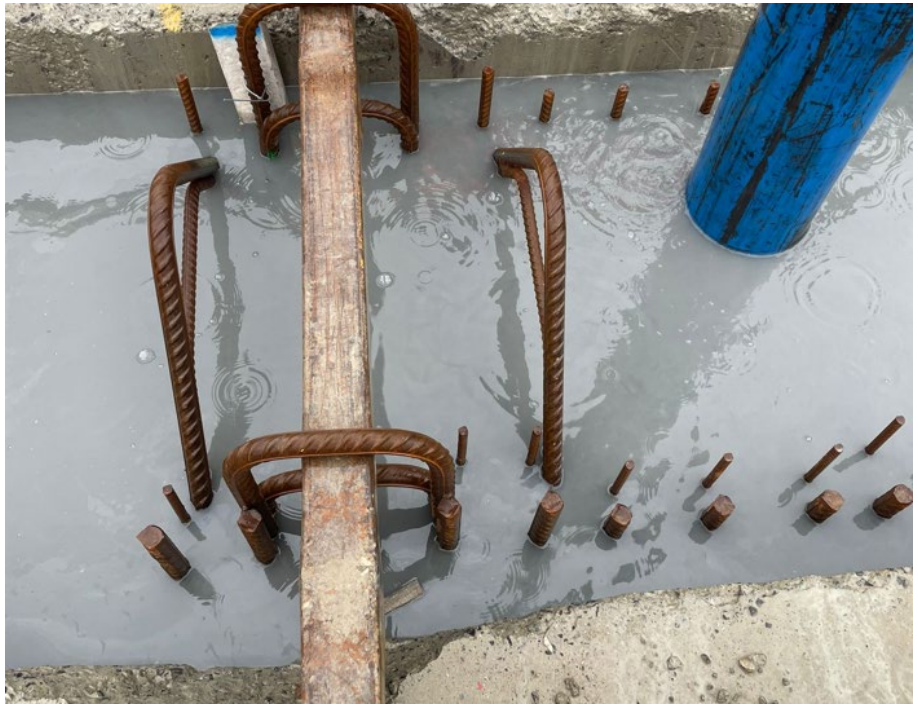
This panel was backfilled with lean mix concrete and the wedges were anchored and rockfill base grouted yet still collapsed due to incorrect fluid management.



Trench Collapse – Single Bite Panel in Fault Zone

Chasing a specification time unreasonably can lead to all manner of quality issues as evidenced by this particular panel panel W2-1.2. A minimum marsh cone result of 32 seconds was proving difficult to achieve post clean and exchange and as a result, an untested polymer was used to try and pass the test. Whilst the result passed, overnight the bentonite completely deflocculated in the panel and dropped to the bottom of the panel leading to these results.





Trench Collapse – Single Bite Panel in Fault Zone







Trench Collapse – Single Bite Panel in Fault Zone



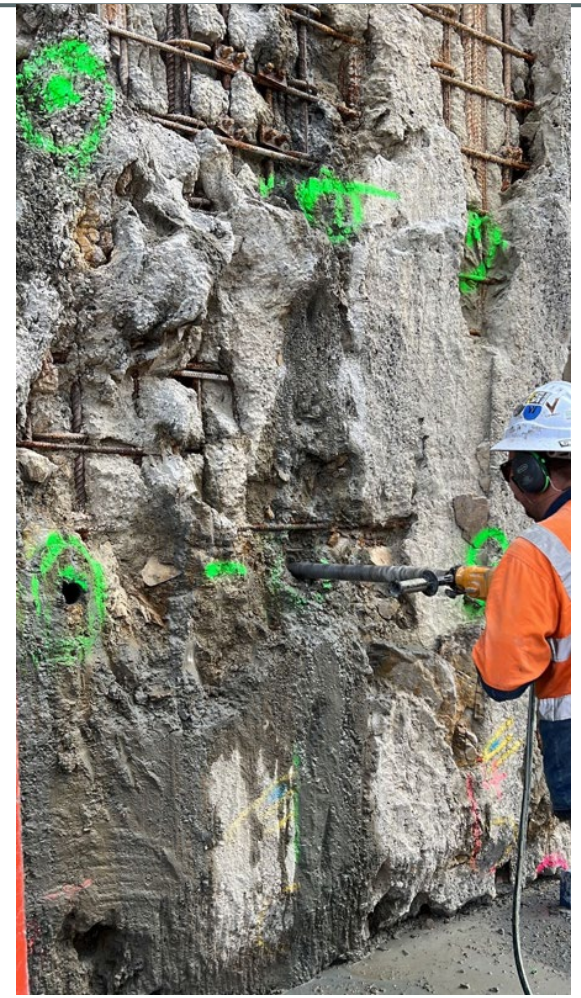




Trench Collapse – Single Bite Panel in Fault Zone



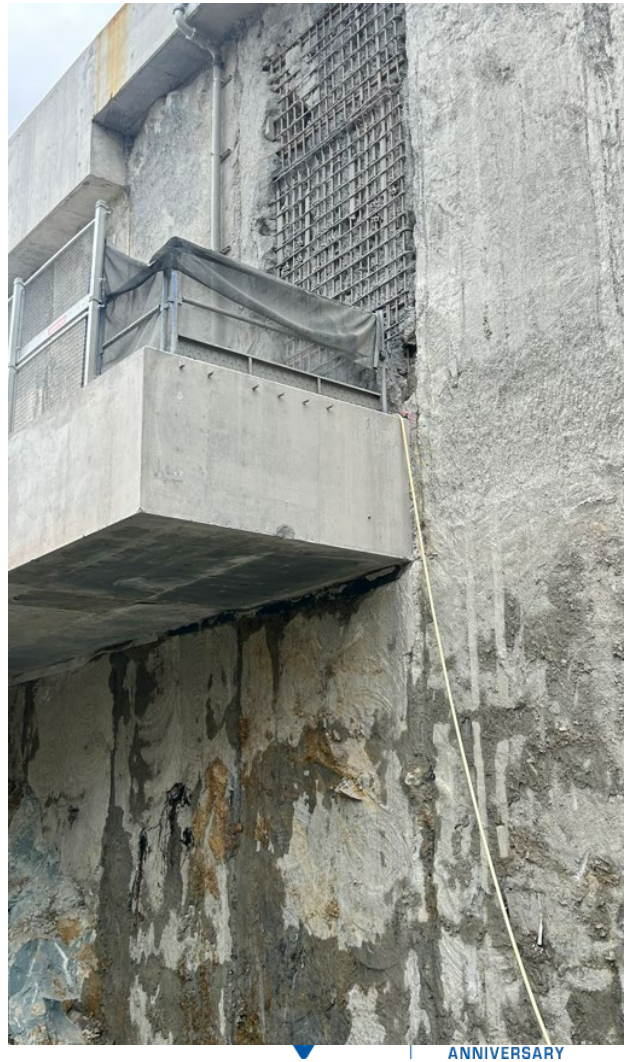




Trench Collapse – Single Bite Panel in Fault Zone

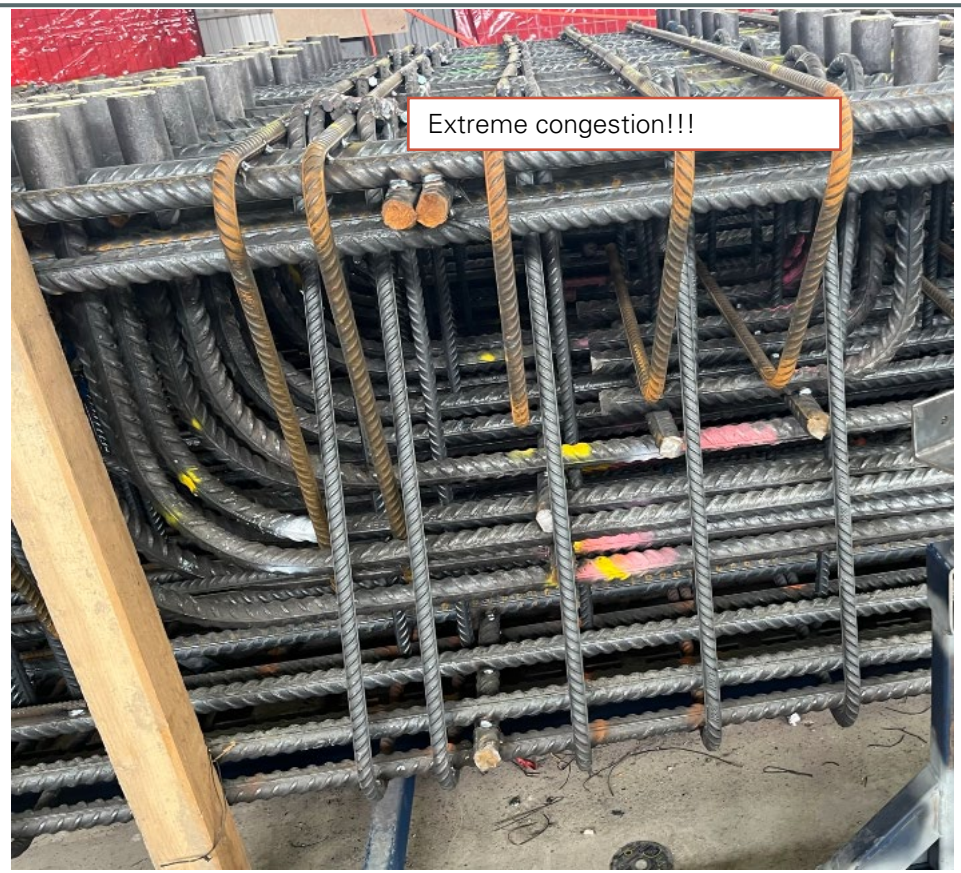






Reinforcement Cages





Reinforcement Cages





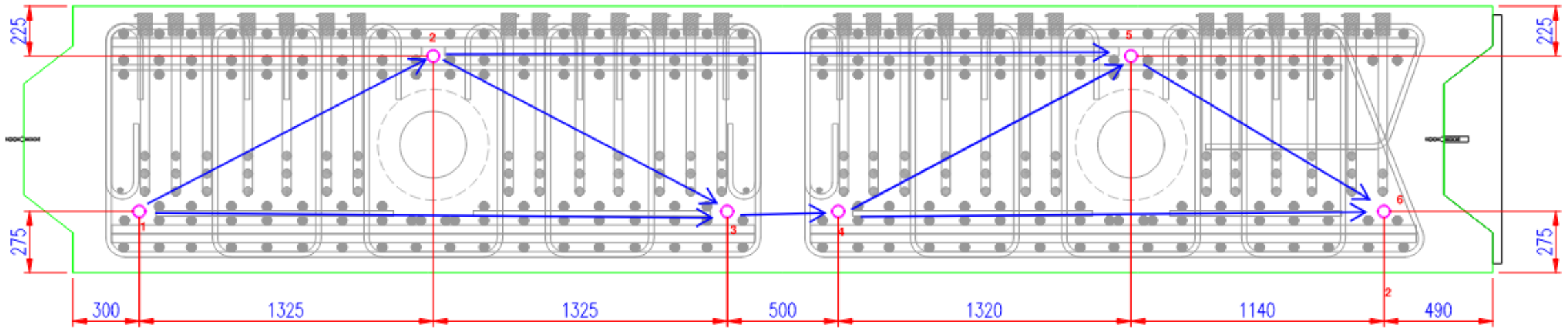
Glass Fibre Cages



Glass fibre cages used for soft eye in southern TBM headwall

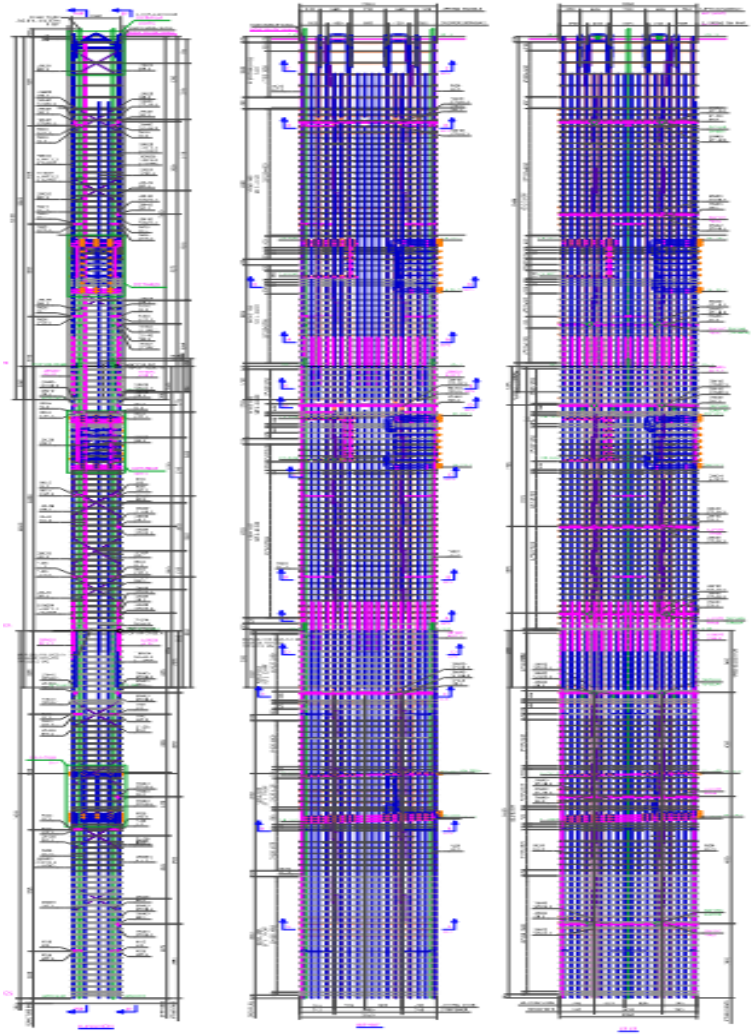


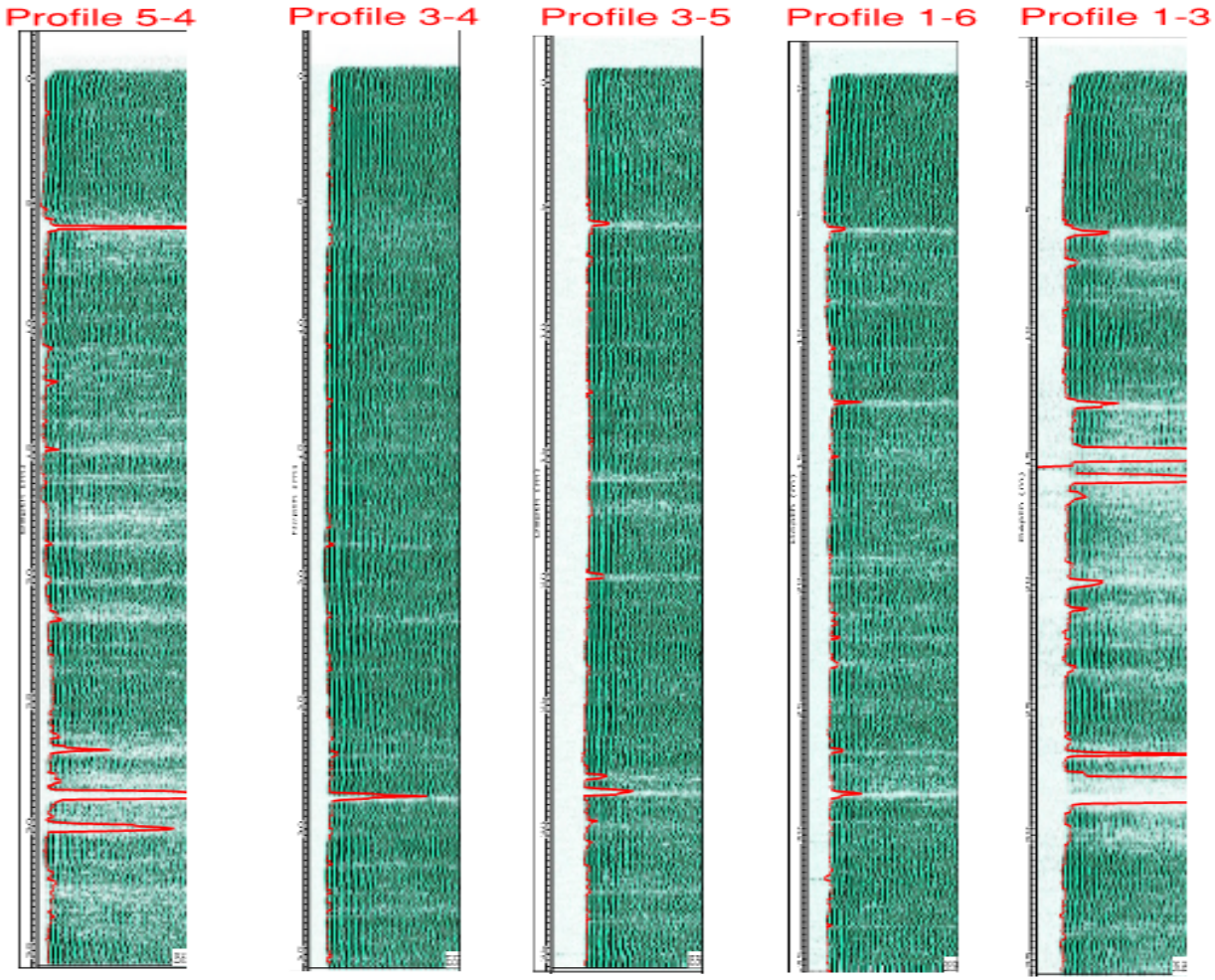
Cross Hole Sonic Testing – Sonic Tube layout with Test Profiles



SONIC LOGGING TUBES POSSIBLE LOCATIONS
(SONITEC-DEXTRA)

Cross Hole Sonic Testing



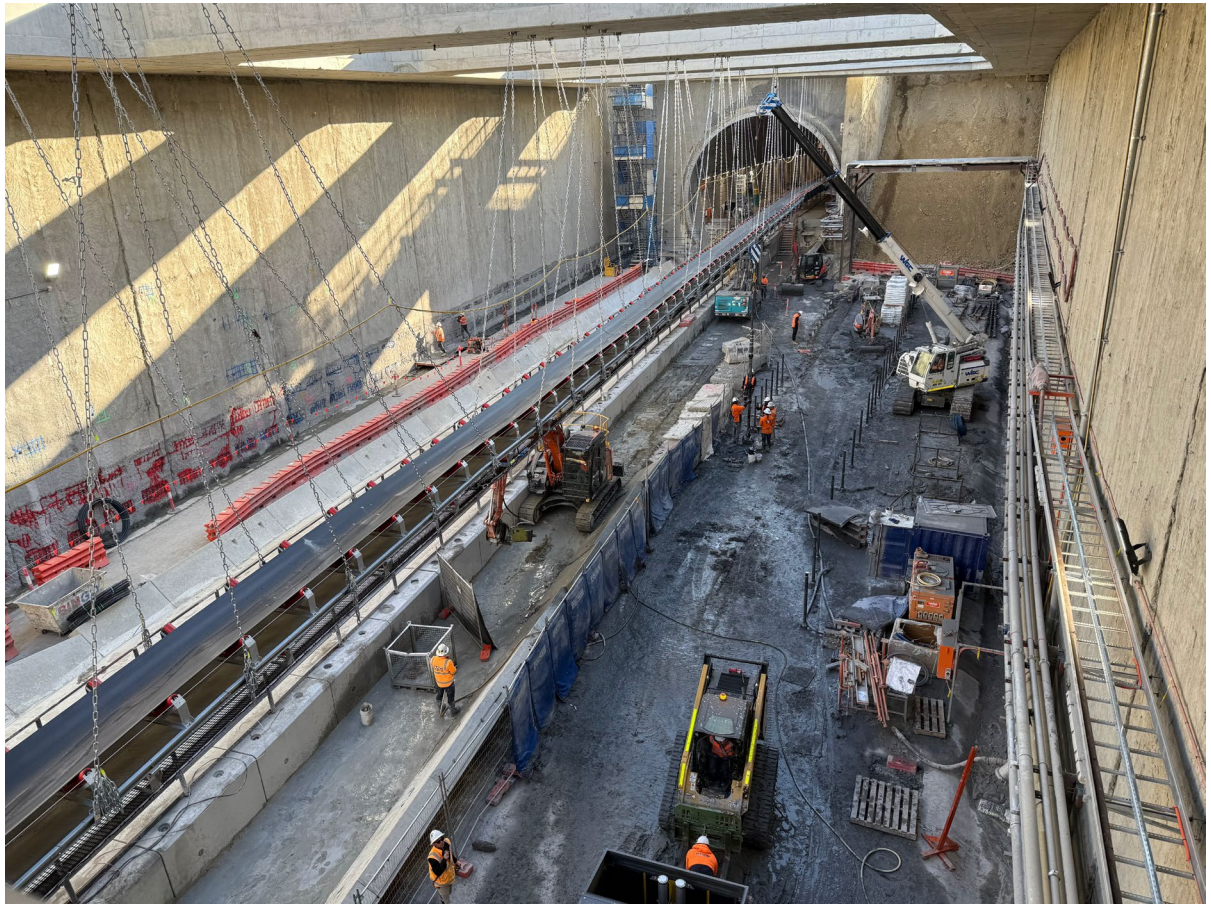


FAT delays and energy losses evident at coupler and splice location indicating either concrete flowability issues or signal issues due to reinforcement

Questions and Answers

Thank you for listening to my presentation!





DFI 50

Complex Logistical Constraints and Their Technical Impact on a Deep Excavation Design

Case Study from Ottawa Light Rail Project, Ontario, Canada

Prepared by

BRIERLEY ASSOCIATES
Creating Space Underground

In association with

KIEWIT EUROVIA VINCI

STAGE 2 ETAPE 2
CONFEDERATION LINE EXTENSION
OTTAWA, ONTARIO, CANADA

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Outline

- Project Overview
- Site Conditions
  - Site Constraints
  - Site Geology
- Shoring Design
  - Utilities
  - Watertight Transition
  - Bulkhead Removal



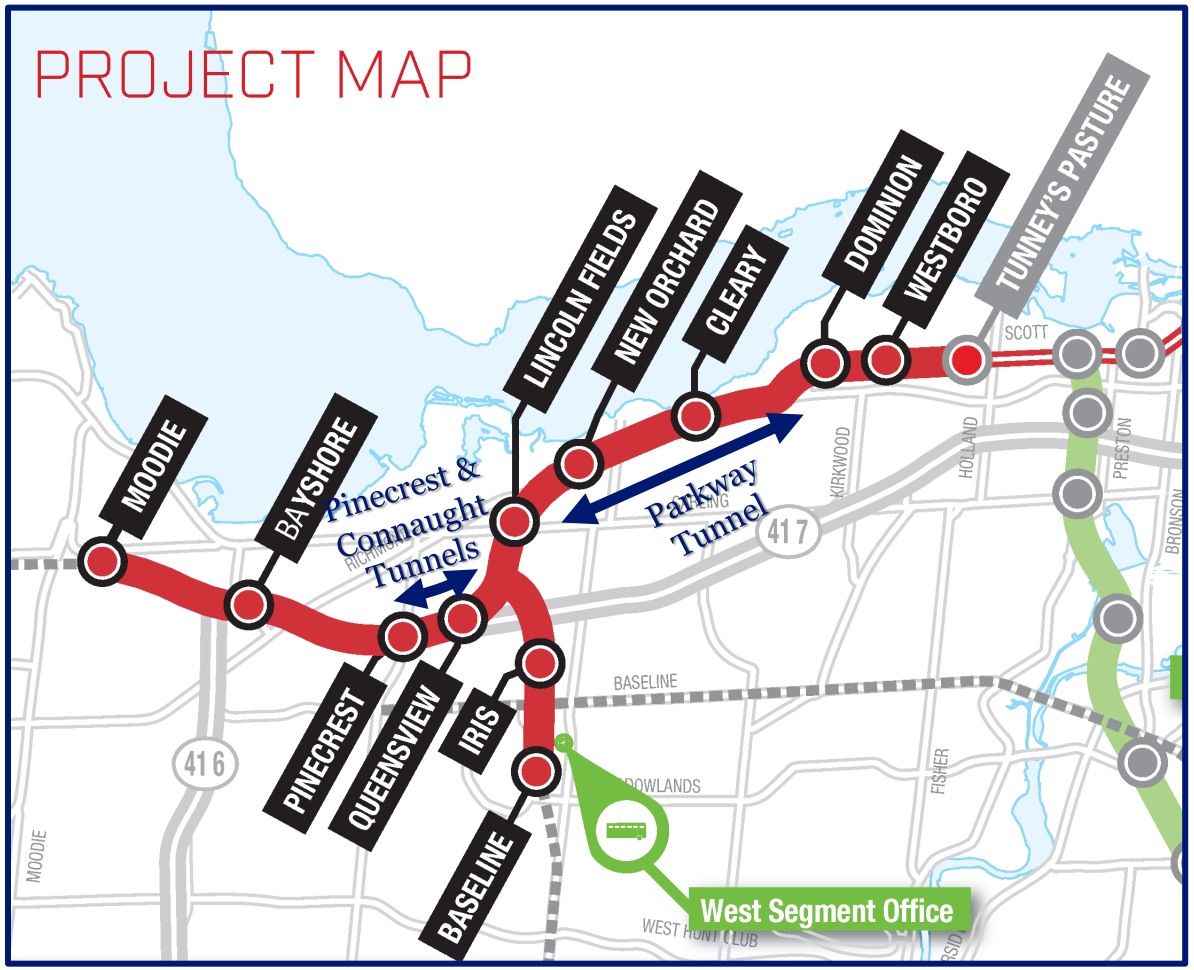
Project Overview



- Project: City of Ottawa Stage 2 O-Train Extension: Confederation Line Extension Project
- Owner: City of Ottawa
- Contract: Design-Build
- Contractor: Kiewit-Eurovia-Vinci Joint Venture
- SOE Designer (Zone 12 & 13a): Brierley Associates

Project Overview

- Confederation Line West
  - 15 km of new rail
  - 11 new Stations
  - 2 Cut & Cover Tunnel Segments
    - 2.9 km Parkway Tunnel
    - 0.6 km Connaught Tunnel
    - 1.0 km Pinecrest Tunnel



Project Overview

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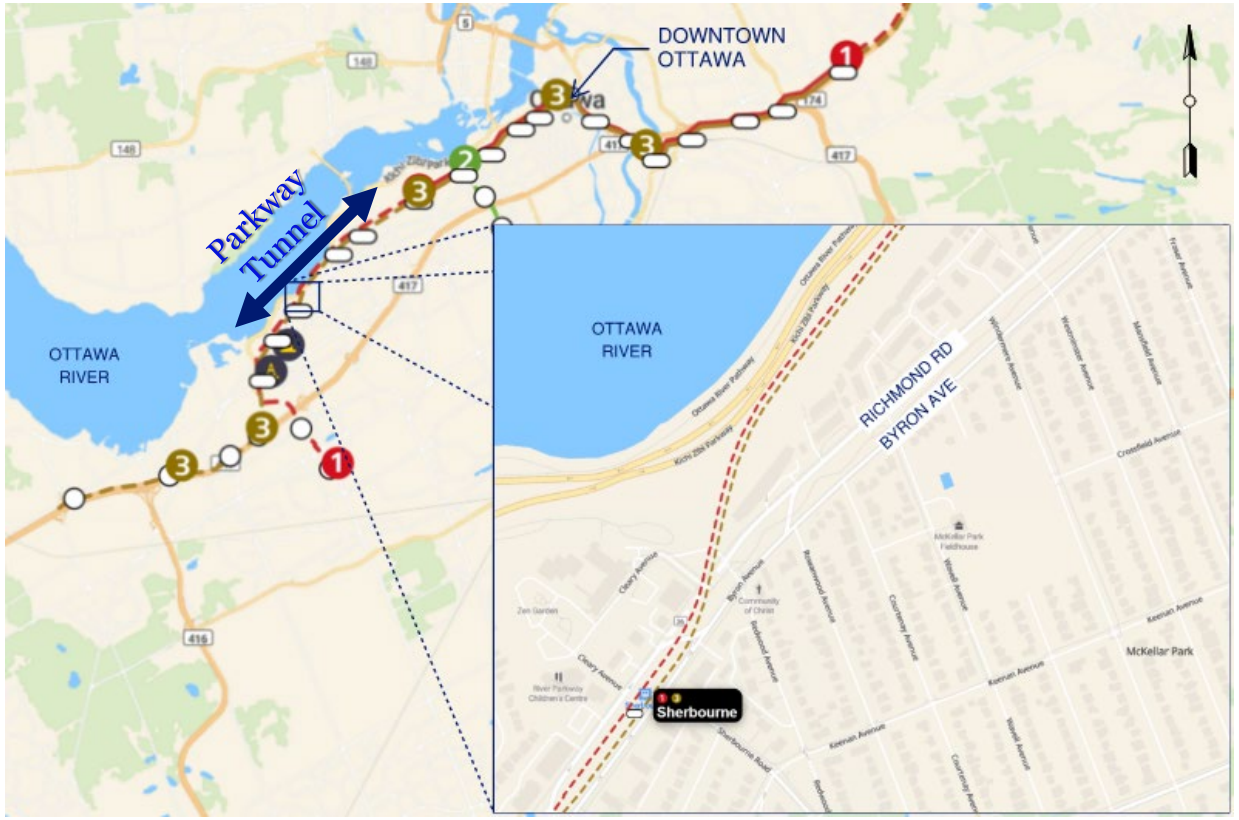
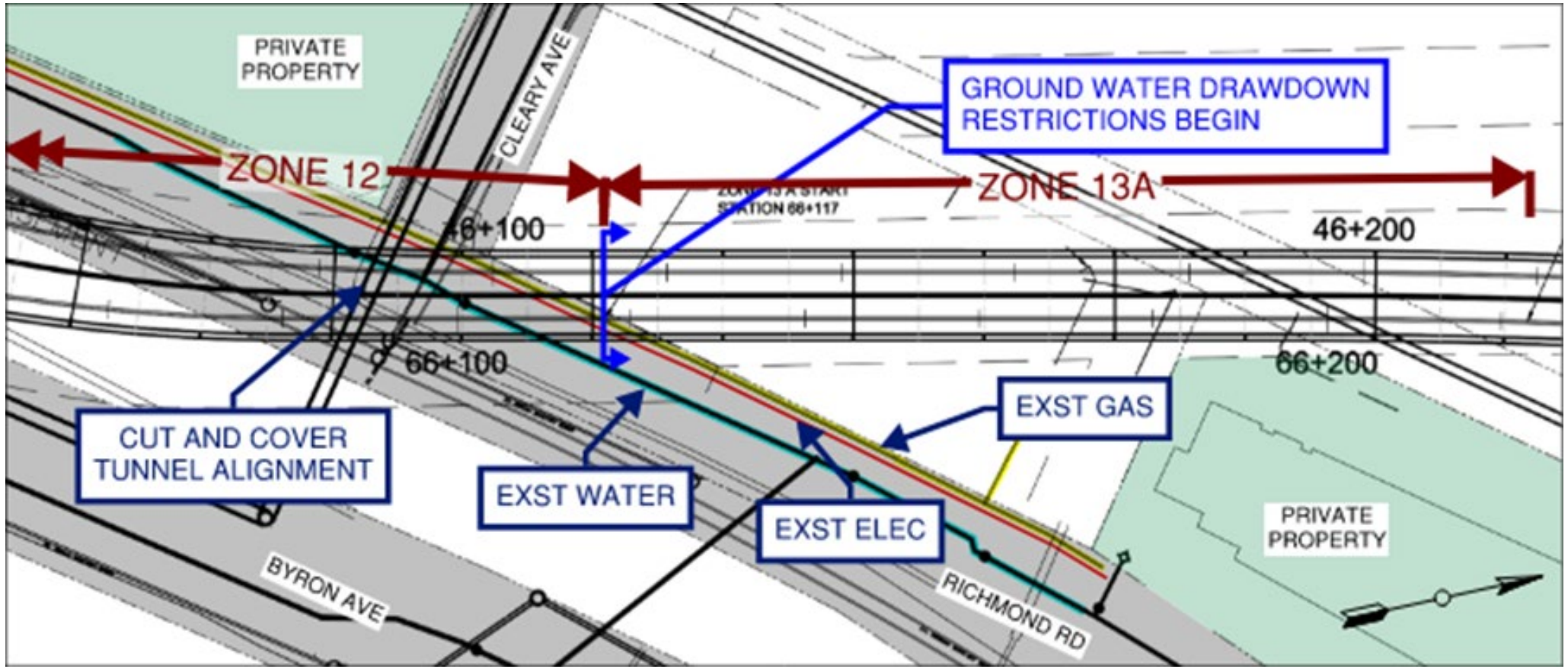
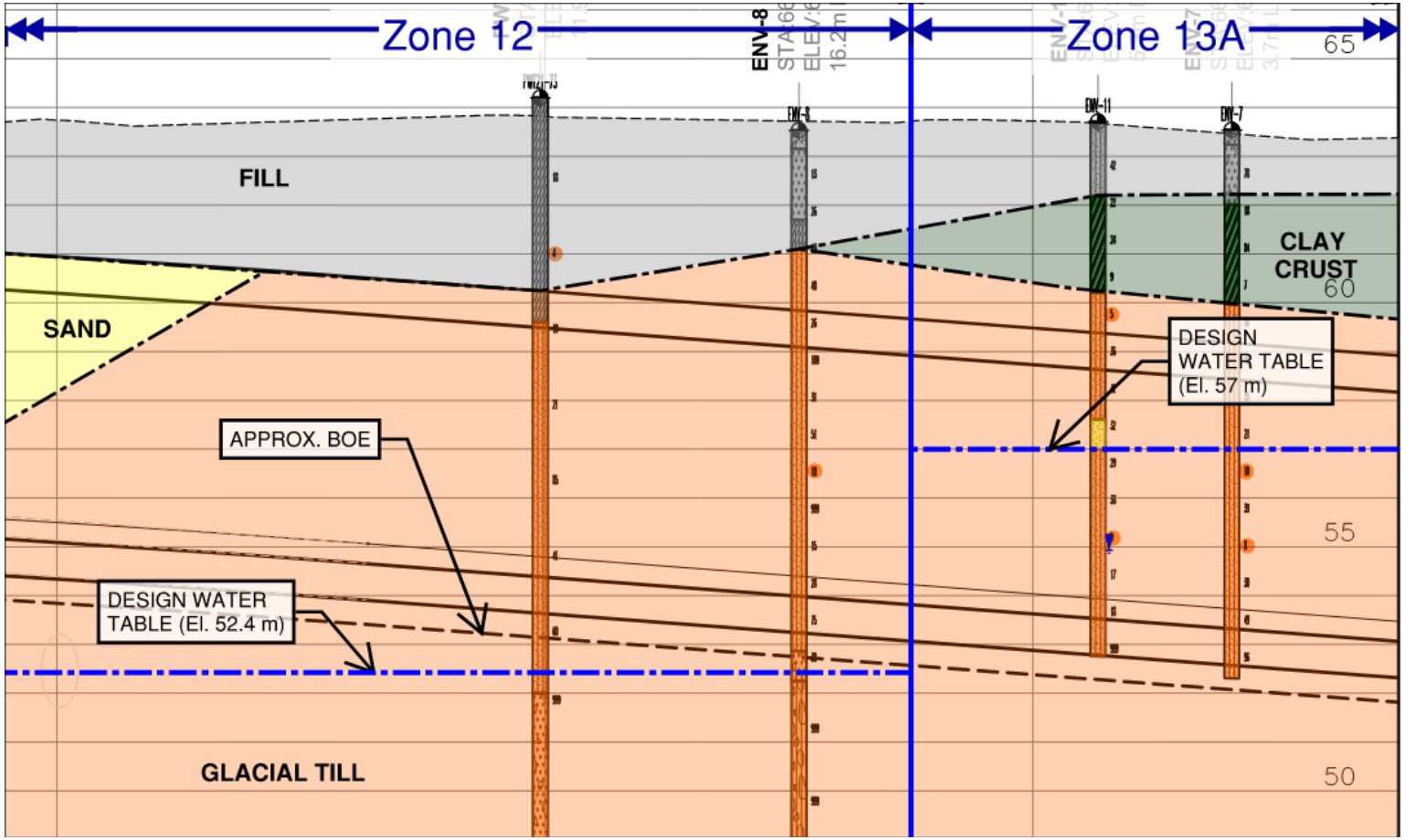


Zone 12 & 13A

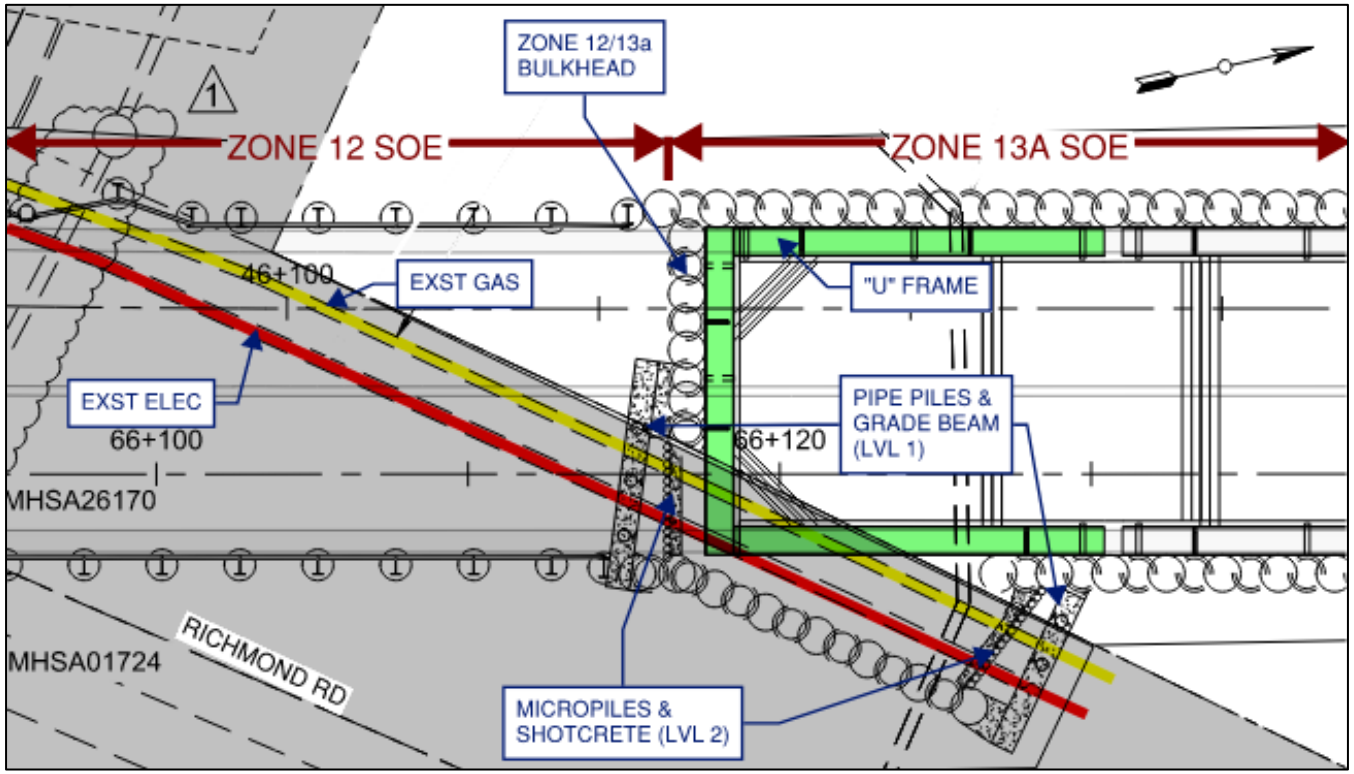
Site Constraints



Site Geology

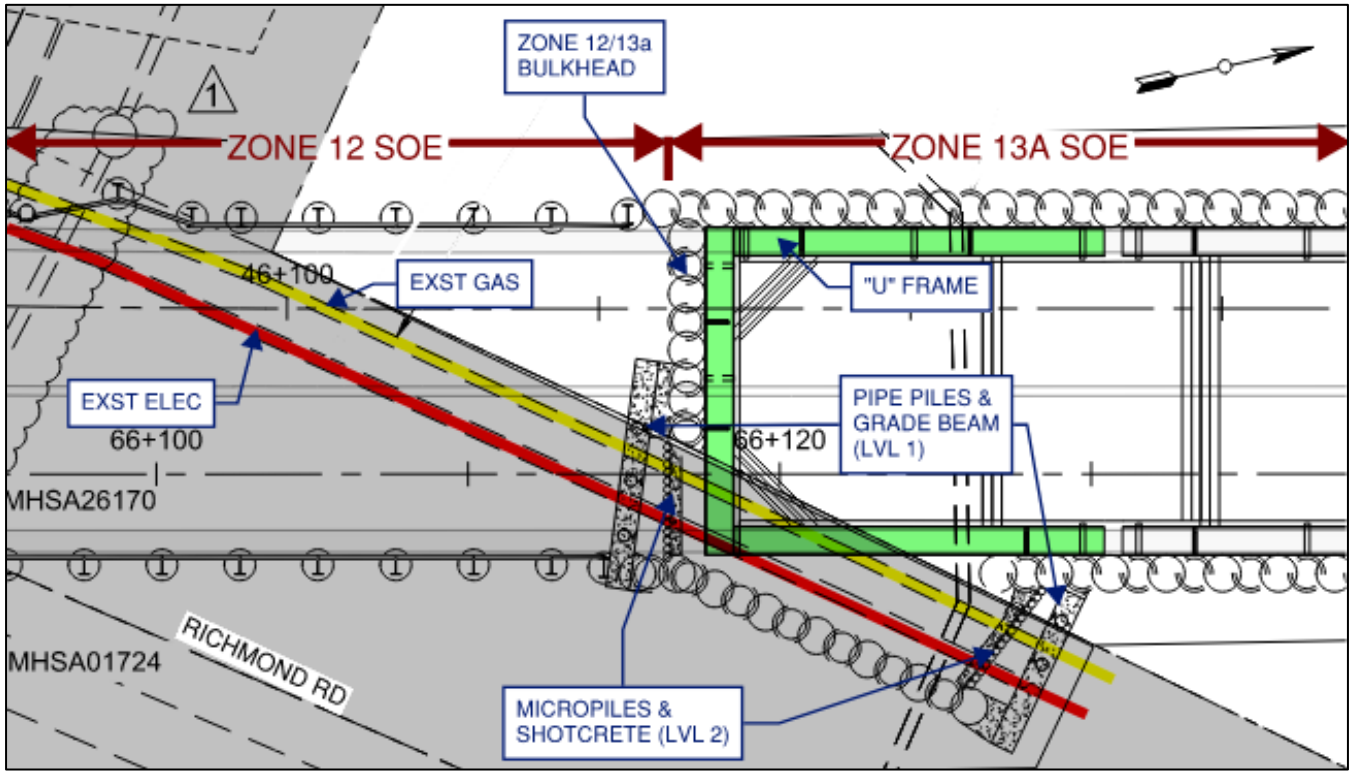


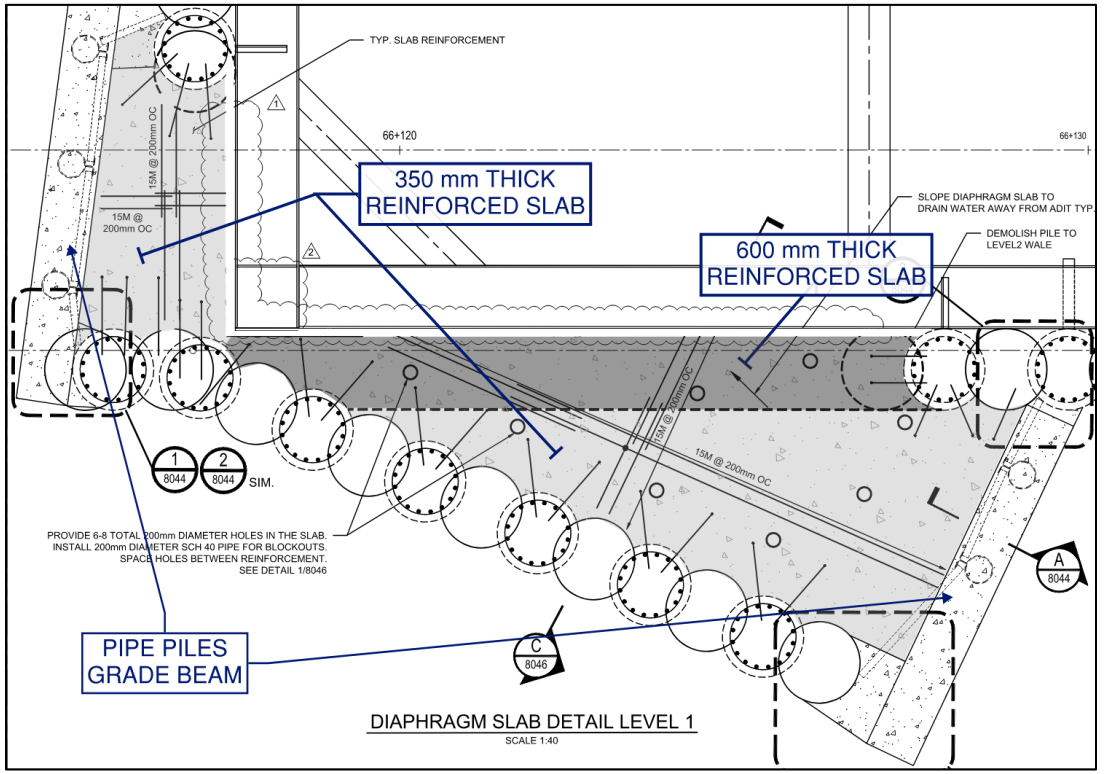
SOE Design



- Zone 13A
  - Secant Piles with internal bracing
- Zone 12
  - Soldier Pile and lagging with internal bracing

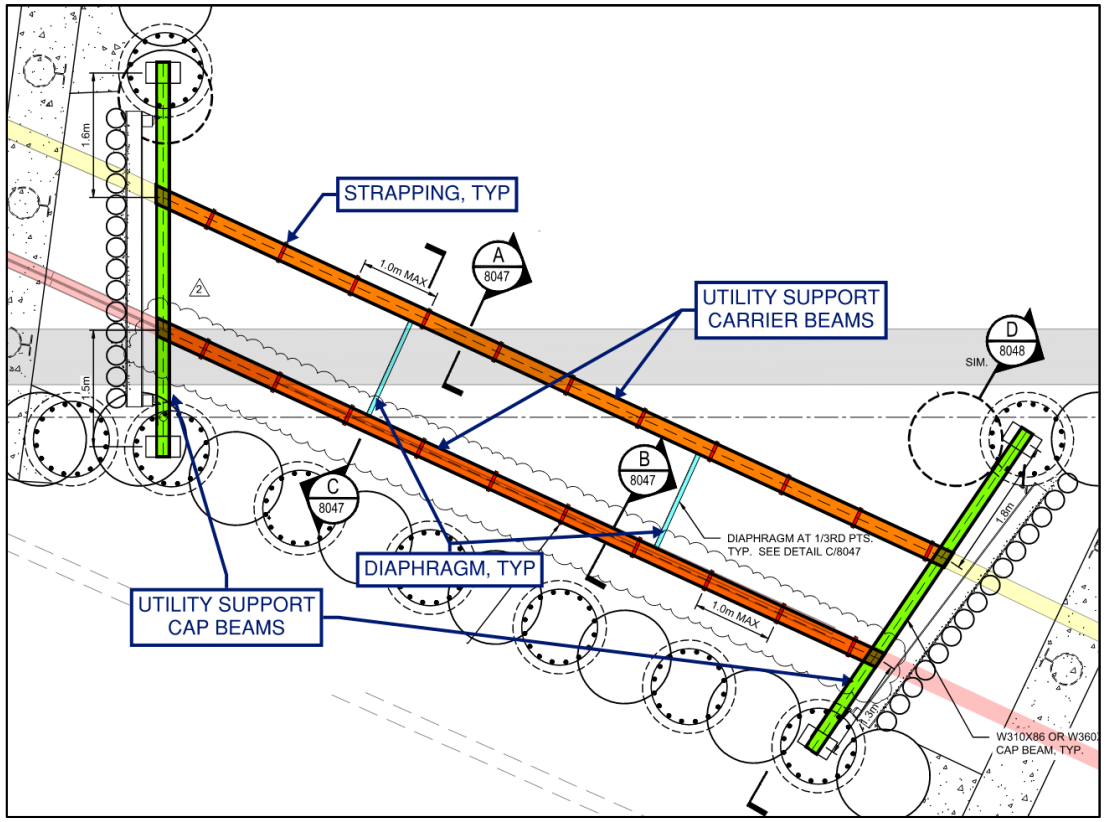
SOE Design

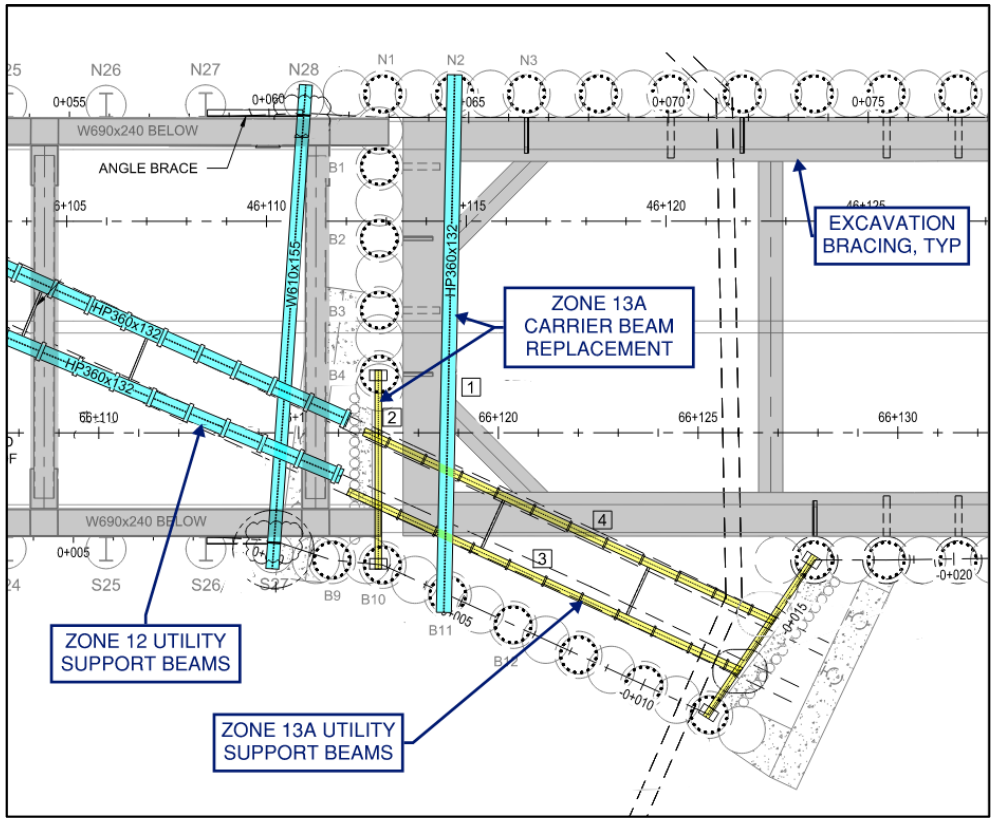

ZONE 12/13a BULKHEAD
ZONE 12 SOE
ZONE 13A SOE
46+100
EXST GAS
"U" FRAME
EXST ELEC
66+100
66+120
PIPE PILES & GRADE BEAM (LVL 1)
MHSA26170
MHSA01724
RICHMOND RD
MICROPILES & SHOTCRETE (LVL 2)


TYP. SLAB REINFORCEMENT
66+120
66+130
350 mm THICK REINFORCED SLAB
600 mm THICK REINFORCED SLAB
SLOPE DIAPHRAGM SLAB TO DRAIN WATER AWAY FROM ADIT TYP
DEMOLISH PILE TO LEVEL2 WALE
15M @ 200mm OC
SIM.
PROVIDE 6-8 TOTAL 200mm DIAMETER HOLES IN THE SLAB. INSTALL 200mm DIAMETER SCH 40 PIPE FOR BLOCKOUTS. SPACE HOLES BETWEEN REINFORCEMENT. SEE DETAIL 1/8046
PIPE PILES GRADE BEAM
DIAPHRAGM SLAB DETAIL LEVEL 1
SCALE 1:40

Additional SOE Logistics

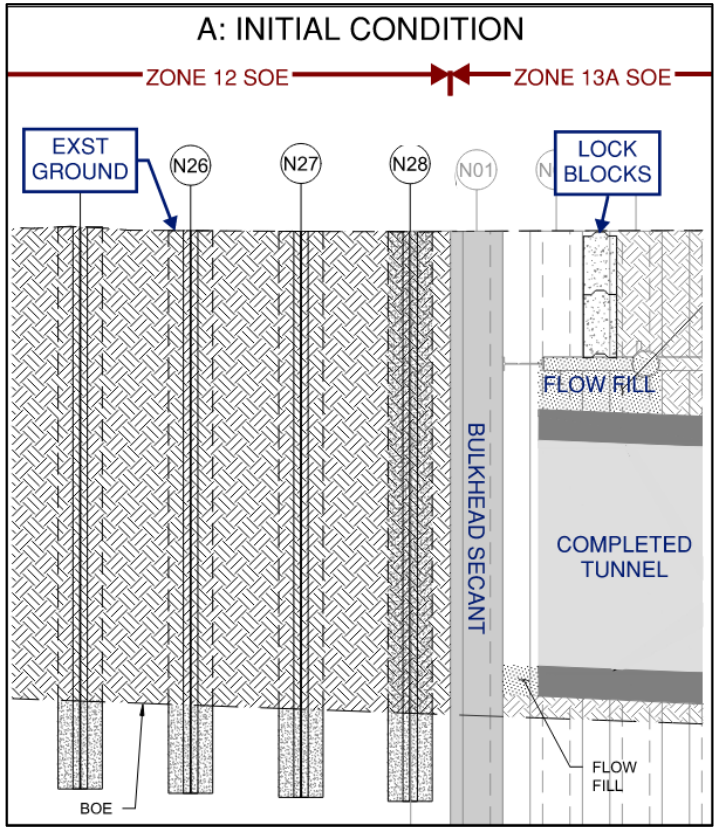
Utility Support

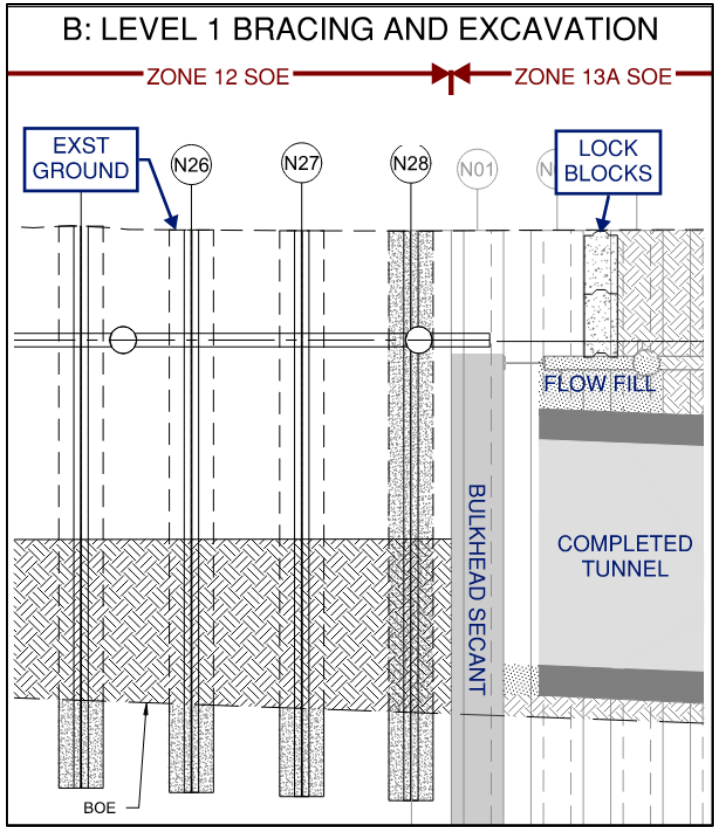


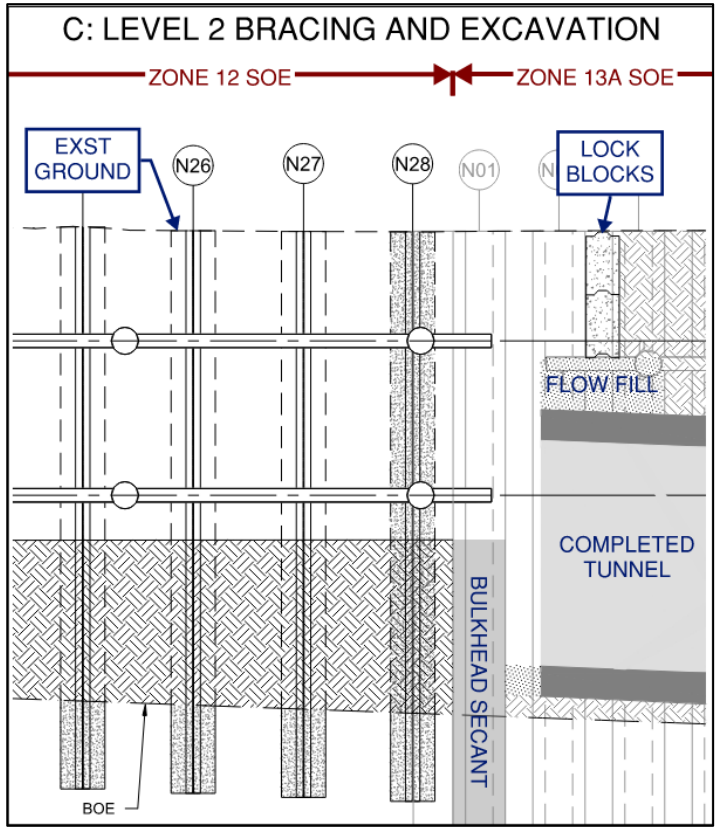


Additional SOE Logistics

Bulkhead Removal







Construction



Photo courtesy of KEV

Construction



Construction







Photos courtesy of KEV

Construction

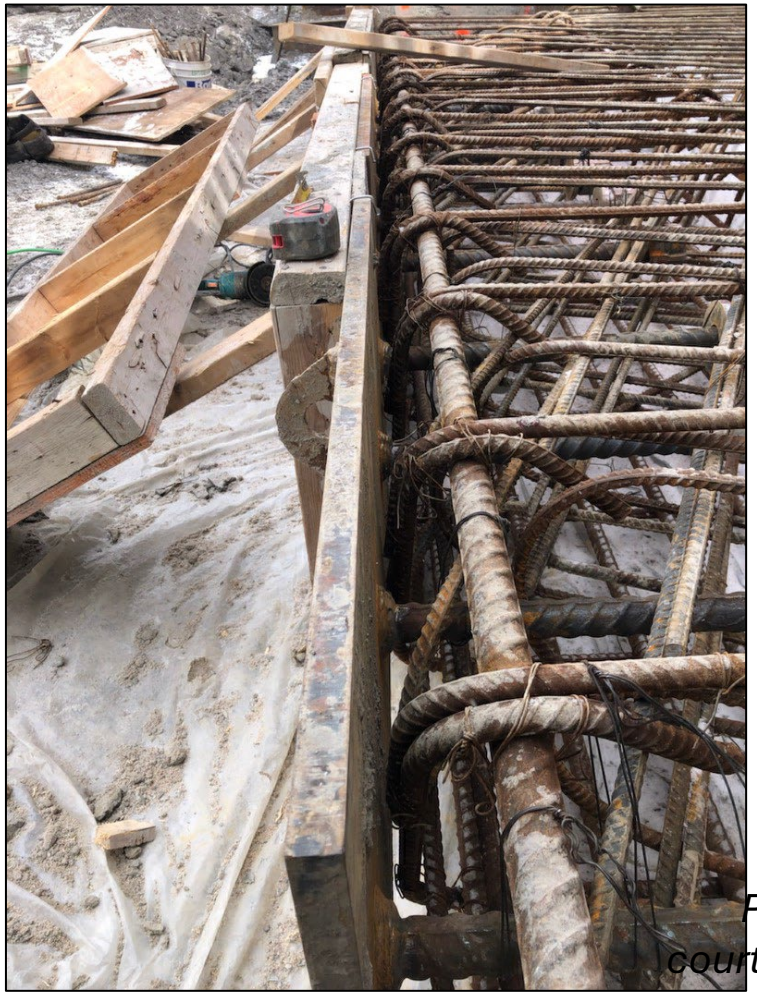


Photo courtesy of KEV

Construction







Photos courtesy of KEV

Construction



Photo courtesy of KEV

Construction



Photo courtesy of KEV

Construction



Construction



Photo courtesy of KEV

Construction



Construction





Construction



Photos courtesy of KEV

Construction

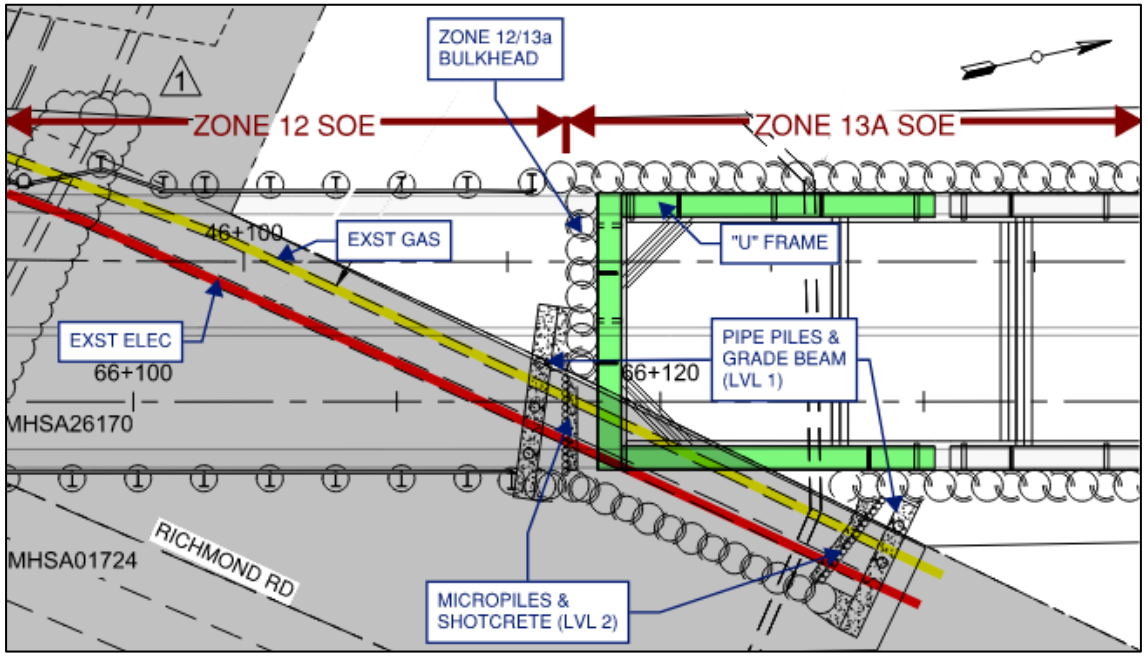


Construction



Construction









Questions?

STAGE 2 ETAPE 2

CONFEDERATION LINE EXTENSION
OTTAWA, ONTARIO, CANADA

PARALLEL SESSION B2
Optimizing for Gold



Deep Foundations | Shoring | Ground Improvement

Berkel and Company

Since 1959

Jake McKean, Morgan NeSmith

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Eli Lilly Lebanon Project

Eli Lilly

One of the World's Top Pharmaceutical Companies

Fluor Corporation

Fourtune 500 Engineering and Construction Company



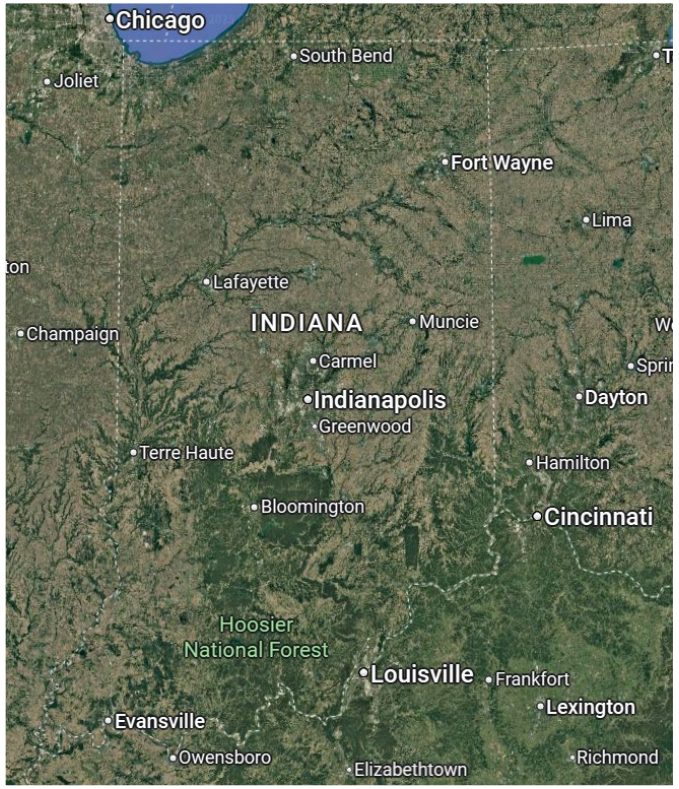
CONTENTS

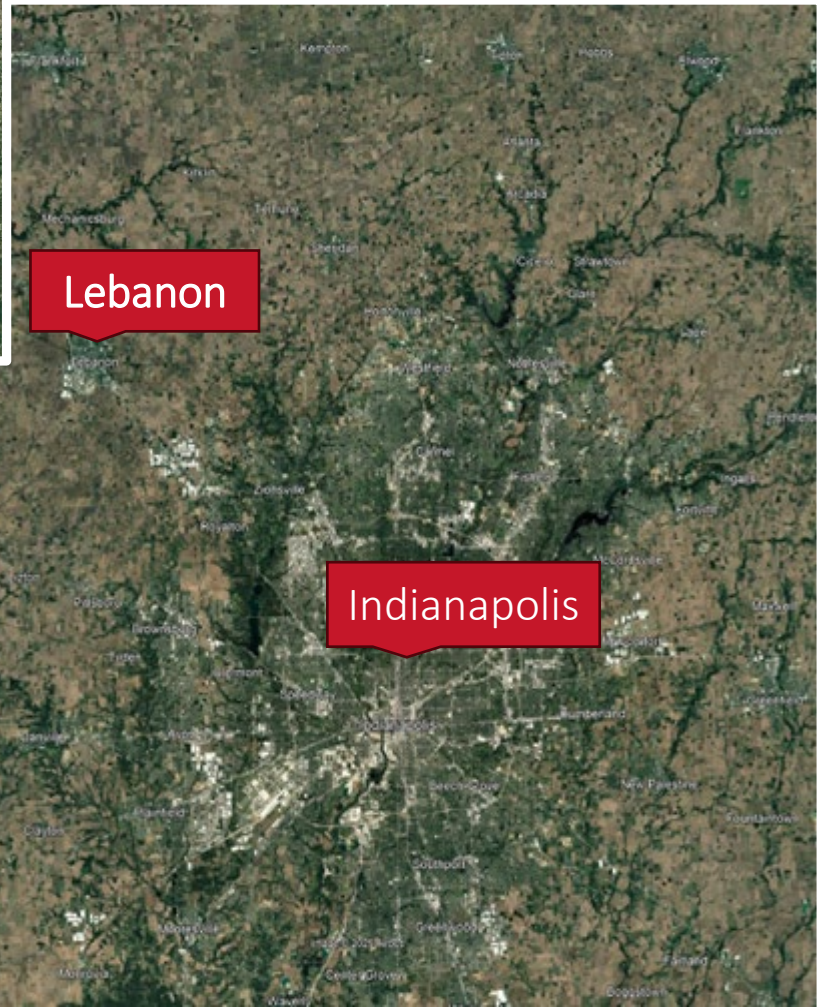
Region and Project Background

Geotechnical Info and Design

Operational Challenges and Logistics

PROJECT LOCATION: Lebanon IN











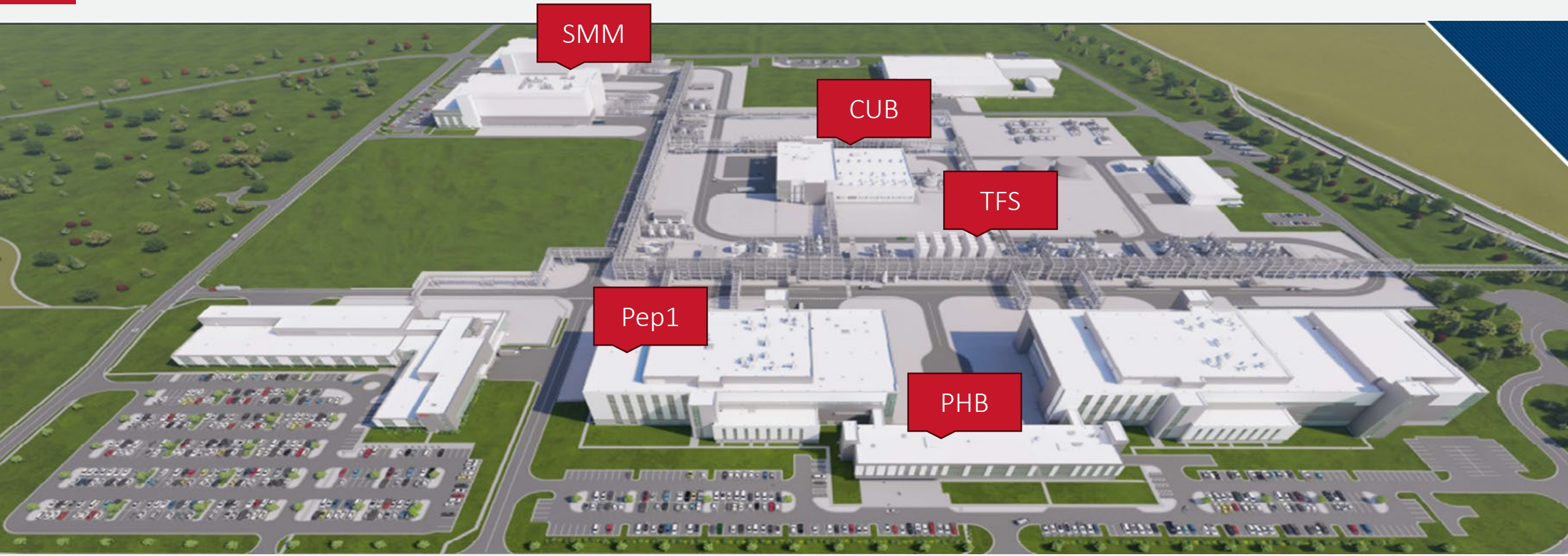
Size and Scale

Building areas range 3,200 sq ft to 330,000 sq ft
(Half Bridgestone Arena's total area)

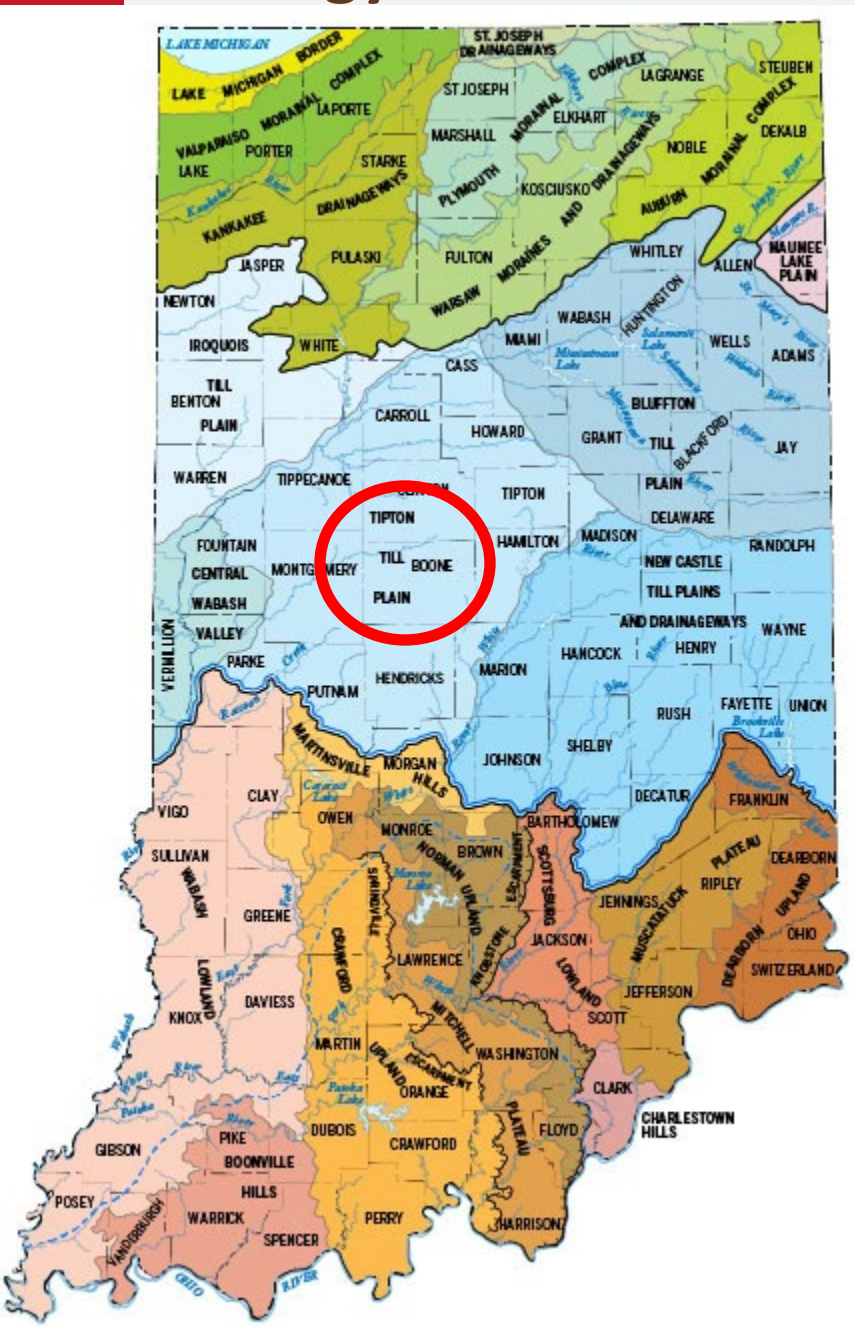
600 Acres – 15x Broadway Strips 1.6 million sq ft – 2X Nissan Stadium



Planned Final Configuration



Geology



Tipton Till Plain Section

of the Central Till Plain physiographic region of Indiana.

Predominant Soil Type: silt or silty clay loams (Crosby-Treaty)

Parent Materials: silty materials or loess over loamy till relatively high clay content (consistent with the glacial deposition history of the region)

Underlying Rock:

Devonian-Mississippian aged New Albany Shale formation

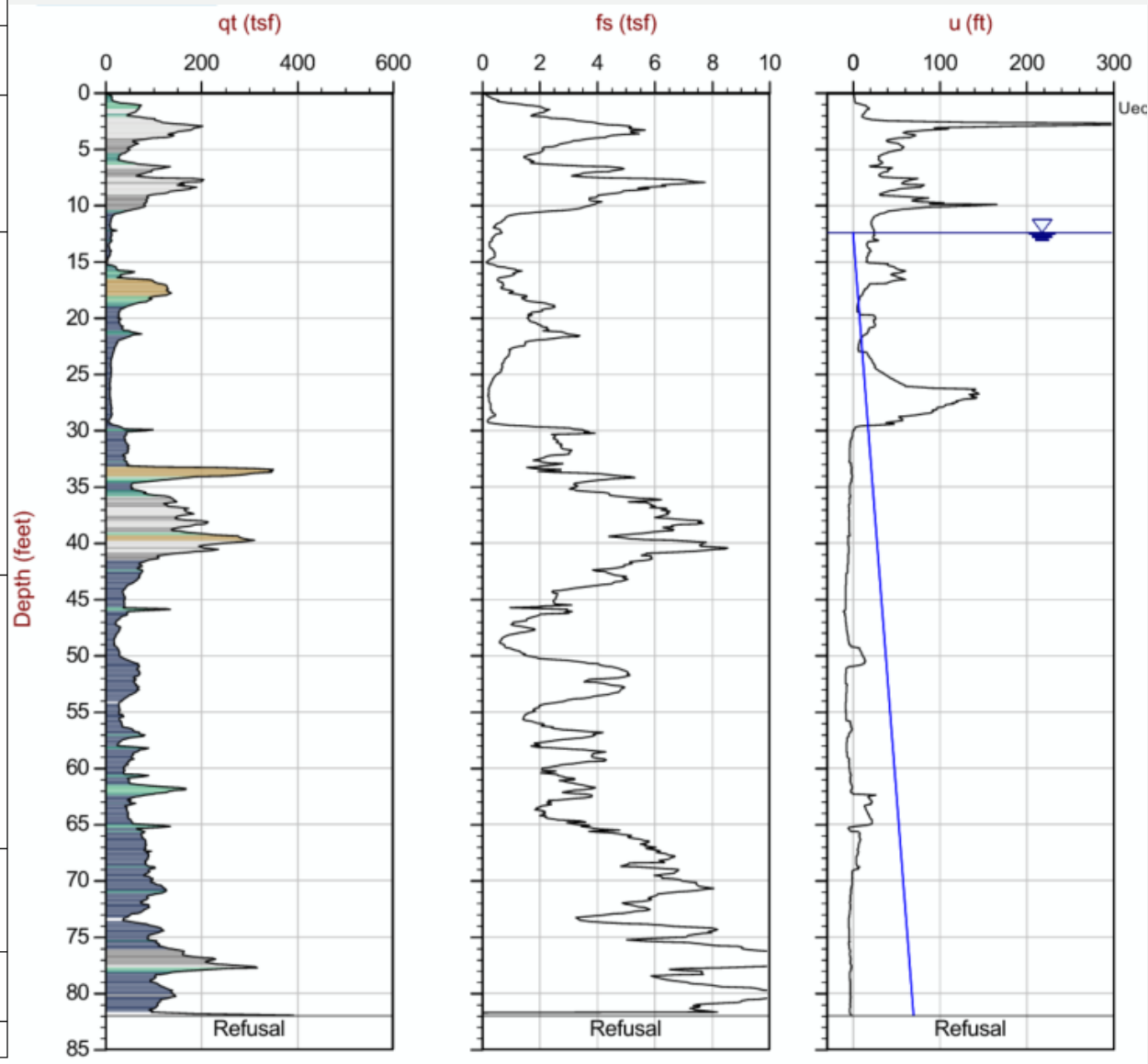
organic-rich black shale with minor amounts of shale and dolostone

Local Bedrock Level: EL 700 – EL 750 ft

Site surface elevation: ~ EL 915 – EL 920 ft

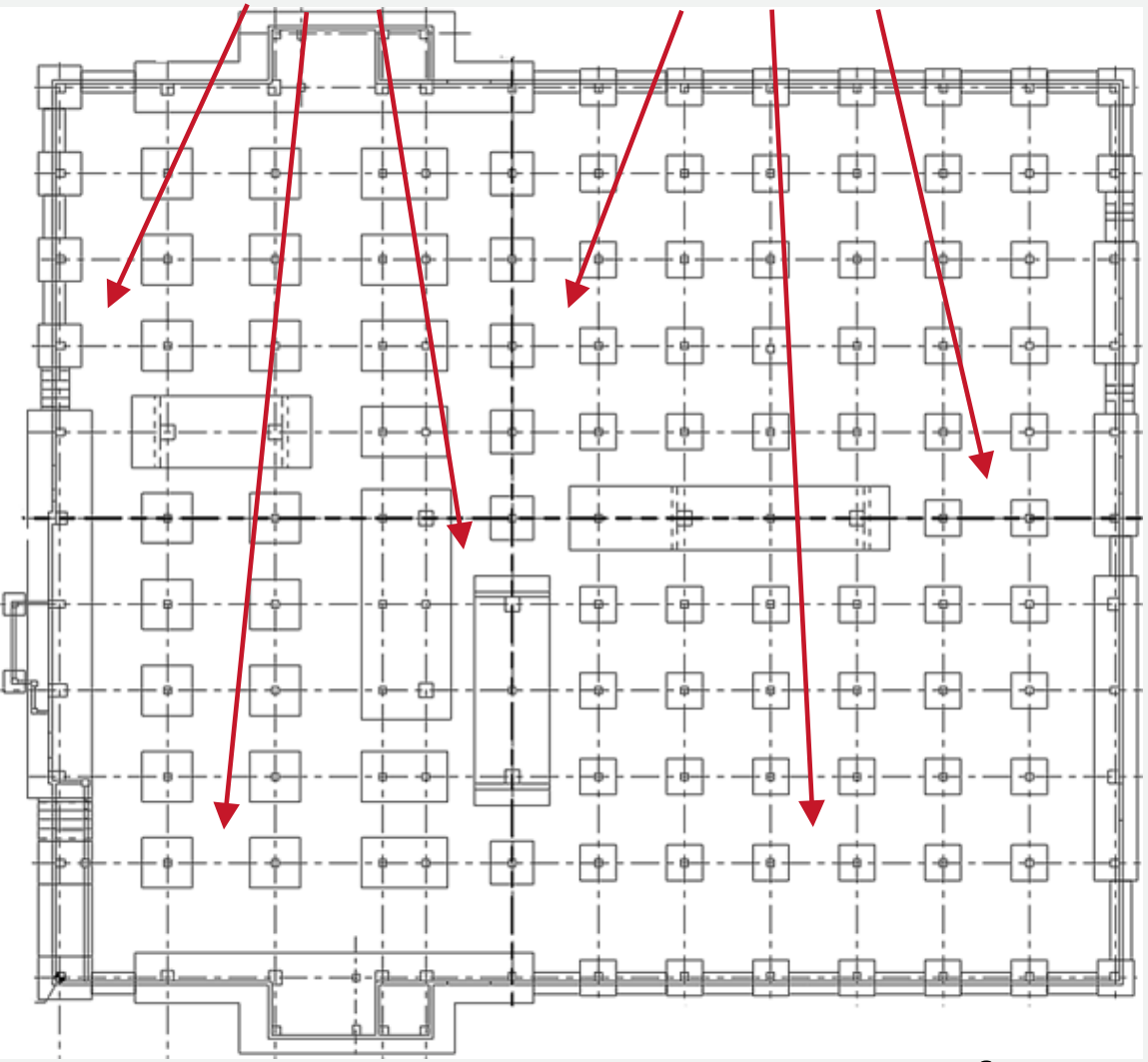
Geotechnical Profile

Elevation [ft]	
925 to 928 (Final Grade)	ENGINEERED FILL: primarily on-site natural soils processed and compacted to meet project specifications.
915 to 920	CULTIVATED ZONE: (historic and active row cropping agricultural activities). Typically, dark brown lean clay containing variable amounts of sand, root hairs, and organic content (1.2 percent to 2.7 percent). **Removed** prior to the placement of the engineered fill mentioned above.
914 to 917	GLACIAL TILL: highly weathered, in soft or loose (to occasionally hard) conditions indicative of intense weathering processes, including freeze-thaw cycles, oxidation, and biological activity, resulting in reduced strength and increased compressibility compared to deeper materials. Low plasticity lean clays with varying amounts of sand and sandy lean clays (CL). Standard Penetration Test (SPT) N-values ranged from 0 to 80 blows per foot (bpf), reflecting the heterogeneous nature of glacial deposits. Natural moisture contents ranged from 8.1 percent to 26.2 percent, with the higher moisture contents generally corresponding to the softer consistency materials.
890	GLACIAL TILL: moderately weathered, firm/stiff or medium dense. Transition between the highly weathered near-surface materials and the less weathered deeper deposits with properties ranging from those described above and below. Low plasticity sandy silty clays (CL-ML). SPT N-values ranged from 3 to 60 bpf, indicating soft to hard consistency, though generally exhibiting greater stiffness than the upper till materials. Natural moisture contents ranging from 9.0 percent to 13.2 percent, consistent with their greater density and lower void ratio.
870	GLACIAL TILL: stiff/hard or dense/very dense, with generally increasing stiffness and density with depth reflecting the reduced weathering and larger overburden pressure.
850	GLACIAL TILL: very hard, minimally weathered with significant overconsolidation
800	Approximate Extent of Exploration



400 psf
Floor Pressure

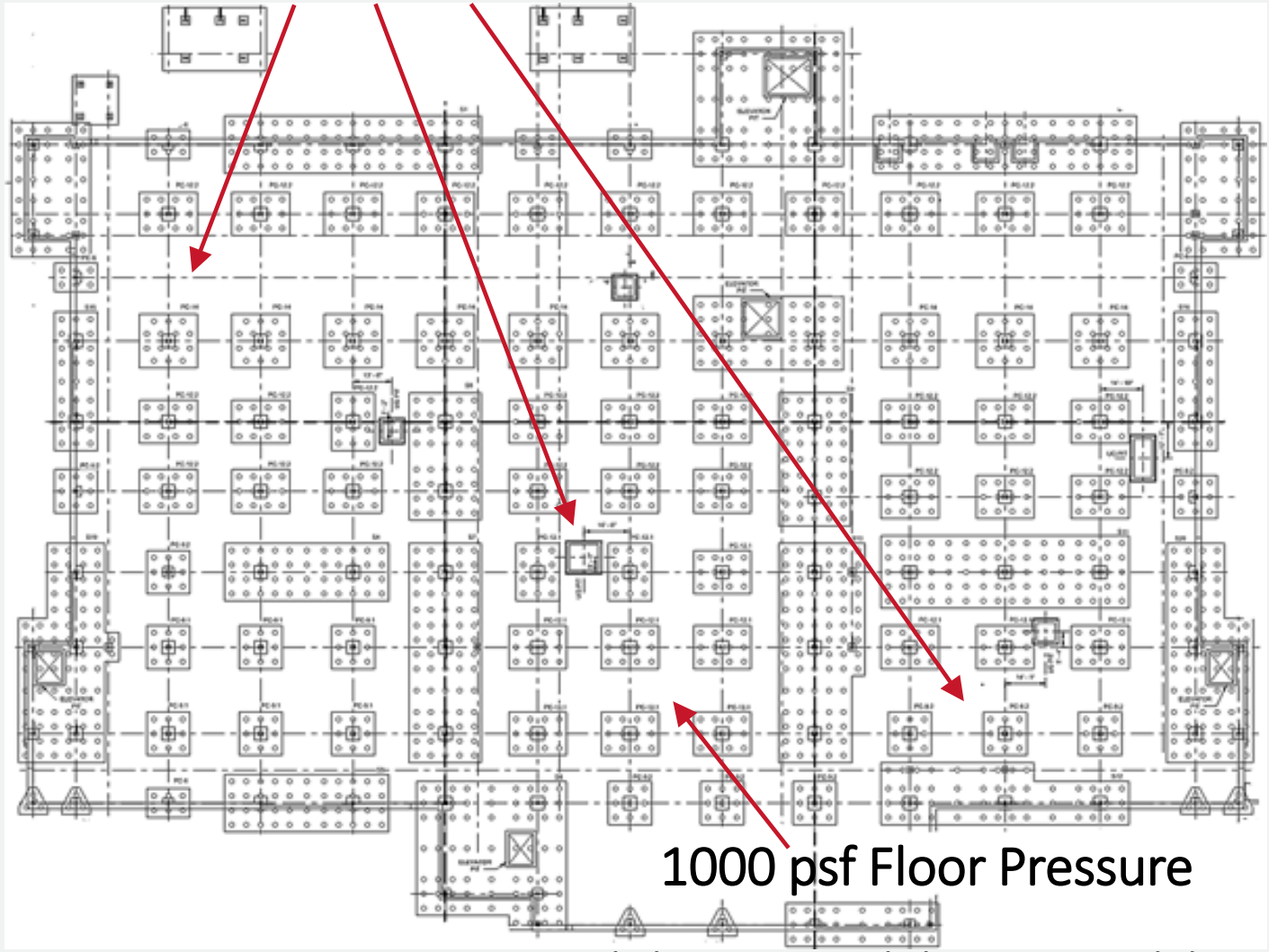
1000 to 2000 psf
Floor Pressure



Footing Pressure: 5,000 to 6,500 psf

CUB

2000 psf
Floor Pressure

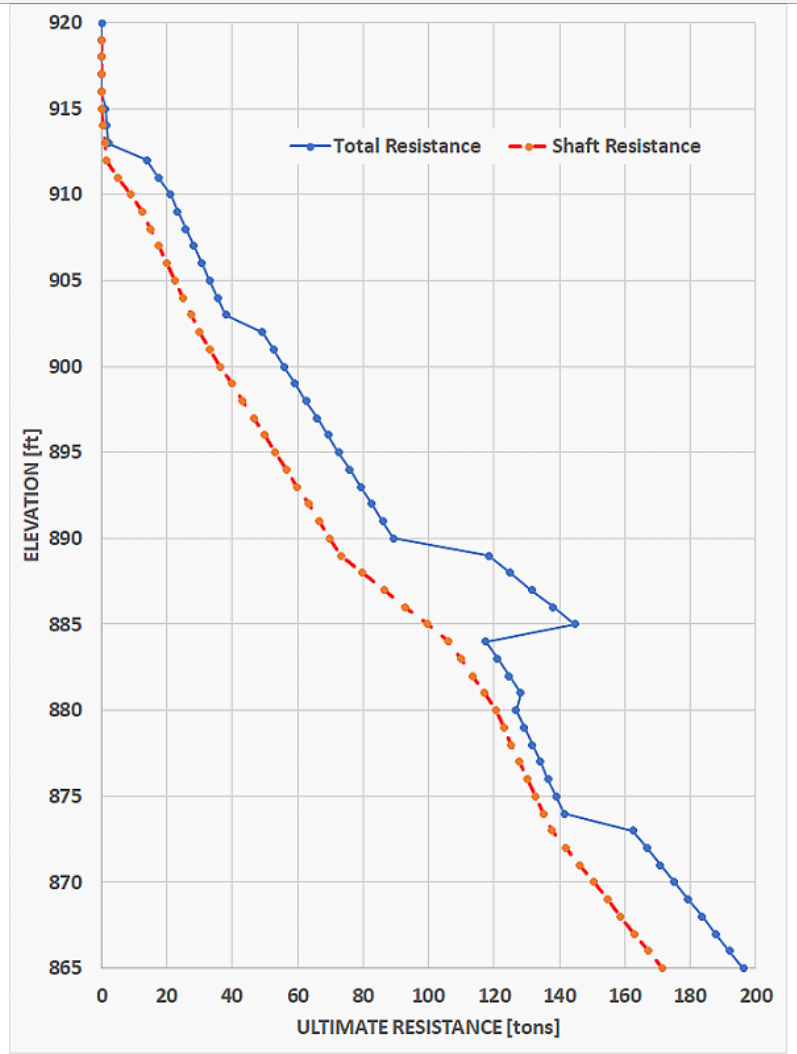


Column Piles: 210 tons (C), 15 tons (T), 11 tons (L)

PEPTIDE I and II

Geotechnical Design - Axial

14-in Diameter APG Rigid Inclusions
CUB, Peptide I and II, Small Molecule

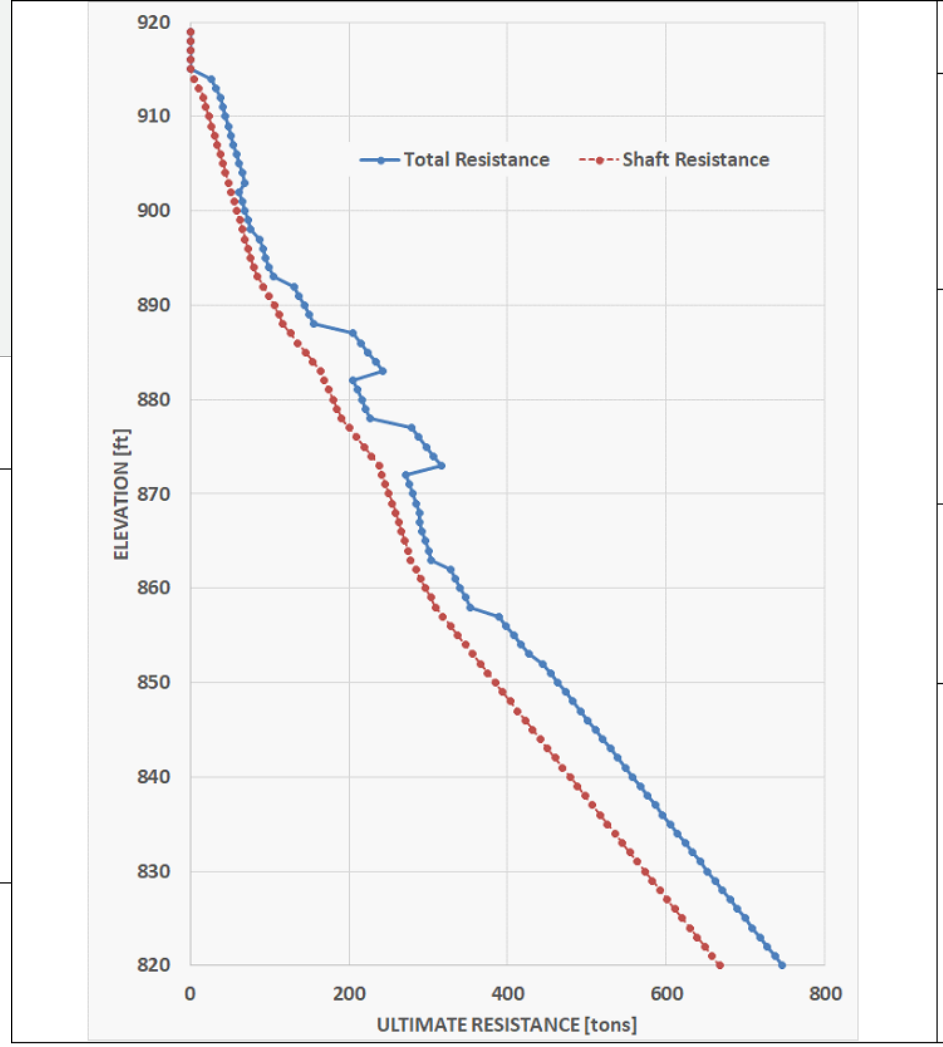


ENGINEERED FILL

WEATHERED GLACIAL TILL

INTERMEDIATE GLACIAL TILL

STIFF/HARD GLACIAL TILL



ENGINEERED FILL

WEATHERED GLACIAL TILL

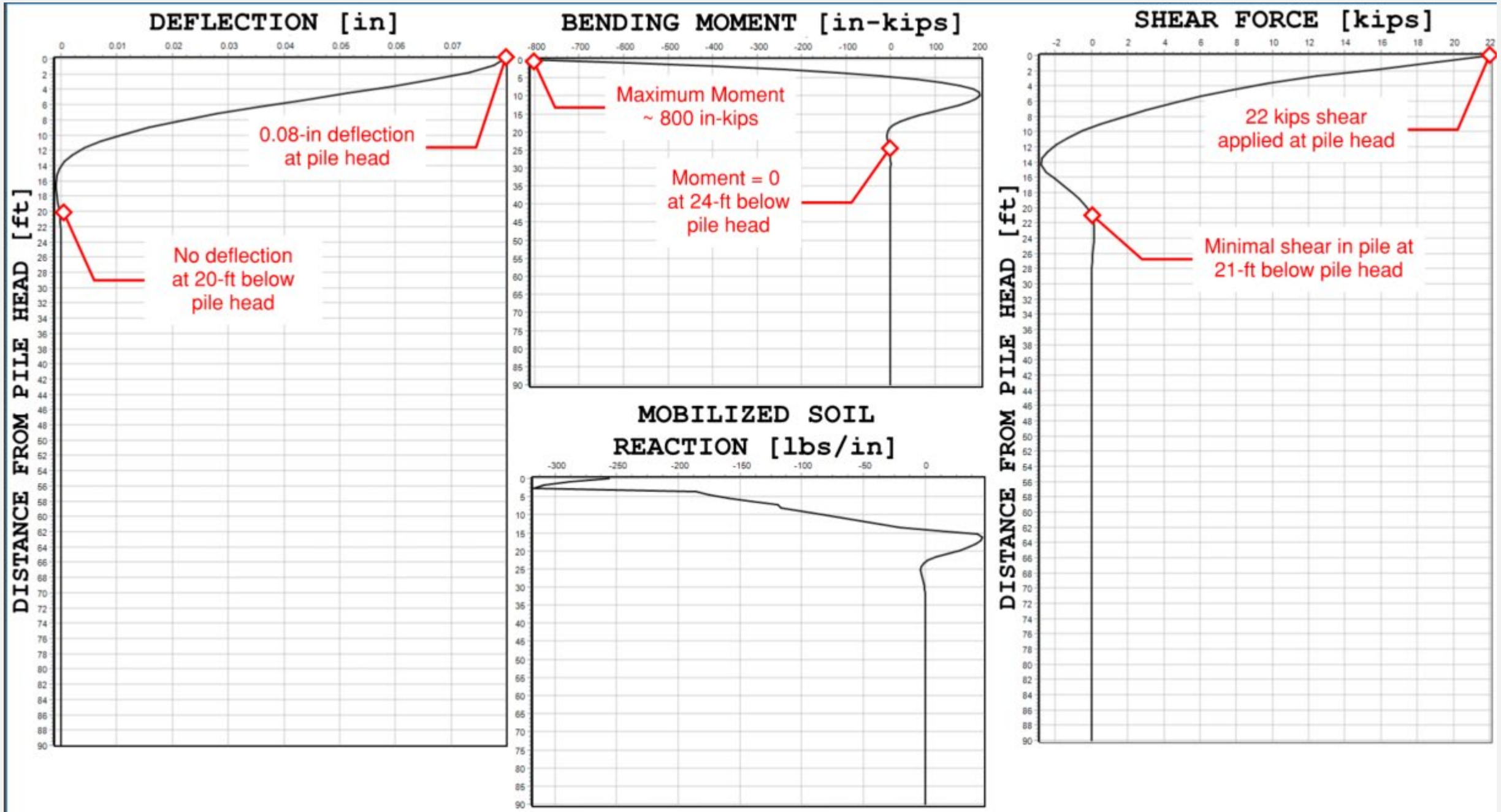
INTERMEDIATE GLACIAL TILL

STIFF/HARD GLACIAL TILL

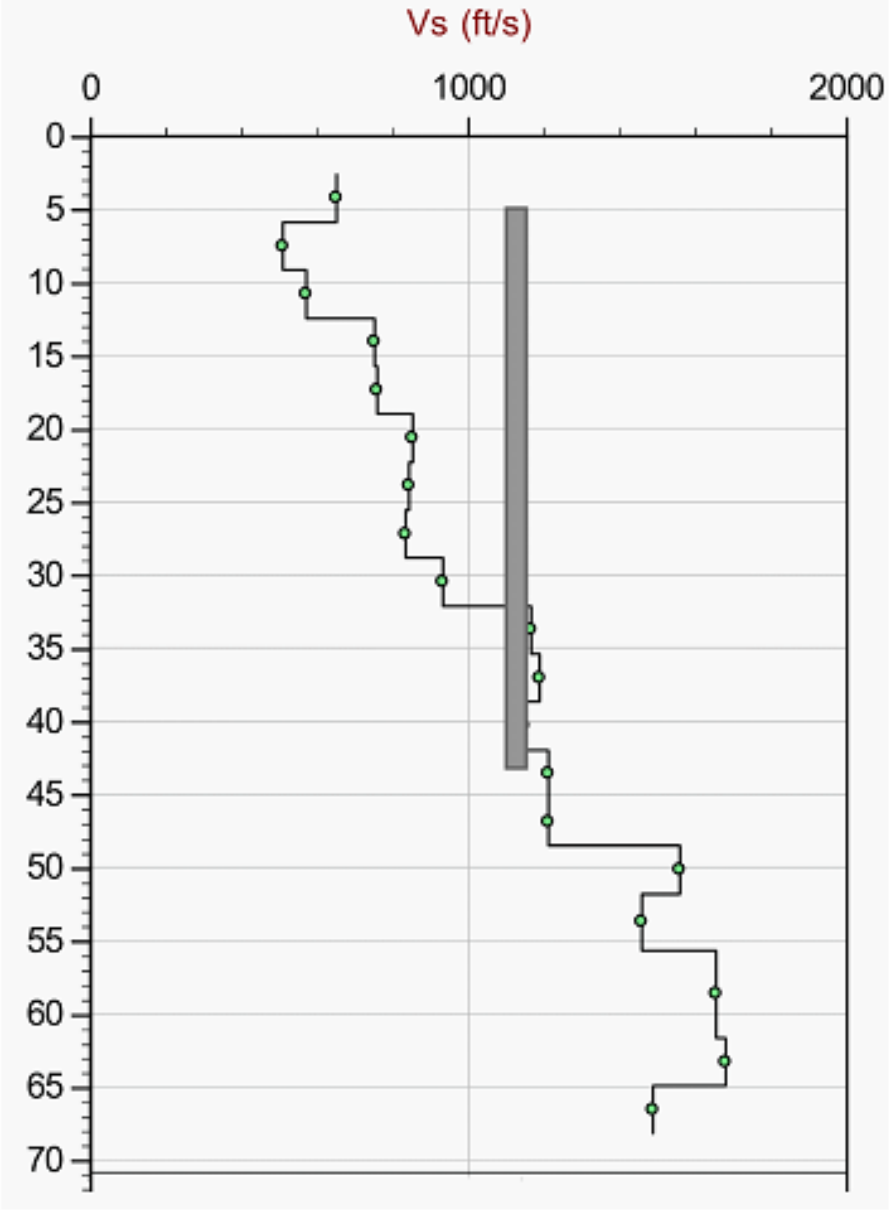
HARD GLACIAL TILL

20-in Diameter APG Piles
Peptide I and II, Small Molecule

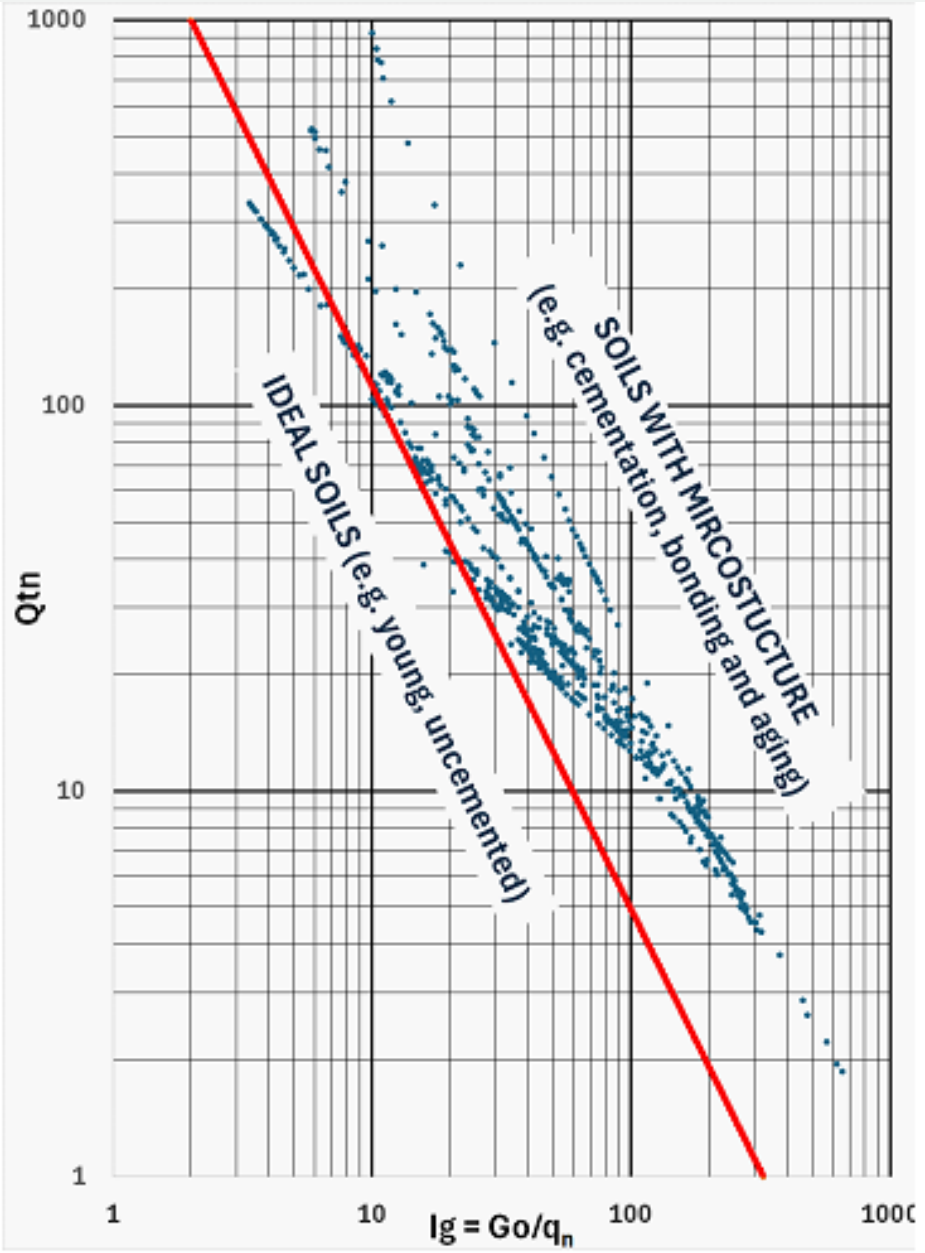
Geotechnical Design - Lateral



Geotechnical Design - Settlement



	E' [psf]	
EL [ft]	Vs	SPT N60
919	1,568,131	401,067
911	1,568,131	401,067
910	1,615,647	103,619
905	1,615,647	103,619
904	2,384,332	205,576
895	2,384,332	205,576
894	2,930,909	301,333
888	2,930,909	301,333
887	5,328,253	721,778
882	5,328,253	721,778
881	5,152,125	473,600
872	5,152,125	473,600
871	9,237,119	876,445
865	9,237,119	876,445
864	9,472,450	1,020,121
855	9,472,450	1,020,121
854	11,331,788	1,041,778
820	11,331,788	1,041,778

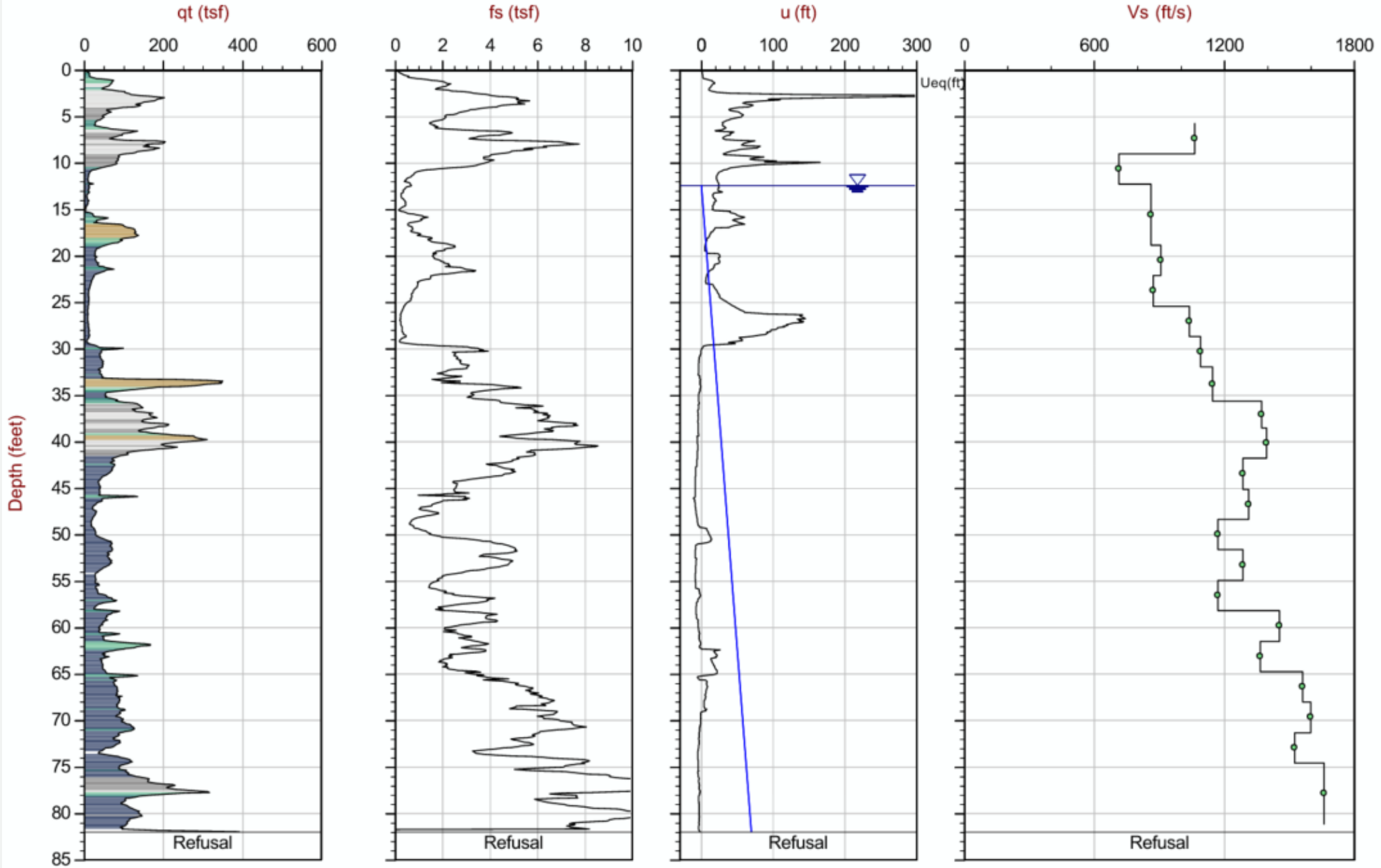


CONETEC

S&ME

Job No: 23-61-26794
Date: 2023-11-01 12:21
Site: Lebanon Plant 1, Lebanon, IN

Sounding: SCPT-2-C-91-PEPI
Cone: 661:T1500F15U35



Abort Pile

Mode: **Grout Head**
Pile ID: **241**
Pile Type: **Tension**

Signal Pump

Grout Pump: **ON**
0.00 cfm
Stroke Counter : **0**

Pump Off

Initial Head Vol 0.0% 0.0%

Grout Pressure 2 63.3% 63.3%

Freeze Depth: ☐

Current: **50.04'**

Max Depth: **50.04'**
Target Depth: **60.00'**
Return Depth: **0.00'**
C.I.E.: **507.5**

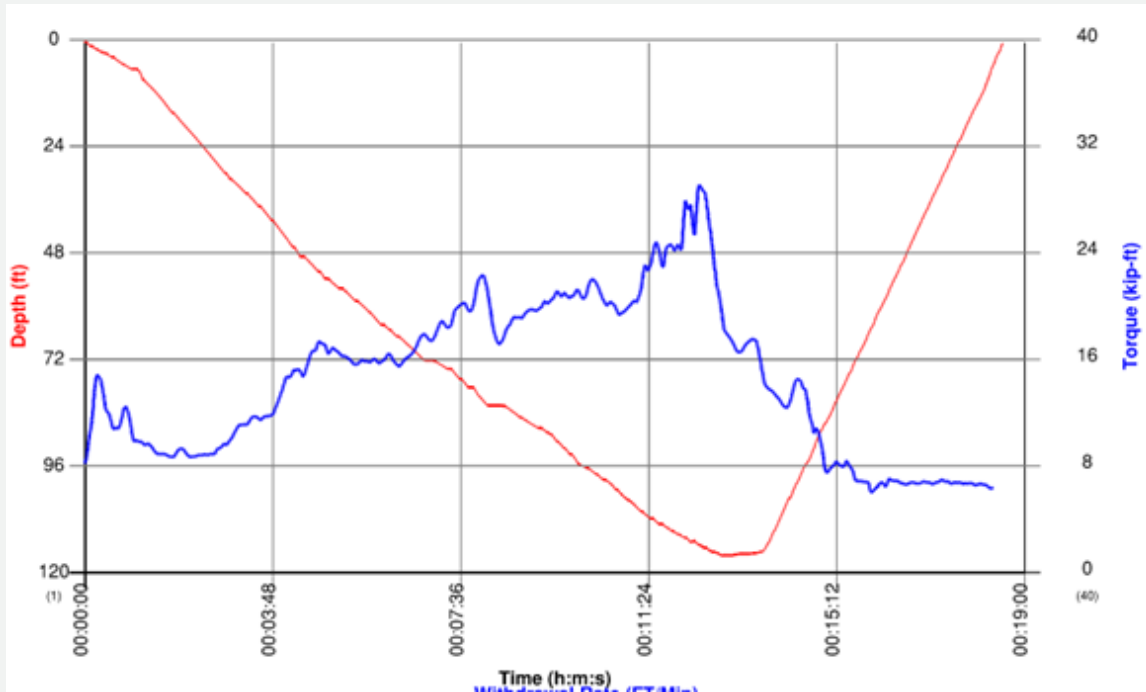
Total GF: **0.000**
Last Inc GF: **0.000**
Total Vol: **0.00 ft³**

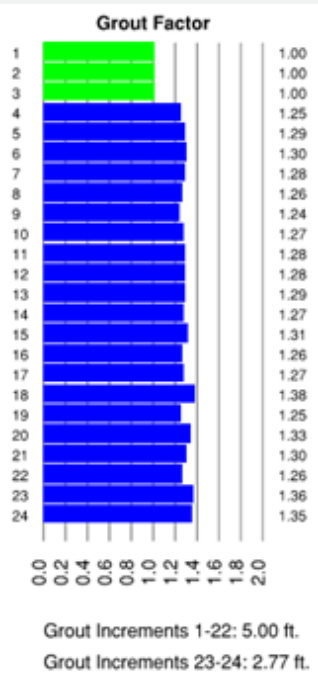
Torque
18.8%

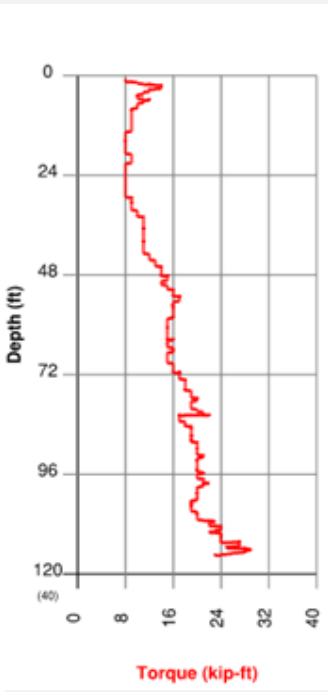
Torque: **37931.02 ft-lb**
RPM: **25 rpm**
Grout Pressure 2 : **19.00 psi**

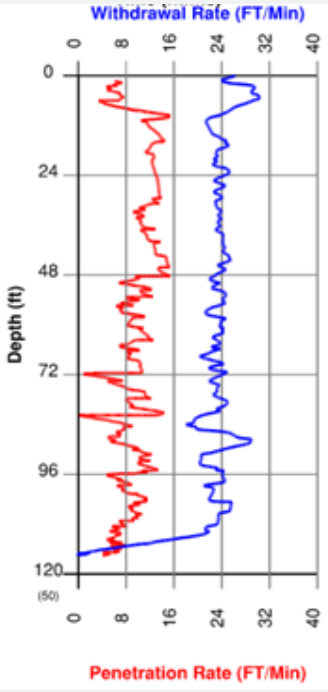
Withdrawal: **-0.0 ft/m**
Drill Time: **6:33**
Grout Time: **0:16**
Total Time: **6:49**

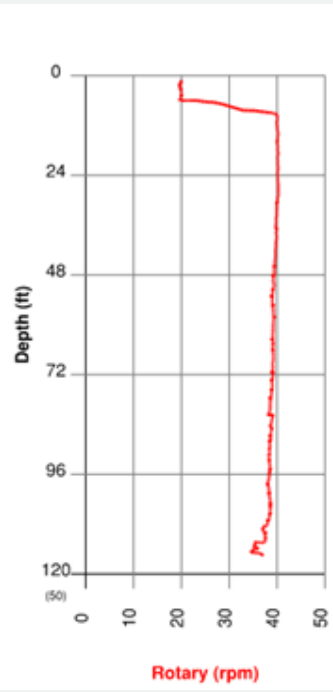
Initial Head Vol: **0.00 ft³**
Theor Vol: **69.87 ft³**
Increment Size: **5.00'**

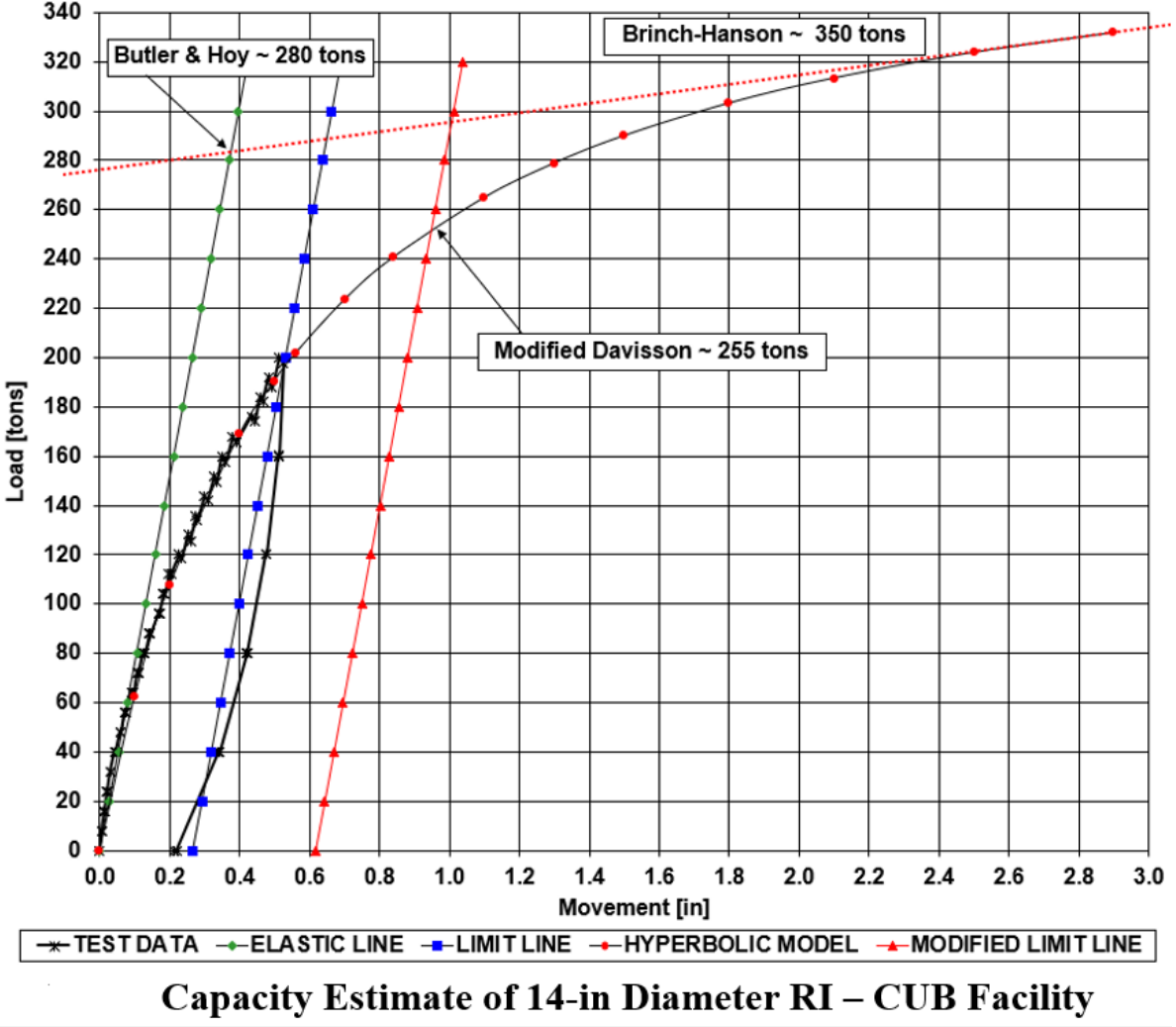




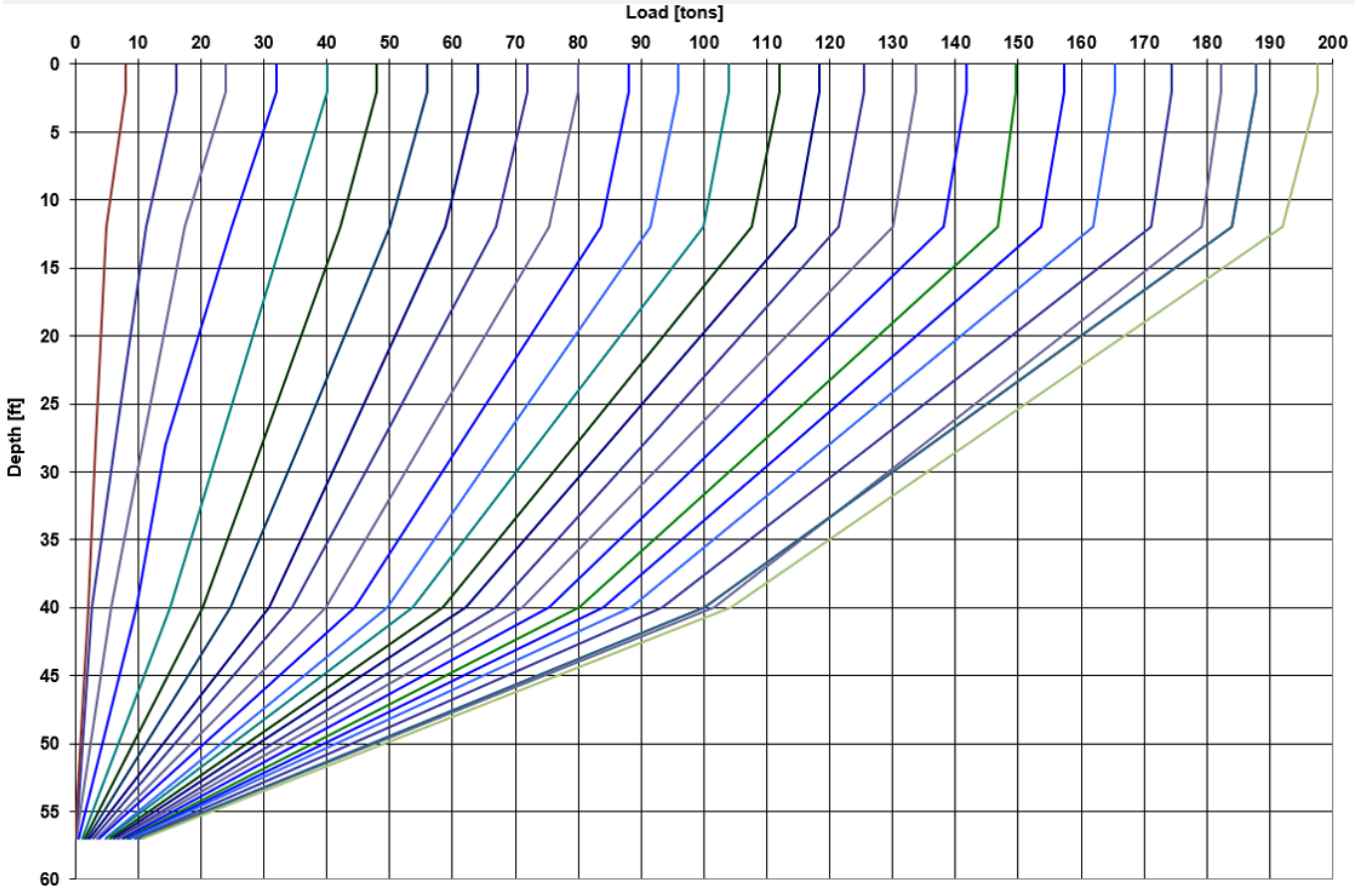




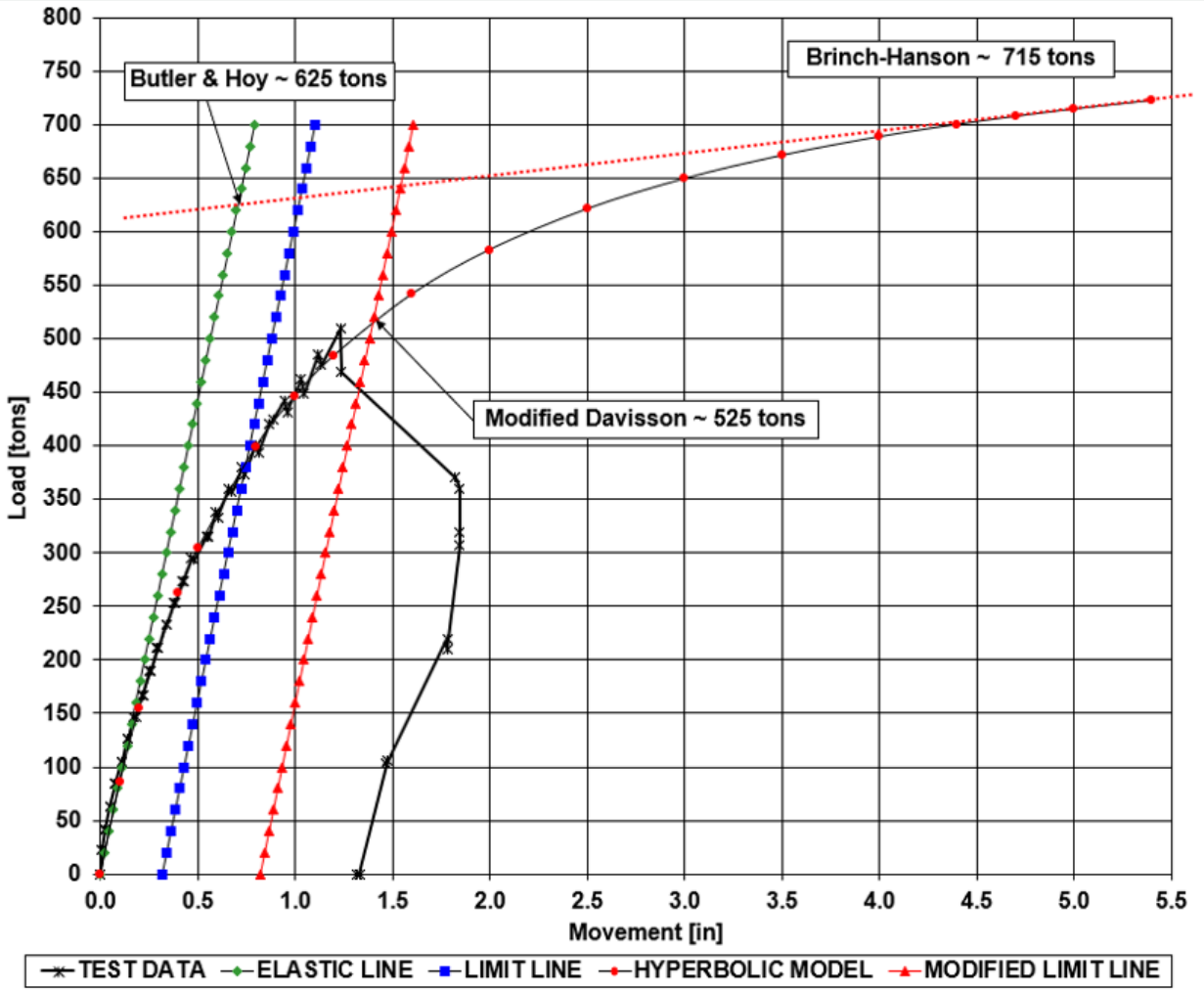




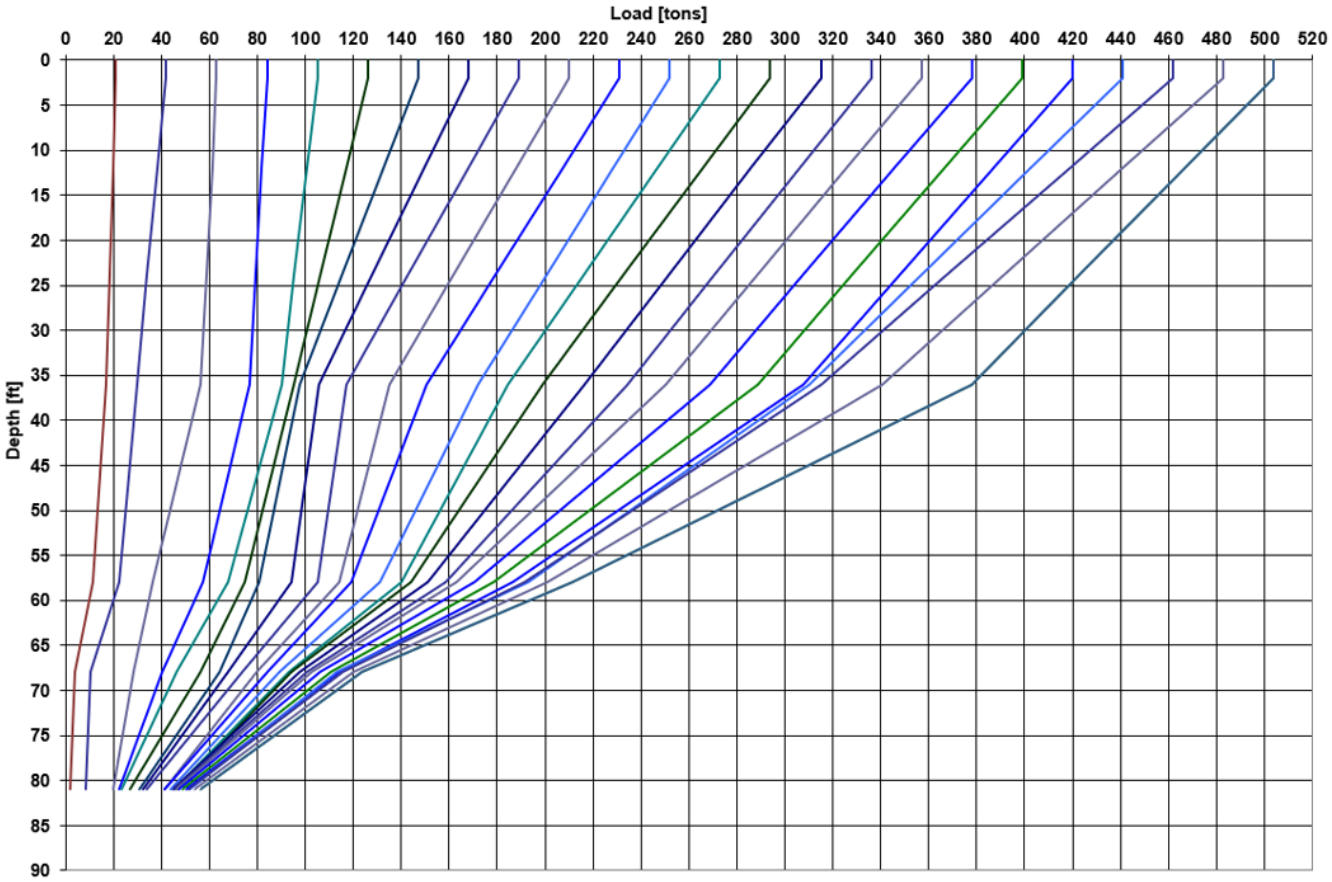
Capacity Estimate of 14-in Diameter RI – CUB Facility



Load Transfer with Depth – 14-in Diameter RI – CUB Facility

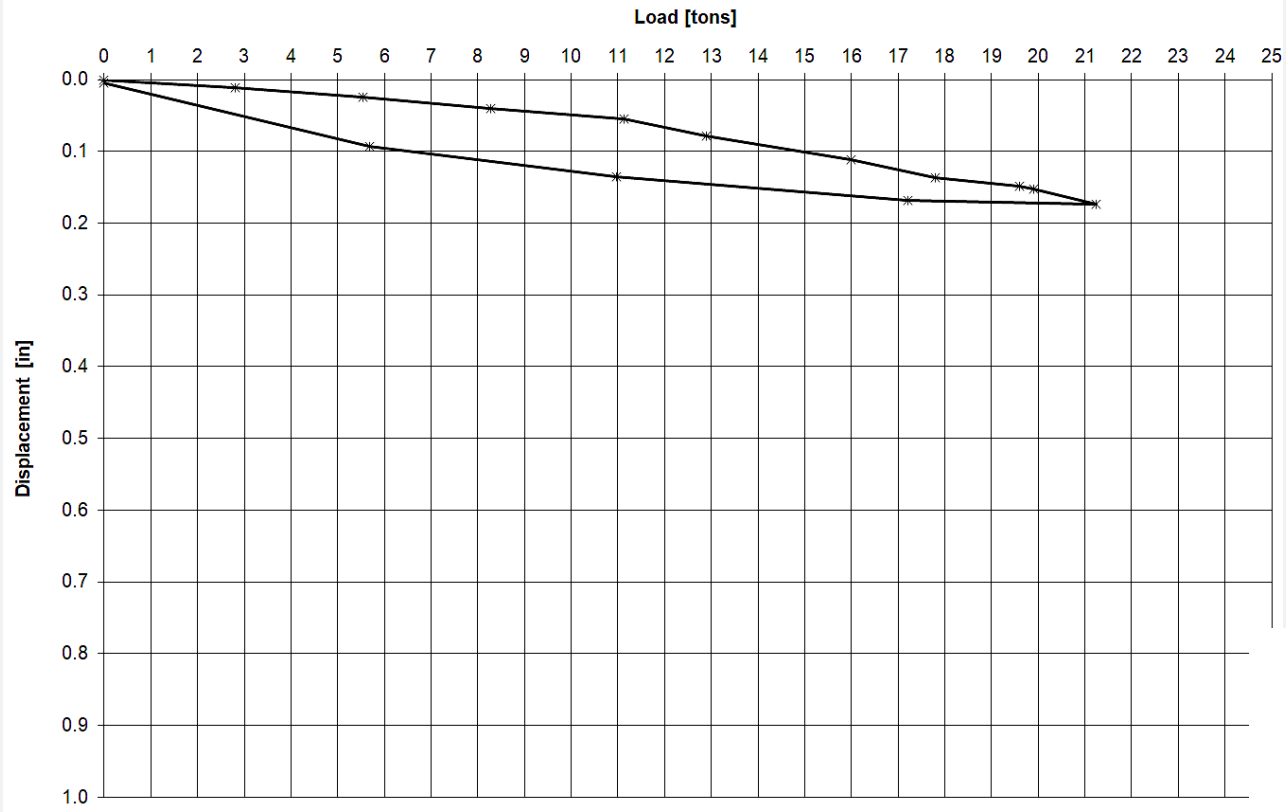


Capacity Estimate of 20-in Diameter ACIP Pile – Peptide



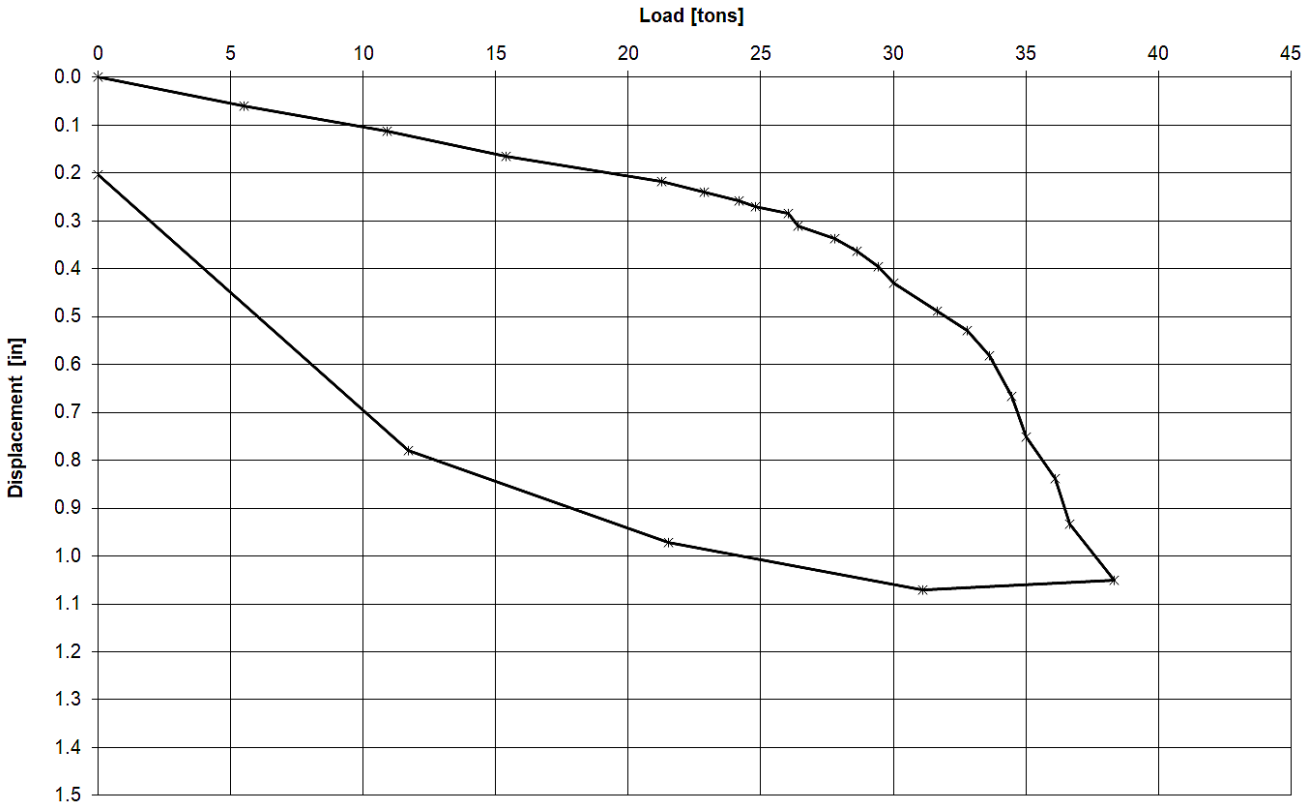
Load Transfer with Depth – 20-in Diameter ACIP Pile – Peptide 1 Facility

PEPTIDE I



Lateral Deflection vs Applied Load – 20-in Dia. ACIP Pile

Lateral Deflection vs Applied Load
To 1-in of Deflection – 20-in Dia. ACIP Pile



For these 3 reasons, this was a waste of time



Operations

Sequencing

Challenges

Lessons Learned

Sequencing

CUB – Rigid Inclusions

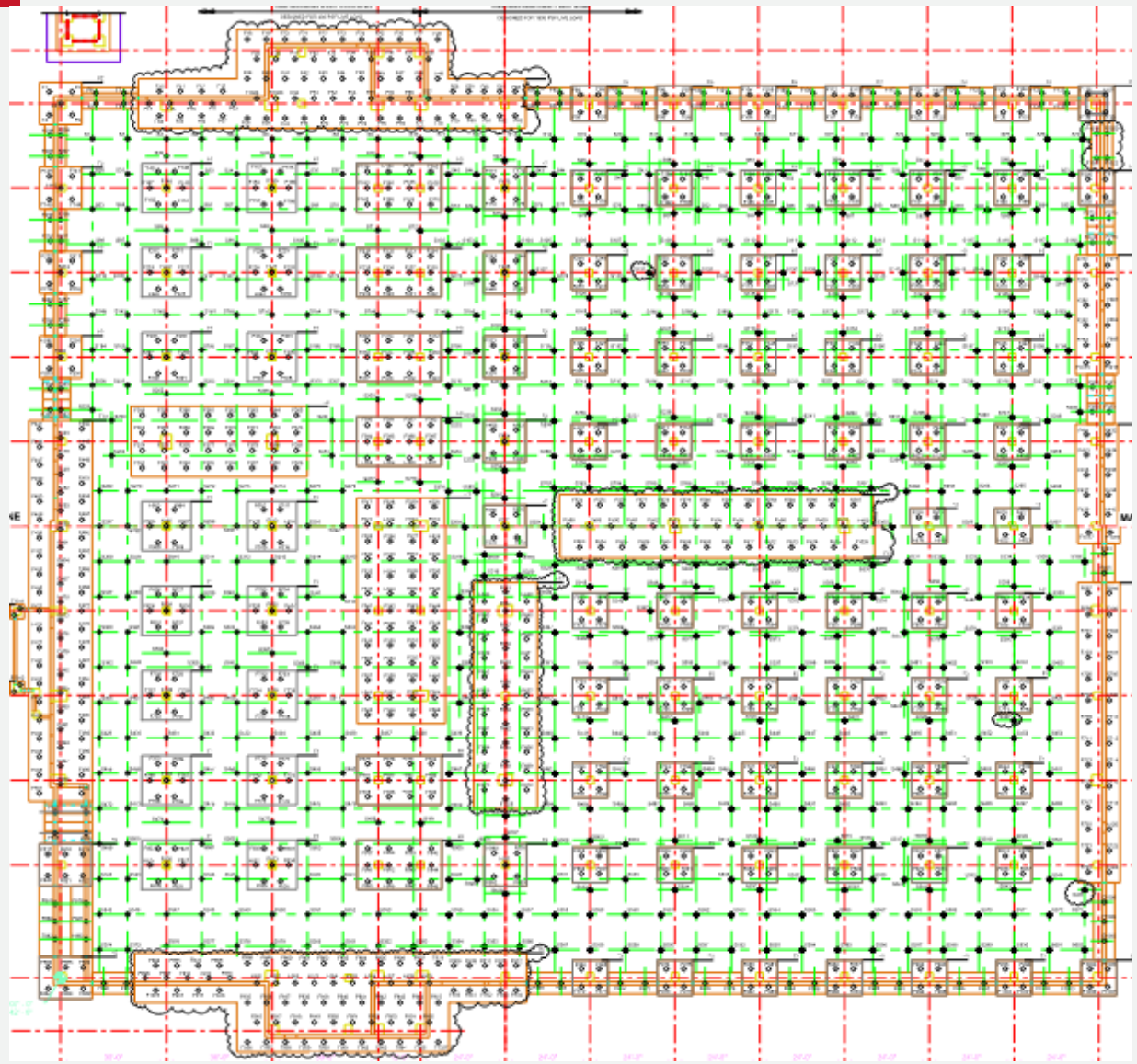
Peptide 1 – APG & RI

Small Molecule – APG & RI

Tank Farm & Pipe Rack



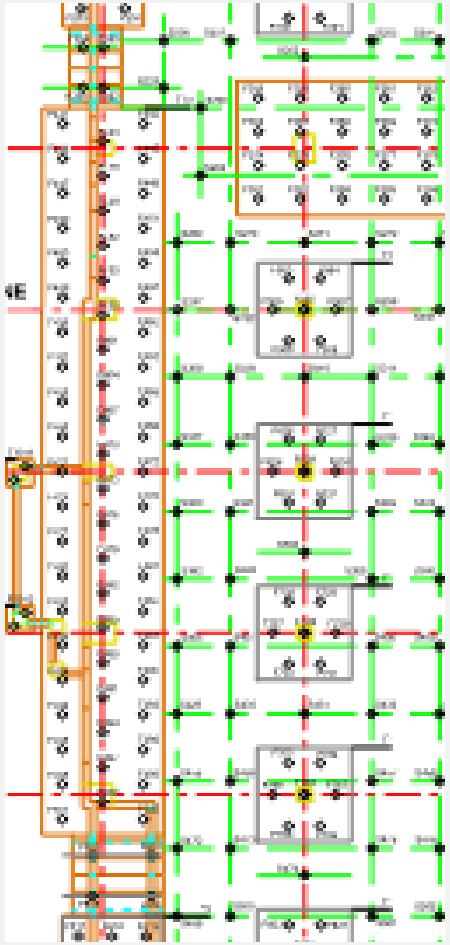
Central Utility Building (CUB)

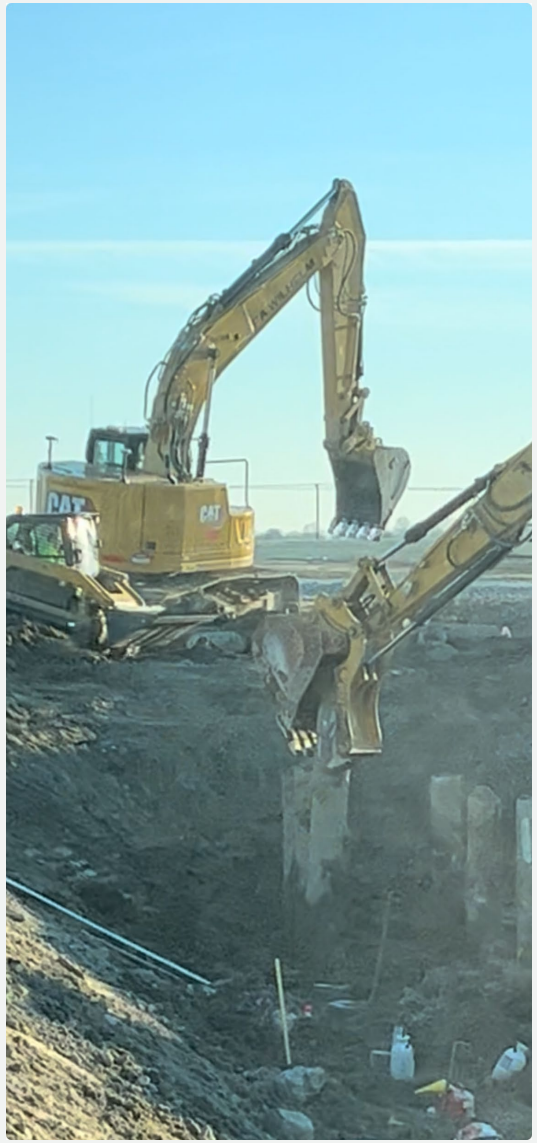






Central Utility Building (CUB)

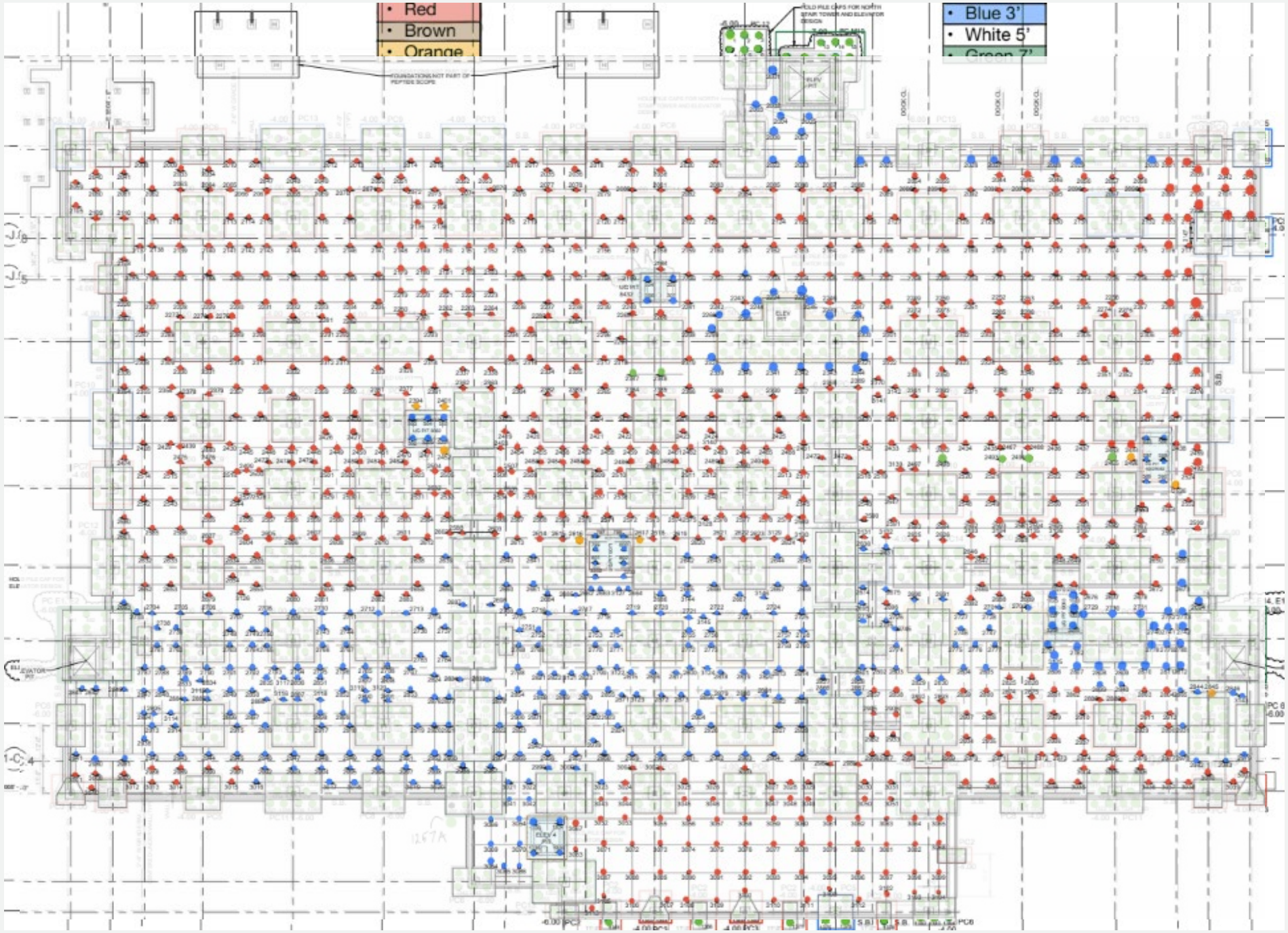




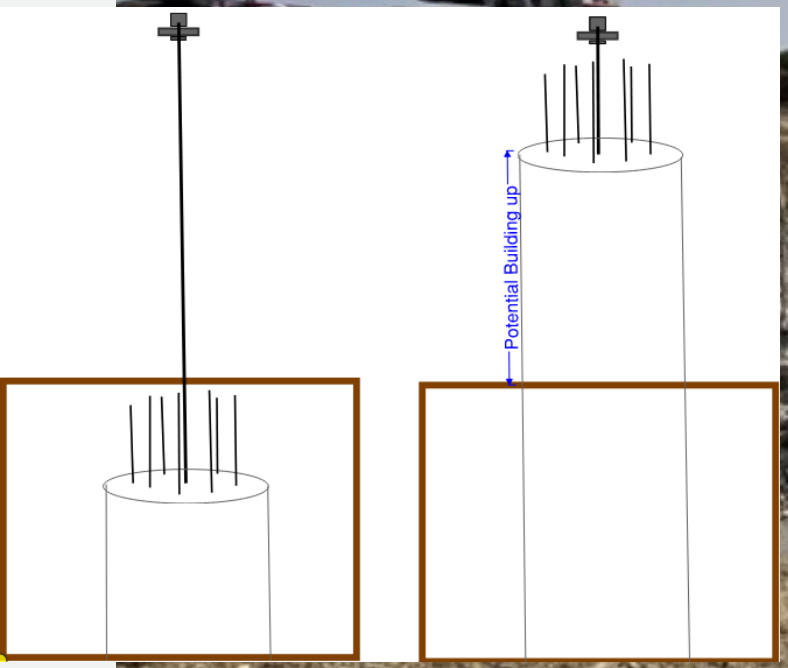


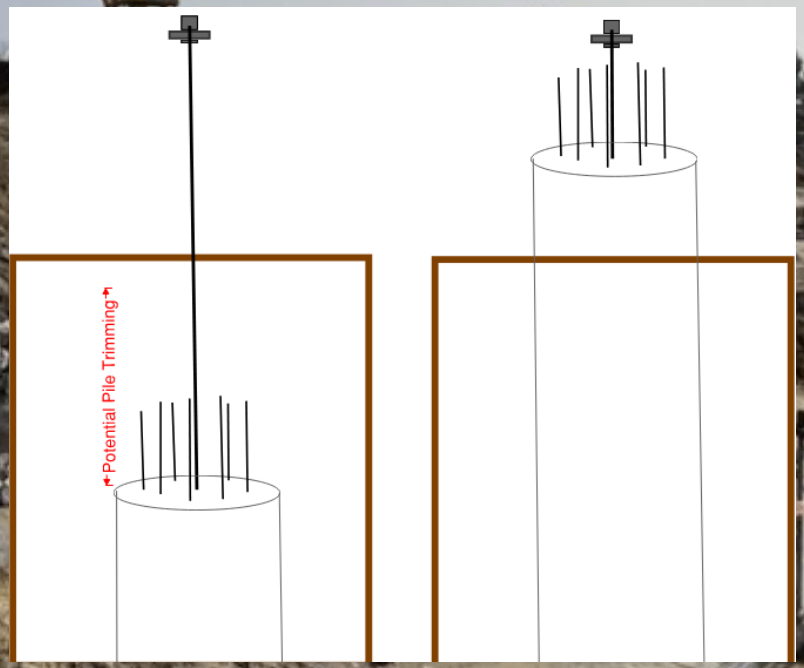
Peptide 1 (APG)

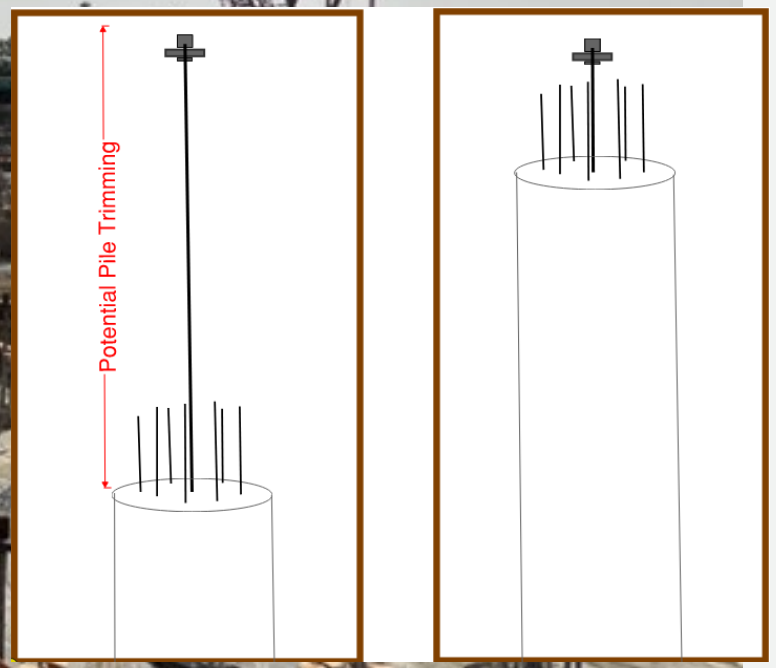
Peptide 1 (Rigid Inclusions)



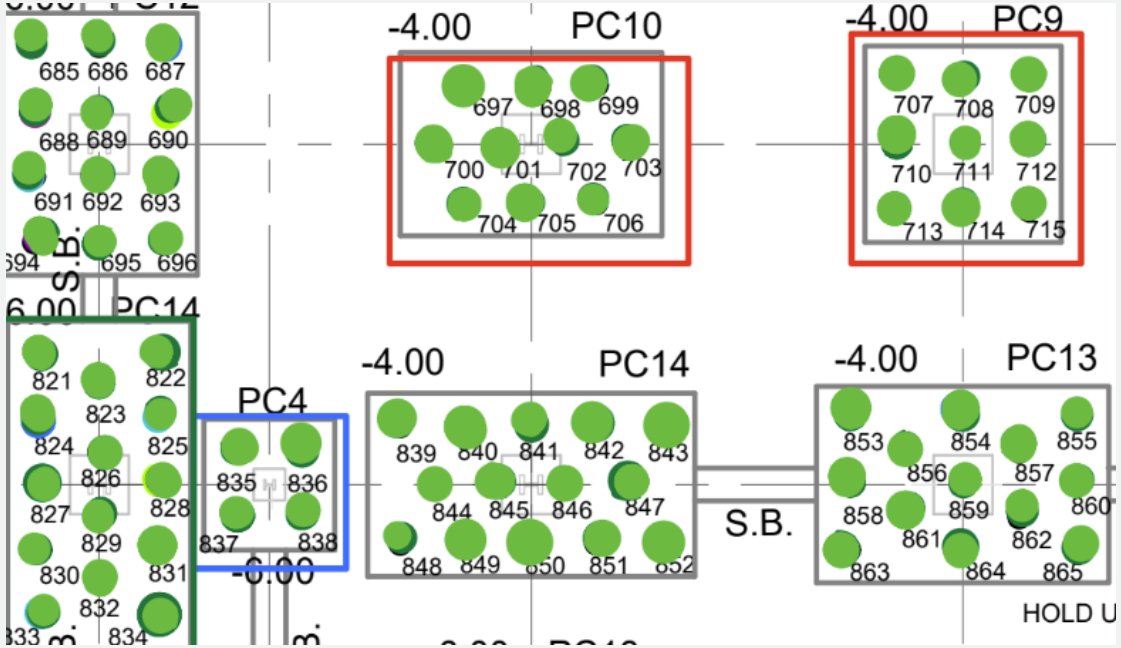
Drill Grade Options







Pile Caps / Projections

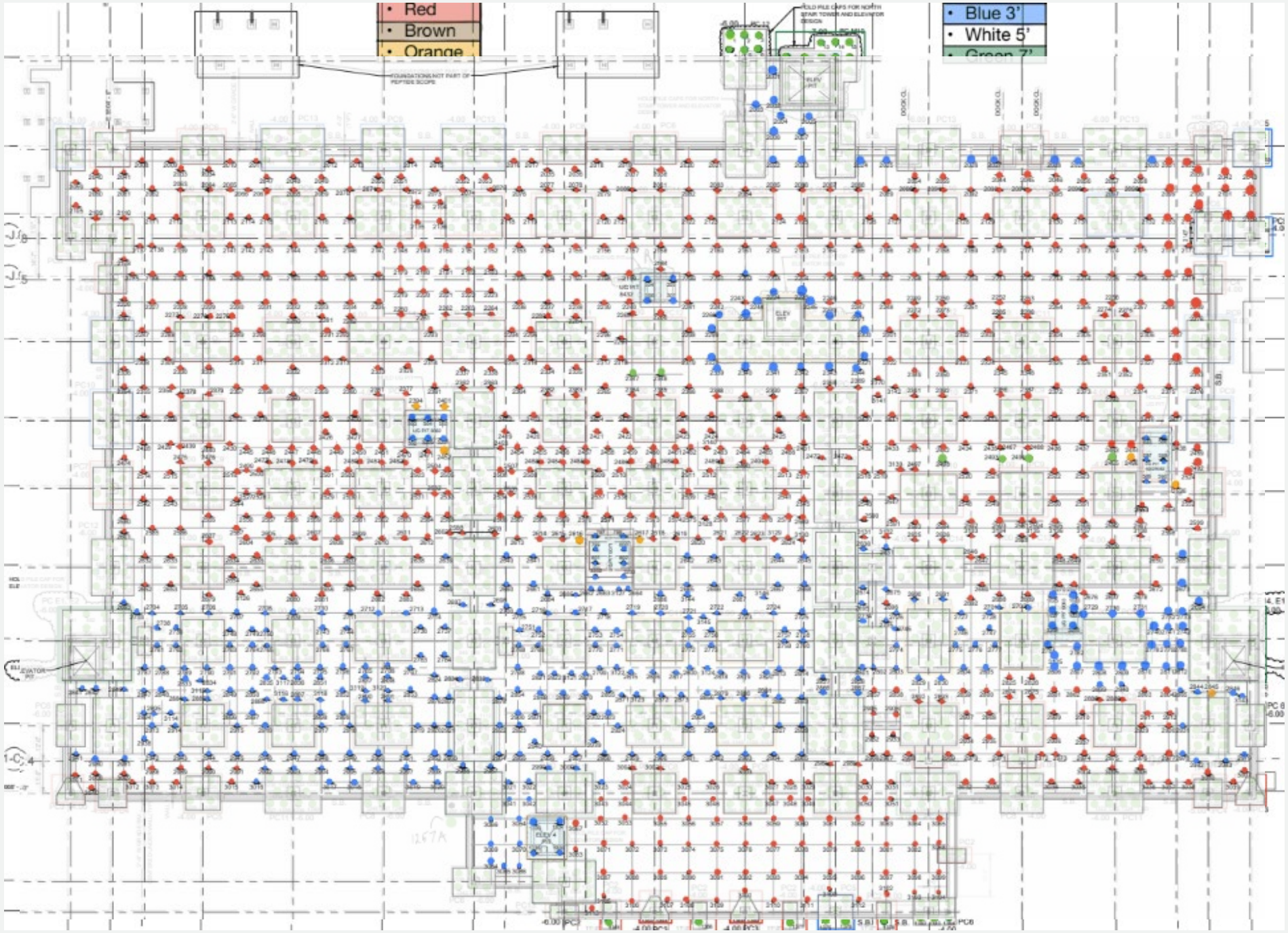






Peptide 1 (APG)

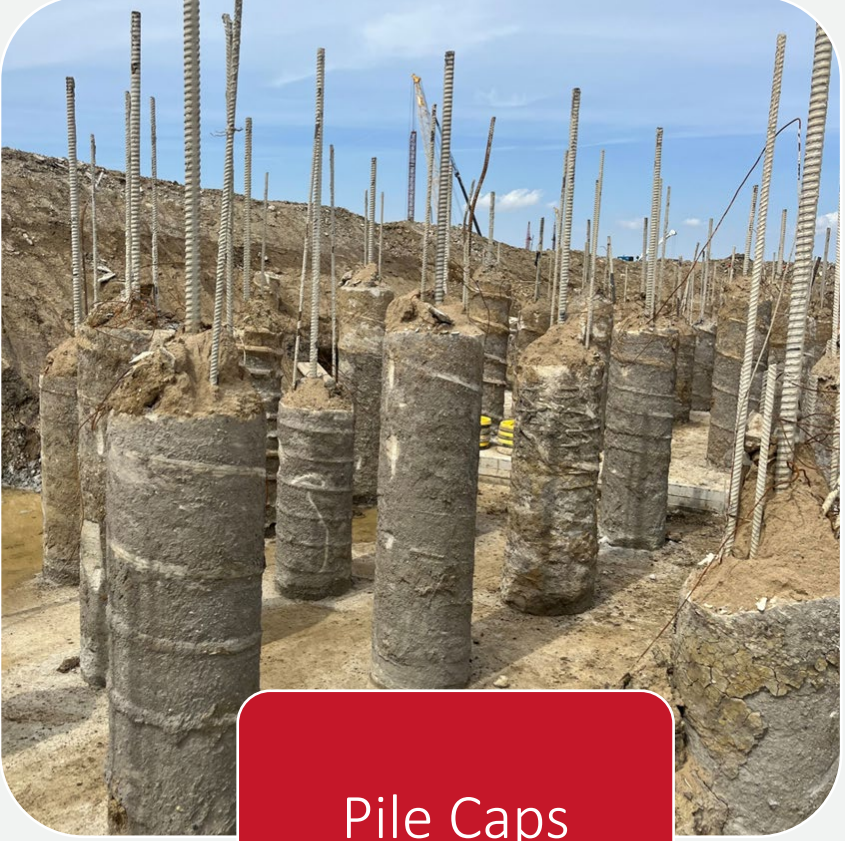
Peptide 1 (Rigid Inclusions)



Peptide 1 Installation







Pile Caps Excavated



Pile Chipping



Pile Caps

- Formed/Poured
- Backfilled

Sequence

Rigid Inclusion Installation









Thank you to our crews!

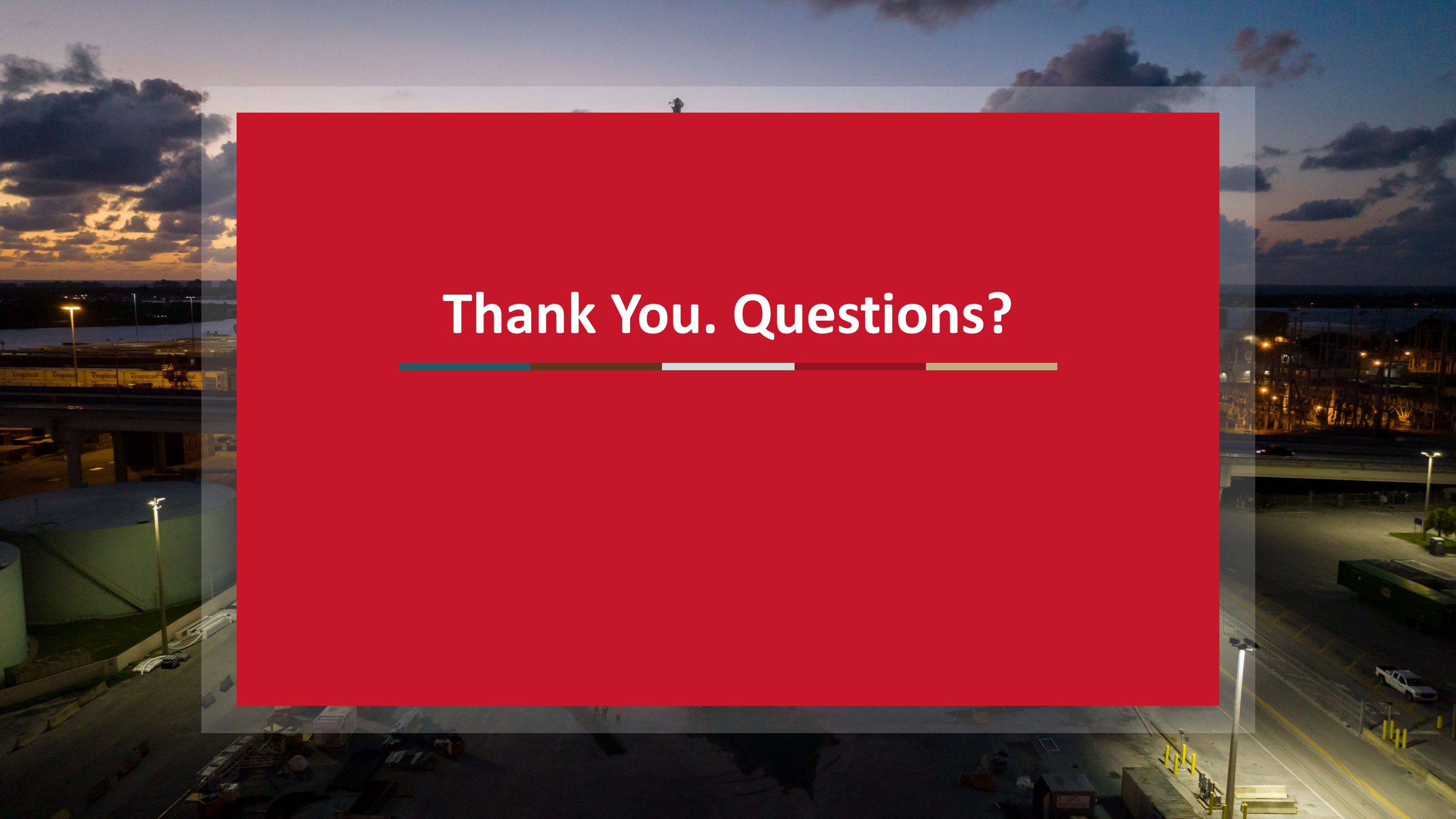












Thank You. Questions?





Parallel Session B2

AN OPTIMIZED APPROACH FOR MANAGING OVERHEAD TRANSMISSION LINE FOUNDATION DESIGN AND CONSTRUCTION RISK

Presenters:

Daniel Rosenbalm, MasTec Power Delivery
Peter Kandaris, retired (formerly DiGioia Gray & Assoc)

Co-Authors:

James P. Adams, Salt River Project
Mariah Paz, Salt River Project

Presentation Organization

- Purpose
- Background
- Risk Challenges & Mitigation Approach
- Subsurface Assessment
- Inspection Program
- Investigation Approach Comparisons

Disclaimer

“The opinions expressed in this presentation and on the following slides are solely those of the presenter(s) and not necessarily those of DFI. DFI does not guarantee the accuracy or reliability of the information nor endorse the recommendations offered by the presenter(s).”

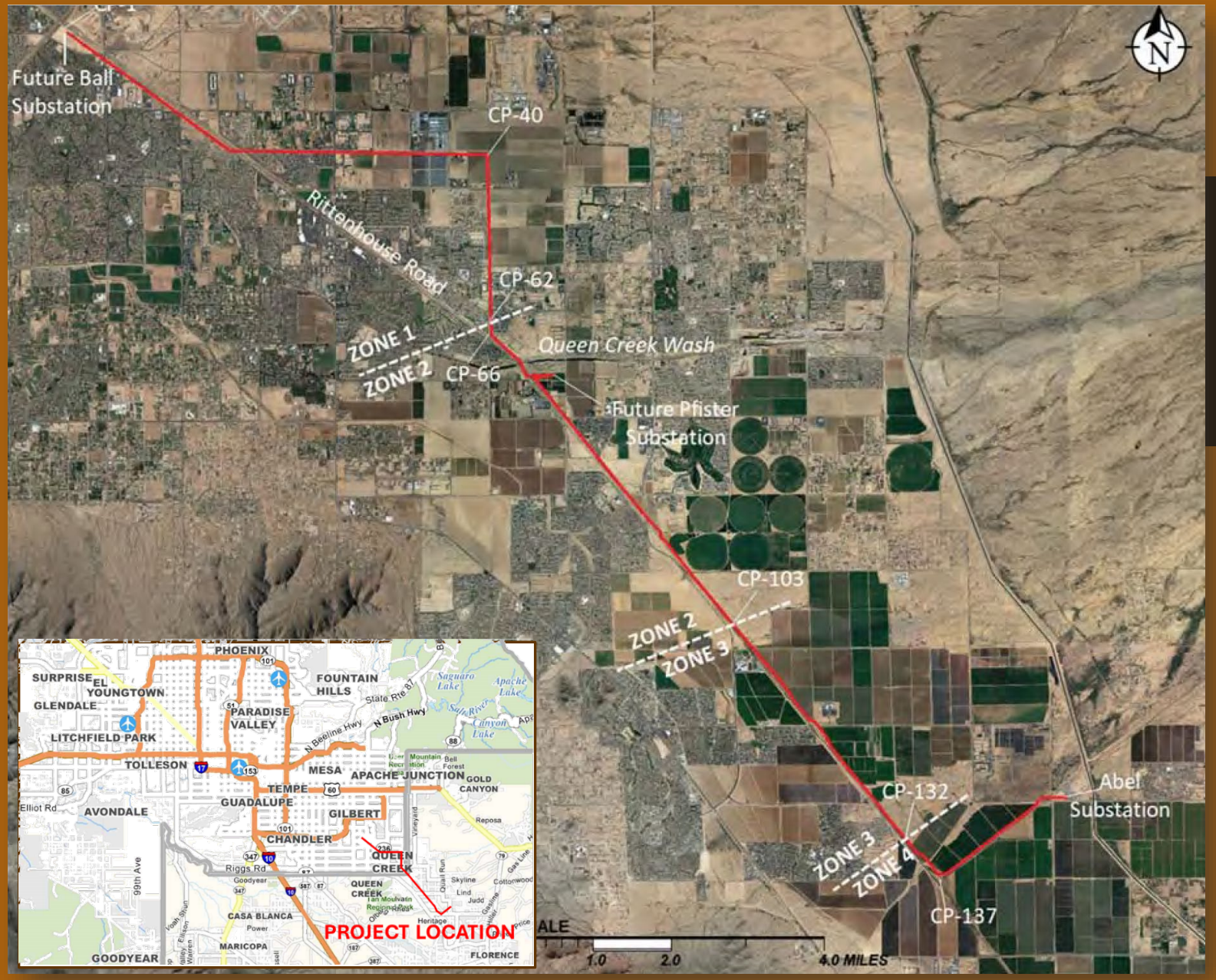
Purpose of Paper

- Investigation Approach Comparisons:
  - Optimization approach used
  - Two alternative methods
- Optimized Program:
  - Detailed pre-exploration survey
  - Strategic bore locations
  - Enhanced QC
- Alternative “A”
  - Bore at all structure locations
  - Standard QC
- Alternative “B”
  - Pre-determined bore sites based on structure type and spacing
  - Standard QC



Background

- New 20-mile double circuit 230/69 kV T/L (Abel-Ball-Pfister, ABP)
- SE region of the Greater Phoenix Metropolitan Area
- Installation of 161 RC drilled shafts
  - Diameters: 6 to 12 feet
  - Depths: 16 to 38 feet
- Piers support predominantly laterally loaded steel monopole structures with anchor bolted base plate



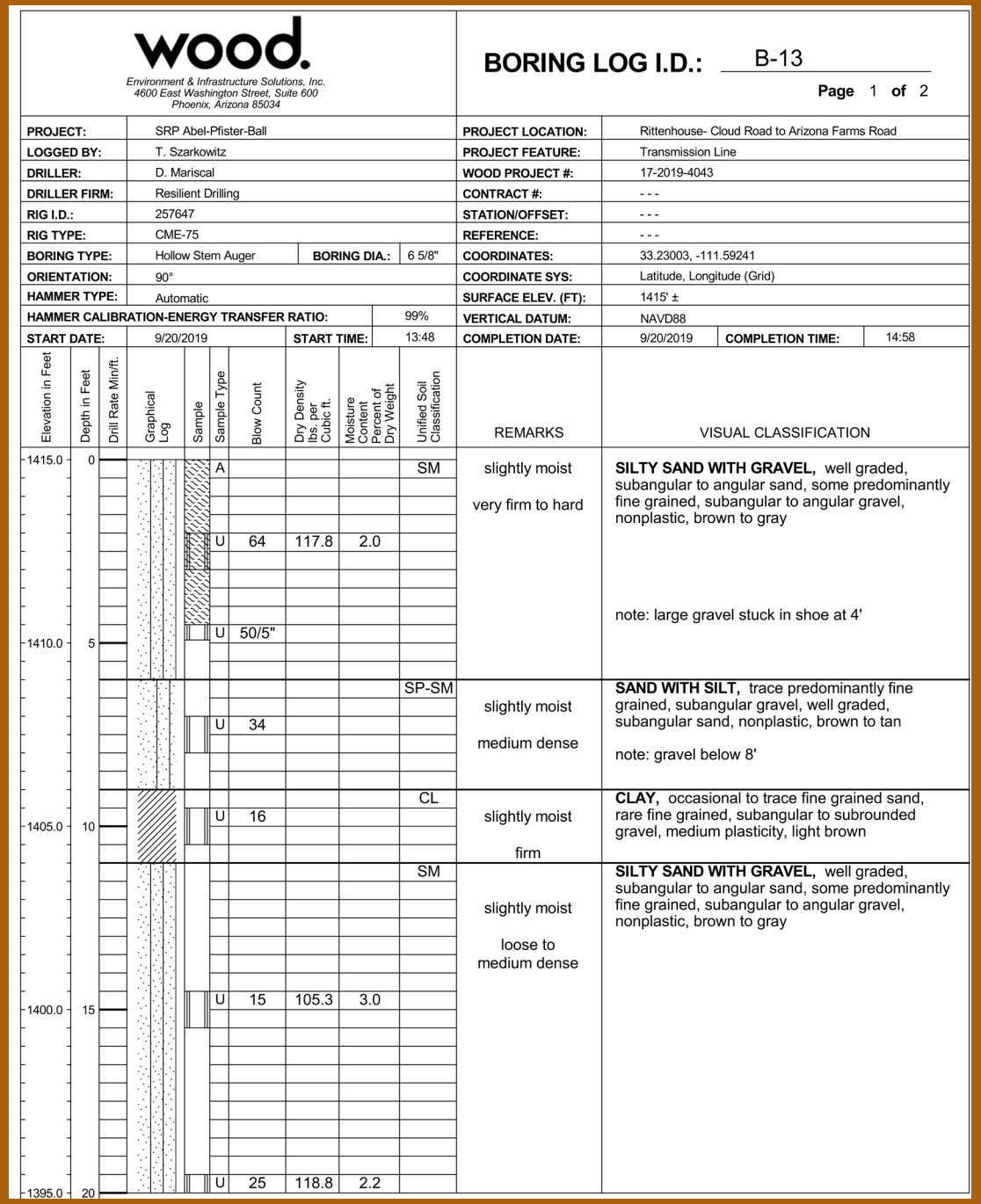
Risk Challenges

- Expedited APB construction schedule:
  - 23 work weeks to meet line in-service needs
  - Required optimized investigation & design process
- Team needed to finish foundation design as quickly as possible w/out sacrificing cost or quality
- Risk Management Approach:
  - Reliability-Based Design
  - Rational selection of bore locations
  - Detailed foundation construction inspection program



Risk Mitigation Approach

- Subsurface Data Location Selection:
  - Focused on alignment geology/soil variation
  - Recognize importance of structure type and spacing
- Construction Inspection Program:
  - Sufficient information recorded during construction to either:
    - ➢ Validate design profile assumptions, or
    - ➢ Update the design profile
  - Updated profiles would be re-analyzed and embedment depth increased, if needed



wood.
Environment & Infrastructure Solutions, Inc.
4600 East Washington Street, Suite 600
Phoenix, Arizona 85034

BORING LOG I.D.: B-13
Page 1 of 2

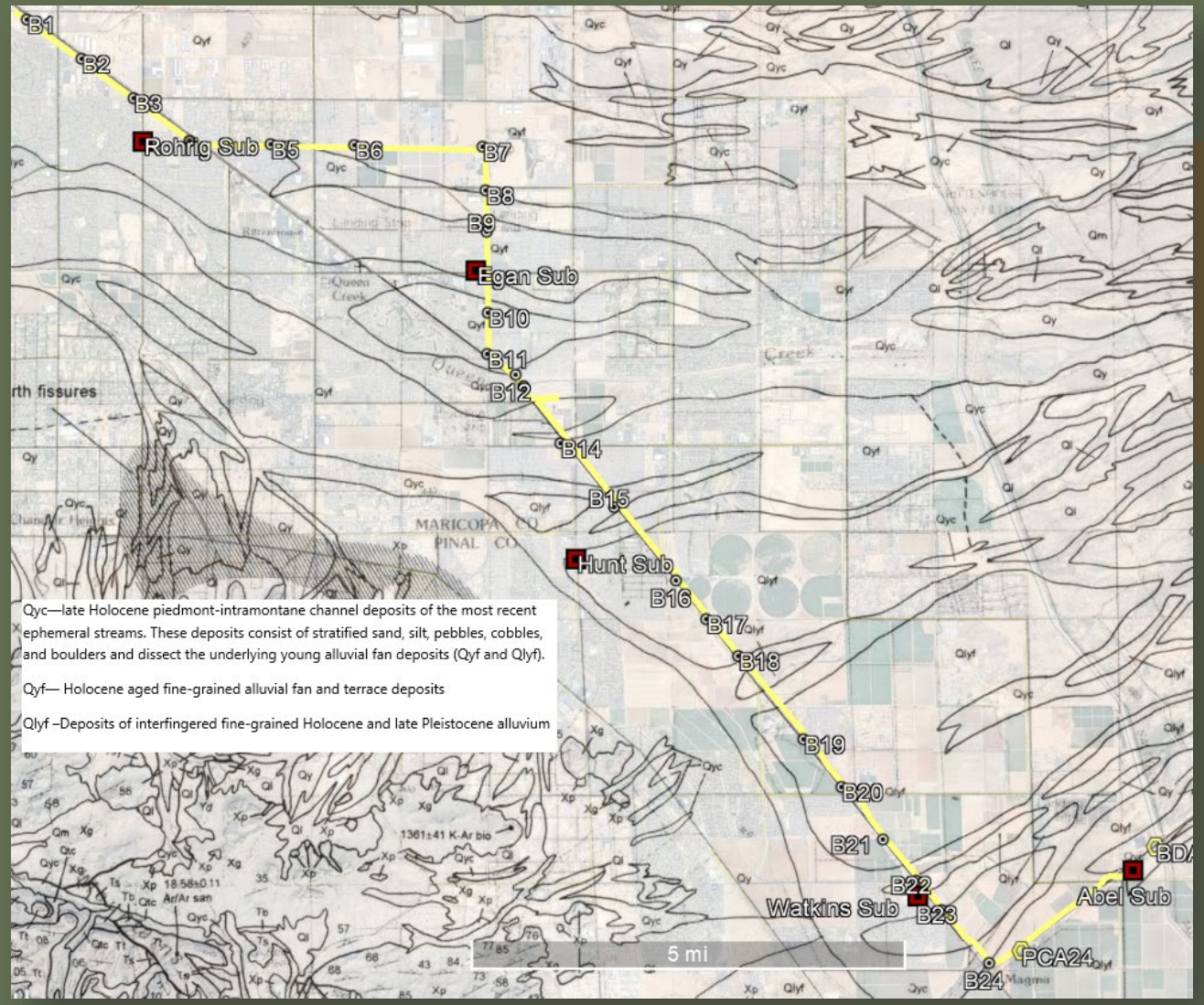
PROJECT:	SRP Abel-Pfister-Ball	**PROJECT LOCATION:**	Rittenhouse- Cloud Road to Arizona Farms Road
LOGGED BY:	T. Szarkowitz	**PROJECT FEATURE:**	Transmission Line
DRILLER:	D. Mariscal	**WOOD PROJECT #:**	17-2019-4043
DRILLER FIRM:	Resilient Drilling	**CONTRACT #:**	- - -
RIG I.D.:	257647	**STATION/OFFSET:**	- - -
RIG TYPE:	CME-75	**REFERENCE:**	- - -
BORING TYPE:	Hollow Stem Auger **BORING DIA.:** 6 5/8"	**COORDINATES:**	33.23003, -111.59241
ORIENTATION:	90°	**COORDINATE SYS:**	Latitude, Longitude (Grid)
HAMMER TYPE:	Automatic	**SURFACE ELEV. (FT):**	1415' ±
HAMMER CALIBRATION-ENERGY TRANSFER RATIO:	99%	**VERTICAL DATUM:**	NAVD88
START DATE:	9/20/2019 **START TIME:** 13:48	**COMPLETION DATE:**	9/20/2019 **COMPLETION TIME:** 14:58

Elevation in Feet	Depth in Feet	Drill Rate Min/ft.	Graphical Log	Sample	Sample Type	Blow Count	Dry Density lbs. per Cubic ft.	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
1415.0	0				A				SM	slightly moist very firm to hard	**SILTY SAND WITH GRAVEL,** well graded, subangular to angular sand, some predominantly fine grained, subangular to angular gravel, nonplastic, brown to gray
					U	64	117.8	2.0			
1410.0	5				U	50/5"					note: large gravel stuck in shoe at 4'
									SP-SM	slightly moist medium dense	**SAND WITH SILT,** trace predominantly fine grained, subangular gravel, well graded, subangular sand, nonplastic, brown to tan note: gravel below 8'
					U	34					
1405.0	10				U	16			CL	slightly moist firm	**CLAY,** occasional to trace fine grained sand, rare fine grained, subangular to subrounded gravel, medium plasticity, light brown
									SM	slightly moist loose to medium dense	**SILTY SAND WITH GRAVEL,** well graded, subangular to angular sand, some predominantly fine grained, subangular to angular gravel, nonplastic, brown to gray
1400.0	15				U	15	105.3	3.0			
1395.0	20				U	25	118.8	2.2			

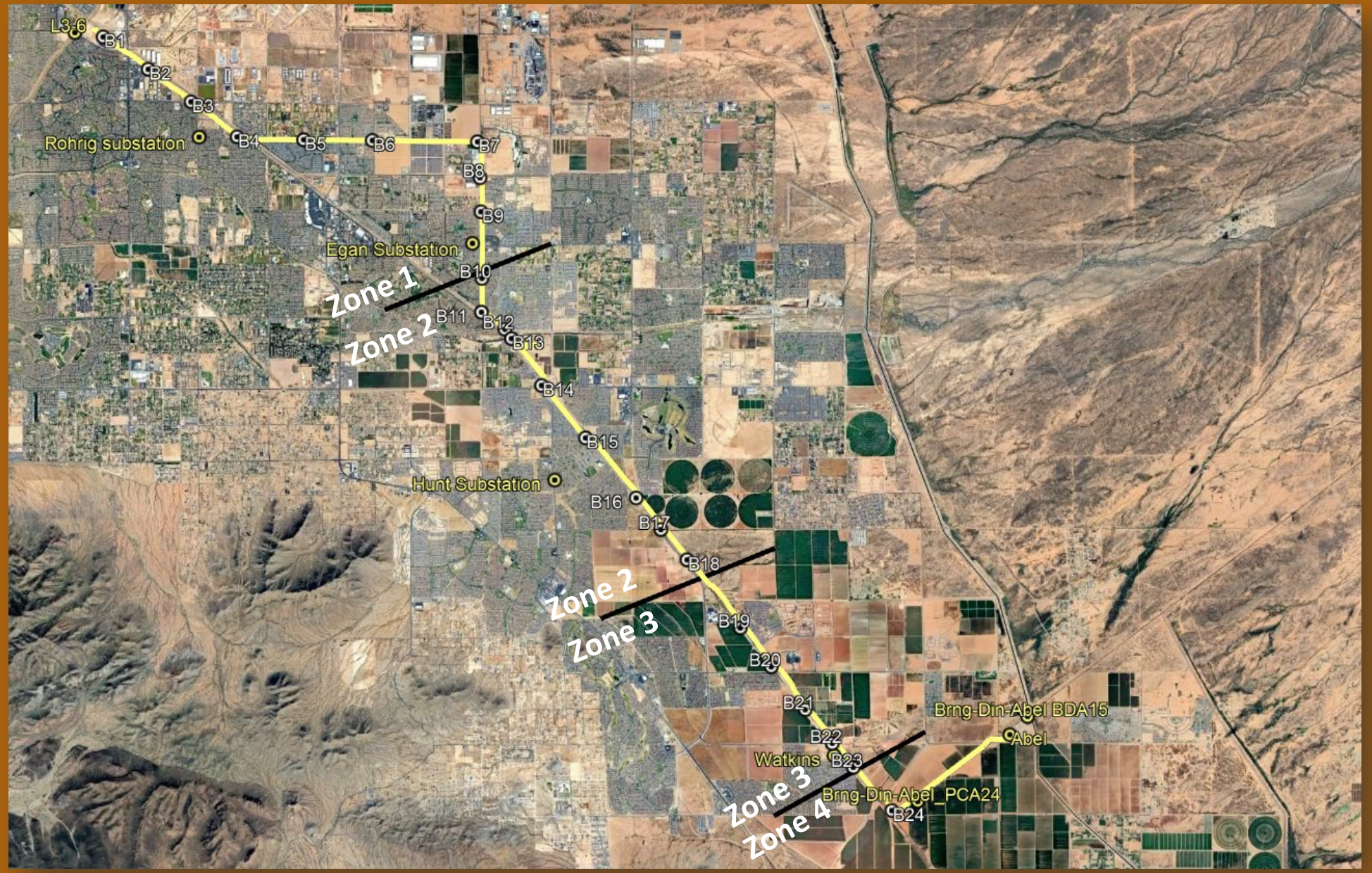
Subsurface Assessment Process – Step 1

Desktop Geologic & Geotechnical Study

- Review of prior data and public geologic maps
- Field reconnaissance to update maps
- Hazard study (geologic, geotechnical, hydrologic)
- Geological zone maps & tables developed



Geologic Zone Grouping



Summary of Geologic Zones

Geologic Zone		Description
1	Young alluvium (Qyc, Qyf)	Non-cemented sands and clay-rich material with surface materials underlain by old, cemented and clay-rich basin fill deposits comprising of material that was eroded from the Superstition and San Tan Mountains.
2	Young alluvium (Qyc) over Intermediate-aged alluvial gravel (Qi)	Same surficial materials as Zone 1; however, underlain by intermediate-aged alluvial gravel deposits that are non-cemented and associated with the sedimentation from Queen Creek. Younger deposits are more prevalent north and south from Queen Creek while older alluvial fan deposits interlayer the subsurface near the channel.
3	Young alluvium Qlyf, Qyc over Old alluvial fans (Qfo)	Dense, weakly cemented sandy clay and clayey sand. The deeper soils in this zone transitions to dense, weakly cemented clay to sandy clay associated with older alluvial fan deposits that predate Queen Creek.
4		Same surficial materials as Zone 3; however, the underlying older alluvium is strongly cemented clay with high plasticity.

Subsurface Assessment Process – Step 2

Investigation Site Selection

- Goal - Select field test sites representative of various subsurface conditions:
  - As demonstrated in prior investigation work
  - As observed by new reconnaissance
- Process:
  - At all dead-end and heavy angle structure locations.
  - At or near geologic transitions.
  - At least one exploration within each mile of the alignment
  - Supplement w/prior data (6 prior adjacent sub or T/L investigations)
  - Sites accessible by standard 2WD drill rig
- Results:
  - 1 boring per 5.4 structures (≈ 6 to 10 structures apart or 0.6 to 1.0 miles apart)
  - All anticipated soil deposits within all geologic zones represented
  - Includes foundations at or adjacent to drainage features or weak ground conditions

Data Evaluation – Soil Layers

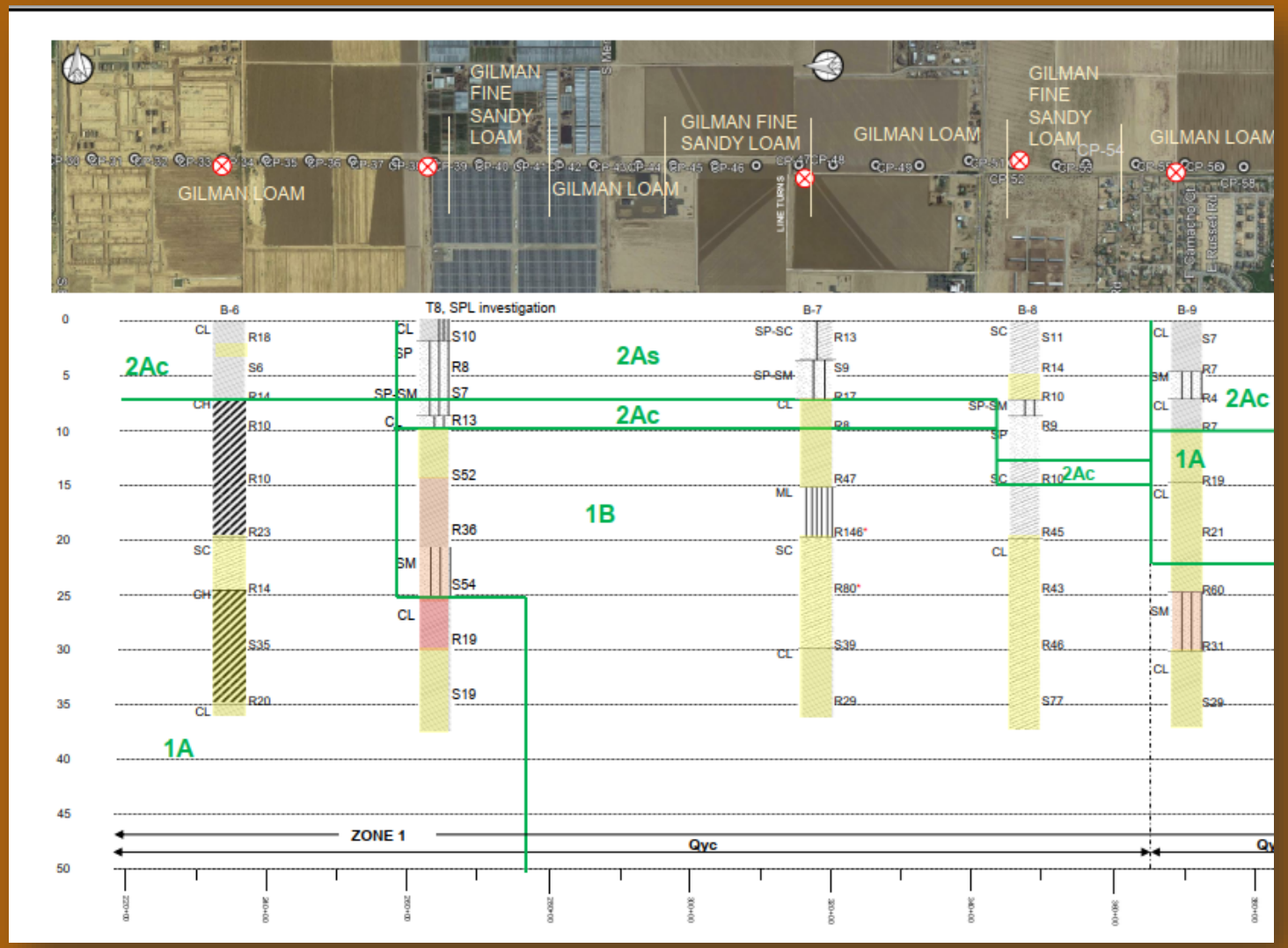
- Derive characteristic properties and design parameters using :
  - Statistical assessment of the blow count data
  - Modified based on laboratory testing
- Determination of characteristic N-value:
  - Data in similar geology and strata combined across the alignment
  - Data sorted by soil type and blow count to identify similar materials
  - Select the lower bound of a 5% exclusion limit of the data mean (≥ lowest N-value)
- 9 distinct geotechnical layers identified along the alignment



Subsurface Assessment Process – Step 3

Approach: Group Adjacent Structures in Similar Conditions

- Horizontal delineations assumed between geotechnical layers with depth (N-value groups)
- Vertical boundaries of layers based on:
  - geologic site reconnaissance observations
  - mapped geologic units
  - NRCS web soil survey soil changes
- 22 unique idealized soil profiles were developed for foundation design



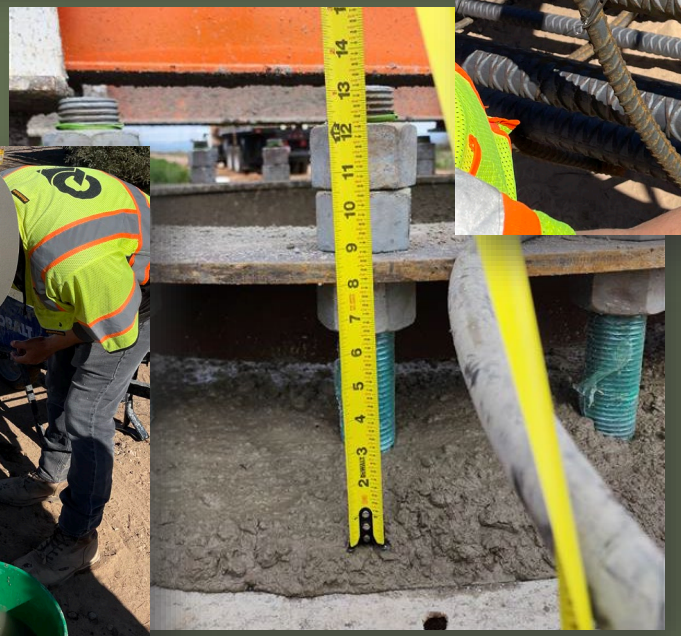
Inspection Program

Purpose:

- Monitor/verify foundations are installed per plans and materials are per plans & specs
- Provide immediate feedback of non-conformance to the foundation design team & owner
- Accurately record as-constructed foundation geometry in real time
- Use highly qualified field inspection staff
  - background in soil engineering
  - knowledgeable on concrete materials
  - understand foundation design









Foundation Construction

- Contractor:
  - verify staking,
  - use appropriate tooling,
  - positioned the drill rig
  - advanced the shaft excavation to the specified depth.
- Excavation tooling (open flight augers):
  - match specified diameter, or
  - pre-drill w/pilot hole, then ream to achieve specified diameter
- All foundations constructed in the dry.
- Caving control:
  - drill and allow limited caving
  - backfill with cementitious slurry
  - redrill to specified diameter







Foundation Construction

- Set full-length anchor bolt cage
  - Rigid support system
  - Verified cage location/orientation
- Place formwork (surface reveal)
- Placed the project specified concrete
- Finished the exposed shaft per the project specifications





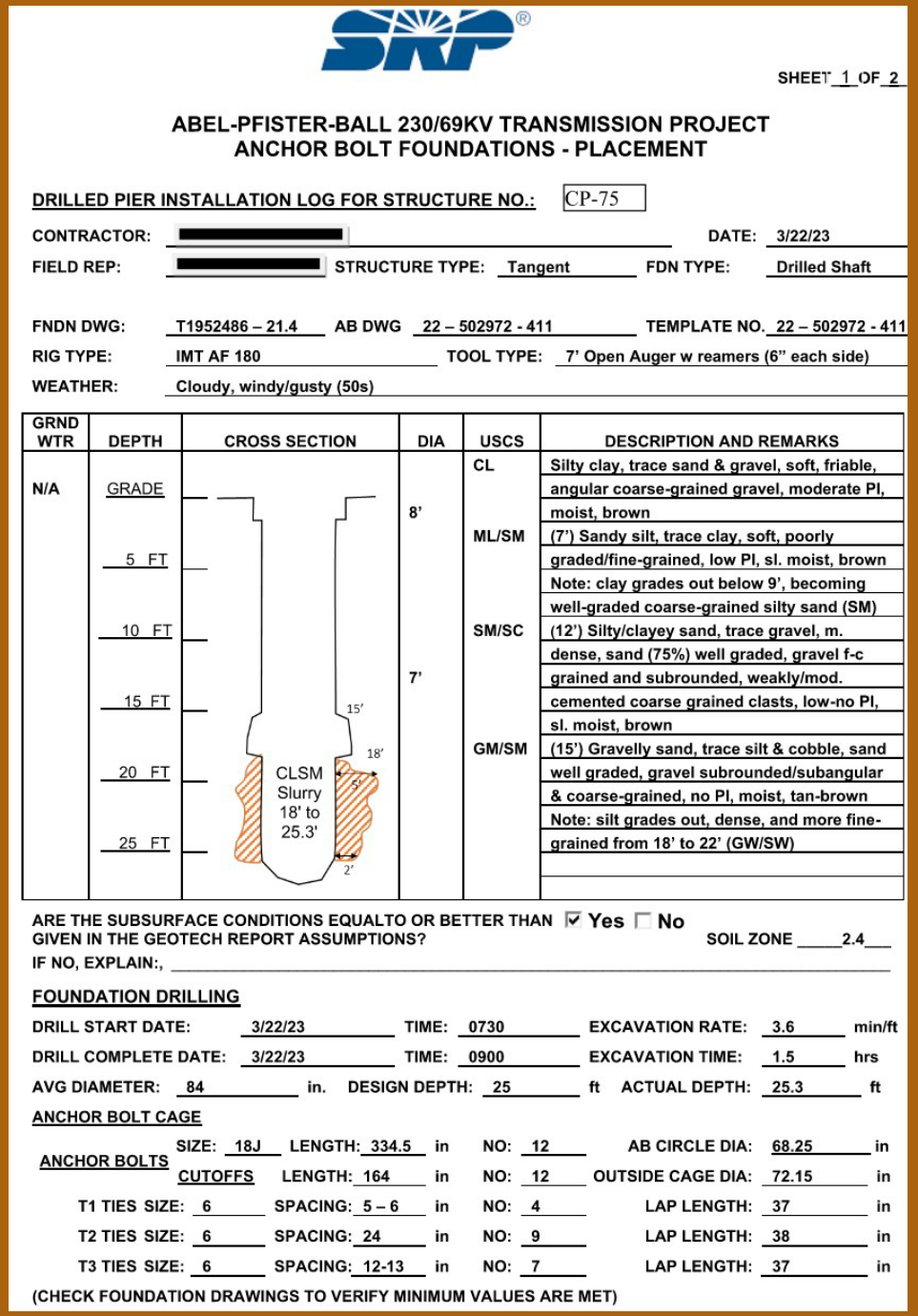






Foundation Inspection

- Create project Field Inspection Manual
- Develop monitoring/inspection forms to document foundation installation
  - excavation geometry
  - drill tooling
  - reinforcement & anchorage
  - concrete placement
  - final foundation appearance and finish
- Forms use common word processing software with data input tables
- Photograph all phases of inspection



SHEET 1 OF 2

ABEL-PFISTER-BALL 230/69KV TRANSMISSION PROJECT
ANCHOR BOLT FOUNDATIONS - PLACEMENT

DRILLED PIER INSTALLATION LOG FOR STRUCTURE NO.: CP-75

CONTRACTOR: [redacted] DATE: 3/22/23

FIELD REP: [redacted] STRUCTURE TYPE: Tangent FDN TYPE: Drilled Shaft

FNDN DWG: T1952486 – 21.4 AB DWG 22 – 502972 - 411 TEMPLATE NO. 22 – 502972 - 411

RIG TYPE: IMT AF 180 TOOL TYPE: 7' Open Auger w reamers (6" each side)

WEATHER: Cloudy, windy/gusty (50s)

GRND WTR	DEPTH	CROSS SECTION	DIA	USCS	DESCRIPTION AND REMARKS
N/A	GRADE		8'	CL	Silty clay, trace sand & gravel, soft, friable, angular coarse-grained gravel, moderate PI, moist, brown
	5 FT			ML/SM	(7') Sandy silt, trace clay, soft, poorly graded/fine-grained, low PI, sl. moist, brown
					Note: clay grades out below 9', becoming well-graded coarse-grained silty sand (SM)
	10 FT		7'	SM/SC	(12') Silty/clayey sand, trace gravel, m. dense, sand (75%) well graded, gravel f-c grained and subrounded, weakly/mod. cemented coarse grained clasts, low-no PI, sl. moist, brown
	15 FT			GM/SM	(15') Gravelly sand, trace silt & cobble, sand well graded, gravel subrounded/subangular & coarse-grained, no PI, moist, tan-brown
	20 FT				Note: silt grades out, dense, and more fine-grained from 18' to 22' (GW/SW)
	25 FT				

ARE THE SUBSURFACE CONDITIONS EQUALTO OR BETTER THAN GIVEN IN THE GEOTECH REPORT ASSUMPTIONS? ☑ Yes ☐ No SOIL ZONE 2.4

IF NO, EXPLAIN:,

FOUNDATION DRILLING

DRILL START DATE: 3/22/23 TIME: 0730 EXCAVATION RATE: 3.6 min/ft

DRILL COMPLETE DATE: 3/22/23 TIME: 0900 EXCAVATION TIME: 1.5 hrs

AVG DIAMETER: 84 in. DESIGN DEPTH: 25 ft ACTUAL DEPTH: 25.3 ft

ANCHOR BOLT CAGE

ANCHOR BOLTS SIZE: 18J LENGTH: 334.5 in NO: 12 AB CIRCLE DIA: 68.25 in

CUTOFFS LENGTH: 164 in NO: 12 OUTSIDE CAGE DIA: 72.15 in

T1 TIES SIZE: 6 SPACING: 5 – 6 in NO: 4 LAP LENGTH: 37 in

T2 TIES SIZE: 6 SPACING: 24 in NO: 9 LAP LENGTH: 38 in

T3 TIES SIZE: 6 SPACING: 12-13 in NO: 7 LAP LENGTH: 37 in

(CHECK FOUNDATION DRAWINGS TO VERIFY MINIMUM VALUES ARE MET)

Concrete Placement Form

SHEET 2 OF 2

ABEL-PFISTER-BALL 230/69KV TRANSMISSION PROJECT
CONCRETE PLACEMENT

CONCRETE PLACEMENT FOR STRUCTURE NO.: CP - 75

CONTRACTOR: [redacted] DATE: 3/23/2023

FIELD REP: [redacted] STRUCTURE TYPE: Tangent 84-27 FDN TYPE: Drilled Shaft

CONC MIX DESIGN: 401110 STRENGTH: 4,000 PSI DESIGN SLUMP 3" – 5"

CONC SUPPLIER: AZMM PLANT LOCATION: Queen Creek, AZ

PLACEMENT METHOD (CHECK ONE): END DUMP X PUMP TRUCK

WEATHER: Cleaar, Sunny AIR TEMP: 50s

FORMS / CAGE FINAL INSPECTION

REVEAL HT: 2 ft DIAMETER: 84 in AB PROJECTION (minus cover) 14 1/8 in.

MIN CLEARANCE SIDES: 4 - 5 in. BOTTOM: N/V in. FORM BELOW GRADE: 2 ft

CURING METHOD (CHECK ONE) COMPOUND X POLY SHEET OTHER

TOP OF CONCRETE: SLOPED TO DRAIN? (Y) N) TYPE OF FINISH Trowel and Broom

INSPECTOR TO VERIFY ANCHOR BOLT ORIENTATION & TOLERANCES ARE PER DESIGN DRAWING ☑ Yes

CONCRETE PLACEMENT POUR DATE: 3/23/2023

LOAD TICKET NO: 155478 CU YDS PLACED: 10.5 RISE IN SHAFT: 7.9 ft

CONCRETE TIMES: BATCH: 0824 START PLACEMENT: 0858 END PLACEMENT: 0907

FIELD ADDITIVES: WATER? (Y (N) IF SO, GALLONS: 0 OTHER ADDITIVES Retardent

SLUMP: 4.50 V in CONC TEMP: 80 Deg. F % AIR ENTRAIN: [redacted] BREAK SAMPLE? (Y (N)

LOAD TICKET NO: 155480 CU YDS PLACED: 10.5 RISE IN SHAFT: 5.5 ft

CONCRETE TIMES: BATCH: 0834 START PLACEMENT: 0910 END PLACEMENT: 0922

FIELD ADDITIVES: WATER? (Y (N) IF SO, GALLONS: 0 OTHER ADDITIVES Retardant

SLUMP: 5.00 in CONC TEMP: 76 Deg. F % AIR ENTRAIN: [redacted] BREAK SAMPLE? (Y) N)

(Note: attach lab test reports to log) (V = visual slump check) (N/V = not visible)

INSPECTOR: [redacted] DATE: 3/23/2023

Concrete Placement Spreadsheet

Structure	CP-75		**Date**	3/23/2023											
Socket Diameter=	40.00	in													
Cage Diameter =	72.15	in	Reveal Height =	2.0	ft			Form Depth =	2.0	ft					
Shaft Diameter =	84.00	in	Embedment Depth =	25.3	ft										
Form Diameter =	84.00	in	Socket Depth=	0.7	ft										

Ticket	Times				Height of Concrete (ft)		Concrete	Concrete Volume (cy)		Shaft Avg. Diameter (inches)		Neat Line	Shaft Avg. Cover (inches)	
	Batch	Start	Empty	Dur	Load Start	Load Finish	Rise (ft)	Per Truck	Cumulative	by volume	nominal	Volume	by volume	nominal
						28.00			0.0					
155478	8:24	8:58	9:07	0:43	28.00	20.10	7.90	10.5	10.5	81.11	80.10	9.6	4.48	3.98
155480	8:34	9:10	9:22	0:48	20.10	14.60	5.50	10.5	21.0	97.21	84.00	7.8	12.53	5.93
155485	8:59	9:24	9:40	0:41	14.60	5.40	9.20	10.5	31.5	75.17	84.00	13.1	1.51	5.93
155487	9:12	9:48	9:58	0:46	5.40	0.00	5.40	7.7	39.2	84.02	84.00	7.7	5.93	5.93
						Total reported volume (cy)		39.2		2.6%	excess of neat line		7.70	
						Neat line volume (cy)		38.2						

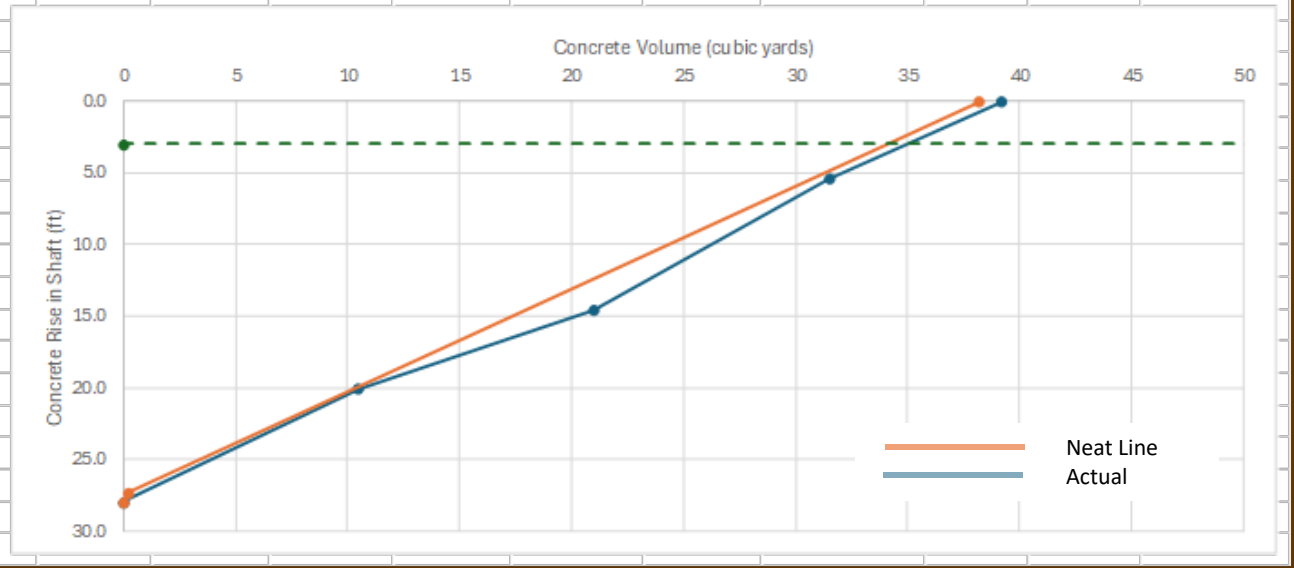
Observations:

Gray shading denotes caving observed in log

Caving reported above slurry section (17'-20' below top of form) prior during drilling. Overage observed in concrete volume measures.

Differences from expected rises due to 1st, 2nd, and 3rd loads likely due to contractor's inability to accurately measure concrete rise. Cover visually checked.

AS BUILT DIMENSIONS		
dia meter	83.7	in
depth	25.3	ft
length	27.3	ft
Shaft Vol	39.0	cy



Investigation Approach Comparisons

- Investigation approaches assessed are based on standard industry practices as summarized by the DFI's Electric Power System Foundations Committee State of Practice report issued this year
- Optimized Approach – Quality Measures

Profile at Structure Different and Weaker than Predicted		Subsurface Not Meeting Minimum Required Resistance		Foundation Analysis Performed to Confirm Resistance		Foundation Volume Exceeding Neat Line plus 10%	
Number	Percent	Number	Percent	Number	Percent	Number	Percent
25	15.5%	4	2.5%	11	6.8%	25	15.5%

- Alternative "A" – Test all structure sites
  - Adds 131 standard borings & limited additional laboratory work - no pre-investigation reconnaissance
  - Soil profile prediction accuracy = 100%, therefore installed foundation volumes reduce by 2.5%
  - Standard QC: concrete testing w/one site monitor
- Alternative "B" – Select test sites, characterize subsurface with nearest boring
  - Same number of borings as optimized program - no pre-investigation reconnaissance
  - Low bound of N-values used to estimate soil properties results in 12% increase in neat line volumes
  - Standard QC: concrete testing w/one site monitor

Investigation Approach Comparisons

- Optimization Approach is comparable cost to Alternative “A” but has advantages:
  - Geological recon + optimized sampling done in less time than testing at all sites.
  - Statistical approach to determining characteristic geotechnical properties = slightly more concrete.
  - Enhanced QC (although higher in cost) rapidly addresses changes in field conditions without slowing construction and improves foundation knowledge.
- Alternative “B” has a higher total cost, has the least information going into construction, and results in the lower reliability as the accuracy of predicted to actual subsurface info is unknown.

Program Element	Optimized Approach	Alternative A	Alternative B
Description	Perform significant pre-exploration survey to strategically locate subsurface investigation locations	Subsurface investigation at each structure site	Subsurface investigation at each large type structure & every 10 small type structures
No. of Geotech Test Sites	30	161	30
Soil Profile Accuracy	84.5%	100%	Not measured
Geotech Characteristic Value	5% LEL	Average	Low Bound
Design Concrete Vol (cy)	8380 (actual)	8363 (estimated)	9368 (estimated)
Installed Concrete Vol (cy)	8870 (actual)	8648 (estimated)	9595 (estimated)
% Concrete Above Neat-line	5.9	3.3	3.3
Quality Control Approach	Enhanced	Standard	Standard
Pre-Construction Engineering Unit Cost (per inst cy conc)	$19/cy	$61/cy	$15/cy
Installed Unit Cost (per cy conc)	$785/cy		
Quality Control Unit Cost (per cy inst conc)	$79/cy	$67/cy	$60/cy
Foundation Cost			
Total Cost	$7.83M	$7.90M	$8.34M
Unit Cost (per inst cy conc)	$883/cy	$913/cy	$860/cy
Unit Cost (per foundation)	$48,653/fndn	$49,051/fndn	$51,802/fndn
Schedule Impact (added time, in months)	Base	2 months	0 months

Questions/Comments?





From Data to Action: Using Testing to Address MWD-Identified Deviations in Rigid Inclusion Installation

DFI50 – Nashville, TN

October 21, 2025

5:35 PM





Ken Kniss, P.E.

Chelsea Riddle, P.E.

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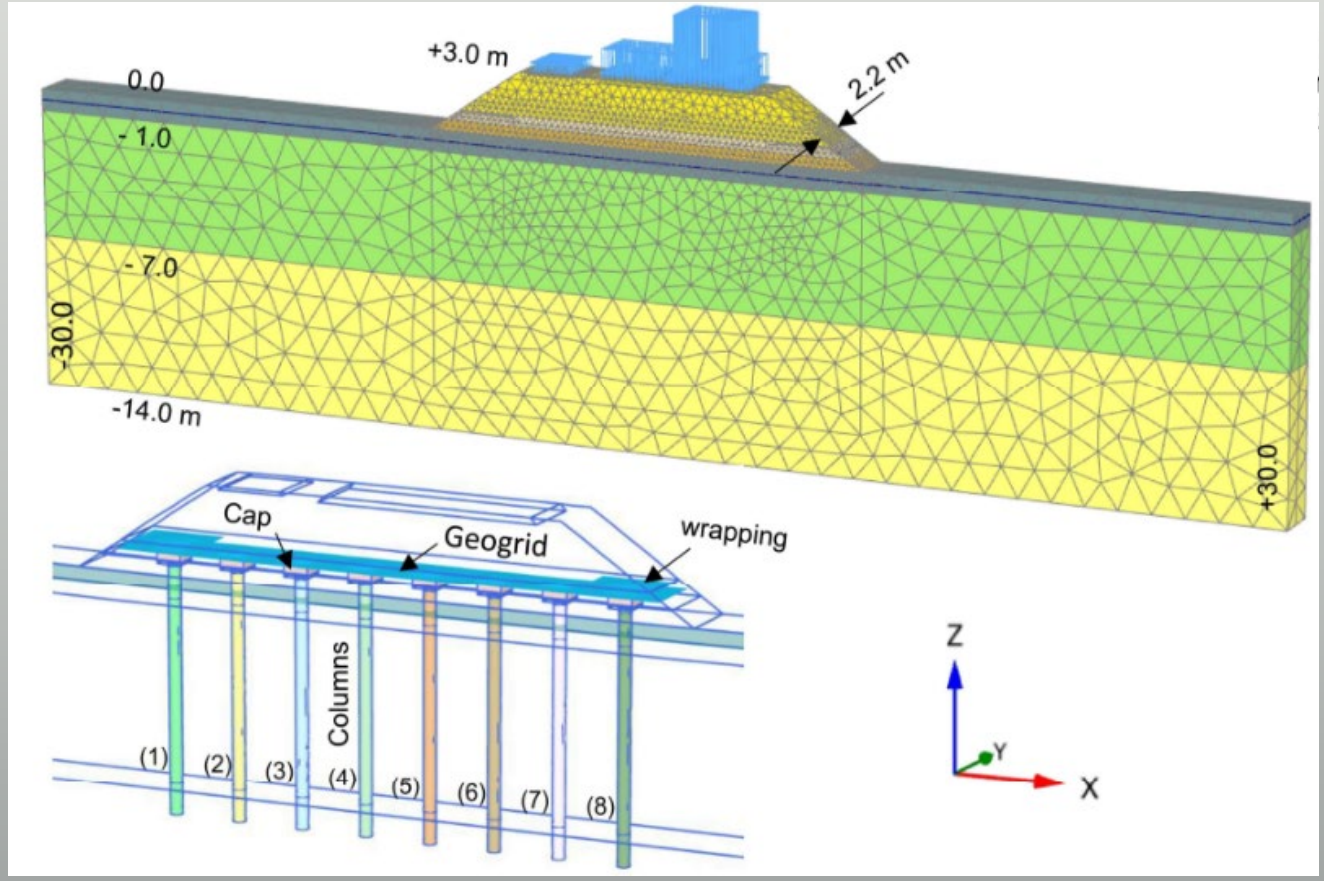
AGENDA

- Background
- Calibration Program
- Examples of Non-Conformance
- Testing Options
- Project Summary – Bethlehem, PA
- Conclusions
- Question and Response

Current Literature

- ASCE Library Search for “Rigid Inclusions”
  - Numerical Analysis
  - Project Summaries
    - Design
    - Performance Monitoring
- ASIRI 2012
  - Most comprehensive
  - 14 of 317 pages dedicated toward quality and testing



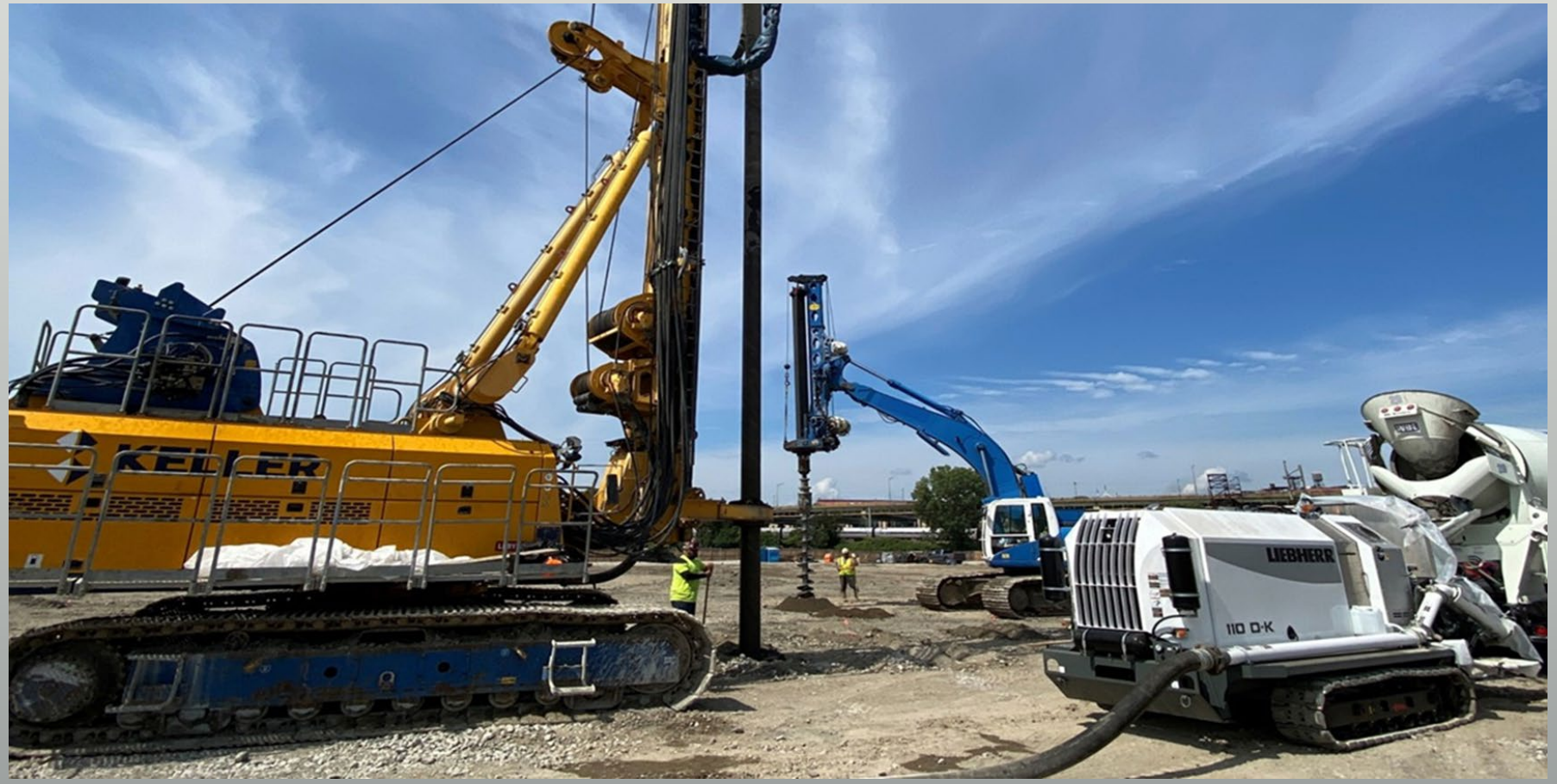
Topolnicki 2024

Rigid Inclusion Guidelines and Standards



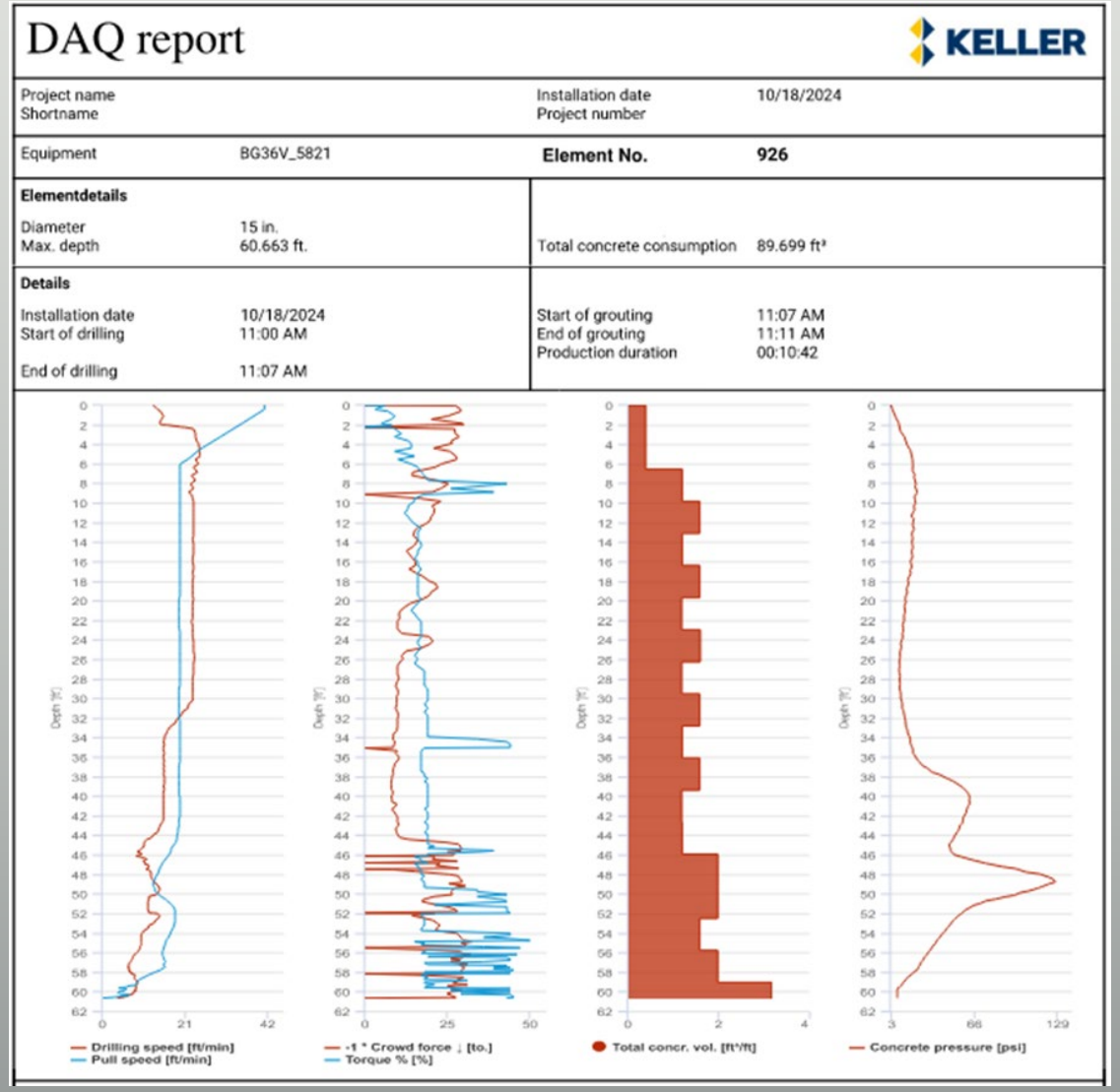


Quality Assurance:
QC Requirements for Rigid Inclusions



- Element Layout
- Element Diameter
- **Drilling Procedure**
  - **Depth**
- Material Properties
- **Grouting Procedure**

Measure While Drilling (MWD) Reports: Processed Installation Data



DAQ report

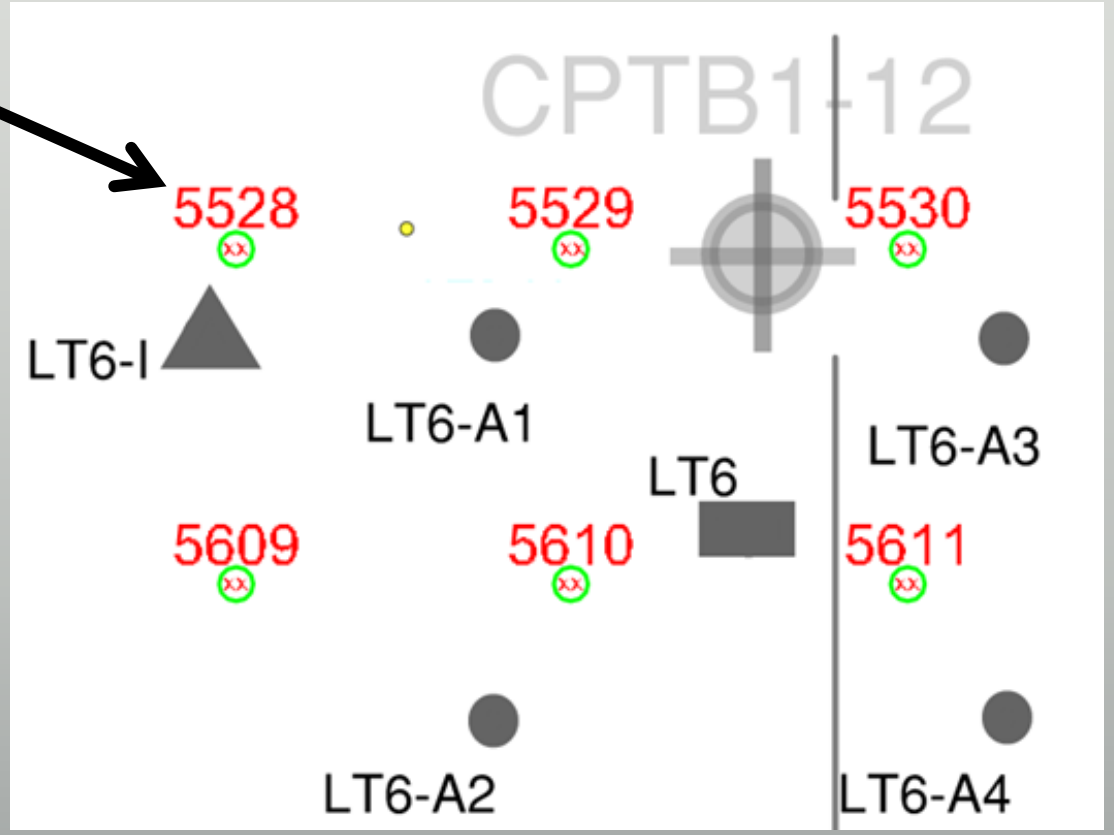
KELLER

Project name Shortname		Installation date Project number	10/18/2024
Equipment	BG36V_5821	**Element No.**	**926**
Elementdetails			
Diameter Max. depth	15 in. 60.663 ft.	Total concrete consumption	89.699 ft³
Details			
Installation date Start of drilling End of drilling	10/18/2024 11:00 AM 11:07 AM	Start of grouting End of grouting Production duration	11:07 AM 11:11 AM 00:10:42

- Date and time of installation
- Duration of installation
- Penetration time and depth
- Drill rig energy and force parameters (crowd force and torque)
- Grout pressure
- Withdrawal rate
- Grout volume placed

Calibration Program

Production Elements



Load Test Element Location

Load Test Anchor Element Location

Sacrificial Calibration Element Location

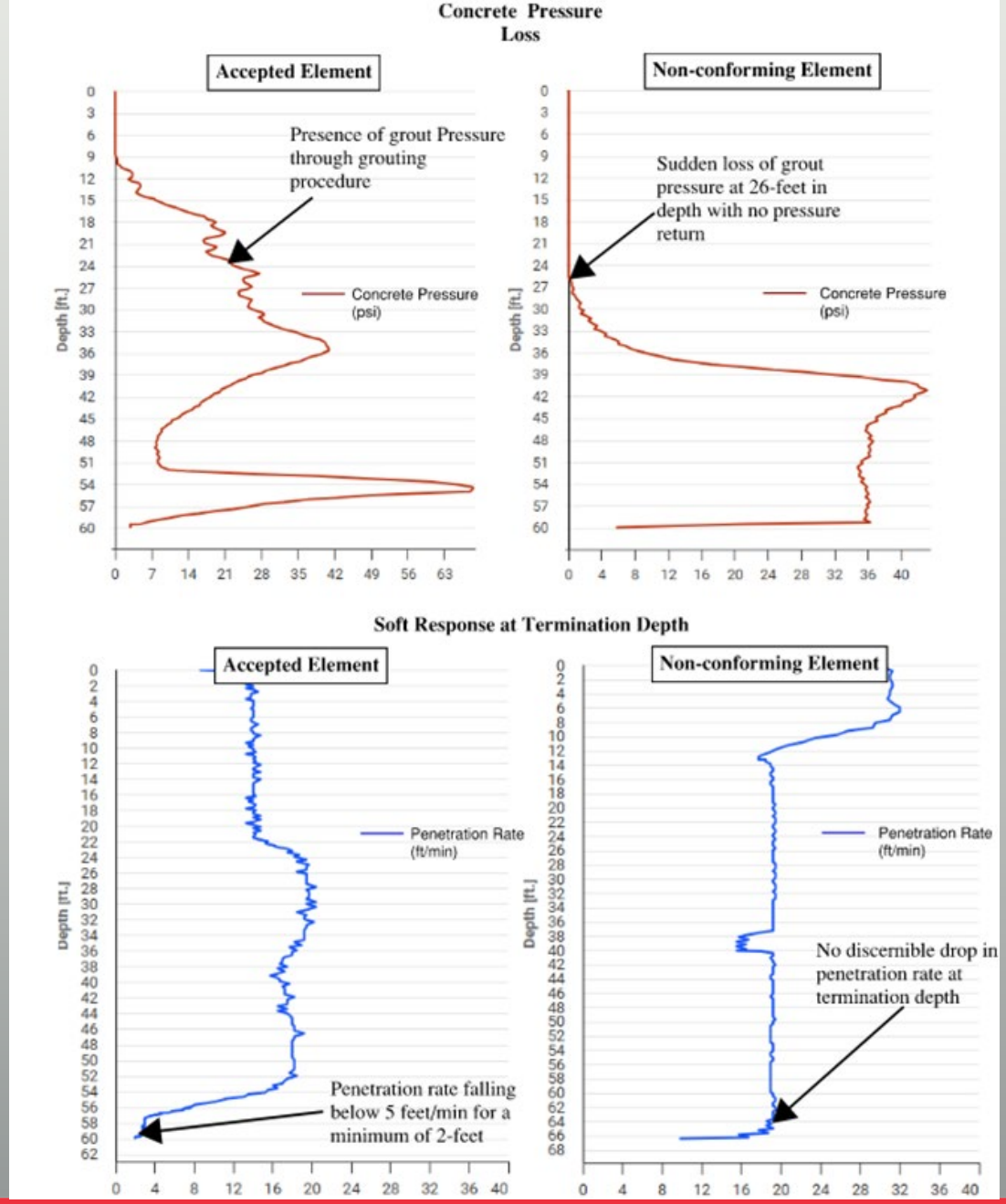
- Ensure soil conditions match expectations
- Calibrate drill rig response, grouting procedure and grout cut-off methods for site specific conditions, grout plastic properties and equipment capabilities
- Install sacrificial elements prior to the test element and production locations.

Test Program

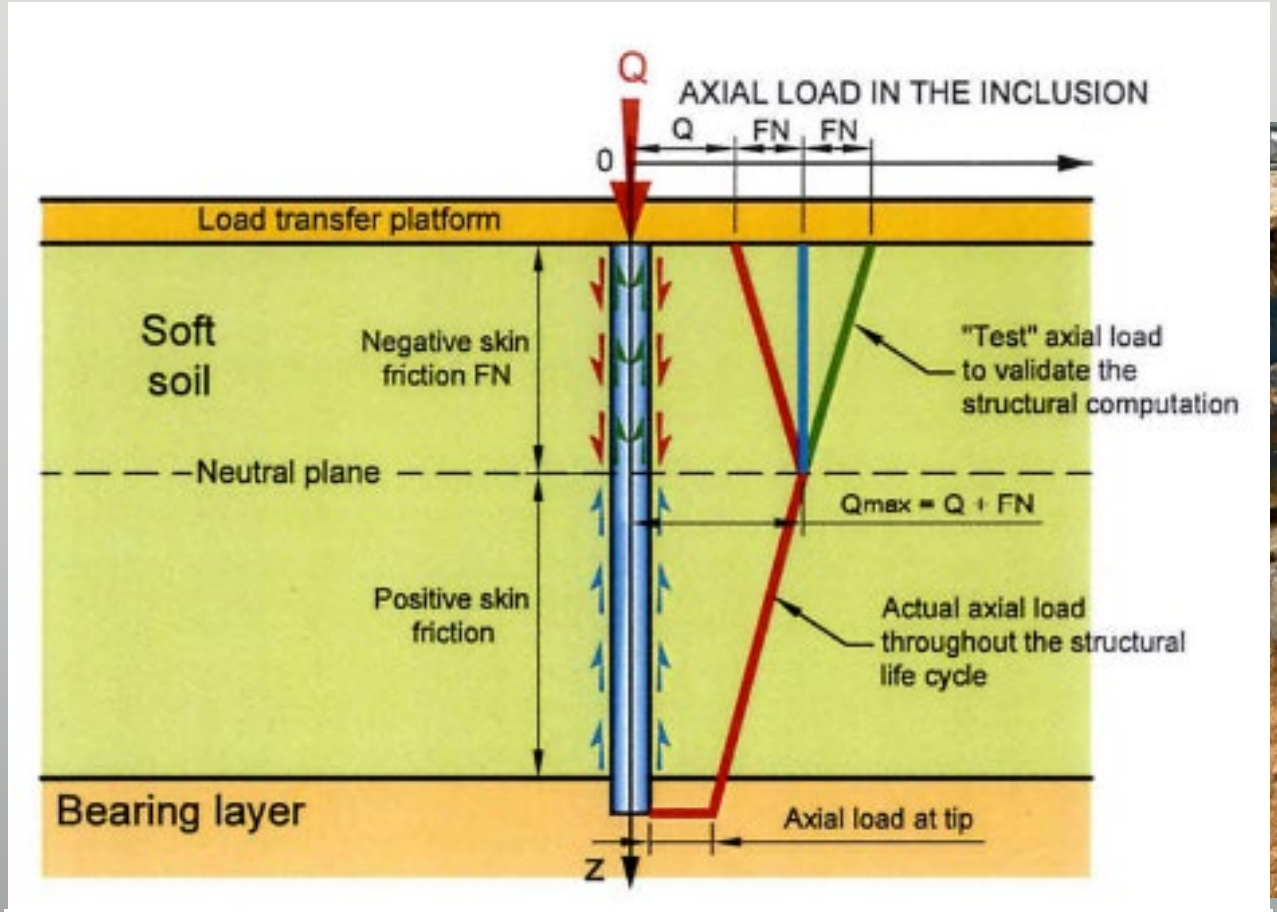
- What to cover?
  - Type of test matches risk for project
  - Load-advancement procedure
  - **Define acceptance criteria**
  - Load Test Frequency
  - Tests for element geometry and integrity



Acceptance Criteria



ASIRI Quality Test (AQT)





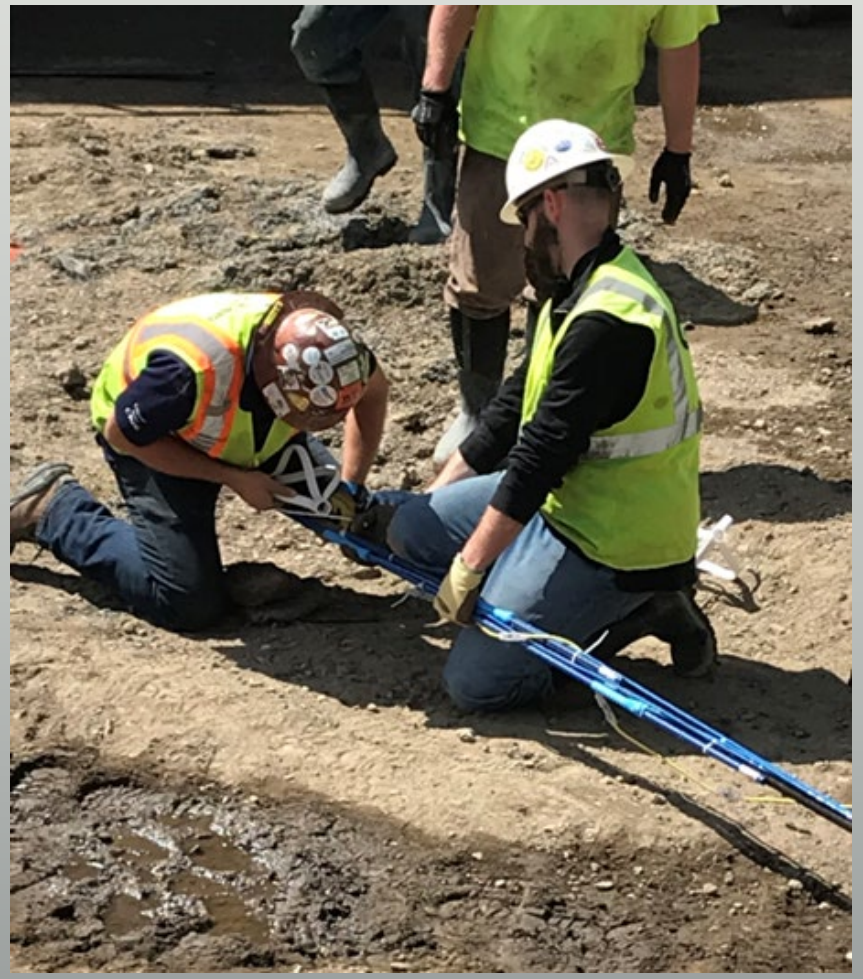
Service situation : Load at the inclusion head Q

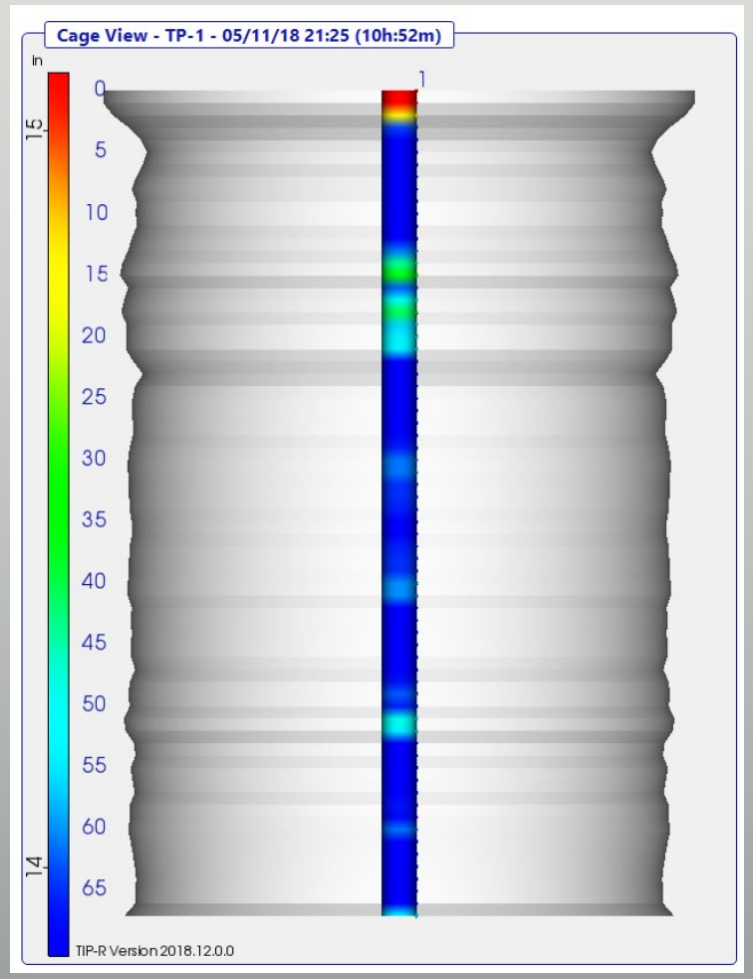
Maximum load $Q_{max} = Q_{ELS} = (Q + FN)$

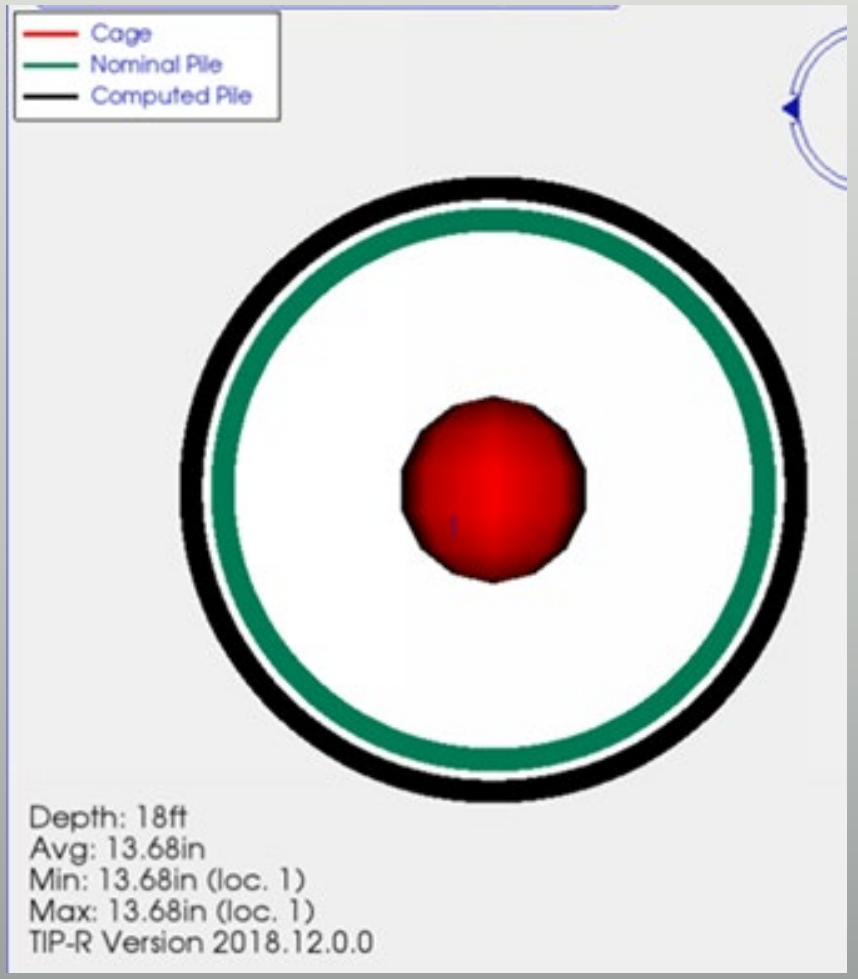
Quality test: Inclusion solely serving to reduce settlement

Test load $Q_{ELS} + FN = Q + 2FN$

Thermal Integrity Profile (TIP) Test



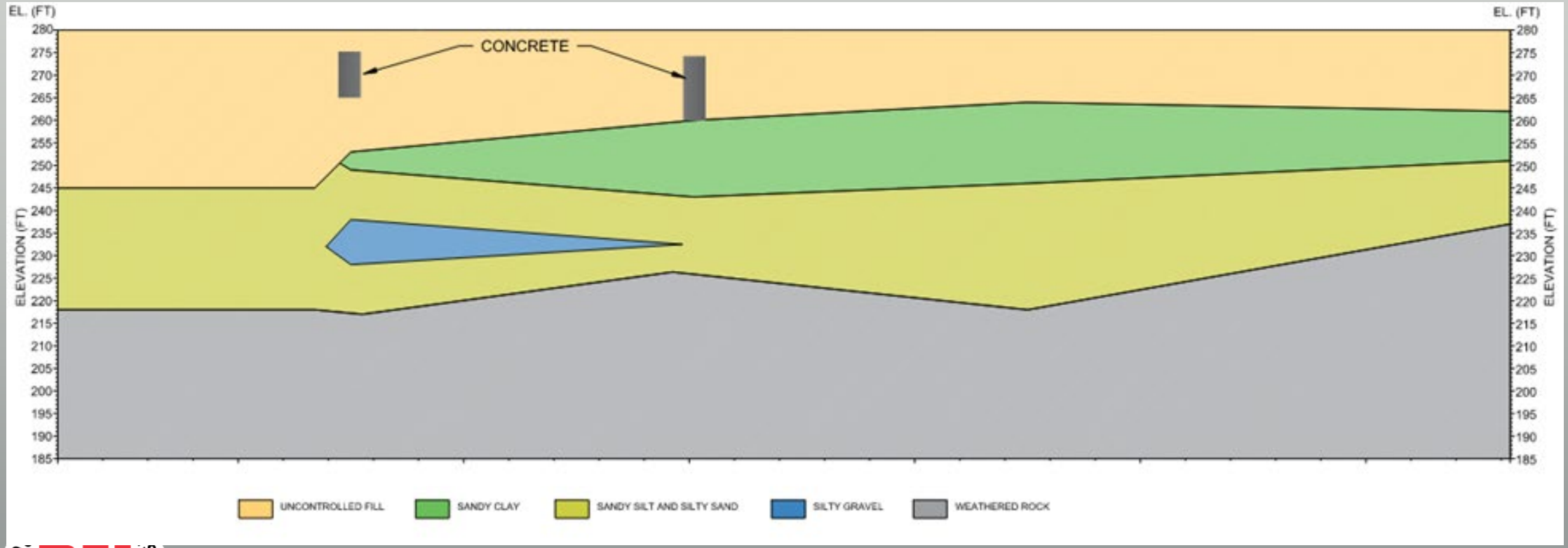




Project Summary: Bethlehem, PA



- **Design Considerations**
  - Large Area Load ≈2000 psf
  - 1-inch total / ½-inch differential settlement
  - Uncontrolled fill
  - Soft clay
  - **<u>Obstructions</u>**
- **Quality Considerations**
  - Loss of grout into voids (bulging/necking)
  - Early refusal on obstructions in fill
  - Relocating elements



Site Challenges

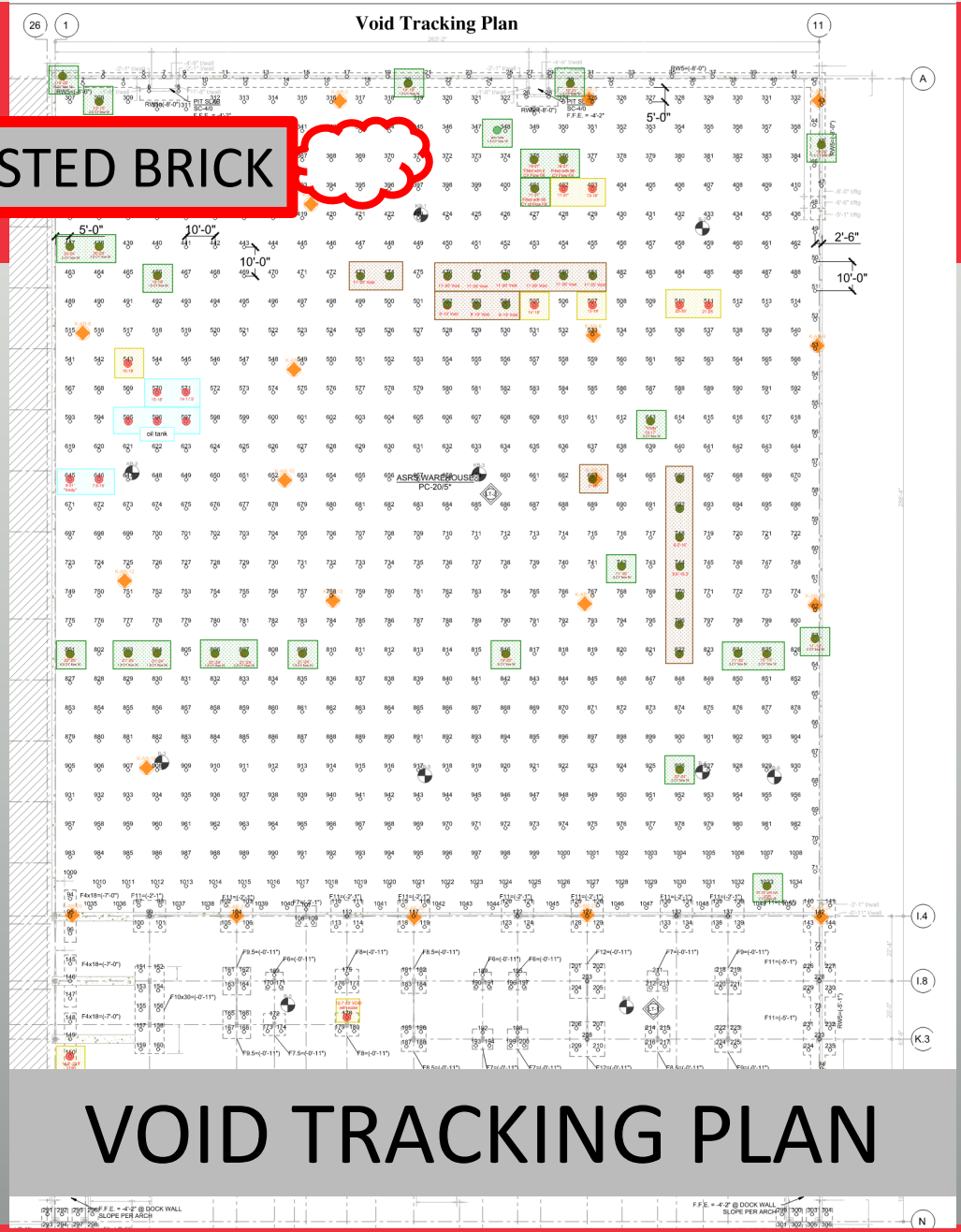




Site Challenges

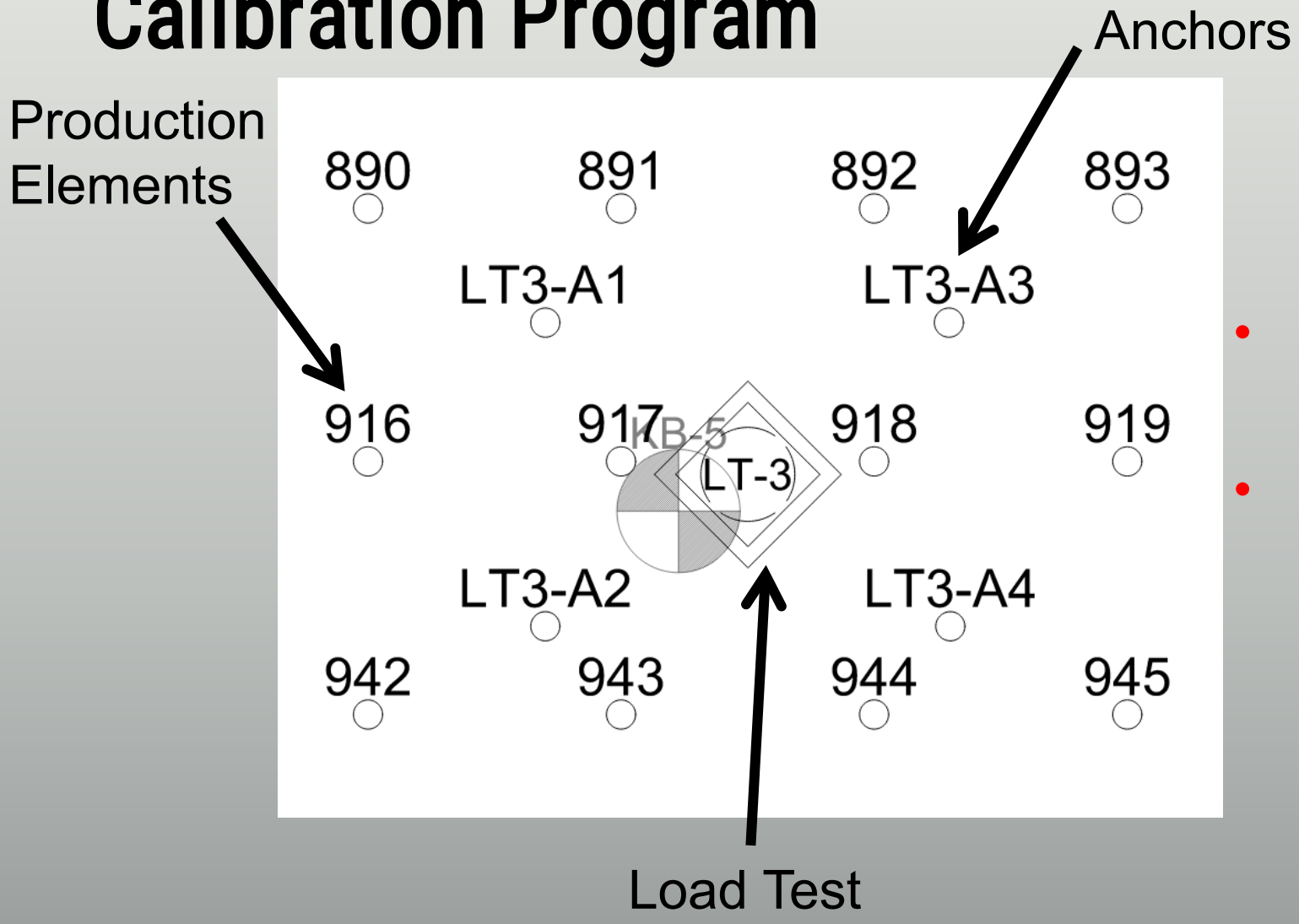


NESTED BRICK



VOID TRACKING PLAN

Calibration Program



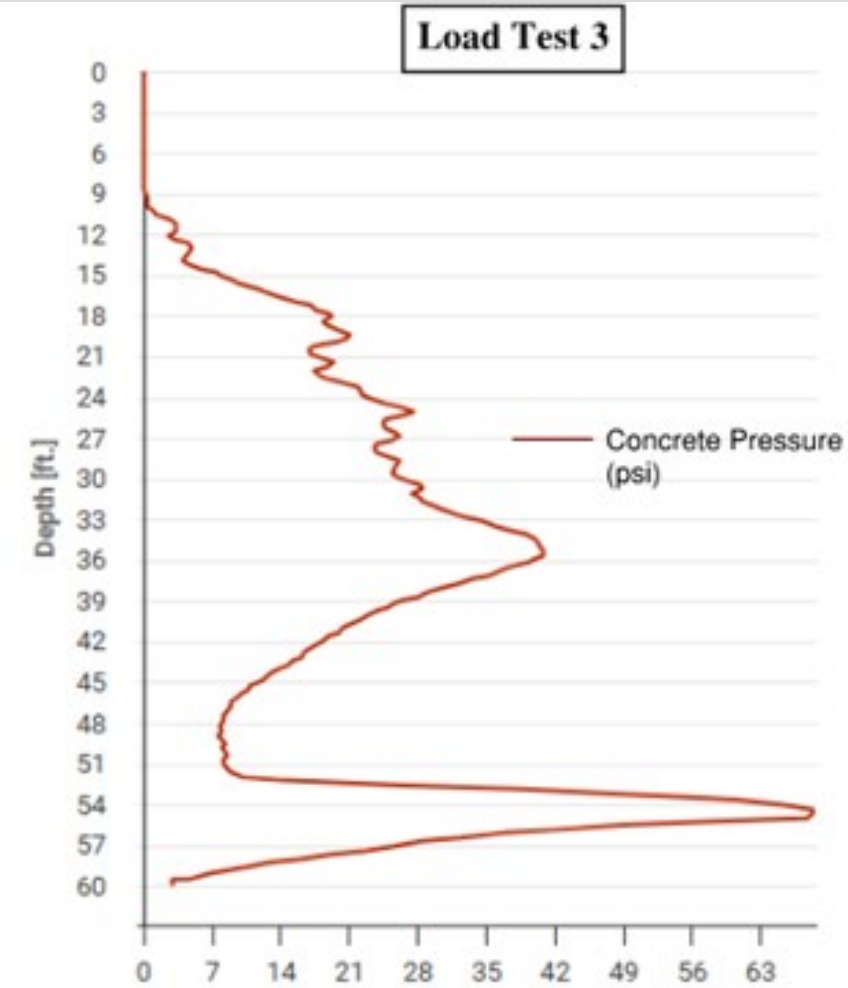
- Everything had to be predrilled…
- DAQ System: The relationships between drilling parameters, soil conditions and element capacity provide a platform for analytic evaluation of variability in soil stratigraphy….

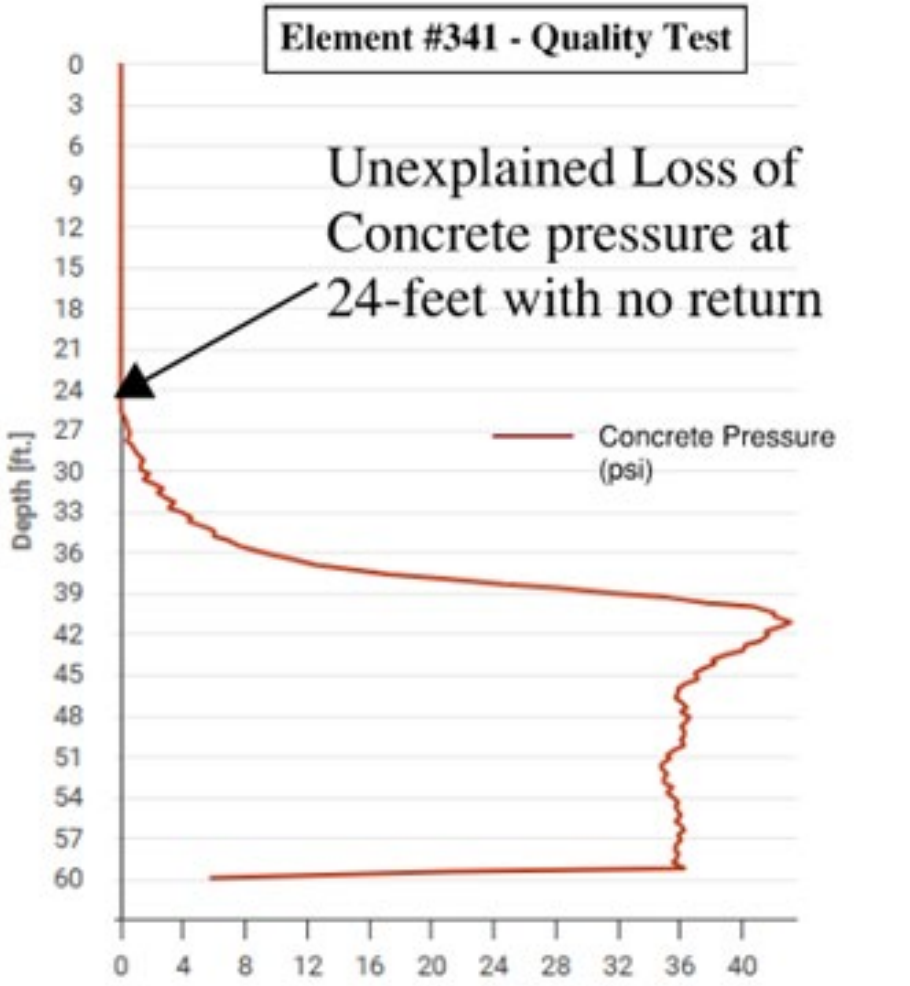
Test Program

- 6 Tests
  - 3 Single-element Tests
    - Confirm geotechnical capacity
    - Evaluate load-deflection behavior
  - 2 ASIRI Quality Tests
    - Confirm performance at design load
    - Reduce nonconformance
  - 1 TIP Test
    - Confirm diameter

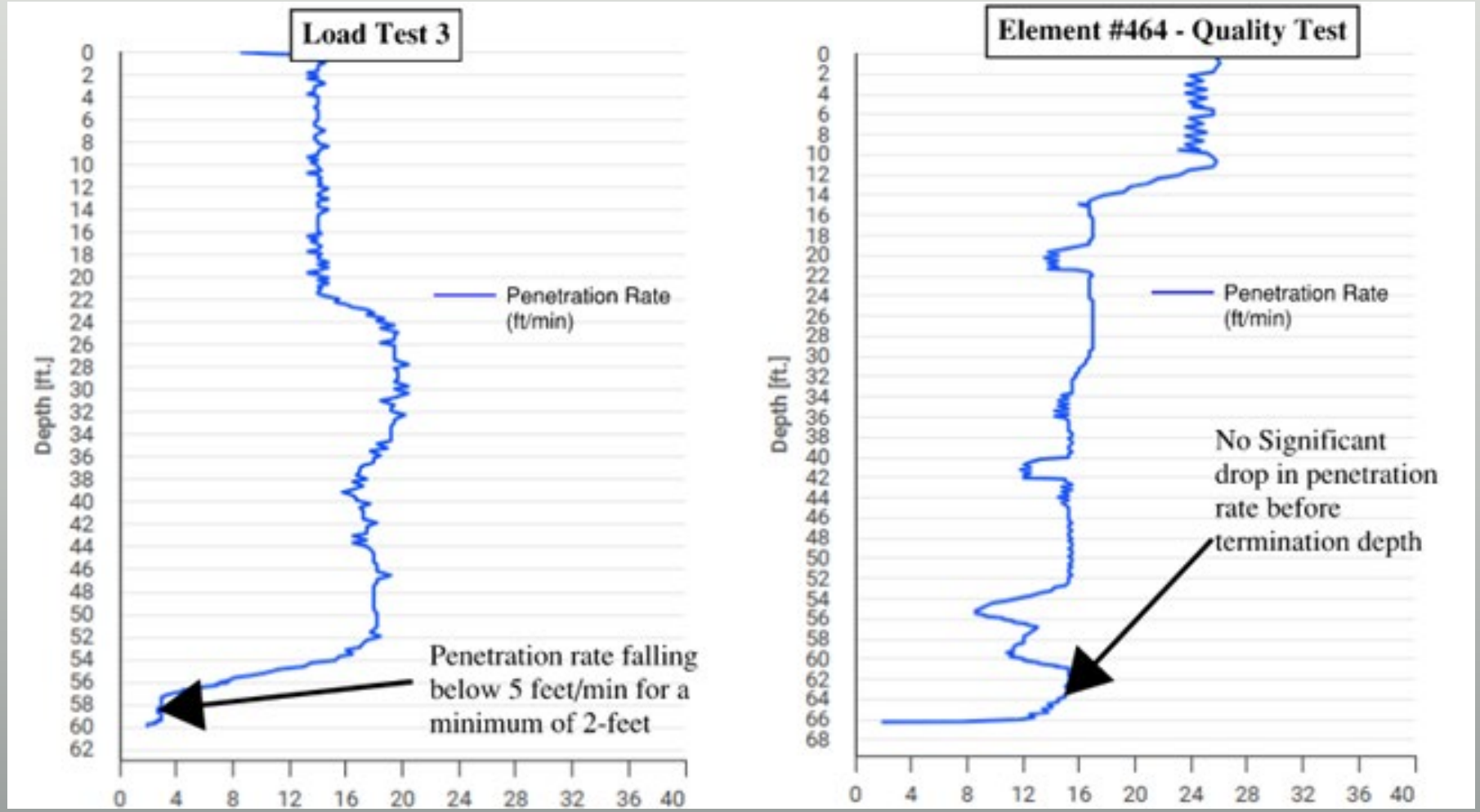


Identify Non-Conformance

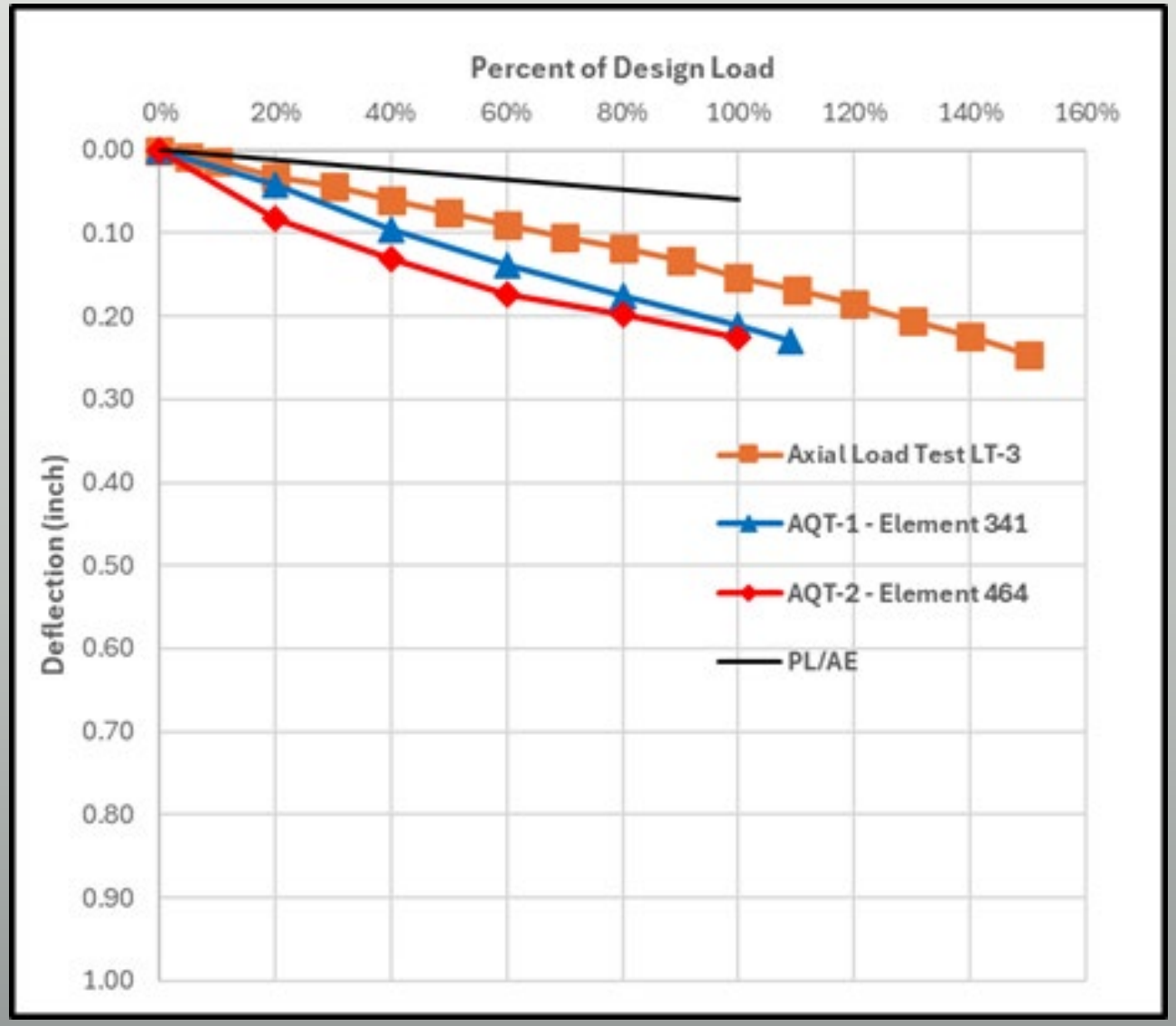




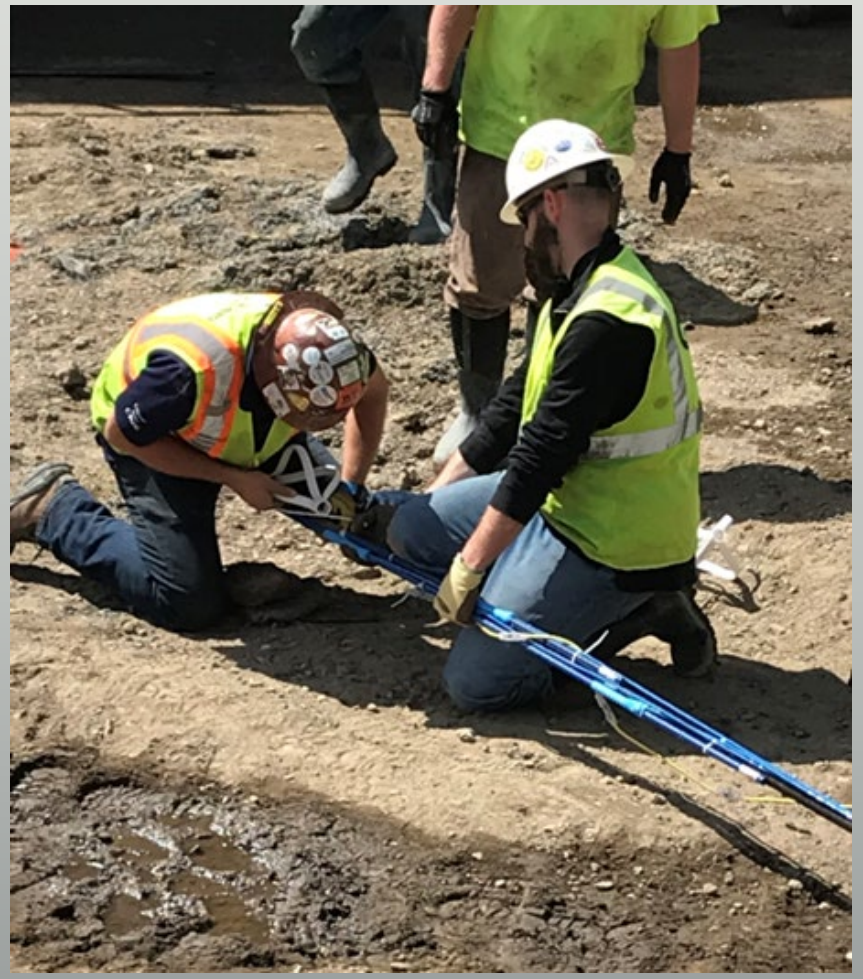
Identify Non-Conformance

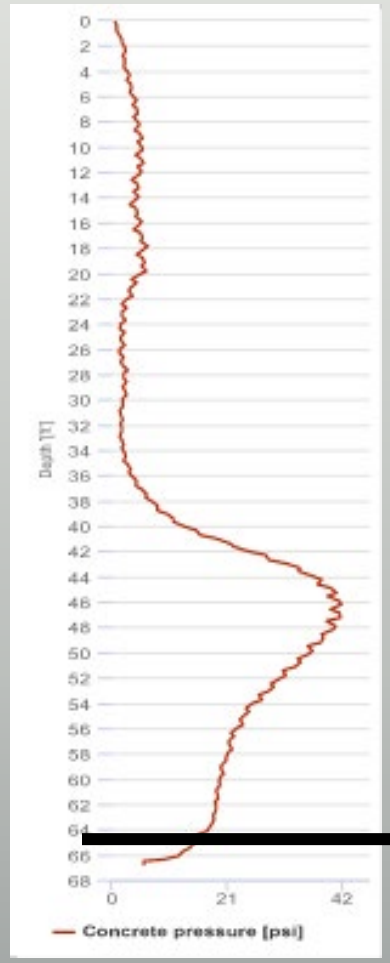


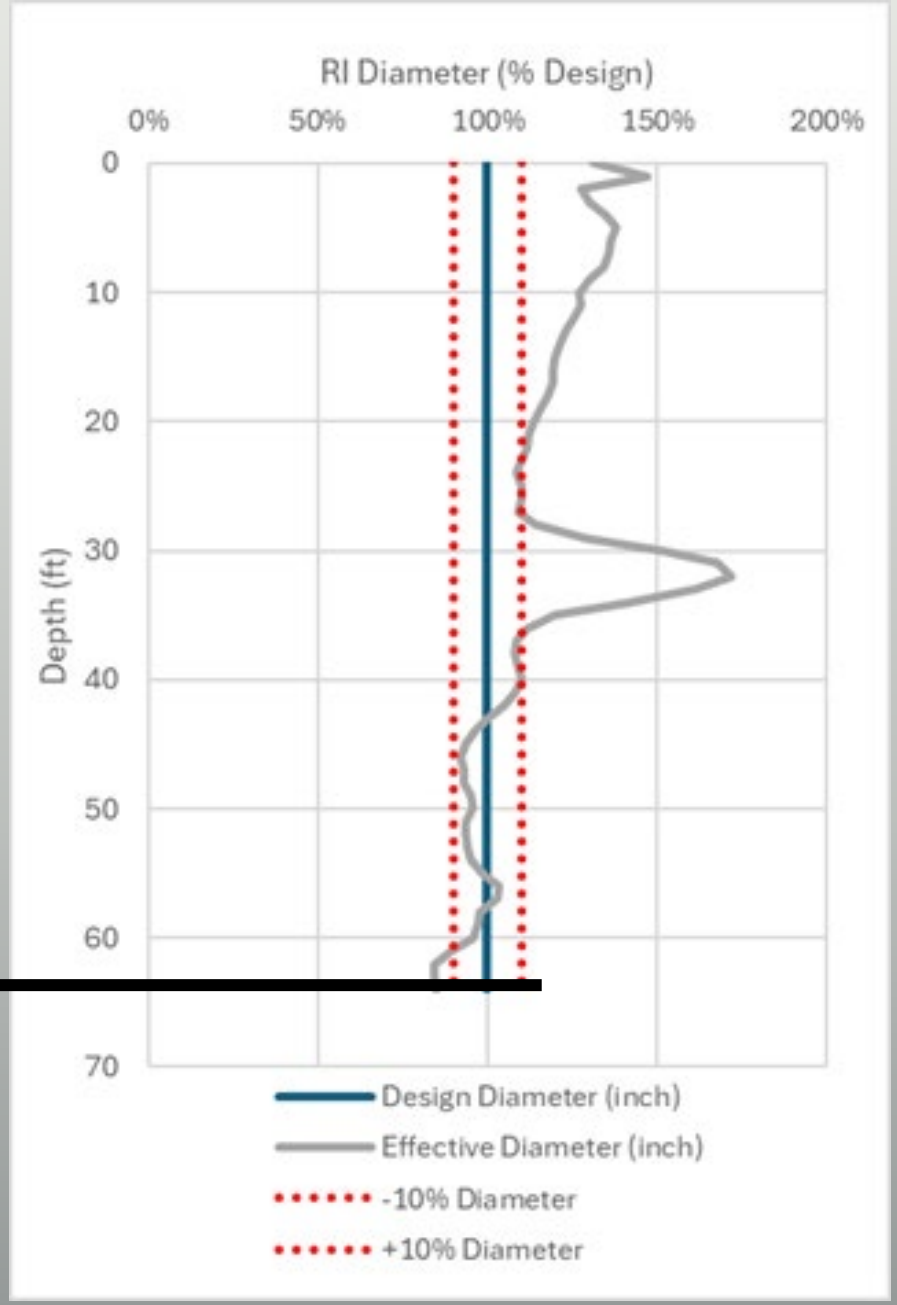
Test Results - AQTs



Test Results - TIP



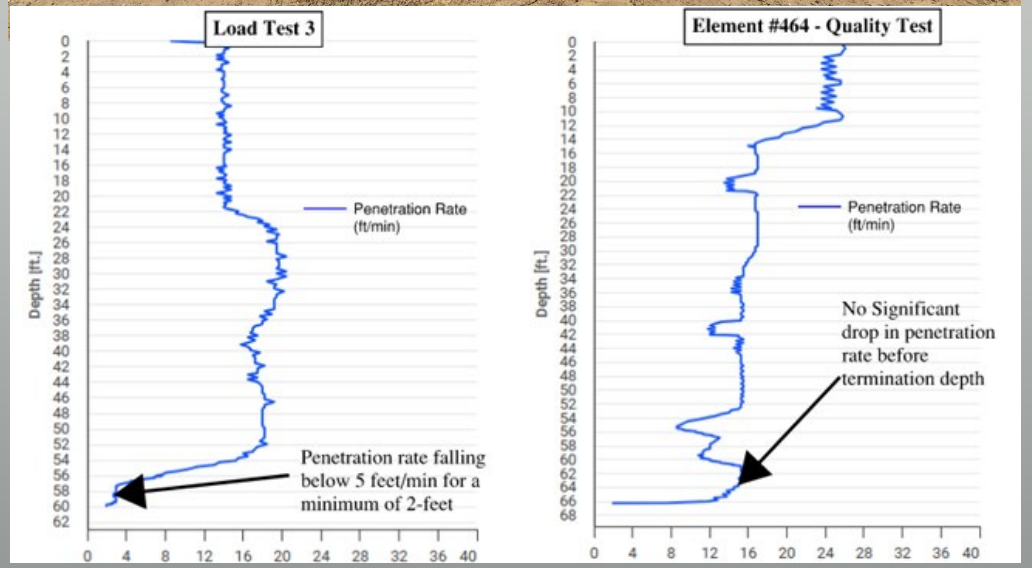




Summary and Conclusions

- Have an expectation of what **“good”** looks like.
- Measure While Drilling (MWD) data offers valuable insights during installation.
- Deviations from acceptance criteria do not necessarily indicate unacceptable performance
- Supplemental tools like the ASIRI Quality Test and Thermal Integrity Profile Test can be used inconjuction with MWD data to assess RI performance more accurately and reduce unnecessary rework.
- A proactive approach to quality on a challenging site demonstrated how testing can be used to reduce rework.







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Thank You!

Question and Response

THINK SAFE



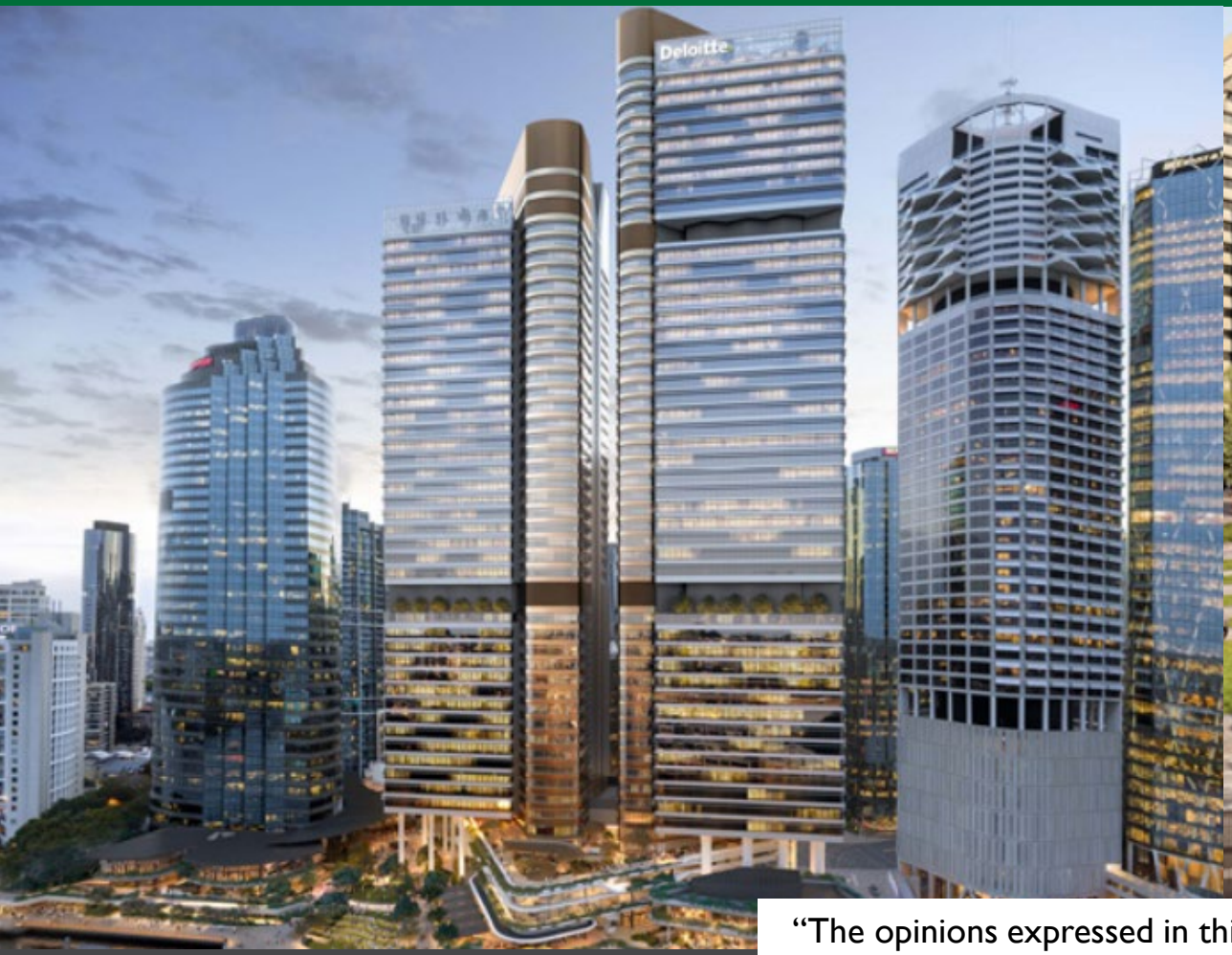


DEEP FOUNDATIONS INSTITUTE
DFI®

Waterfront Brisbane, Australia

David Cunliffe, EDG Consulting Pty Ltd, Brisbane, QLD, Australia
Daniel Schutt, A-Squared Studio Engineers Pty Ltd, Brisbane, QLD, Australia

EDG consulting





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TYPSAGroup TYPSA ENGINEERING SERVICES

Waterfront Brisbane – Project Location



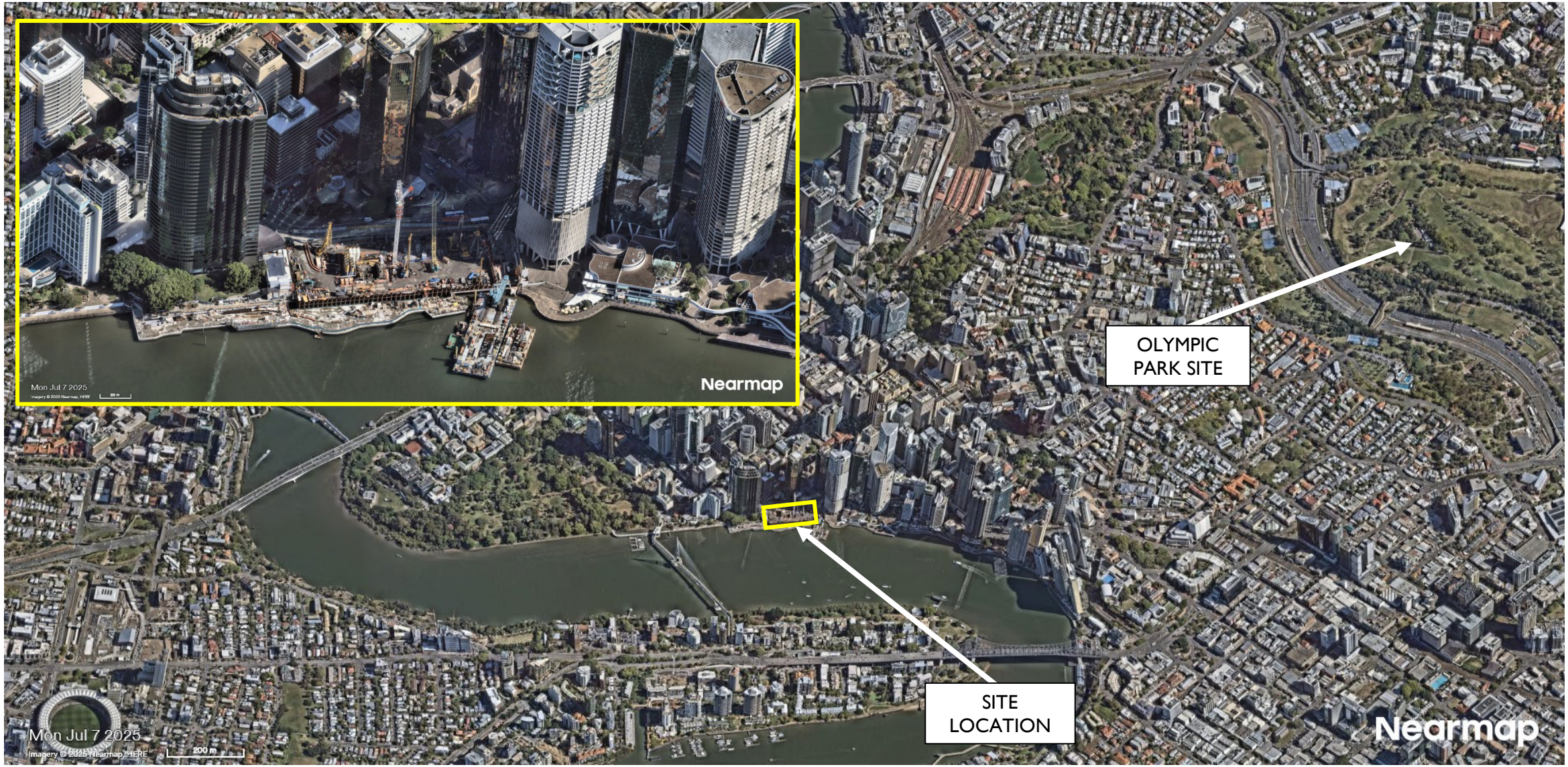
Brisbane Predicted Growth

- QLD has the highest population increase over the previous 5 years, compared to other states
- High overseas and interstate migration
- Most migration is focussed on southeast QLD, in areas such as the Gold Coast, Sunshine Coast and Brisbane
- Brisbane is the host city of the 2032 summer Olympic and Paralympic games

Brisbane 2032
Olympic Games Host
Queensland

Brisbane 2032
Paralympic Games Host
Queensland

Waterfront Brisbane – Project Location



TYPSA

Construction Sequence

Waterfront Brisbane – Construction Sequence







- **Pre-existing Condition:** Three-level building including a single-level basement.
- **Demolition:** From barges and from existing structure.
- **Temporary works:** Steel Combi-wall, 6m deep Mass Soil Mixing (MSM), 17m deep, 2.2m diameter Deep Soil Mixing (DSM) columns and piling platform.

Waterfront Brisbane – Construction Sequence





Installation of d-wall and barrettes: Installed using grab excavators (within the soil strength materials) and hydrofraise cutters (within the rock strength materials). The excavations are supported with a polymer support fluid.

Establish jump form and advance north tower: The jump form is established allowing the north core to progress from ground level in advance of the basement excavation

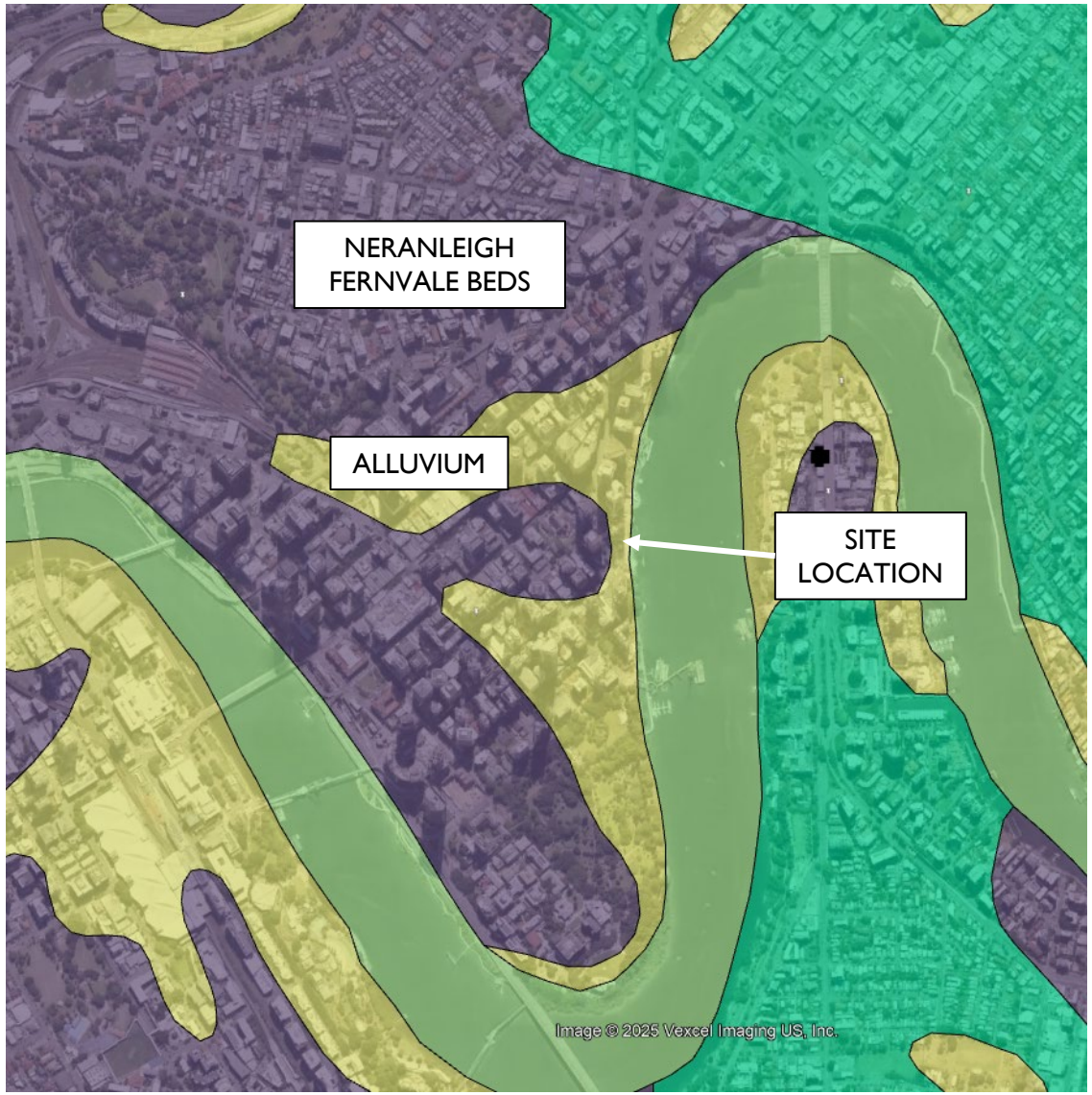
Piling: Piles are installed to support the tower and basement columns as well as the south tower core. Pile excavations supported by polymer fluid

Excavate basement: Excavated in stages with lateral support provided by the permanent basement slabs as the excavation progresses. A temporary access hole located over the south tower core provides access for removal of excavation spoil, requiring temporary anchors to support the diaphragm wall.

Advance South tower and construct podium levels: A pile cap is constructed to connect all south tower core piles, allowing the south tower to progress and the basement and podium levels to be completed.

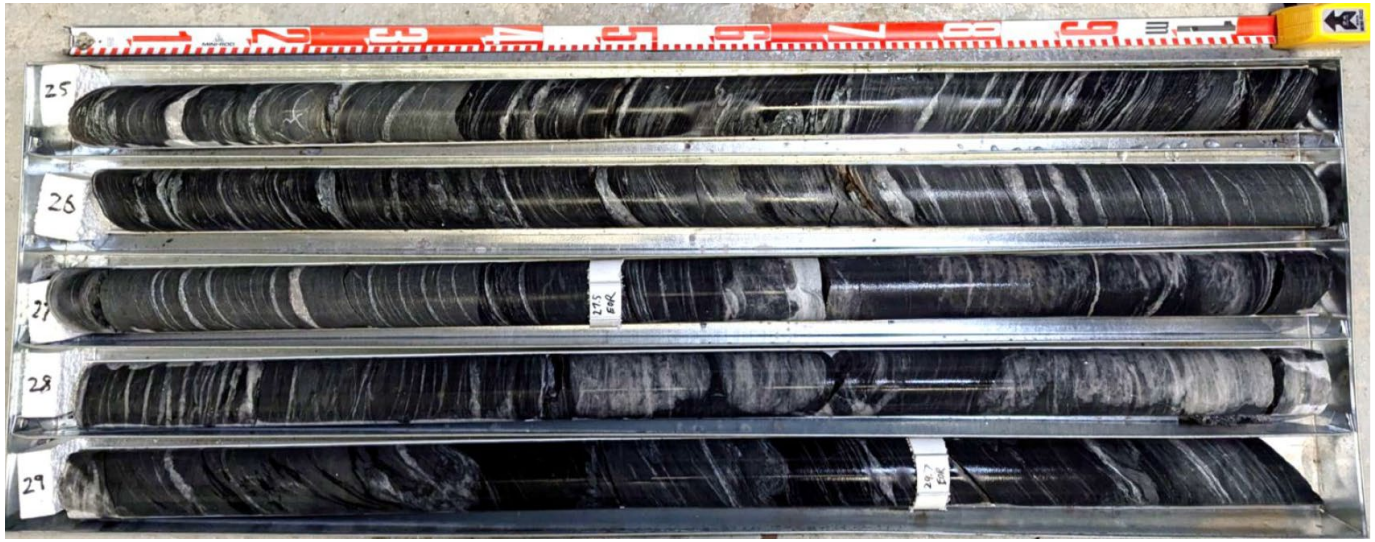
Ground Conditions

Waterfront Brisbane – Ground Conditions

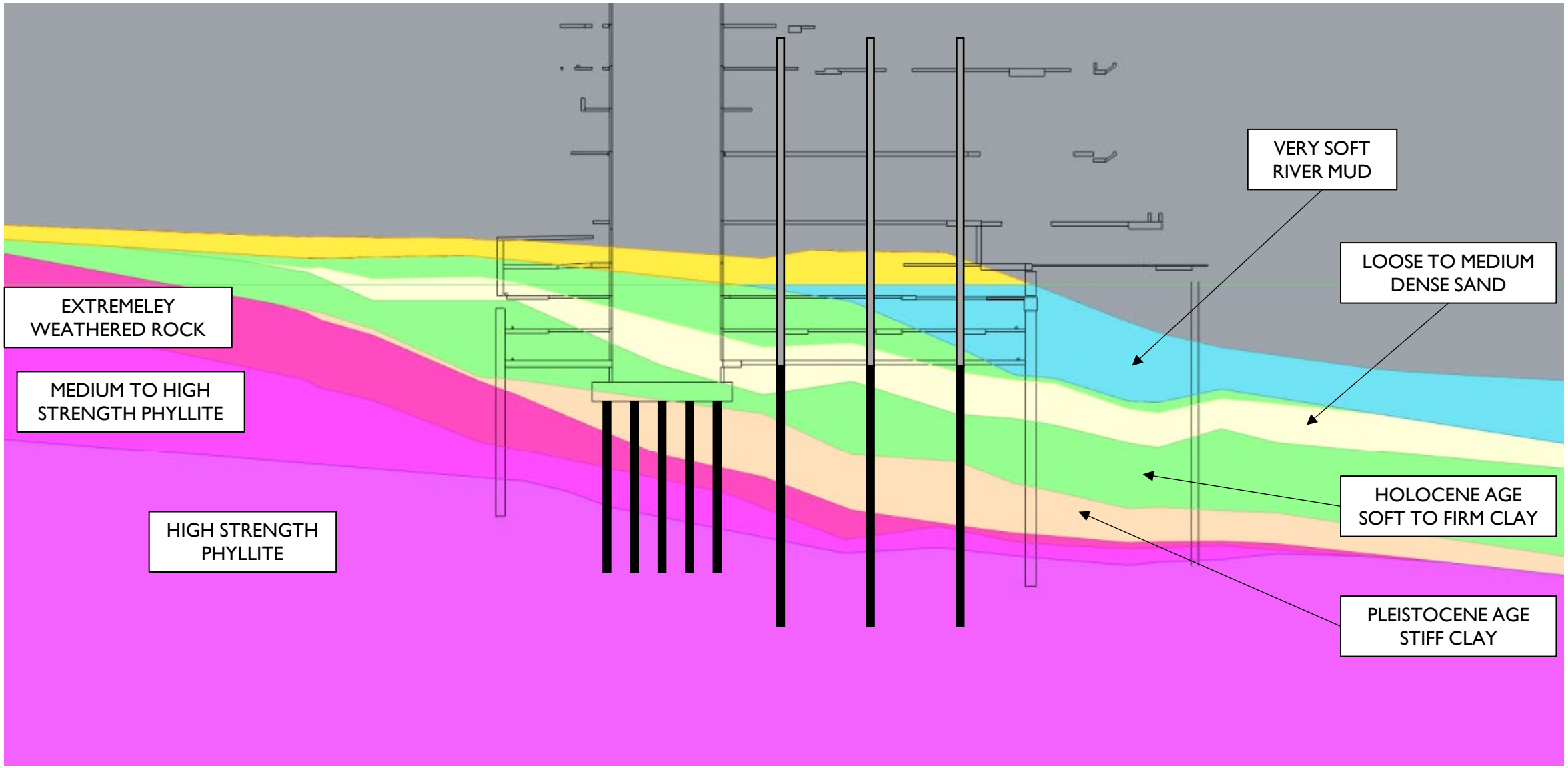


Material Descriptions

- Soils are typically clays and very soft to firm (Su = 5kPa to 35kPa or 1PSI to 5PSI)
- Fine to medium grained, medium dense sand exists between the clay layers
- Rock is typically described as Phyllite and/or Greywacke, distinctly foliated which generally dips between approximately 30° and 70° sub-horizontal, towards the northeast.

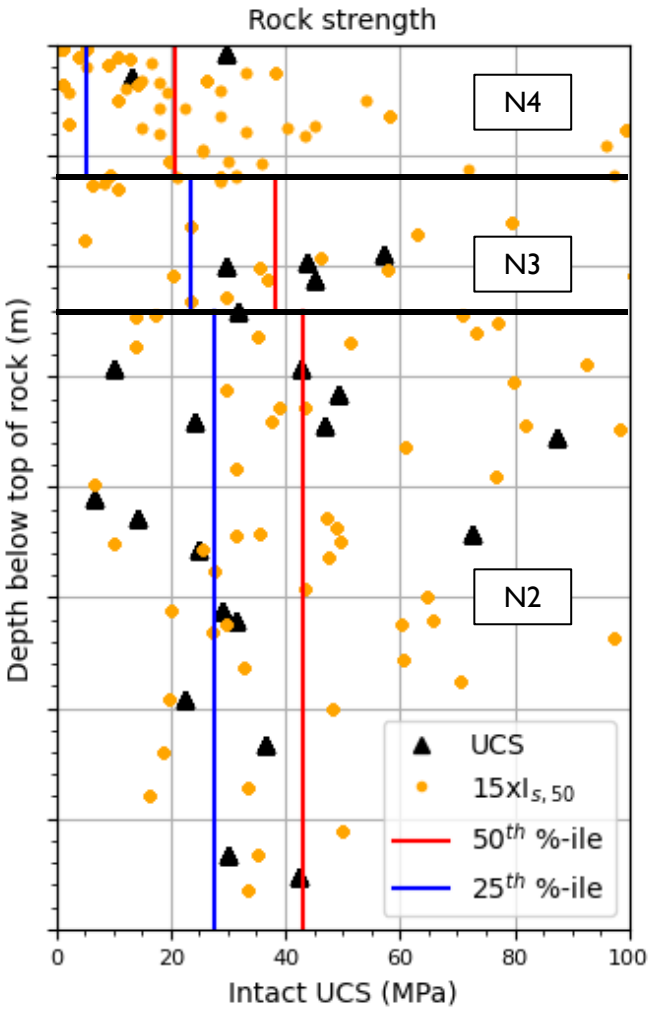


Waterfront Brisbane – Ground Conditions



TYPSA

Foundation Design



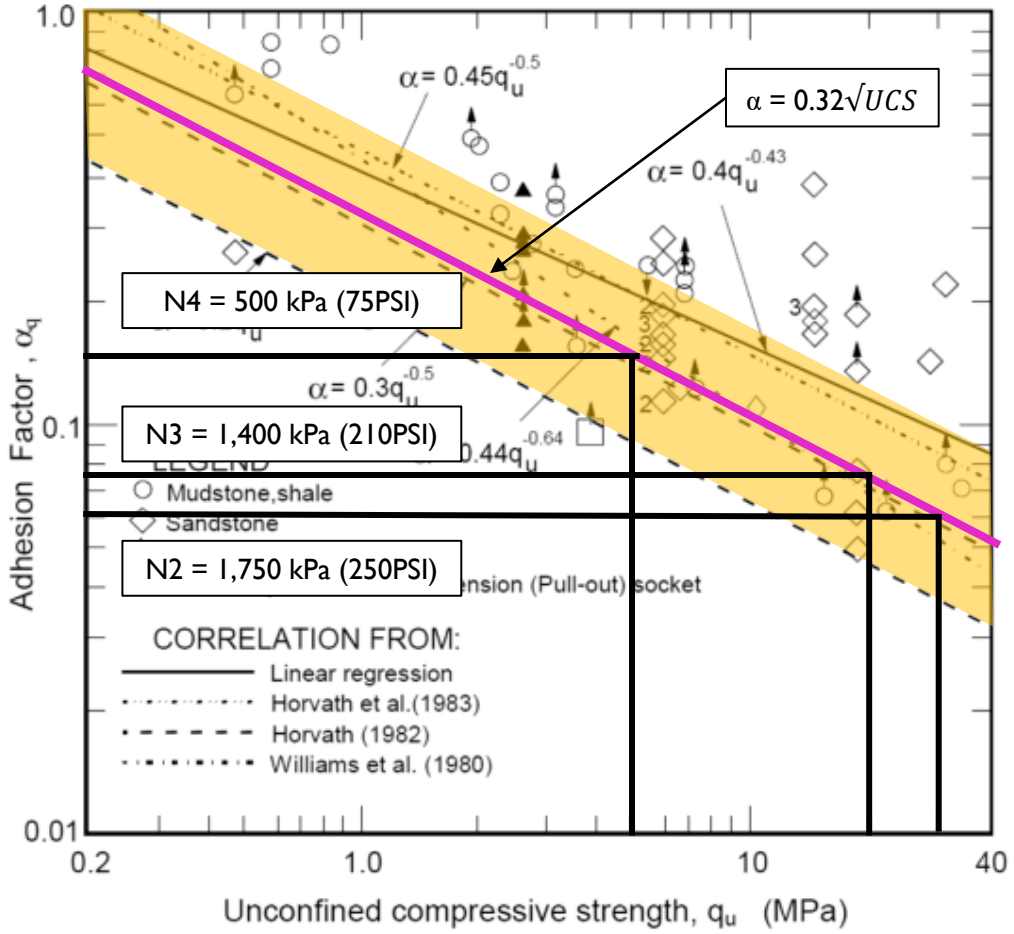
Rock Units

- N4 – Low Strength
  UCS = 5MPa (725PSI)
- N3 – Medium to High Strength
  UCS = 20MPa (2,900PSI)
- N2 – High Strength
  UCS = 30MPa (4,350PSI)

Skin Friction

$\alpha = 0.32$ was selected based on

- Anticipated socket roughness
- Recommended by Rowe & Armitage
- Level of redundancy in foundation system
- Agreement to carry out Osterberg Cell test to help verify foundation design parameters



Haberfield, C.M. and Lochaden, A.L.E., 2019. Analysis and design of axially loaded piles in rock. *Journal of rock mechanics and geotechnical engineering*, *11*(3), pp.535-548.

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Socket Roughness

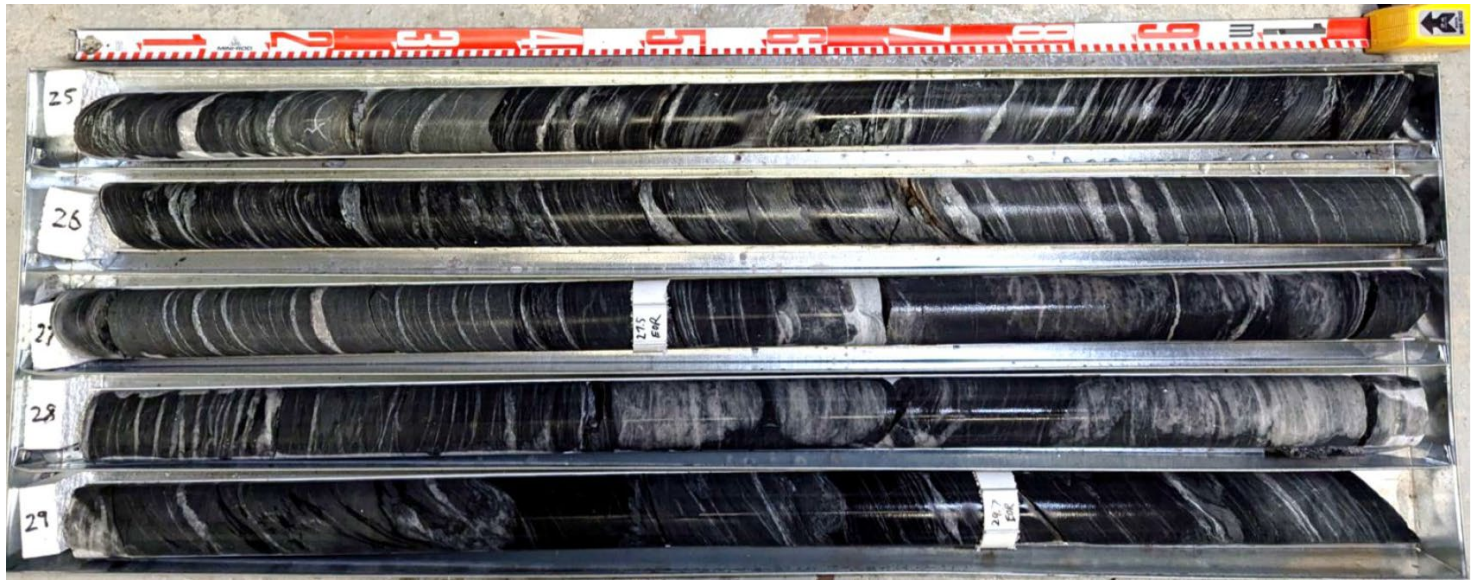
- Distinct foliation helps promote non-planar rough surfaces
- Pile excavations supported by polymer, reducing the likelihood of filter cake build up compared with bentonite
- Pile drilling observed by EDG including assessment of base cleanliness





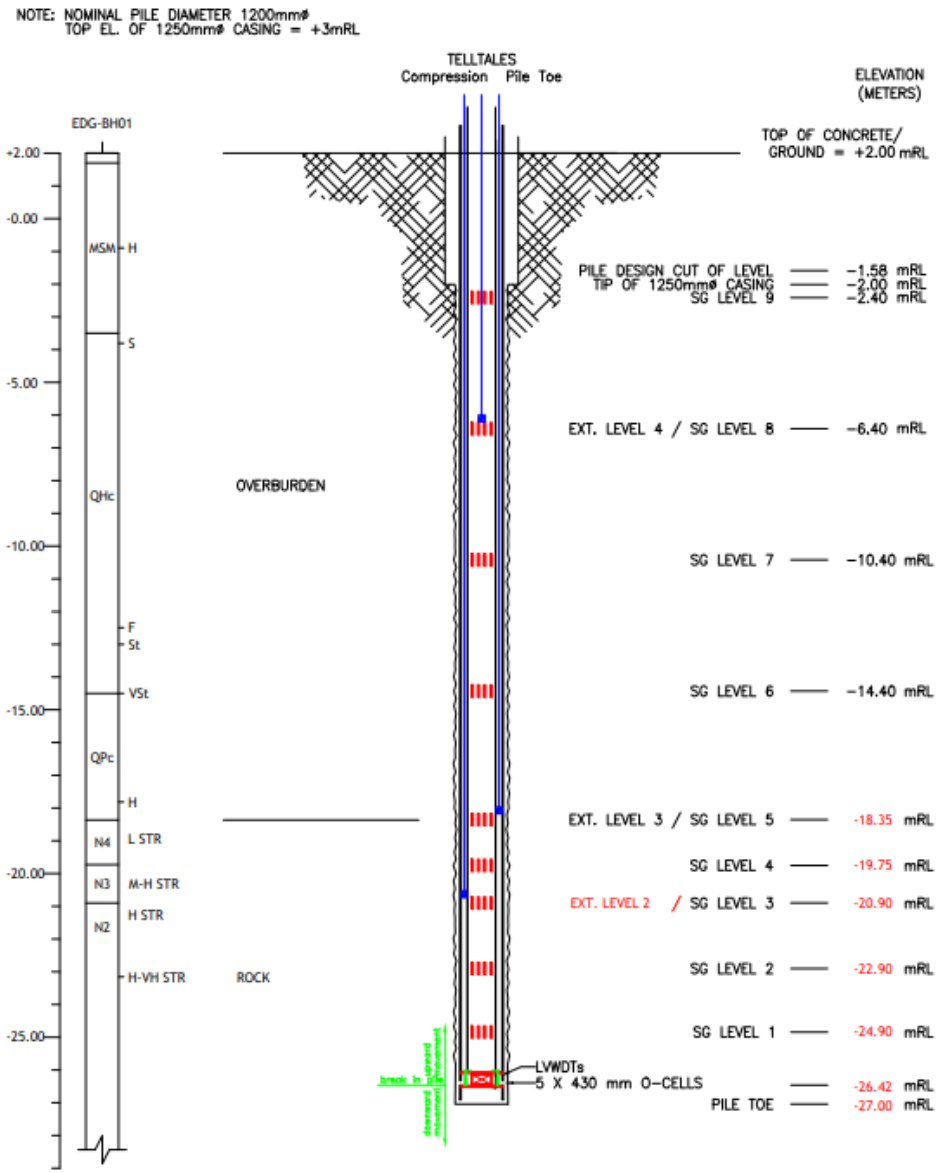






Osterberg Cell Testing and Interpretation

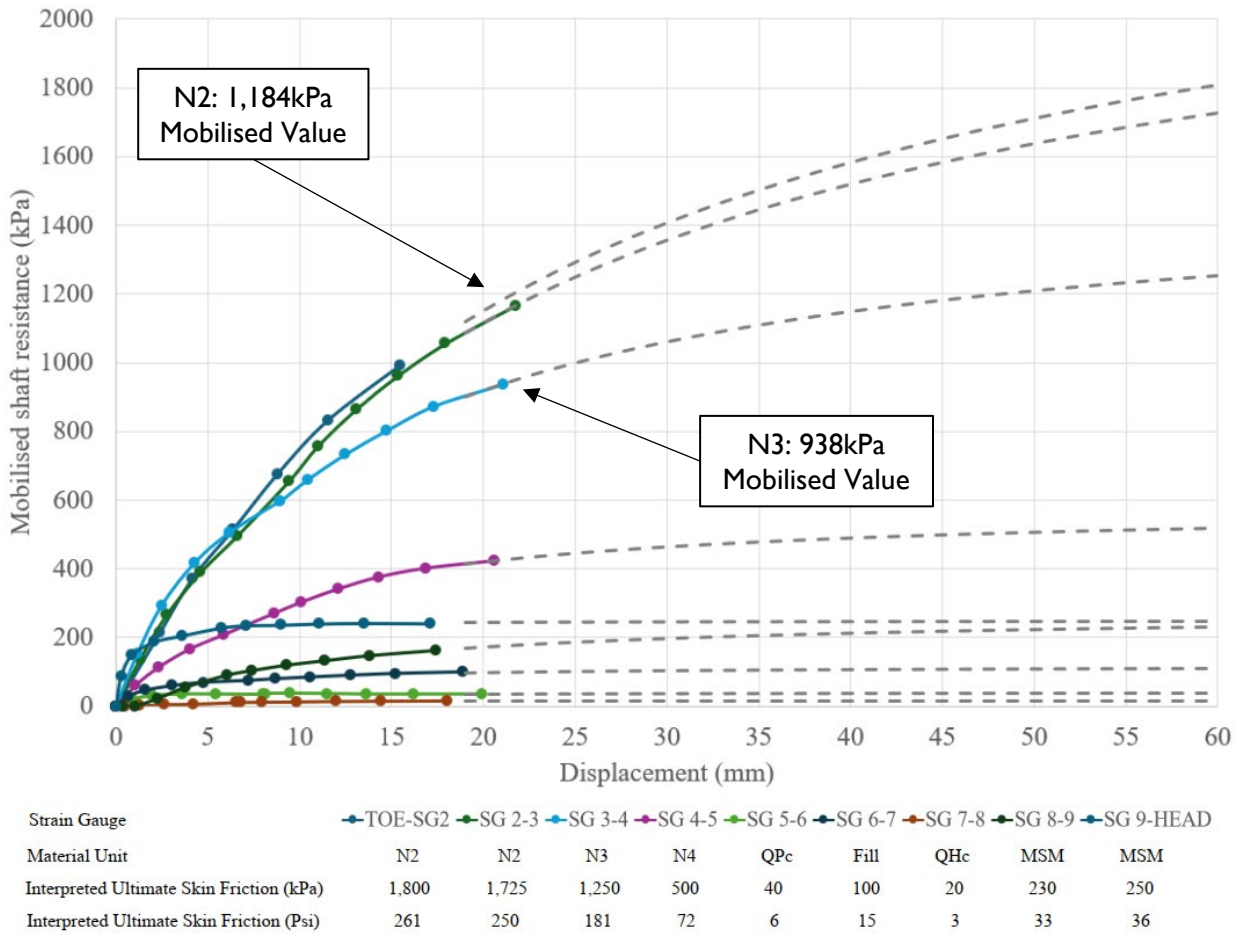
Waterfront Brisbane – Osterberg Cell Testing and Interpretation



Osterberg Cell Instrumentation

- 1.2m diameter pile
- Five 430mm diameter load cells
- Max load = 31.8MN (up and down)
- 9 levels of strain gauges positioned to assess all materials
- Strain gauge spacing closest at the base of the pile (nearest to the O-Cells)
- Telltales
- Extensometers





Strain Gauge	TOE-SG2	SG 2-3	SG 3-4	SG 4-5	SG 5-6	SG 6-7	SG 7-8	SG 8-9	SG 9-HEAD
Material Unit	N2	N2	N3	N4	QPc	Fill	QHc	MSM	MSM
Interpreted Ultimate Skin Friction (kPa)	1,800	1,725	1,250	500	40	100	20	230	250
Interpreted Ultimate Skin Friction (Psi)	261	250	181	72	6	15	3	33	36

Osterberg Cell Interpretation

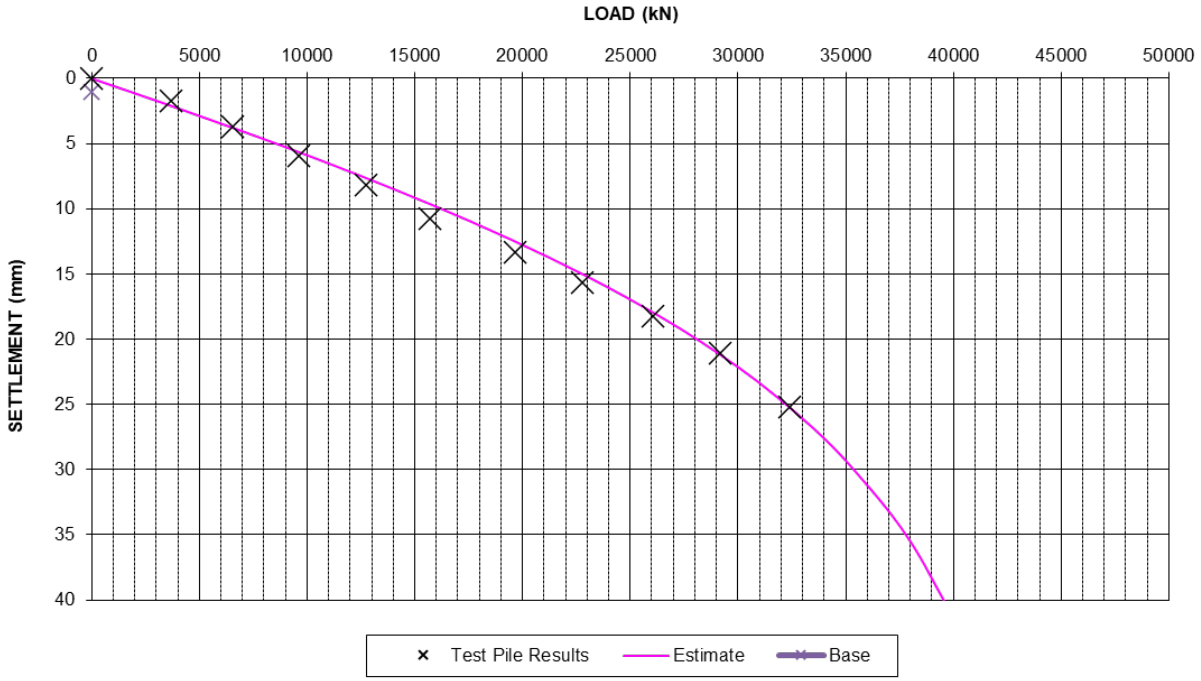
- Ultimate values of skin friction were mobilised in the soil strength materials
- Mobilised values of skin friction were not reached in the rock strength units
- Ultimate skin frictions were assessed via extrapolation using a non-linear least squares hyperbolic regression function

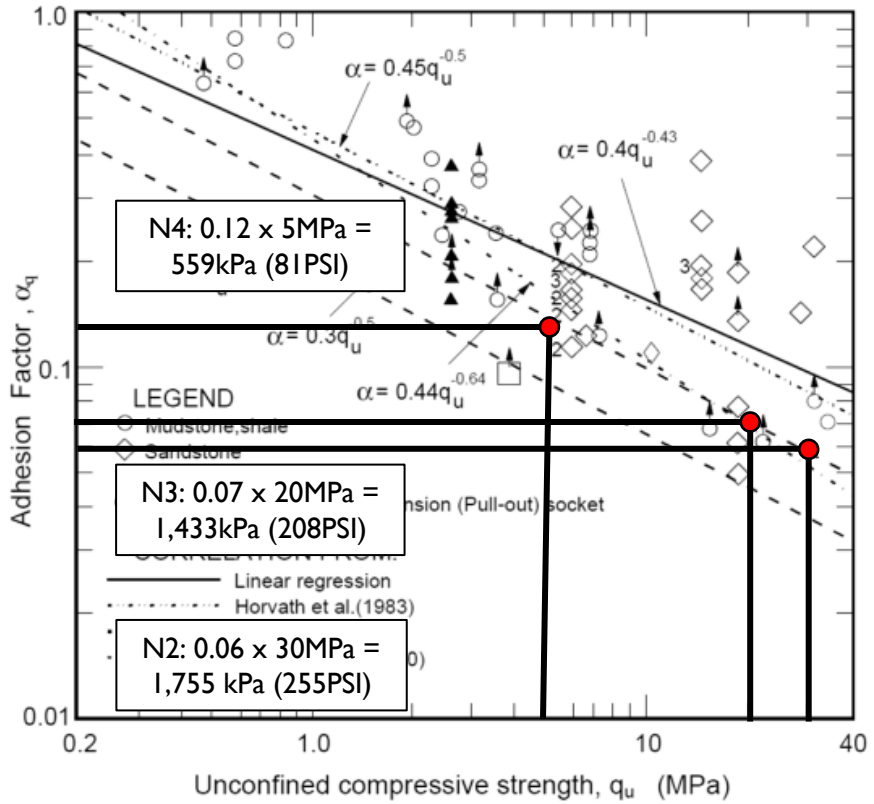
Unit	Shaft resistance (kPa)		
	Maximum mobilised	Design Values	Extrapolated Values
N4	425	500	520
N3	938	1,450	1,250
N2	1,184	1,750	1,800

Osterberg Cell Interpretation

- Engaged with Fugro Load Test to assess total ultimate pile (upward) capacity / skin friction
- Assessed the distribution of skin friction across all soil and rock units

Unit	Shaft resistance (kPa)		
	Maximum mobilised	Design Values	Interpreted Values
N4	425	500	559
N3	938	1,450	1,433
N2	1,184	1,750	1,755





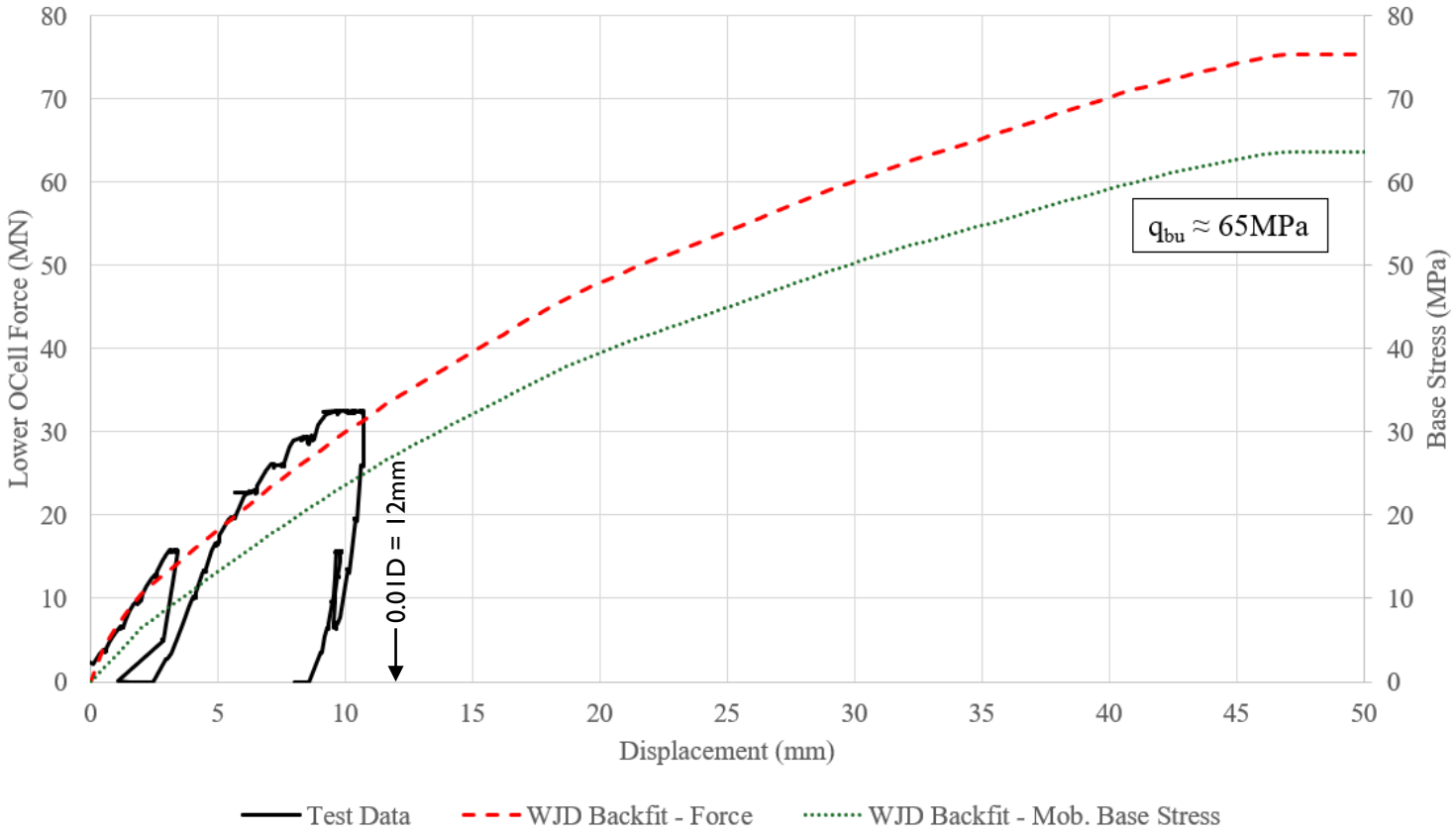
Reached Skin Friction = 35MN approx.

Ultimate Skin Friction = 50MN approx.

TYPSA

Osterberg Cell Interpretation

- Ultimate end bearing not reached due to high strength of rock mass and capacity of hydraulic jacks
- Test data was calibrated with a semi-empirical method by Williams, Johnston and Donald
- Mobilised base pressures in excess of 25MPa at <0.01D base displacement.
- Mobilised base pressure exceeded that expected under working loads.
- Interpreted ultimate end bearing exceeded design values
- Interpreted rock mass modulus was consistent with selected design values and Pressuremeter testing



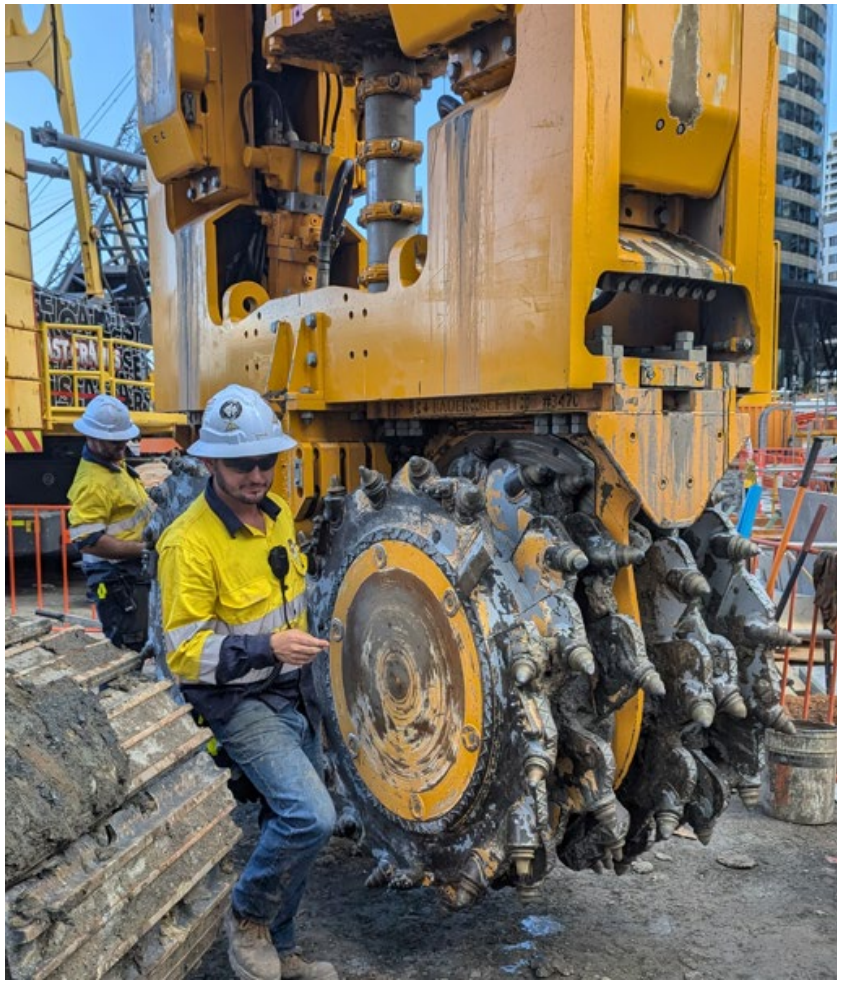
TYPSA

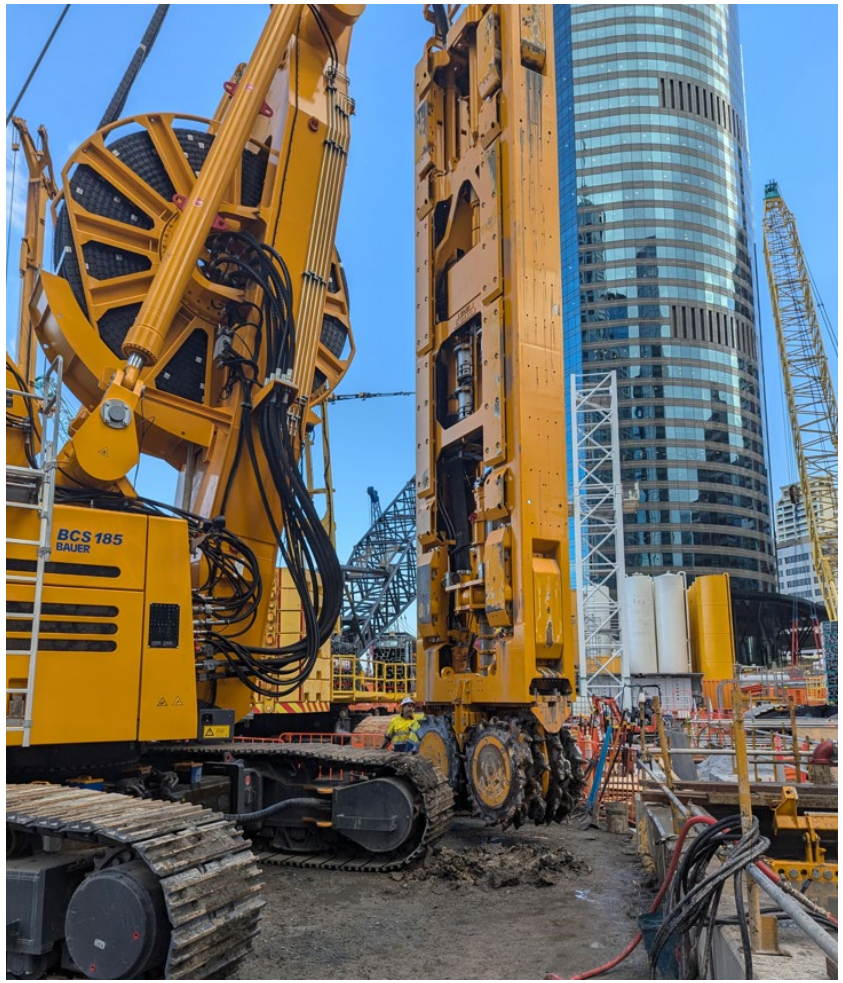
Conclusions

- Ultimate skin friction and end bearing parameters were required for the design of heavily loaded pile barrette foundations
- Initial parameters were selected based on our experience and recommendations in published literature
- "Best guess" values were selected on the basis that verification testing would be carried out using the Osterberg Cell method
- Mobilised values of skin friction were not reached in the rock strength units, however were assessed by:
  - Extrapolation using a non-linear least squares hyperbolic regression function
  - Estimation of total ultimate skin friction (assisted by Fugro Load Test) and assessment of the distribution of total skin friction across all soil and rock units
- O-Cell test results successfully verified design skin friction values
- Mobilised base pressure exceeded that expected under working loads
- Interpreted ultimate end bearing exceeded design values
- Interpreted rock mass modulus was consistent with selected design values and Pressuremeter testing

Waterfront Brisbane – Construction Phase Photos







Thank you

PARALLEL SESSION C2

MSSP Equipment & Testing

Understanding Vibratory Pile Driver/Extractor Force

Presented by John L White

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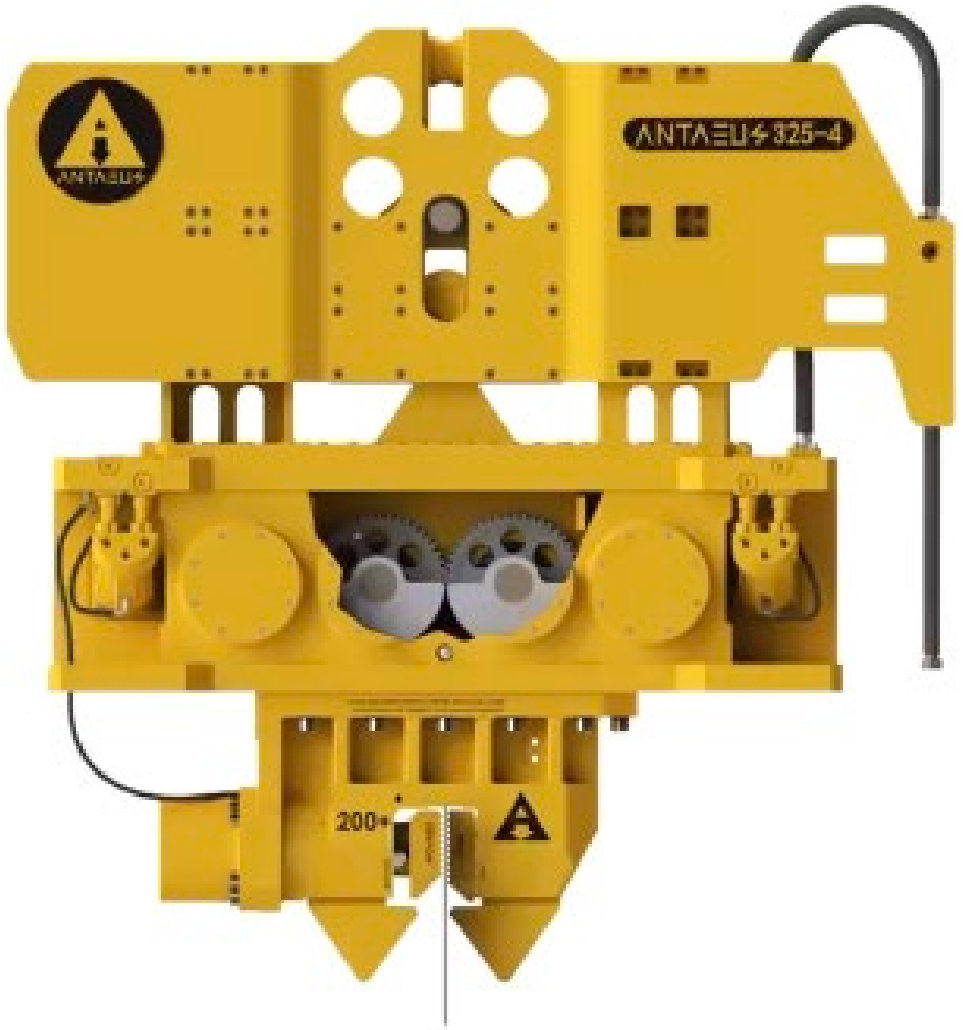
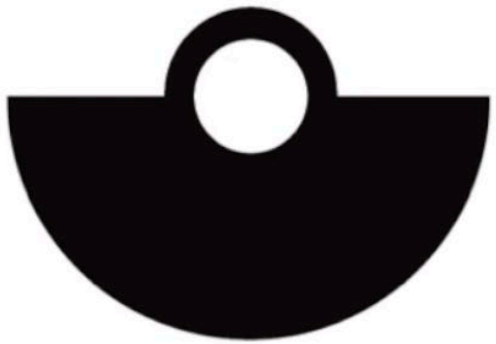
Vibratory eccentrics



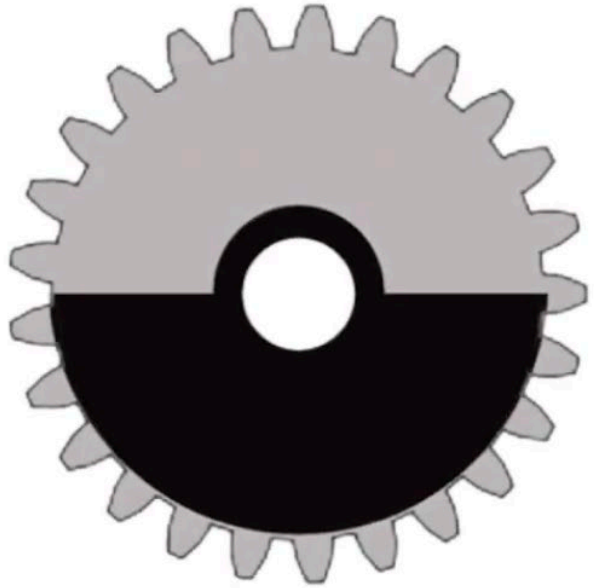
Eccentrics



Eccentric

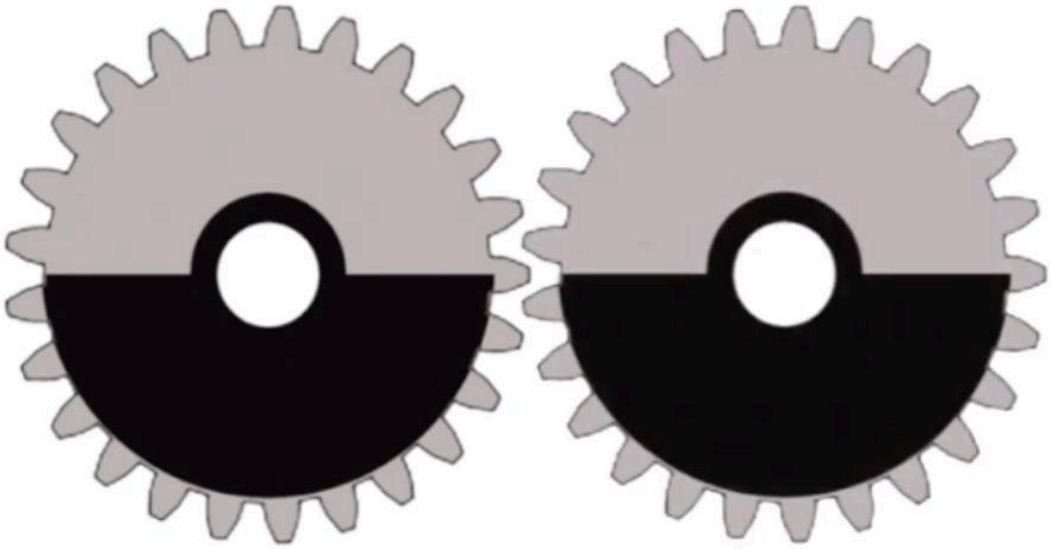


Eccentric with gear attached



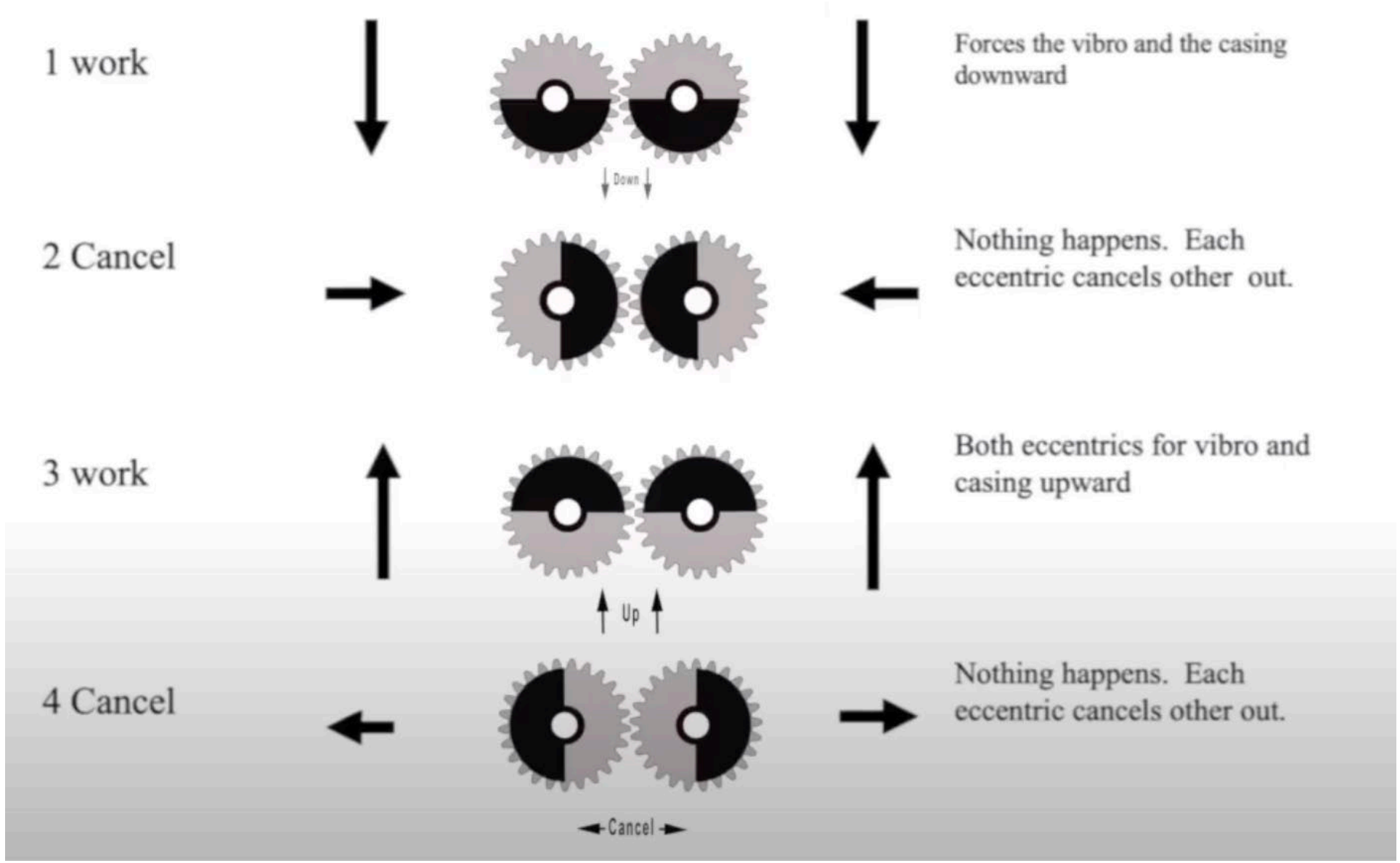
Attaching A Gear To The Eccentric

Two eccentrics

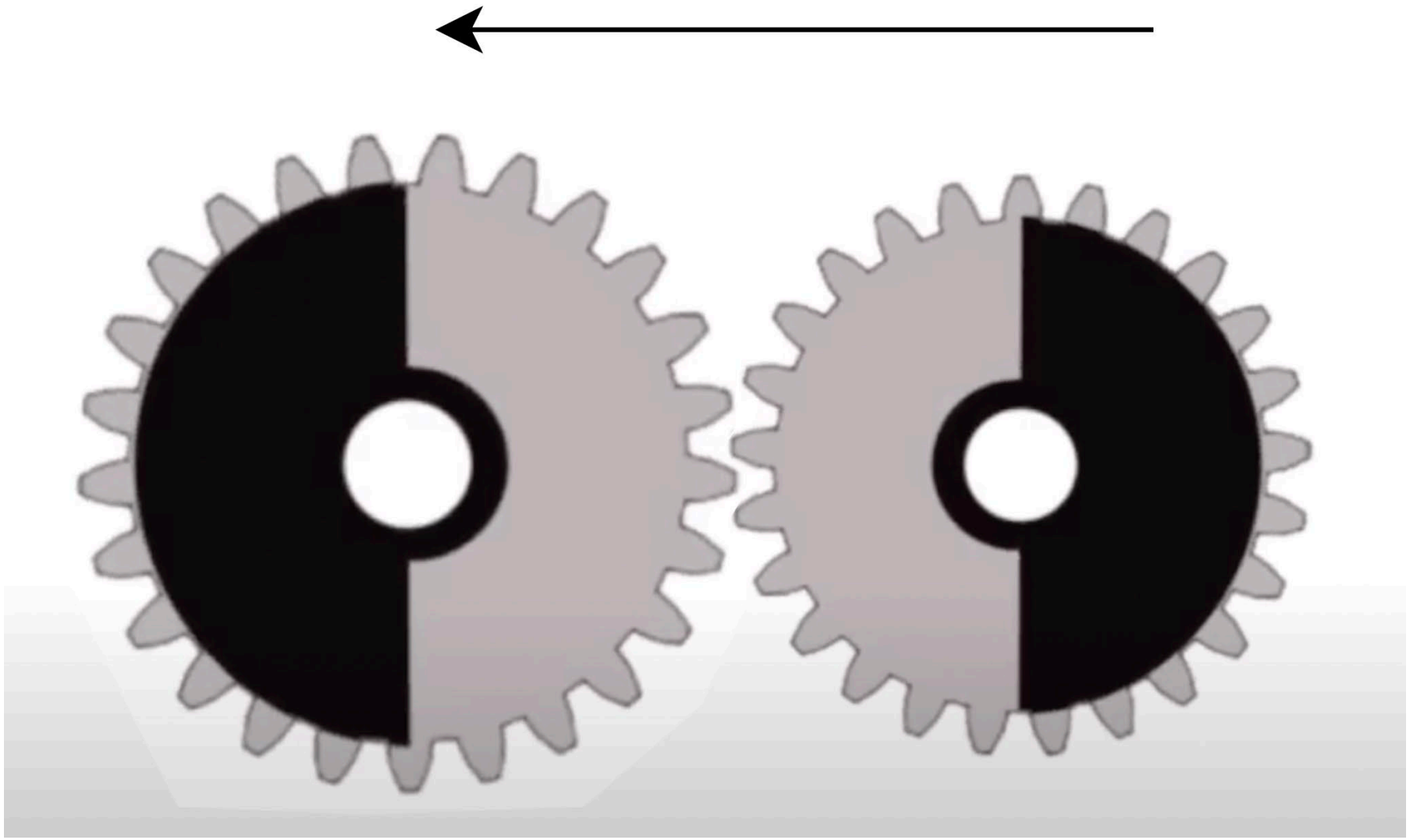


Gear Timing Two Pairs Of
Eccentrics

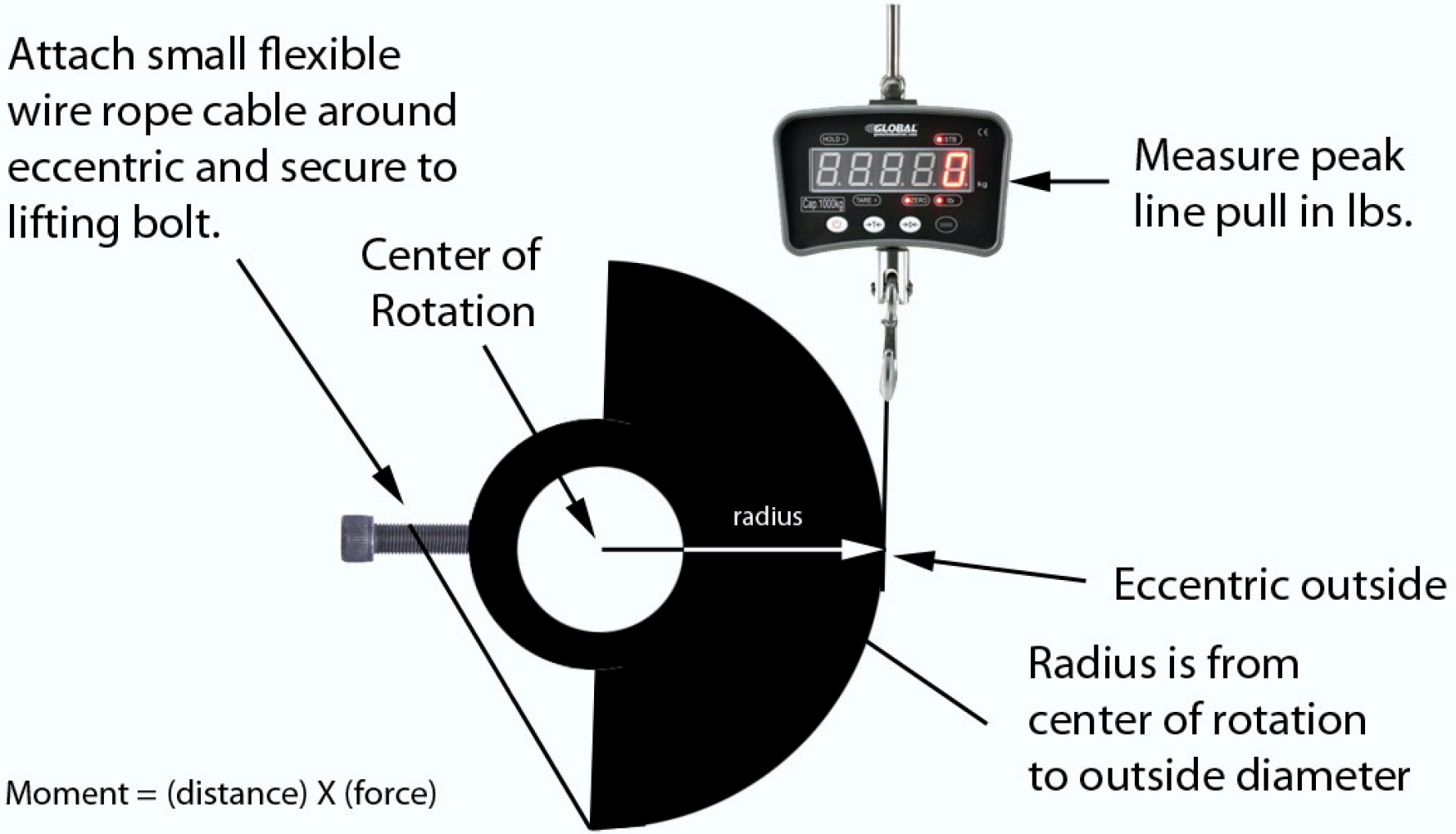
Four strokes of the paired eccentrics



What if they are not balanced?



HOW TO MEASURE ACTUAL ECCENTRIC MOMENT

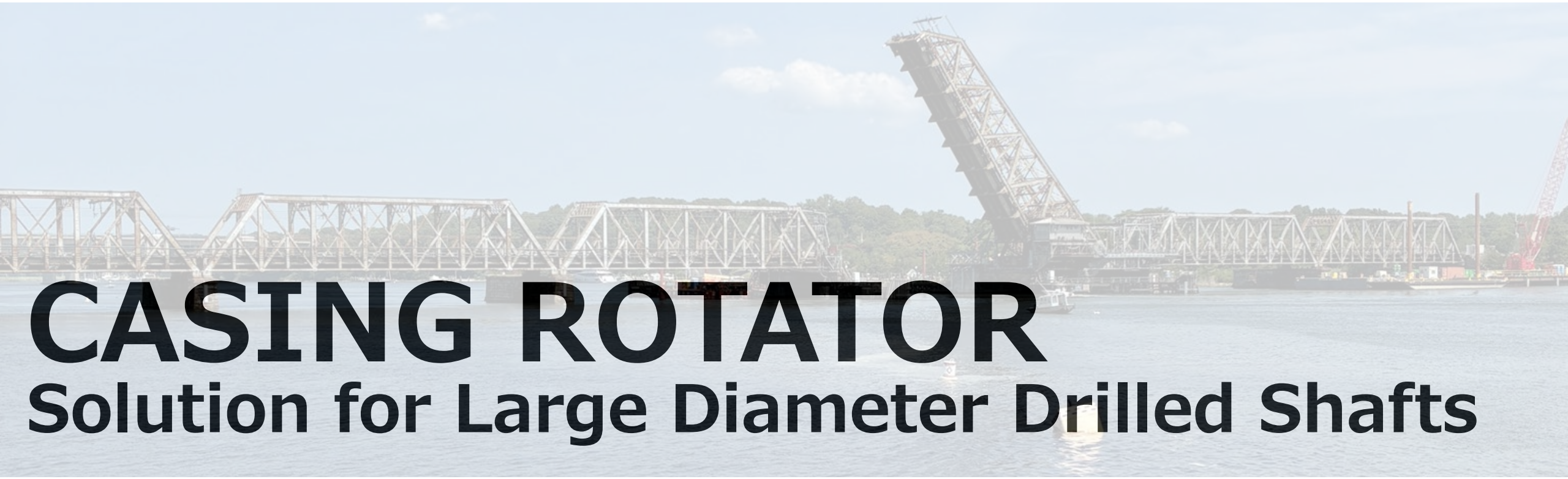


Eccentric Moment = (radius inches) X (Peak line pull lbs)

John White

Thank you!!

John White – CEO
Antaeus Foundation Equipment
forestledge@gmail.com
206.495.7030



CASING ROTATOR
Solution for Large Diameter Drilled Shafts

Yujiro Kobayashi

NIPPON SHARYO

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What dose this number stand for?

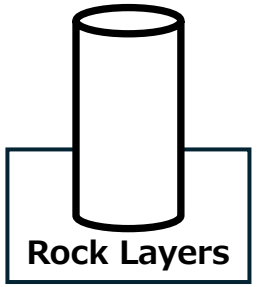
800


We have manufactured
800
Casing Rotators

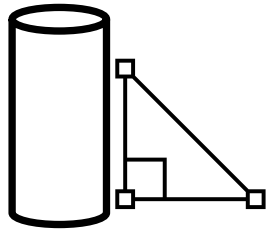
Casing Rotator

Proven Method

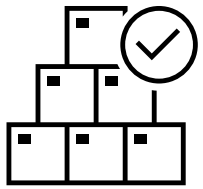




Advance Casing through Boulders into Sloping Hard Rock



High Degree of Plumbness



>800units



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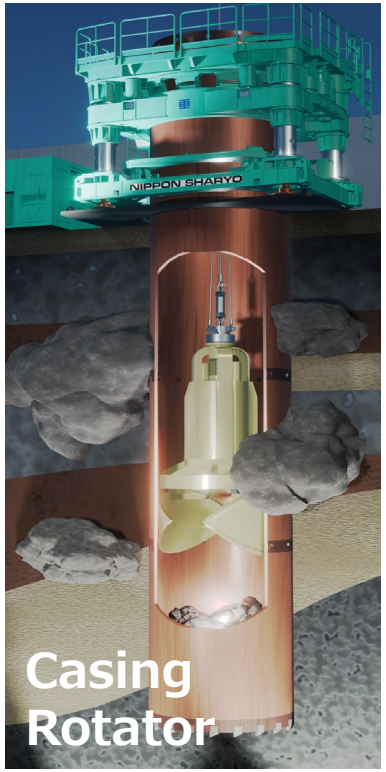
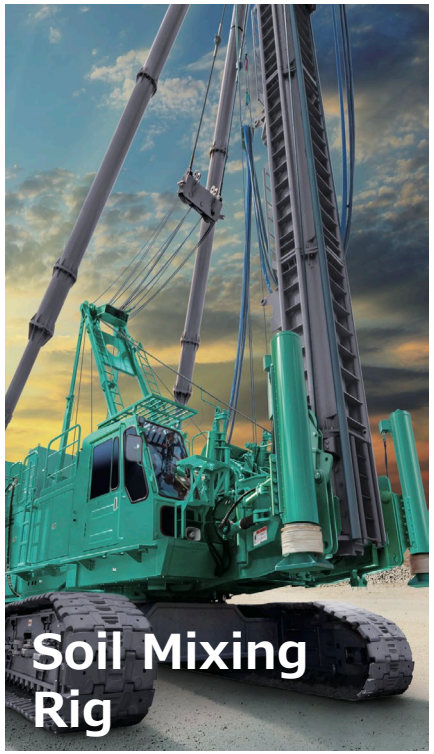
About Us

We are a Manufacturer of Foundation Drilling Equipment

Products











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Challenges in Foundation Industry

Condition of U.S. highway Bridges



Year:2024 「National Transportation Statistics」 Bureau of Transportation Statistics

40,000 bridges are Poor

Infrastructure upgrade is crucial

Challenges in Foundation Industry

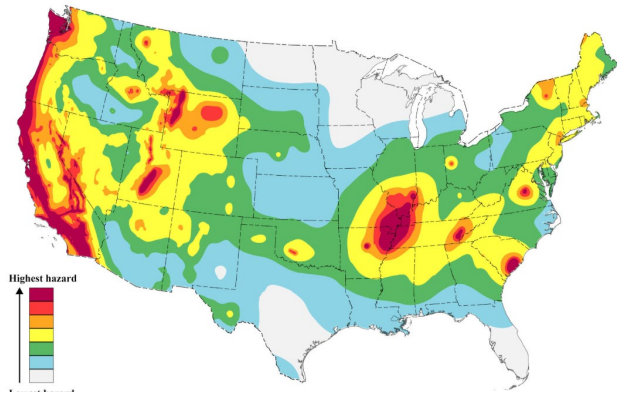
Shifting Design Requirements

Lateral loads



Vessel Collision at Baltimore Key Bridge

Seismic loads

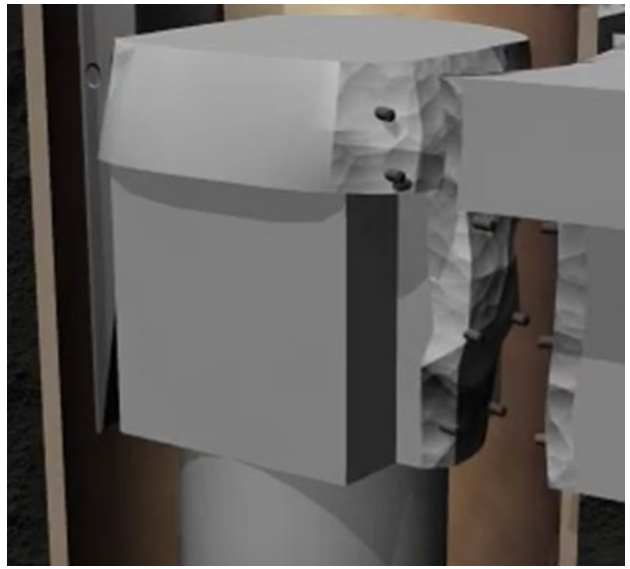


Earthquake Hazard

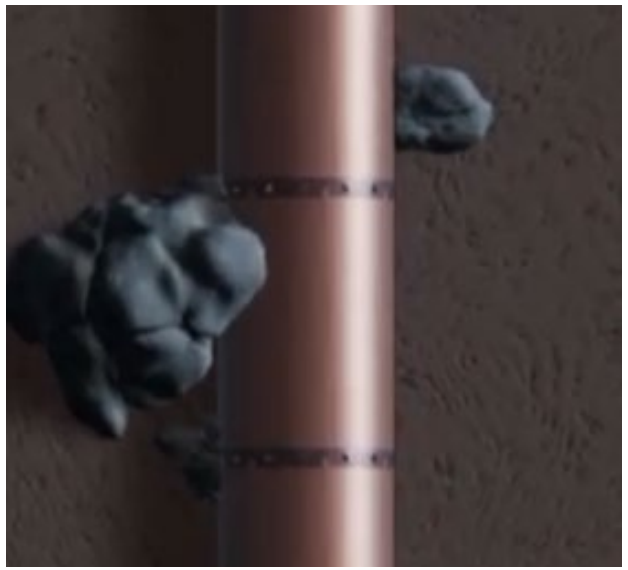
Larger & Deeper Foundations

Challenges in Foundation Industry

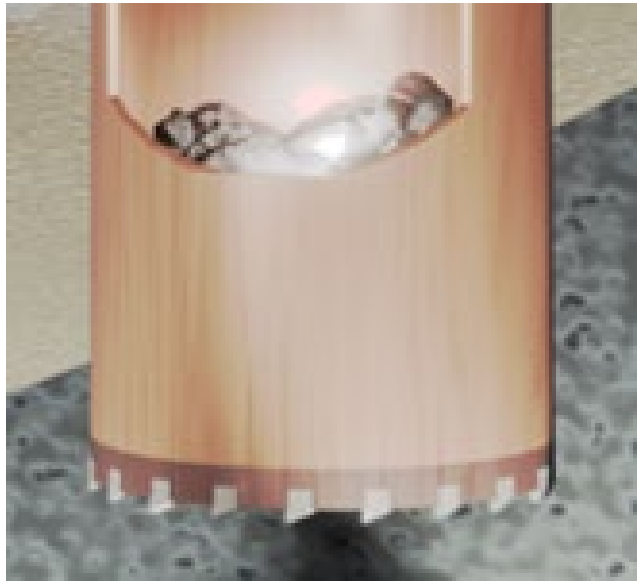
Cutting obstructions



Cutting boulders



Rock Socketing into sloping hard bedrock



Foundations are challenging and complex



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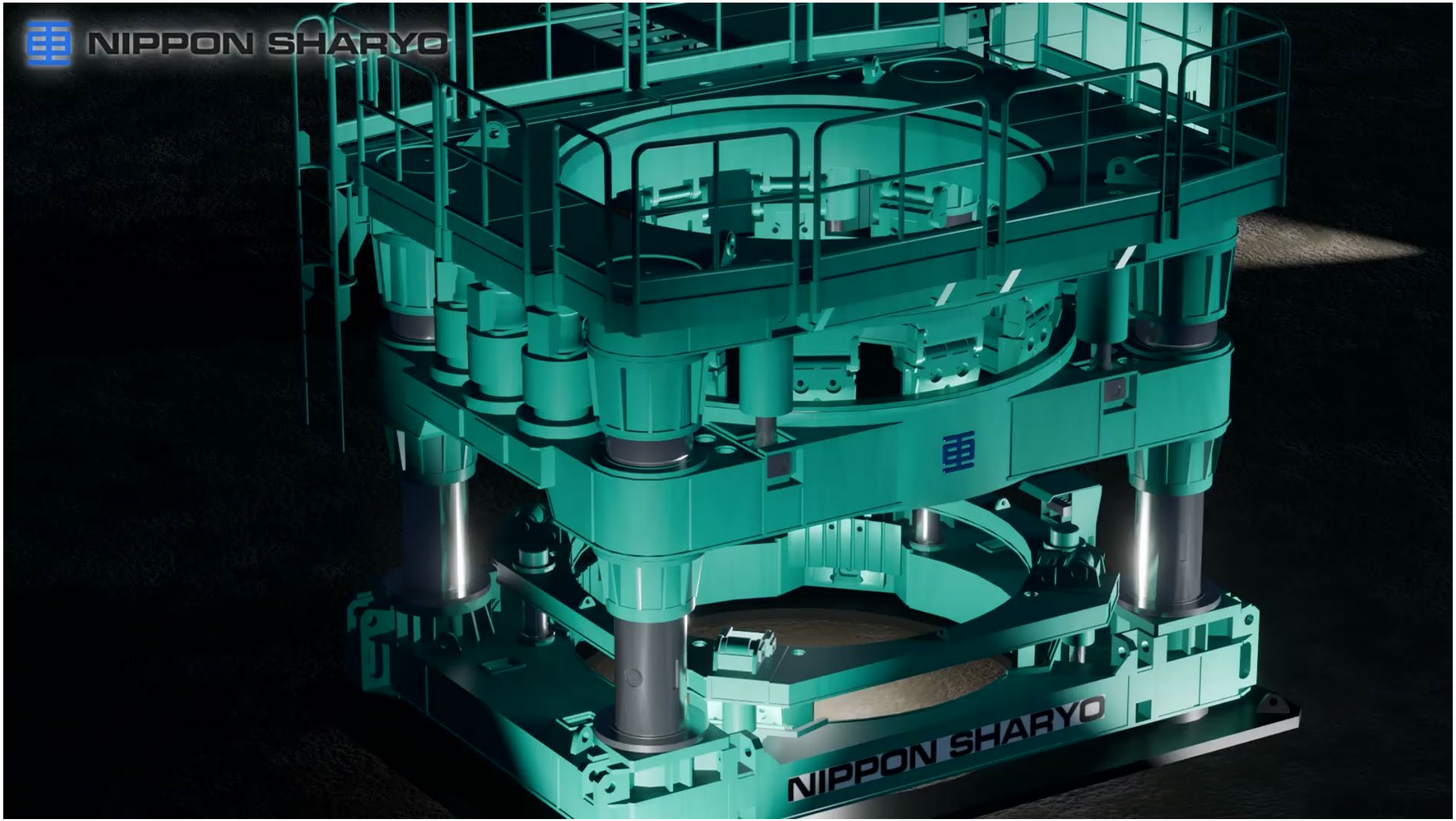
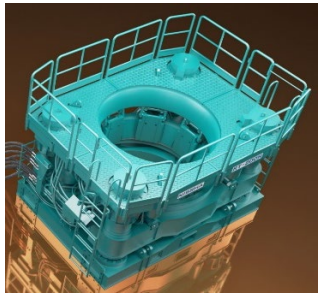
What is the Casing Rotator?



What is the Casing Rotator?



What is the Casing Rotator?





	Casing Rotator	Rotary Drilling Rig
Rotation Torque	4,000,000 ft-lbs	300,000 ft-lbs
Crowd Force	300,000 lbf	67,000 lbf
Extract Force	950,000 lbf	67,000 lbf
Drilling Diameter	Up to10.4 ft	4 ft (Double Rotary)
Drilling Depth	Over 420 ft	100 ft

(Specification NIPPON SHARYO DHR70)



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Proven Method

Test 1	Rock Cutting Efficiency
Test 2	Deep Drilling Efficiency
Test 3	Drilling Plumbness

Proven Method

Test 1 **Rock Cutting Efficiency (test site)**

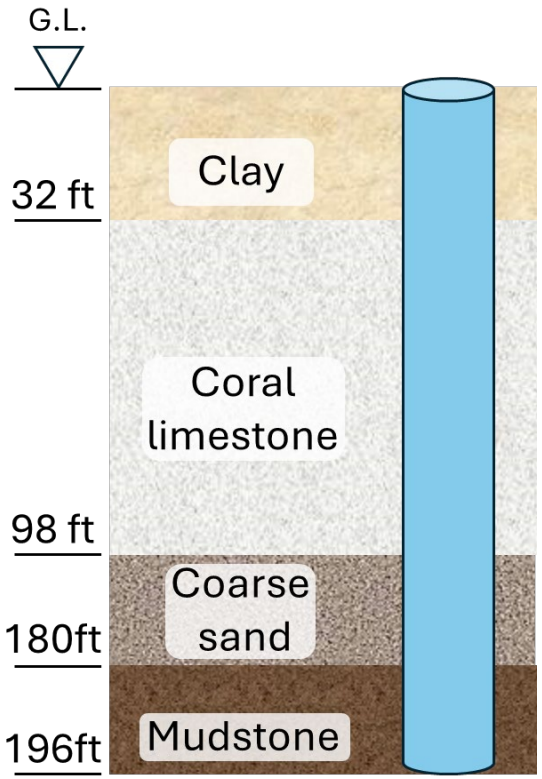


- Granite with UCS of **30,000 psi**
- Penetration rate: **3ft/ 100min**

Casing Rotator Cuts the Extremely Hard Rock

Proven Method

Test 2 Deep Drilling Efficiency

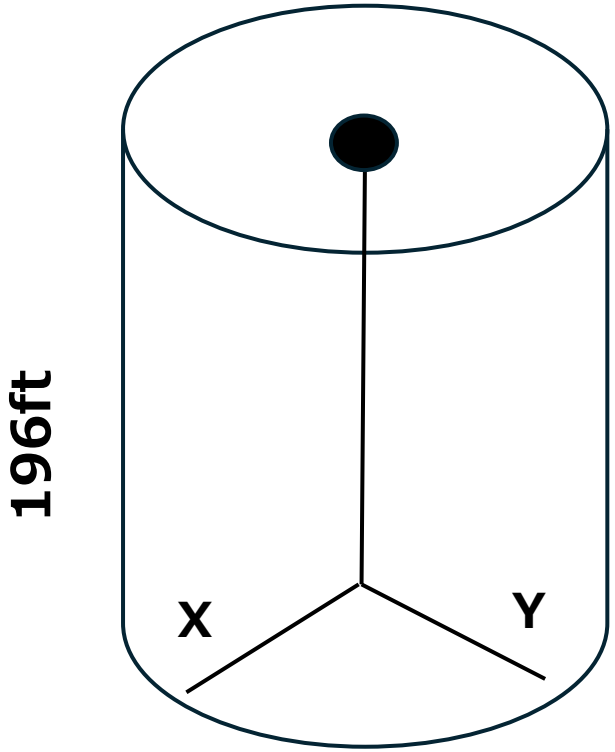


- Drilling 5 shafts to depth of **196ft**
- Average installation time **22** hours

Casing Rotator Drills Deep Shaft Efficiently

Proven Method

Test 3 **Drilling Plumbness**



✓ **Measured offset at 195ft**

- **X axis 2 inch**
- **Y axis 1 inch**

Casing Rotator Maintains High Degree of Plumbness



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Case History

Moses Wheeler Bridge (Milford-Stratford, CT)



©Raito Inc

Foundation design

- 9’10” Permanent casing
- Up to 141’ deep

The challenge

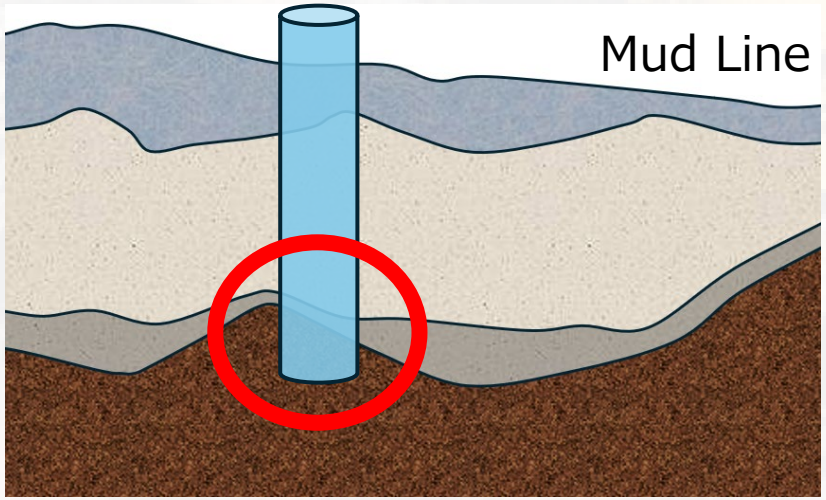
- Mica schist with UCS of **20,000**psi
- **Boulders and refuse**
- 3’ into **sloping rock**

Case History

Connecticut River Bridge (Old Saybrook, CT)



©Becho Inc



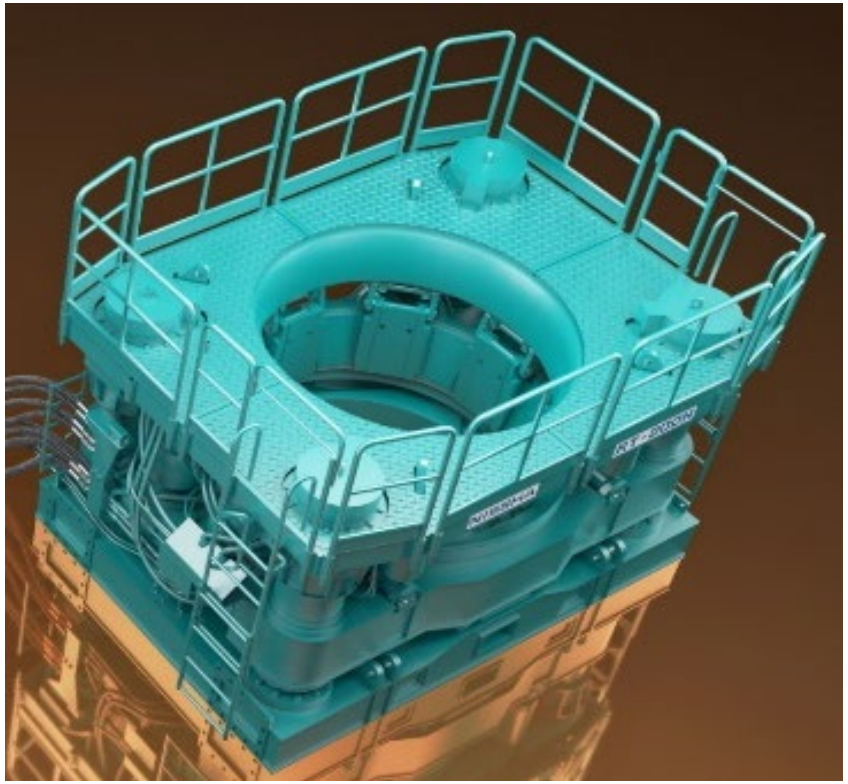
Foundation design

- 6′ & 8′ Permanent Casing
- Up to 160′ deep

The challenge

- **Steeply sloping bedrock**

Contact



Yujiro Kobayashi
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E-mail: y-koba@n-sharyo.co.jp



From Data to Action: Using Testing to Address MWD-Identified Deviations in Rigid Inclusion Installation

DFI50 – Nashville, TN

October 21, 2025

5:35 PM





Ken Kniss, P.E.

Chelsea Riddle, P.E.

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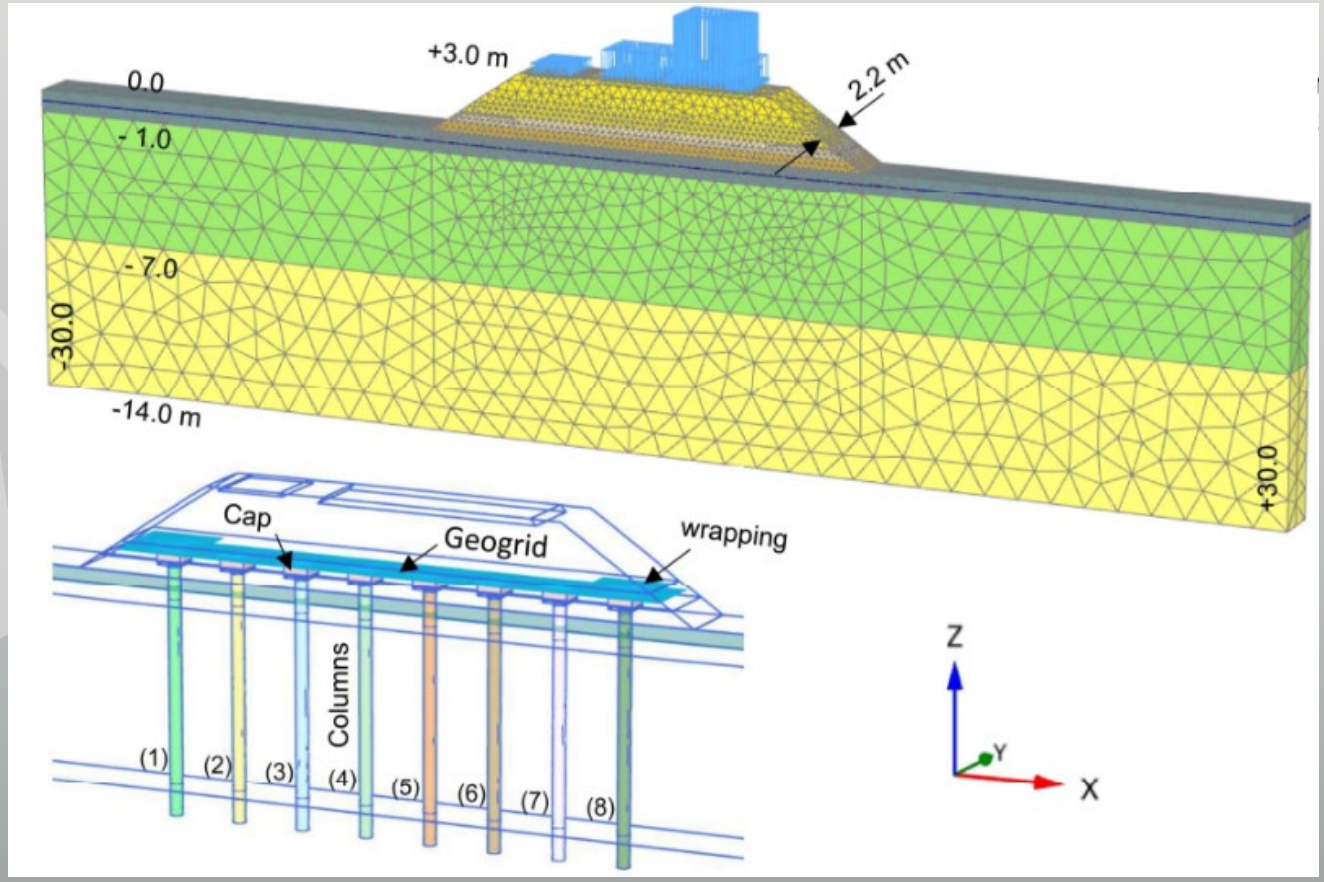
AGENDA

- Background
- Calibration Program
- Examples of Non-Conformance
- Testing Options
- Project Summary – Bethlehem, PA
- Conclusions
- Question and Response

DRAFT

Current Literature

- ASCE Library Search for “Rigid Inclusions”
  - Numerical Analysis
  - Project Summaries
    - Design
    - Performance Monitoring
- ASIRI 2012
  - Most comprehensive
  - 14 of 317 pages dedicated toward quality and testing

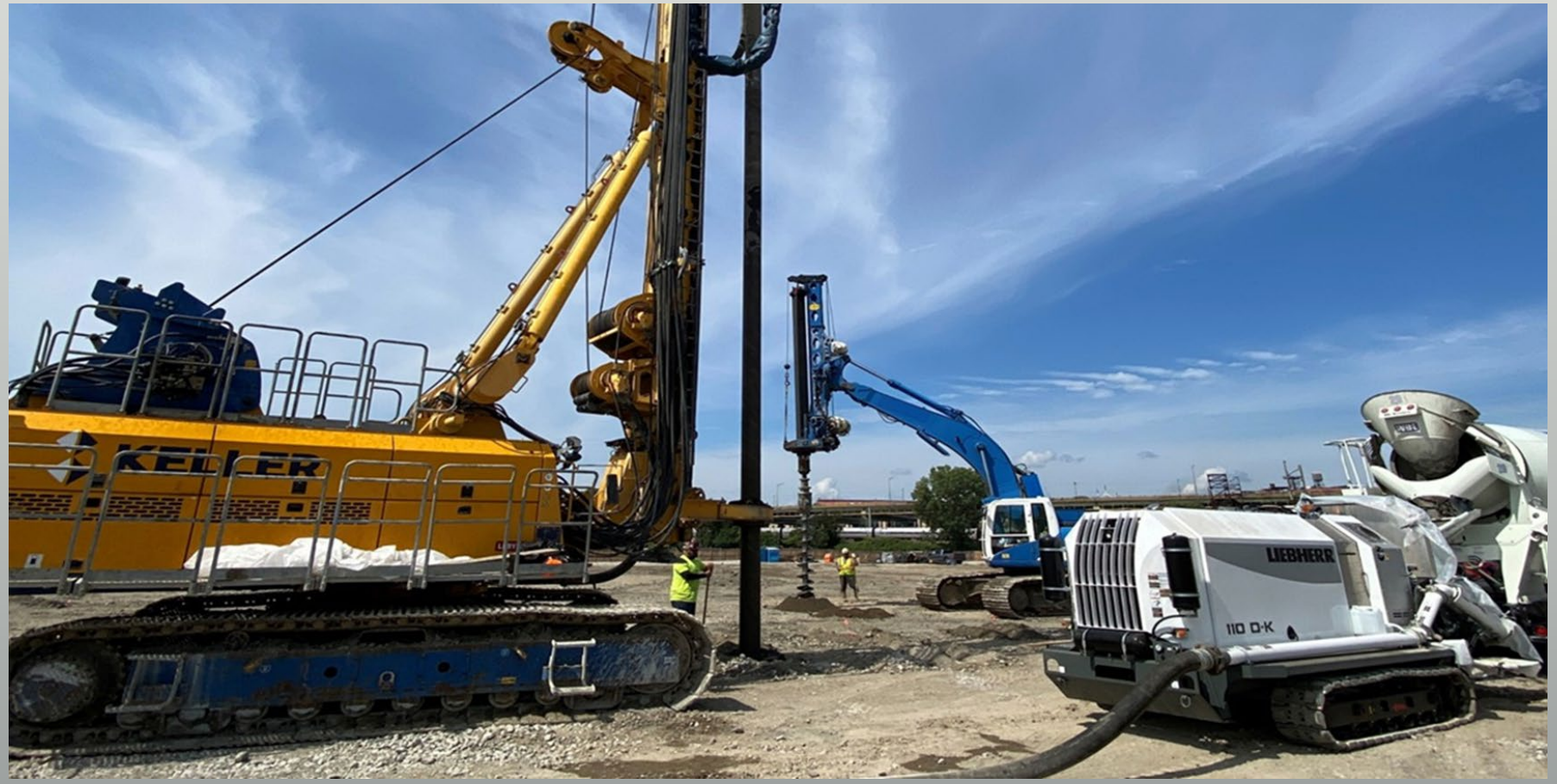


Topolnicki 2024

Rigid Inclusion Guidelines and Standards

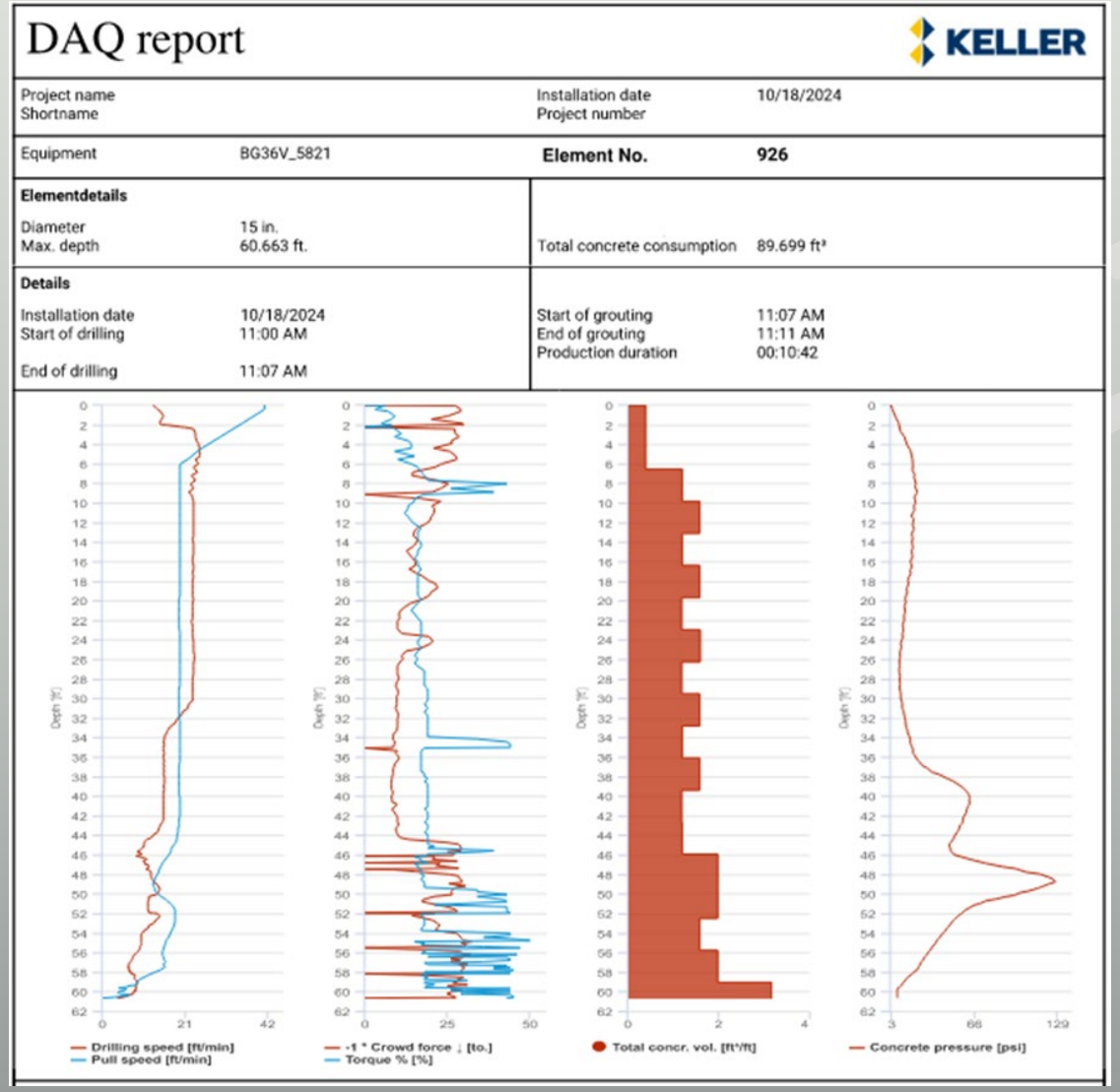
DRAFT

Quality Assurance:
QC Requirements for Rigid Inclusions



- Element Layout
- Element Diameter
- **Drilling Procedure**
  - **Depth**
- Material Properties
- **Grouting Procedure**

Measure While Drilling (MWD) Reports: Processed Installation Data



DAQ report

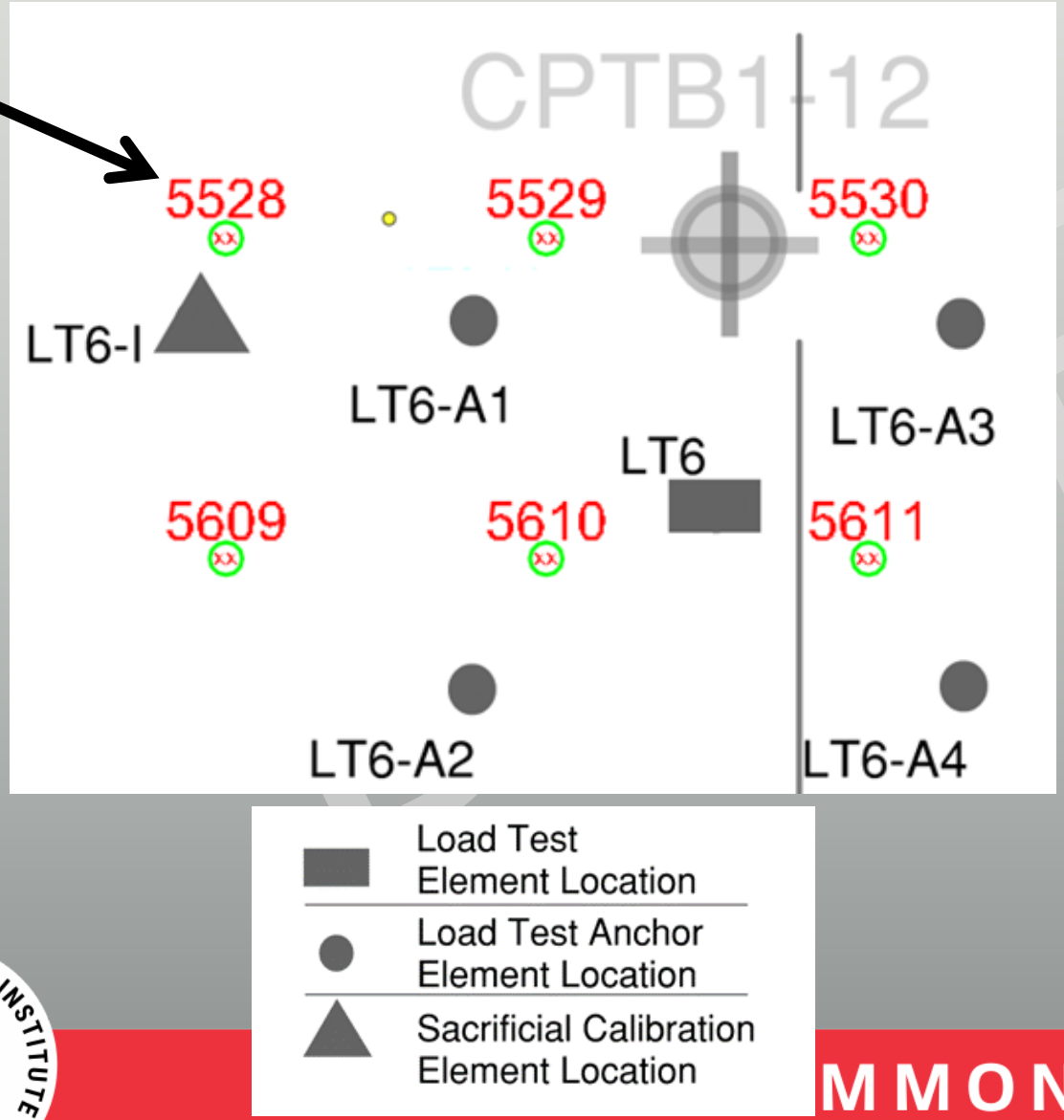
KELLER

Project name Shortname		Installation date Project number	10/18/2024
Equipment	BG36V_5821	**Element No.**	**926**
Elementdetails			
Diameter Max. depth	15 in. 60.663 ft.	Total concrete consumption	89.699 ft³
Details			
Installation date Start of drilling End of drilling	10/18/2024 11:00 AM 11:07 AM	Start of grouting End of grouting Production duration	11:07 AM 11:11 AM 00:10:42

- Date and time of installation
- Duration of installation
- Penetration time and depth
- Drill rig energy and force parameters (crowd force and torque)
- Grout pressure
- Withdrawal rate
- Grout volume placed

Calibration Program

Production Elements



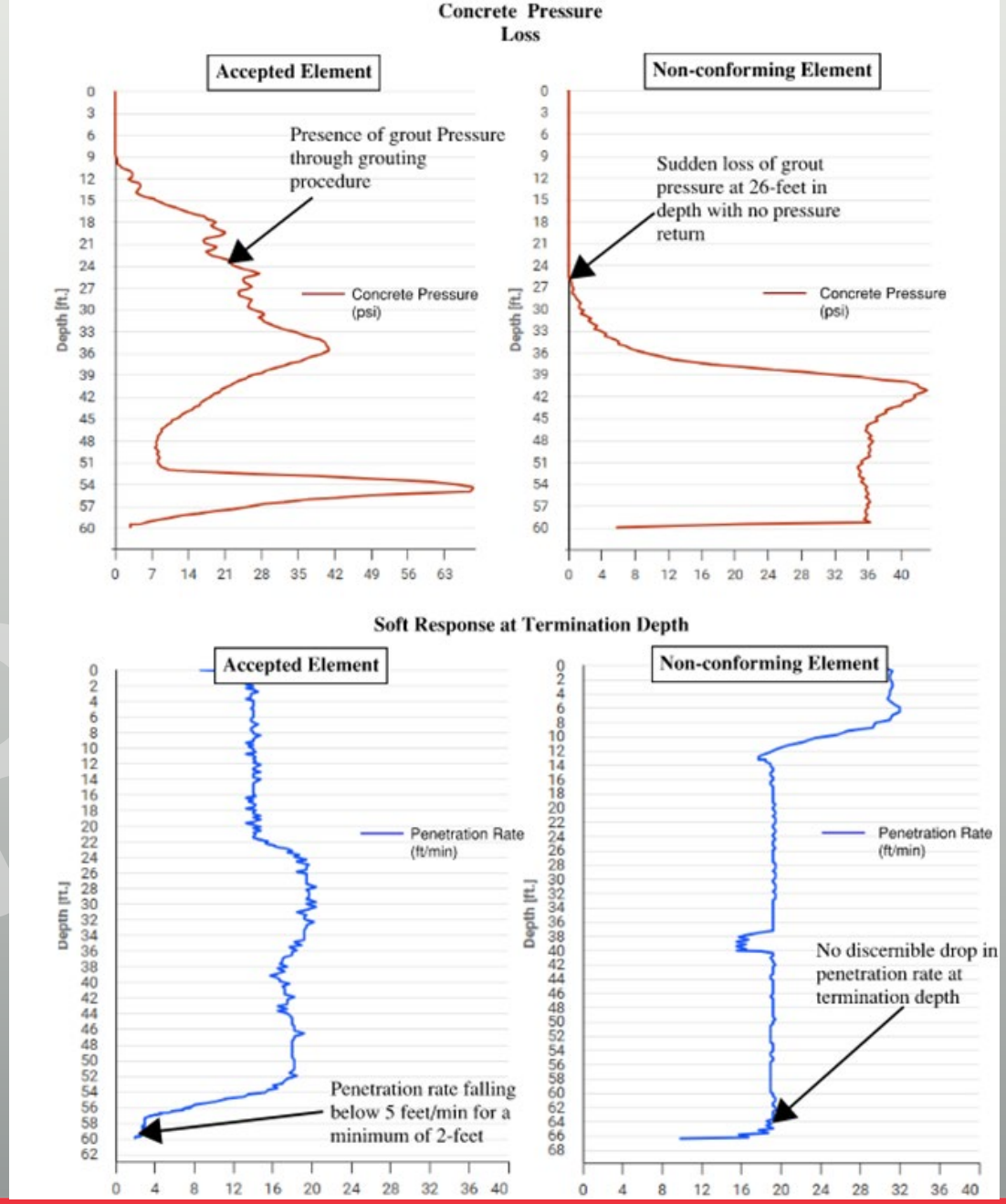
- Install sacrificial elements prior to the test element and production locations.
- Ensure soil conditions match expectations
- Calibrate drill rig response, grouting procedure and grout cut-off methods for site specific conditions, grout plastic properties and equipment capabilities

Test Program

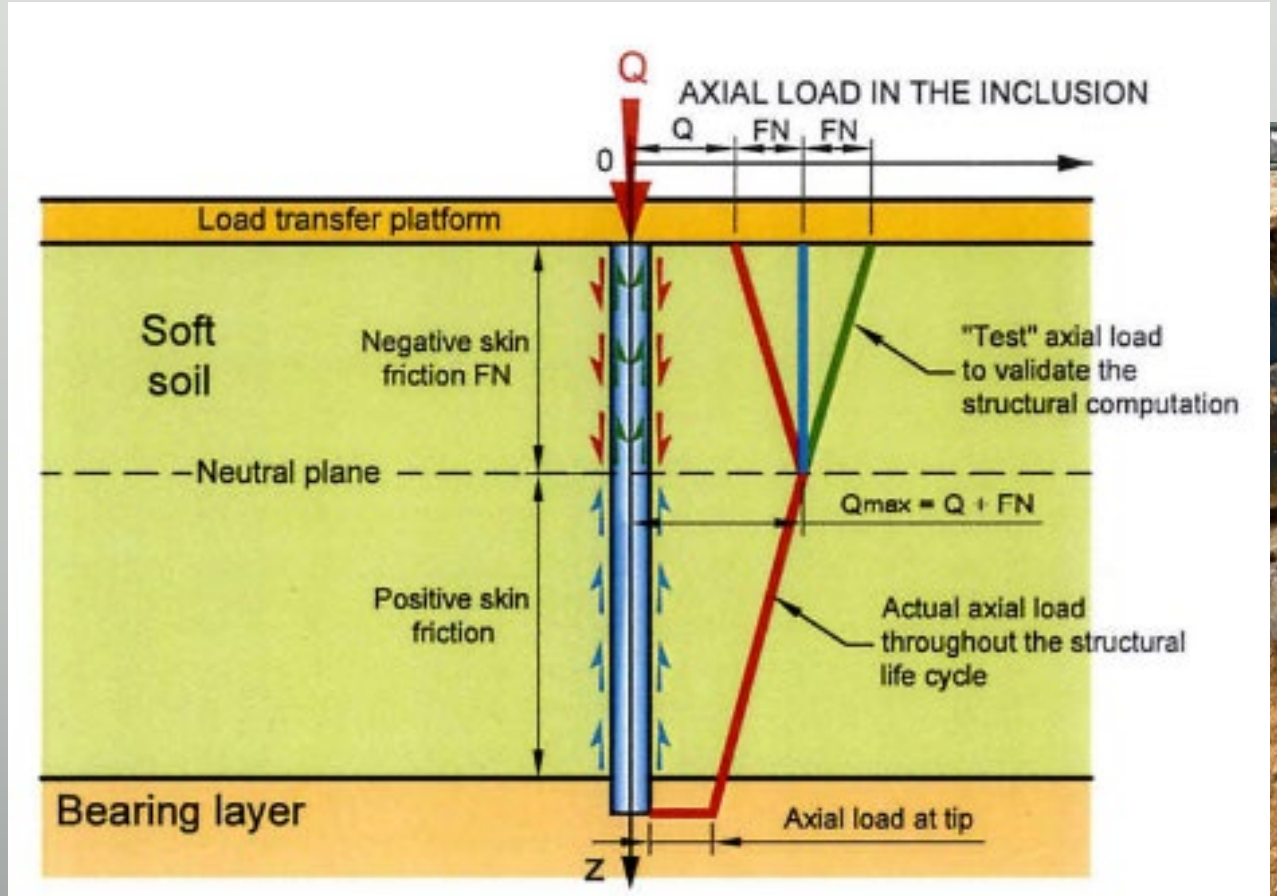
- What to cover?
  - Type of test matches risk for project
  - Load-advancement procedure
  - **Define acceptance criteria**
  - Load Test Frequency
  - Tests for element geometry and integrity



Acceptance Criteria



ASIRI Quality Test (AQT)





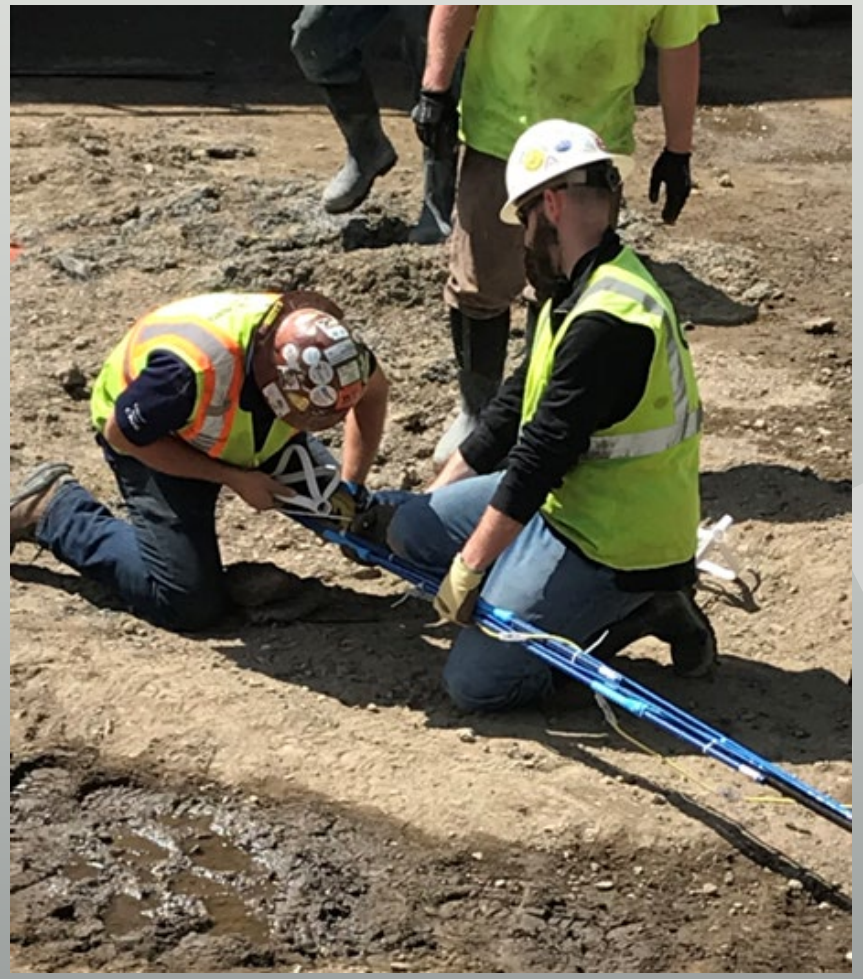
Service situation : Load at the inclusion head Q

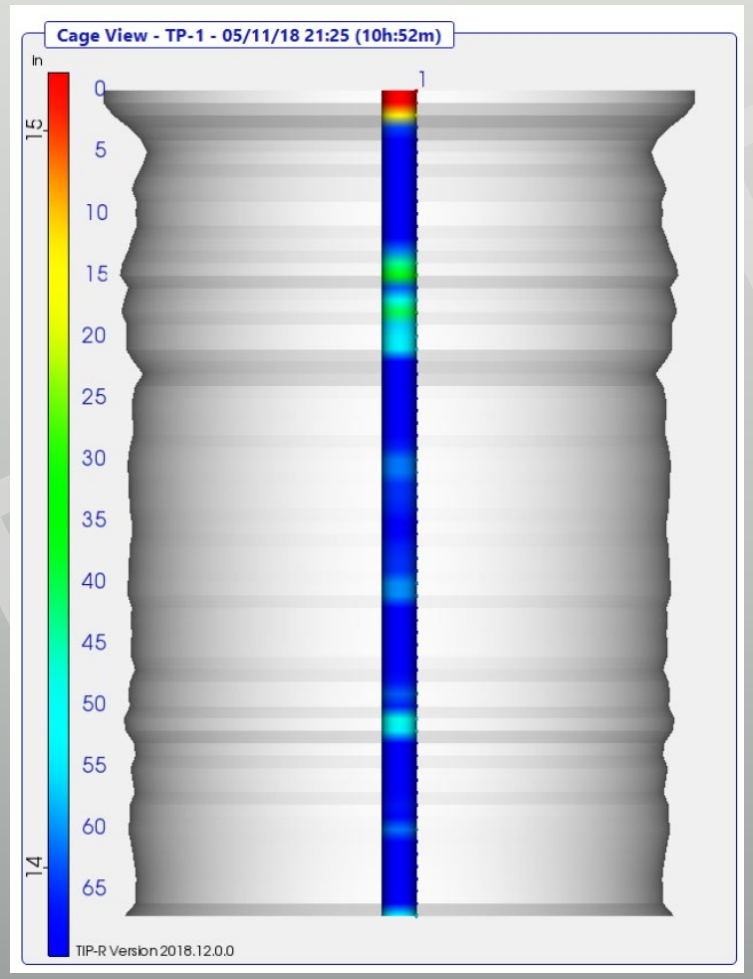
Maximum load $Q_{max} = Q_{ELS} = (Q + FN)$

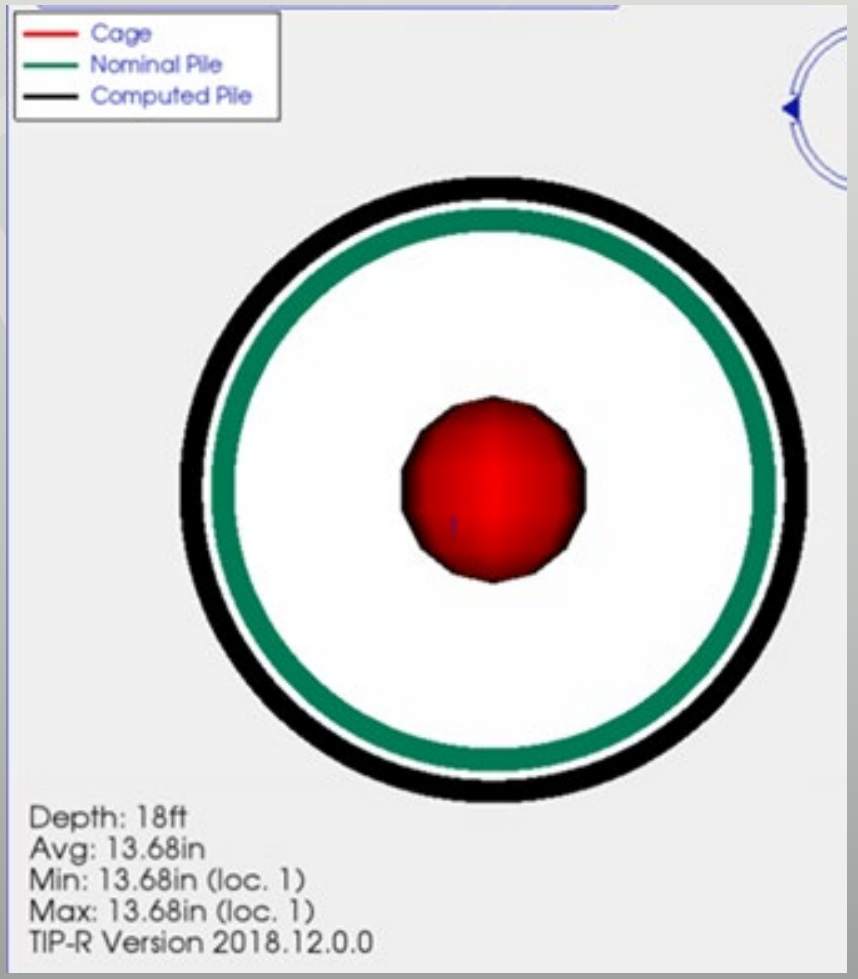
Quality test: Inclusion solely serving to reduce settlement

Test load $Q_{ELS} + FN = Q + 2FN$

Thermal Integrity Profile (TIP) Test





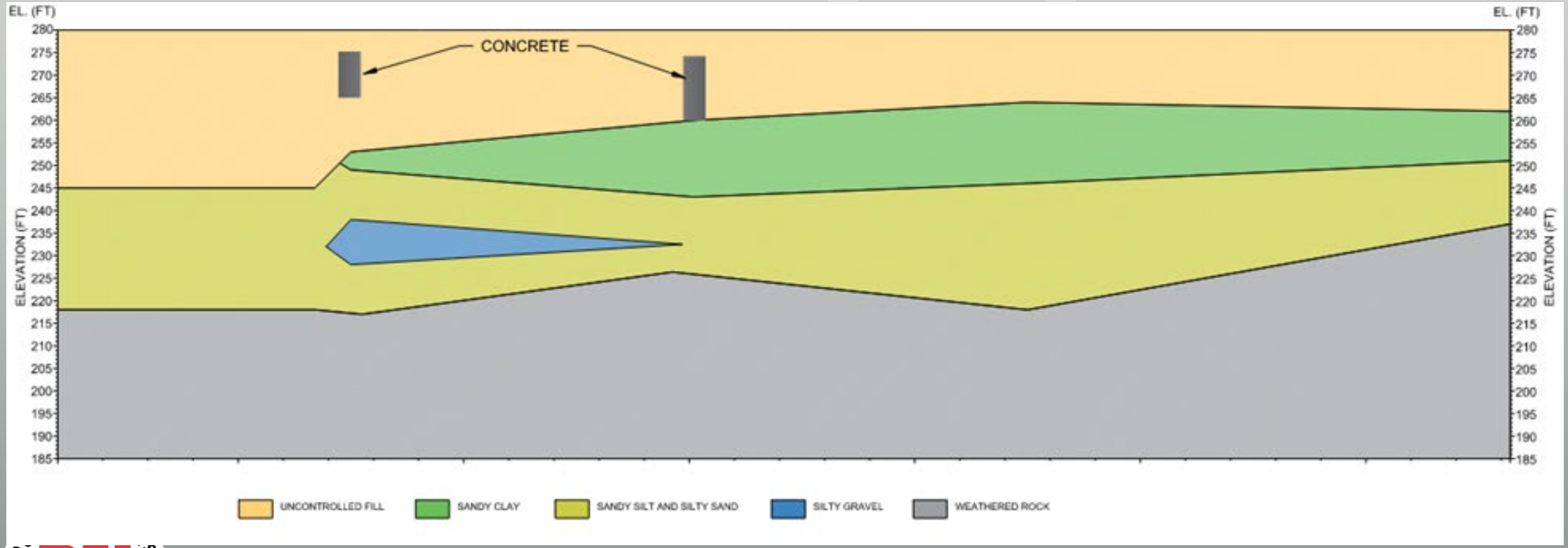


Project Summary: Bethlehem, PA



- **Design Considerations**
  - Large Area Load ≈2000 psf
  - 1-inch total / ½-inch differential settlement
  - Uncontrolled fill
  - Soft clay
  - **<u>Obstructions</u>**

- **Quality Considerations**
  - Loss of grout into voids (bulging/necking)
  - Early refusal on obstructions in fill
  - Relocating elements



Site Challenges

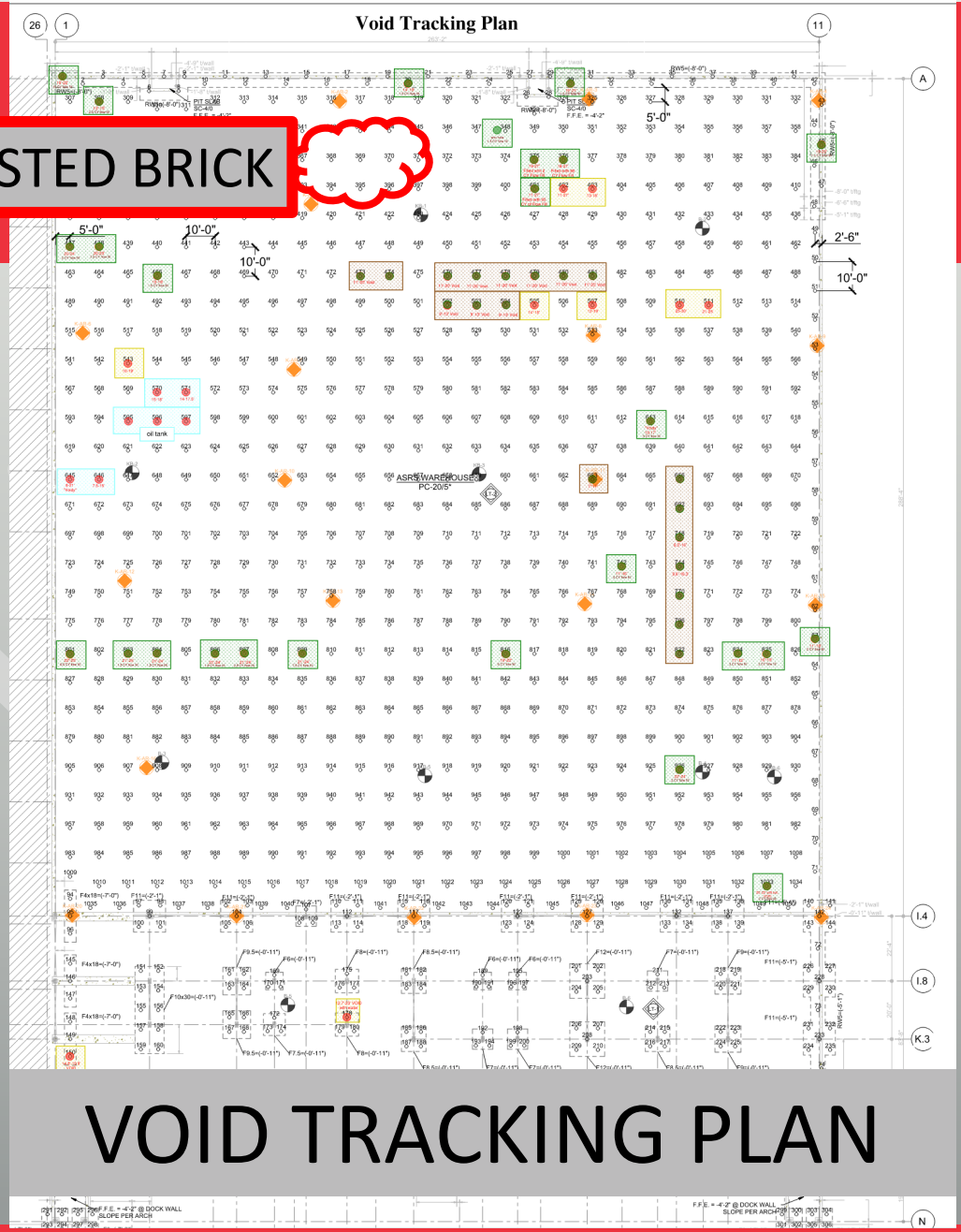




Site Challenges



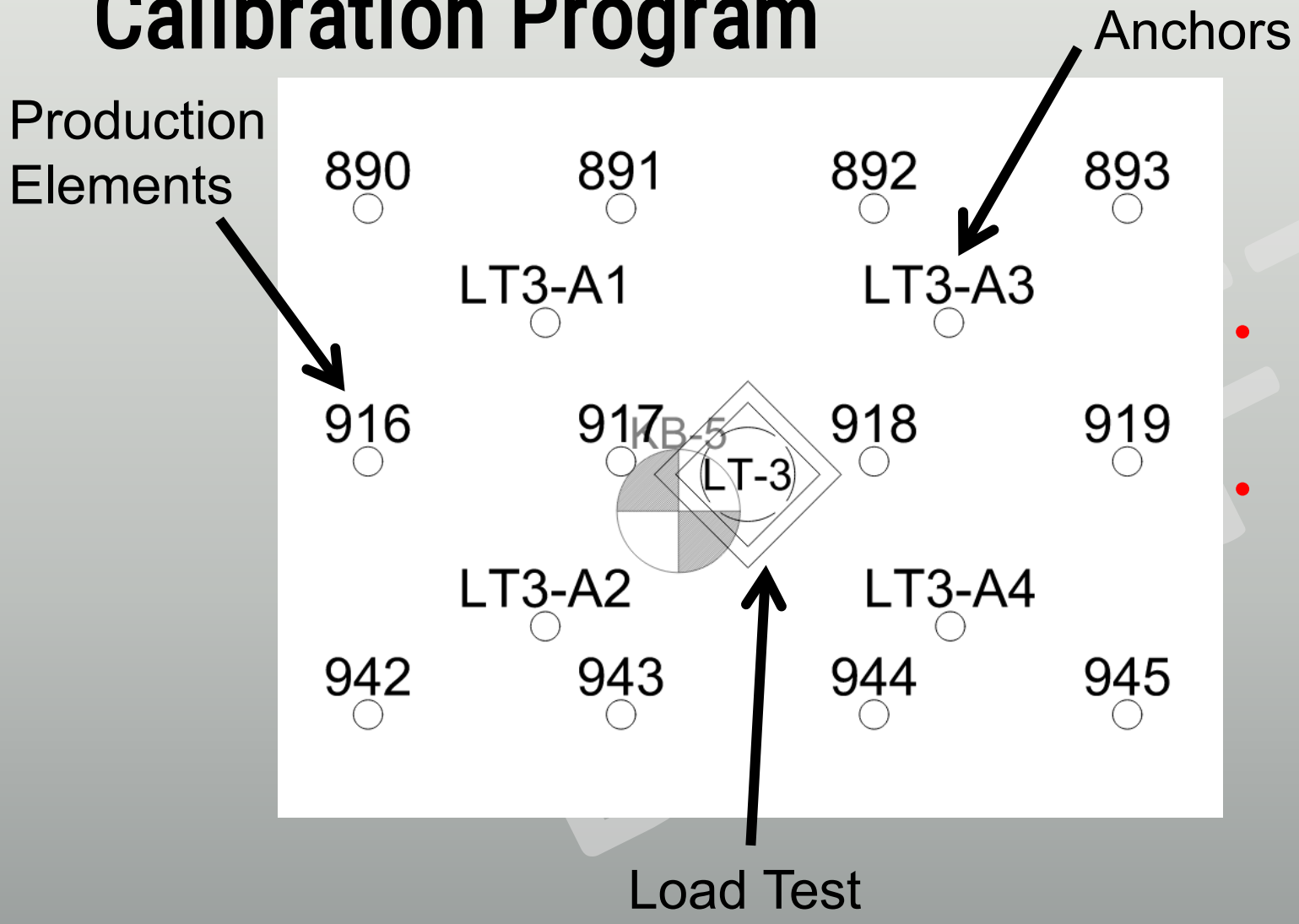
NESTED BRICK



VOID TRACKING PLAN

FINDING COMMON GROUND

Calibration Program



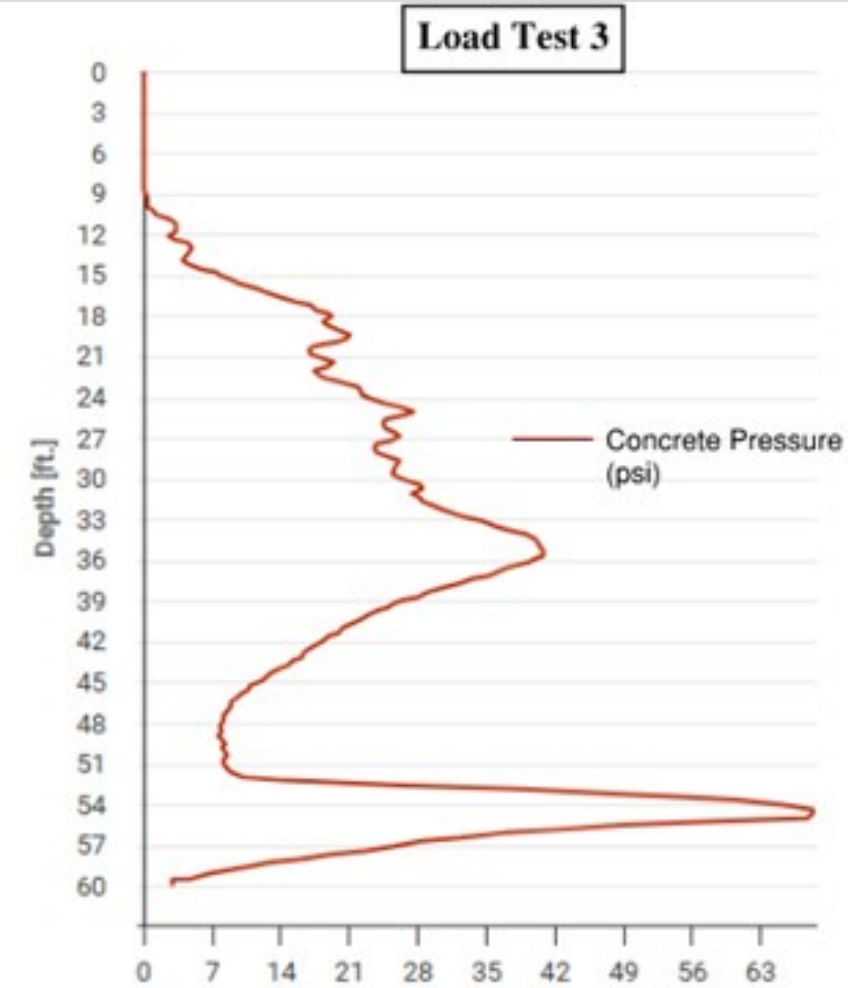
- Everything had to be predrilled…
- DAQ System: The relationships between drilling parameters, soil conditions and element capacity provide a platform for analytic evaluation of variability in soil stratigraphy….

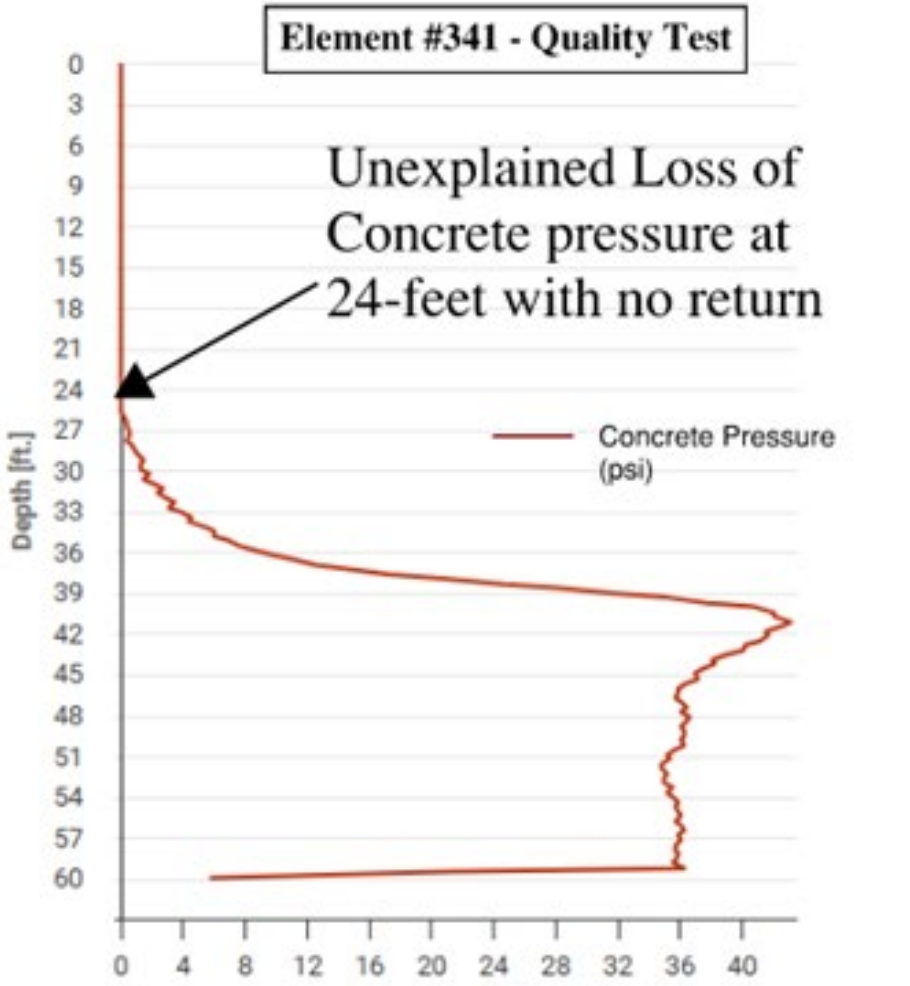
Test Program

- 6 Tests
  - 3 Single-element Tests
    - Confirm geotechnical capacity
    - Evaluate load-deflection behavior
  - 2 ASIRI Quality Tests
    - Confirm performance at design load
    - Reduce nonconformance
  - 1 TIP Test
    - Confirm diameter

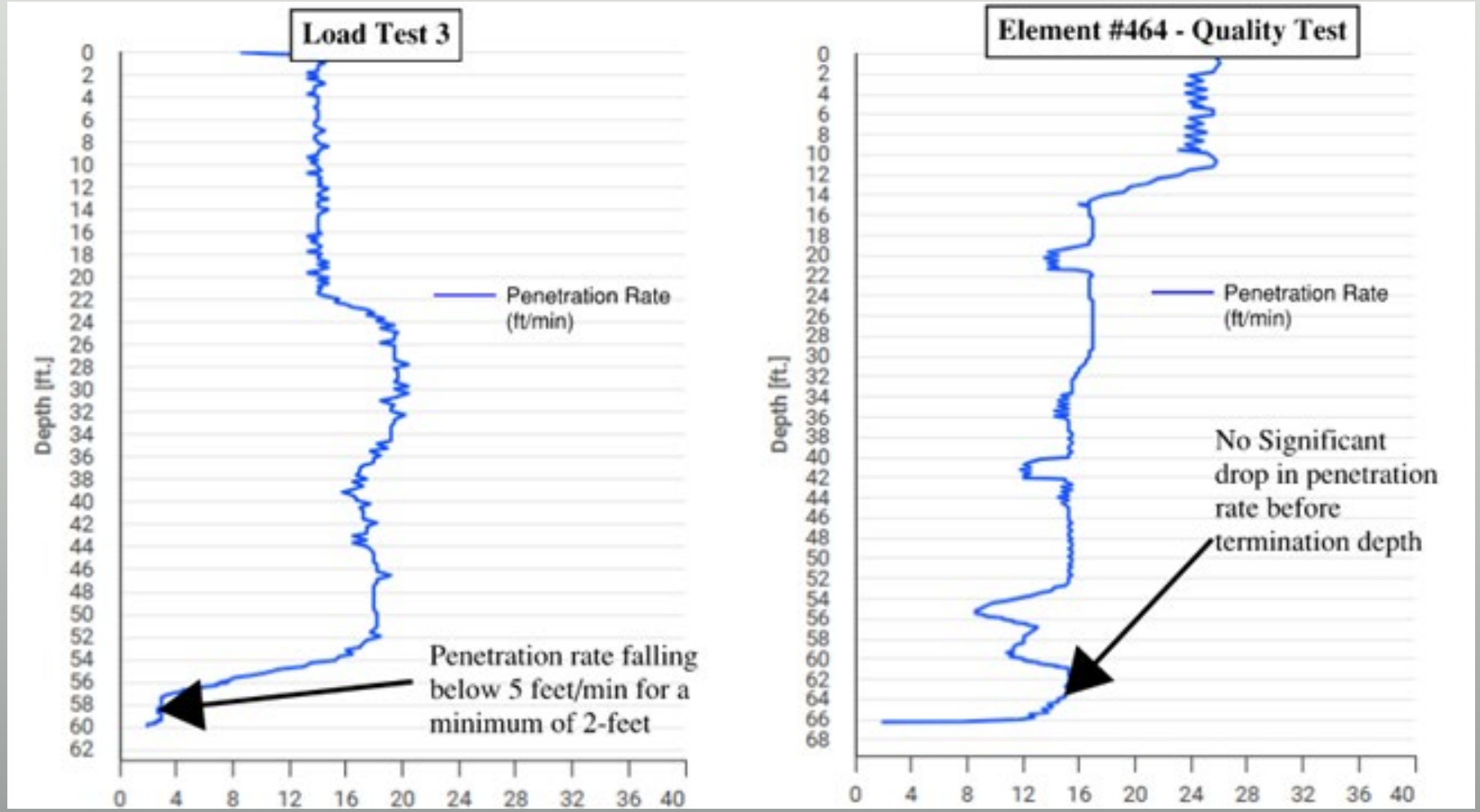


Identify Non-Conformance

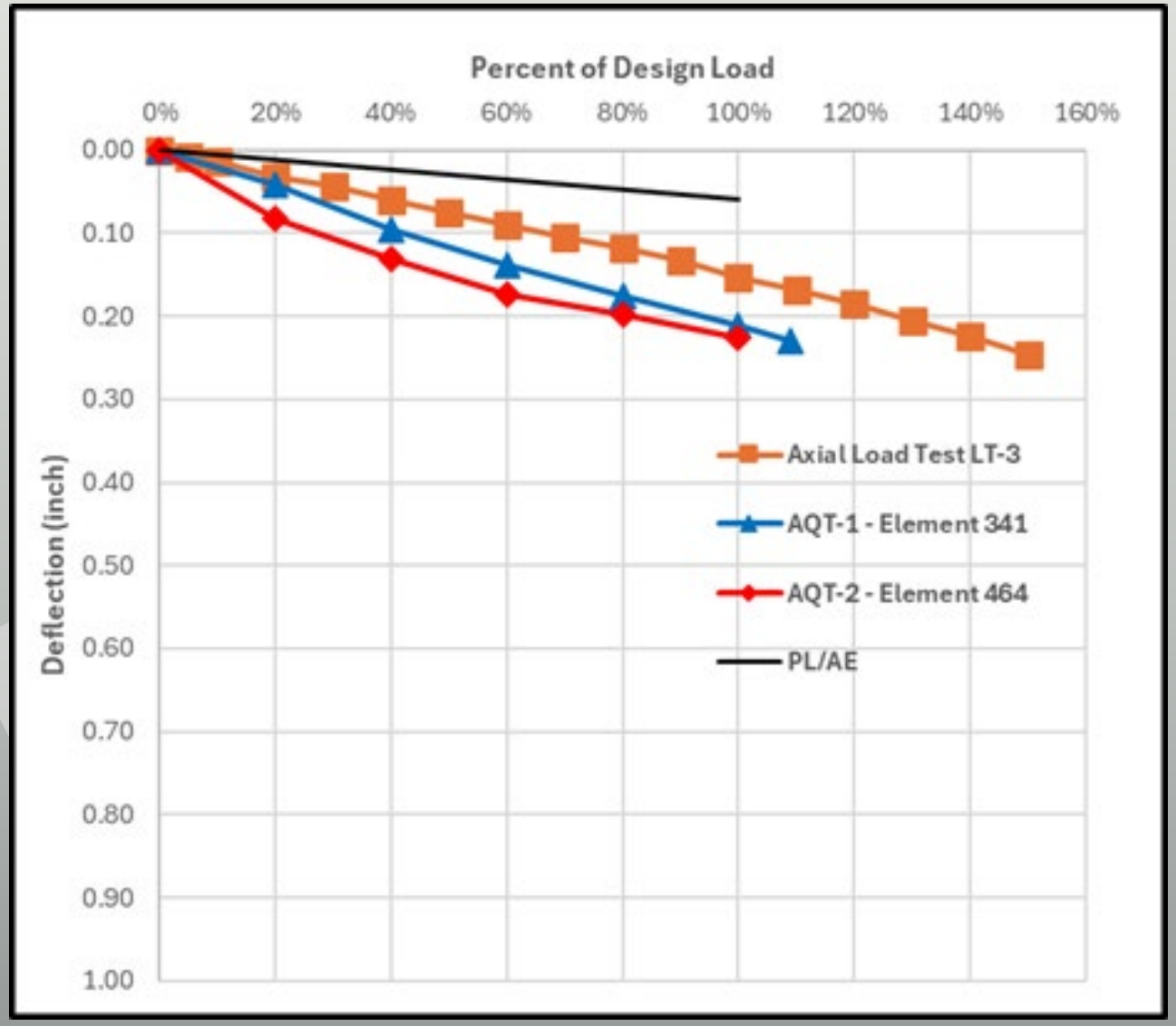




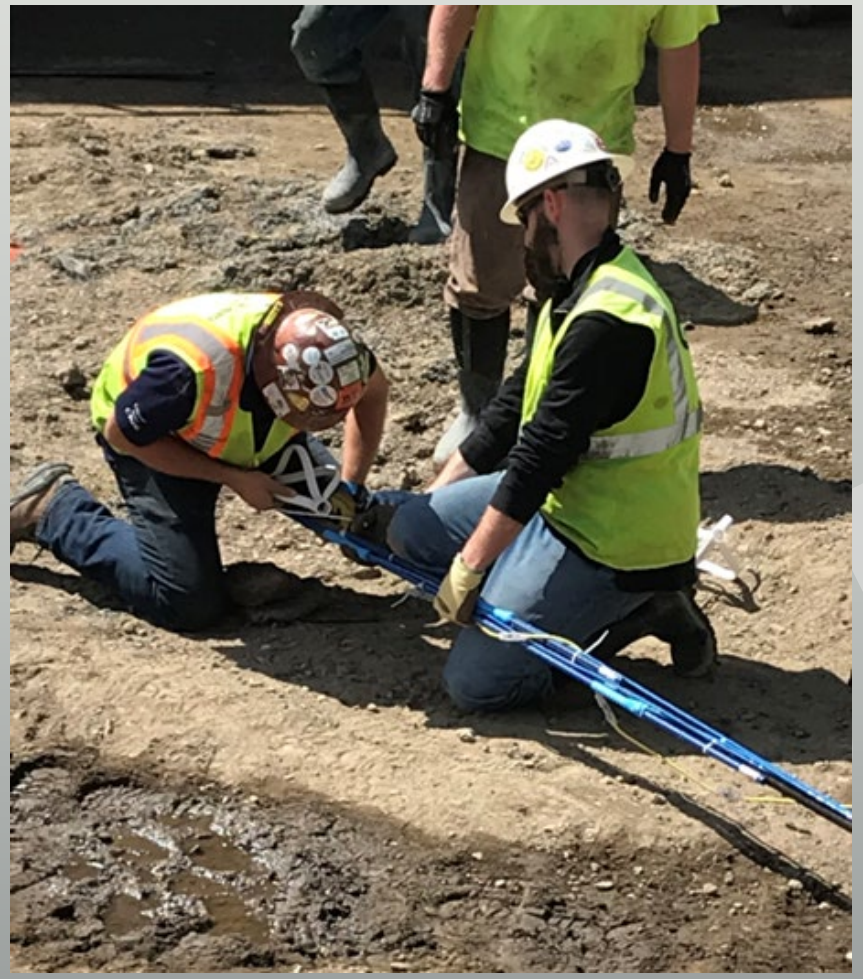
Identify Non-Conformance

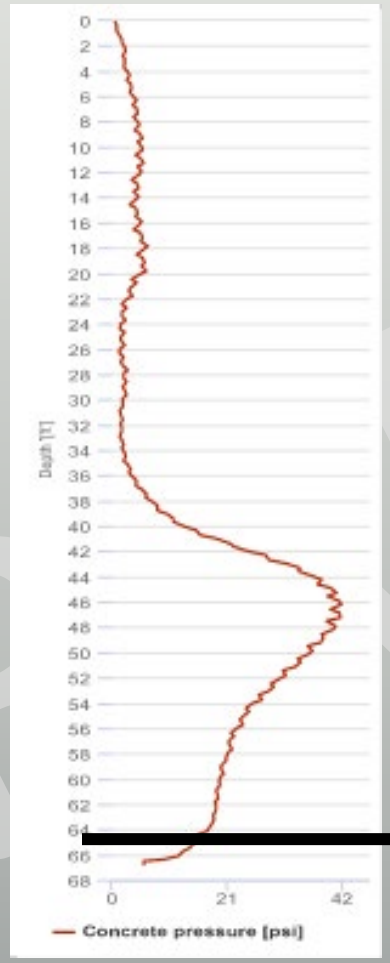


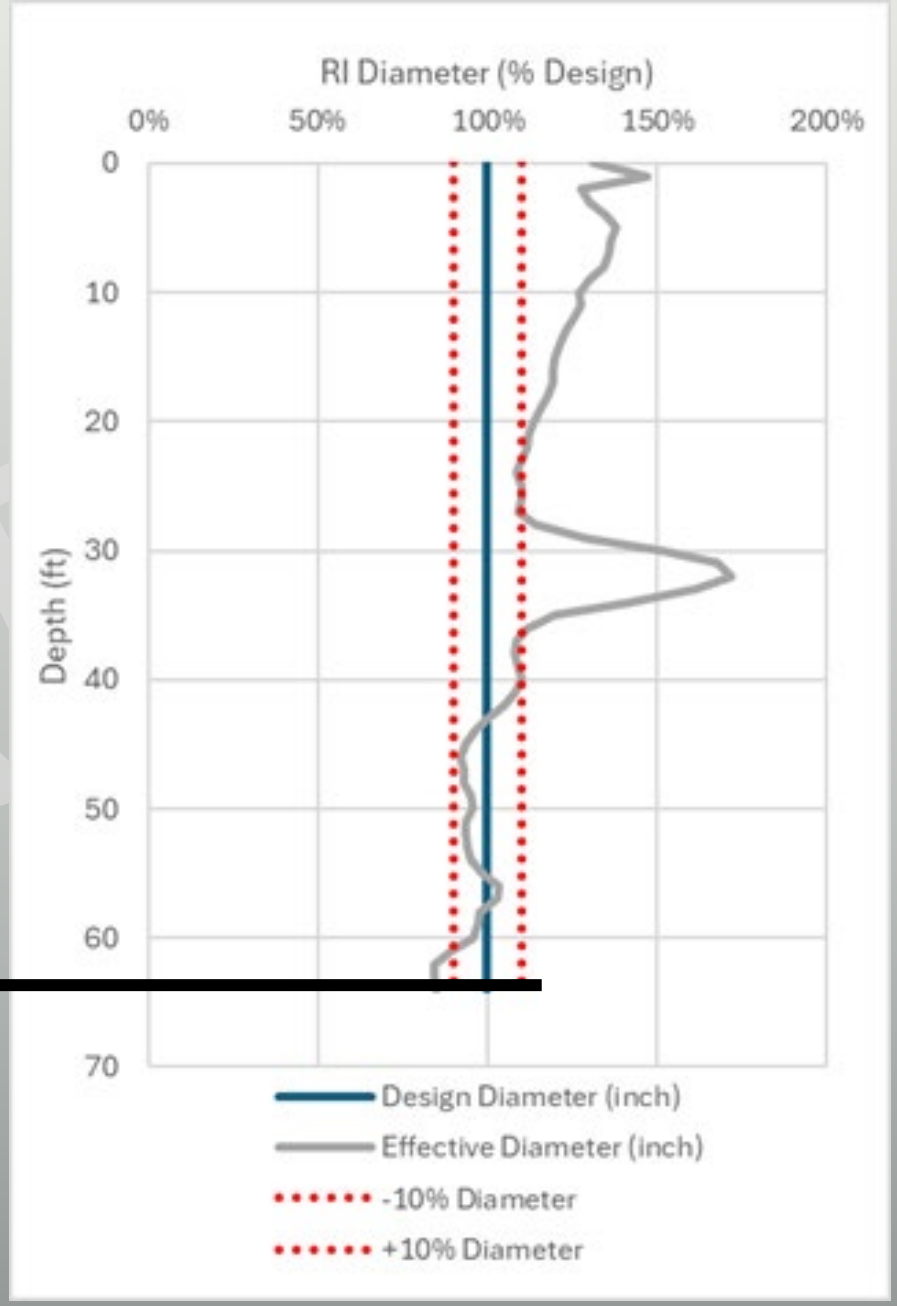
Test Results - AQTs



Test Results - TIP



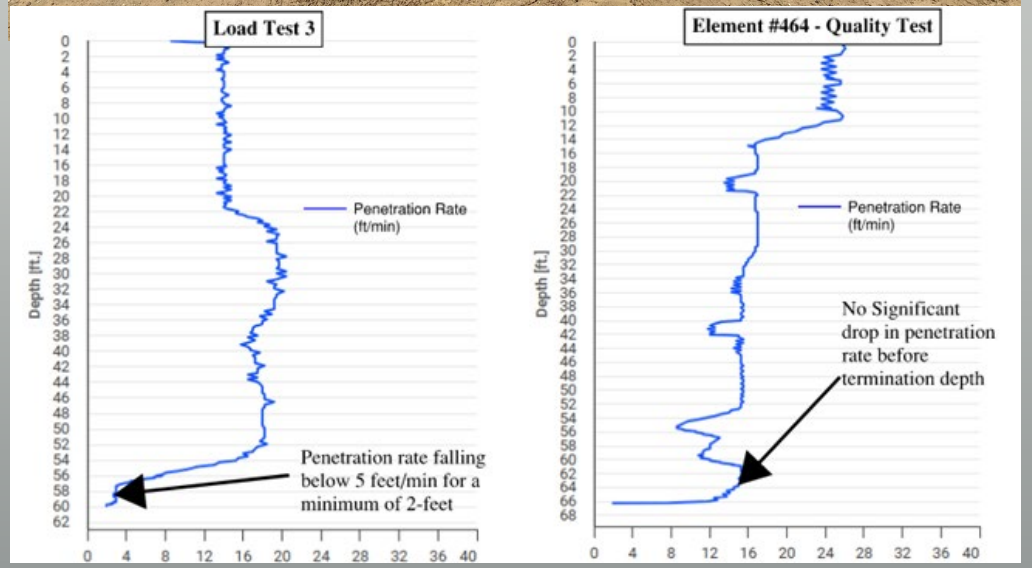




Summary and Conclusions

- Have an expectation of what **“good”** looks like.
- Measure While Drilling (MWD) data offers valuable insights during installation.
- Deviations from acceptance criteria do not necessarily indicate unacceptable performance
- Supplemental tools like the ASIRI Quality Test and Thermal Integrity Profile Test can be used inconjuction with MWD data to assess RI performance more accurately and reduce unnecessary rework.
- A proactive approach to quality on a challenging site demonstrated how testing can be used to reduce rework.





DRAFT



KELLER

Thank You!

Question and Response

THINK SAFE





DEEP FOUNDATIONS INSTITUTE
DFI®



United Rentals®

Smarter Shoring for a Greener Future

Chelsea N. Miller, P.E.

United Rentals Trench Safety

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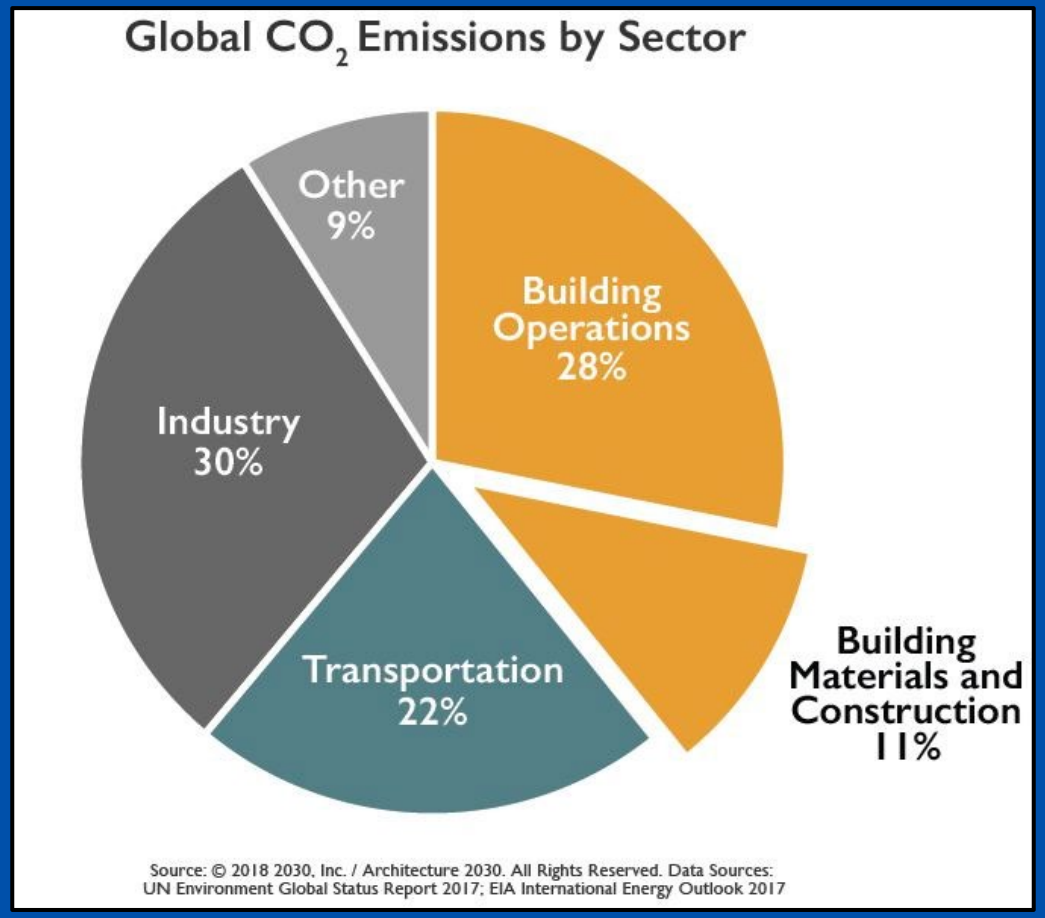
Presentation Roadmap

- *Why Sustainable Shoring?*
- *What is Sustainable Shoring?*
- *Case Study*
  - *Life Cycle Assessment (LCA) Methodology*
  - *Results and Carbon Savings*
  - *Economic Benefits*
- *Concluding Remarks*

Why Sustainable Shoring?



Why Sustainable Shoring?



Why Sustainable Shoring?







Why Sustainable Shoring?











What is Sustainable Shoring?

Embodied Carbon = the sum of all greenhouse gas emissions associated with a product or processes that contribute to the global warming potential over 100 years.

Materials: Steel, concrete, aluminum, glass

Processes:

Driving 3 miles or using a light bulb for 10 hours



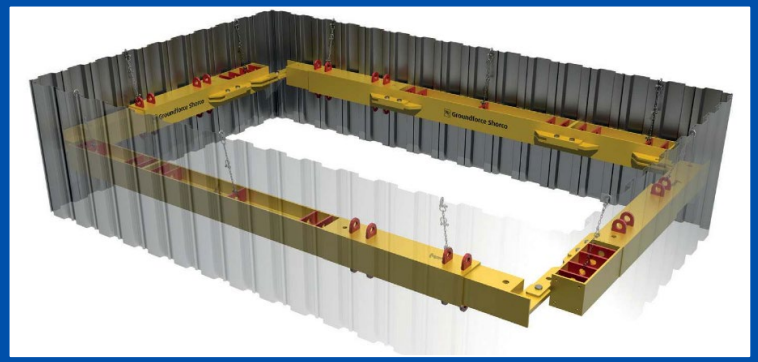
The Hidden Carbon of Temporary Works





Groundforce Shorco’s Mega Brace System

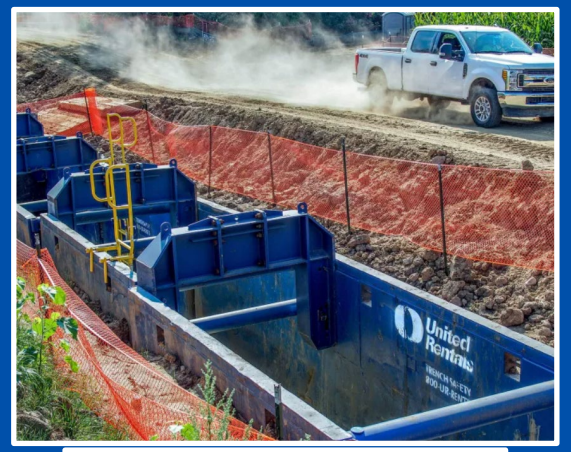






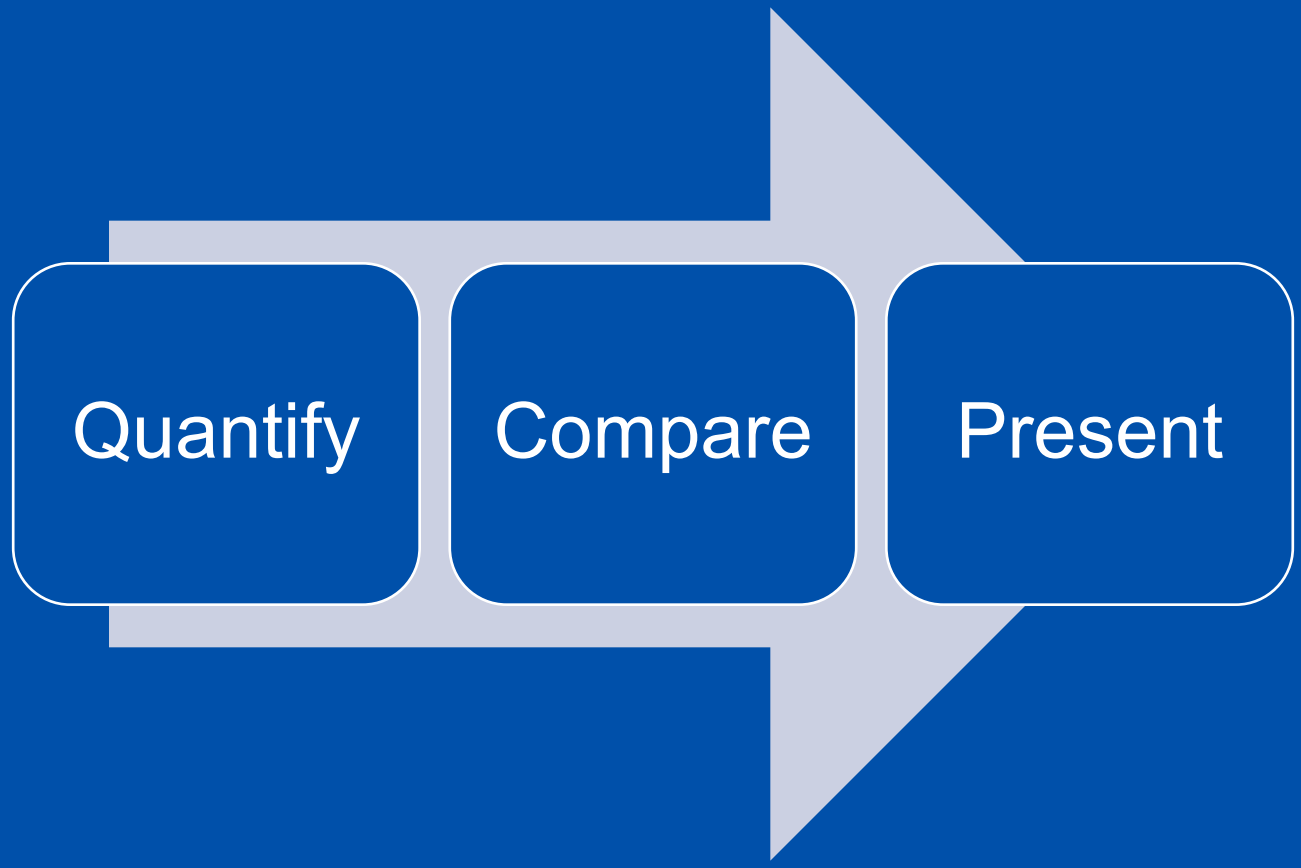
GME|E+S Slide Rail System



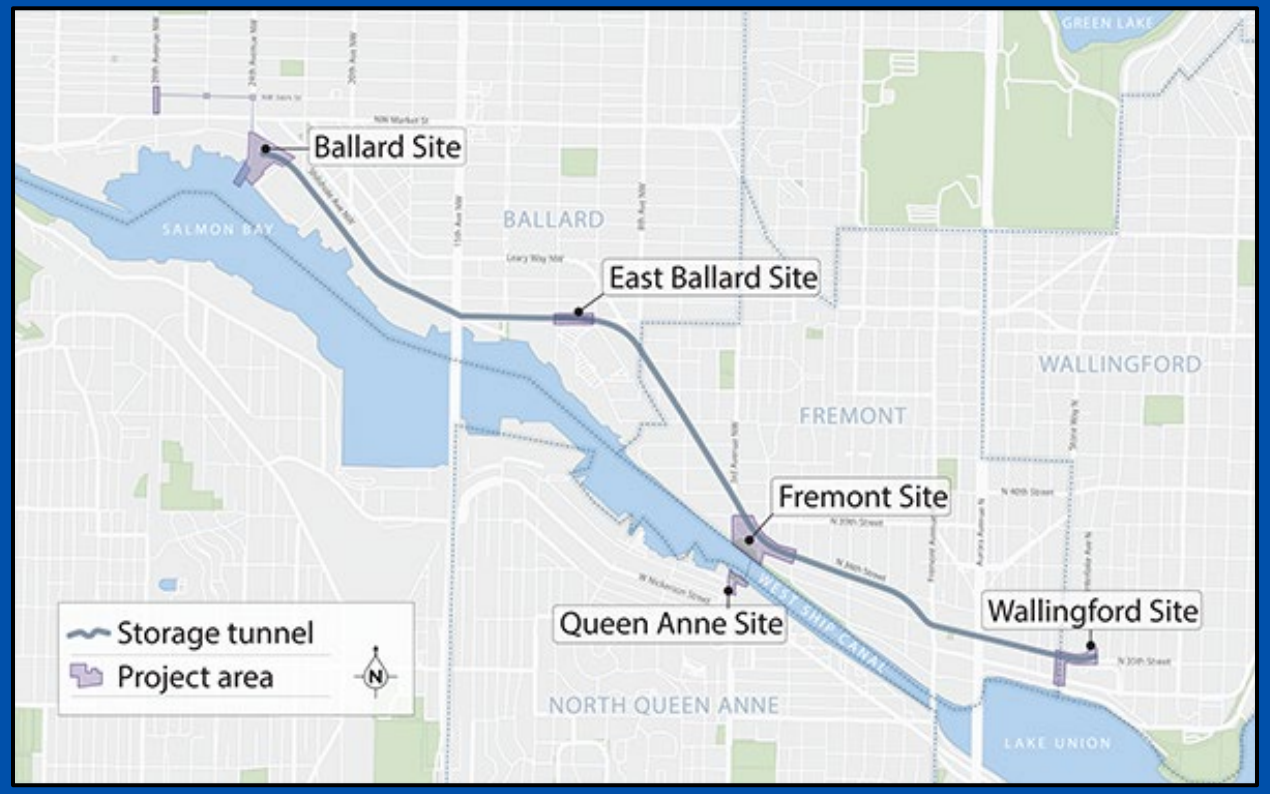




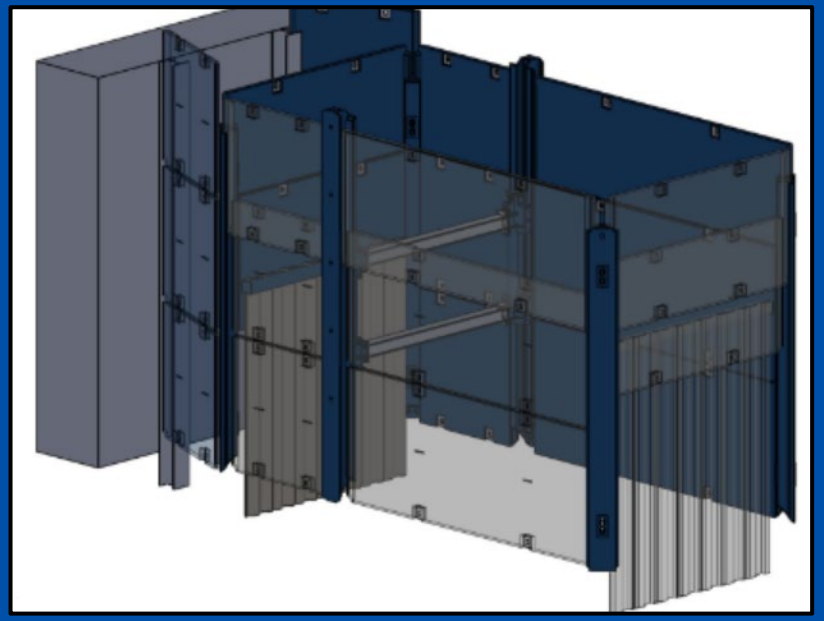
Study Objectives

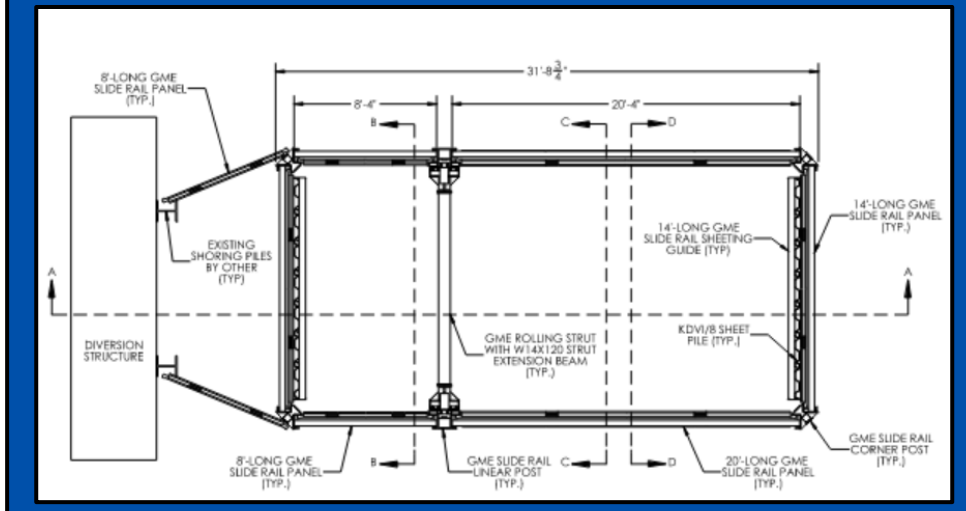


Case Study – Ship Canal Water Quality Project



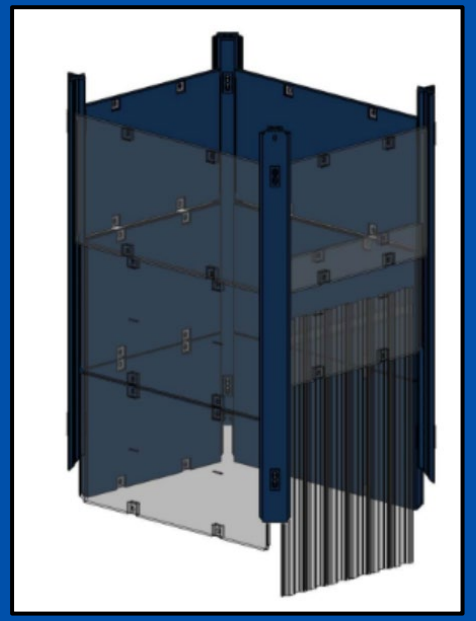
Case Study – Ship Canal Water Quality Project

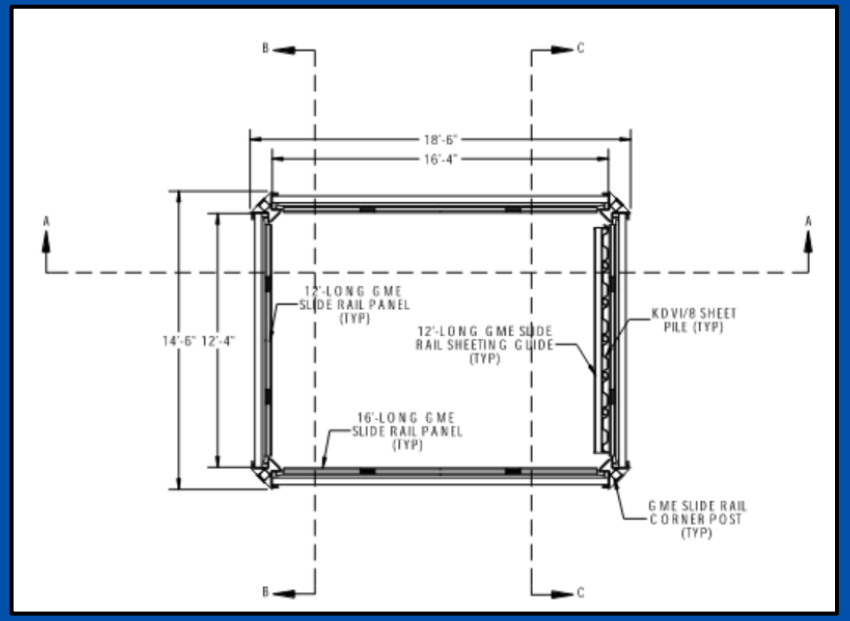




Boring Pit – Two Bay Slide Rail

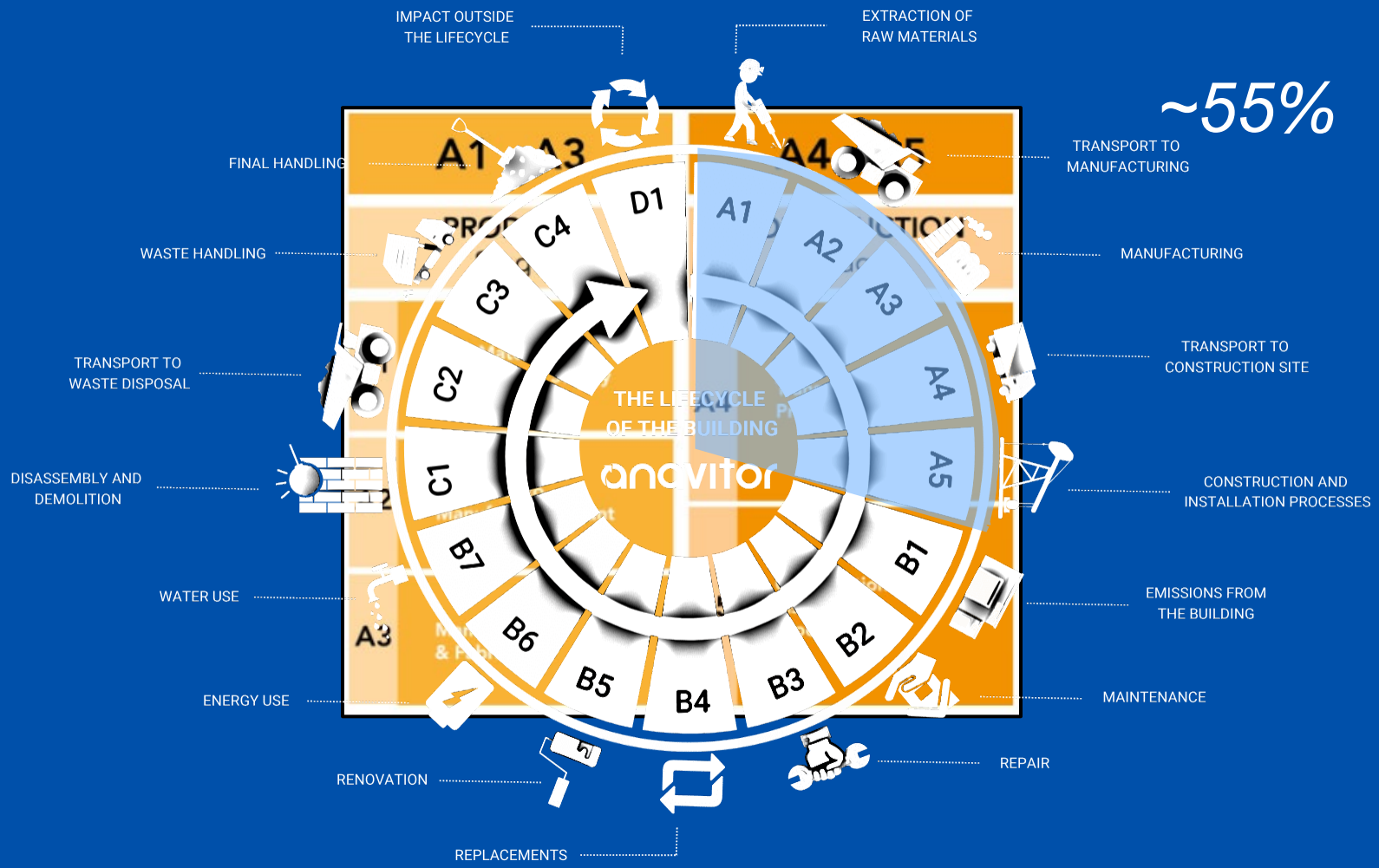
Case Study – Ship Canal Water Quality Project





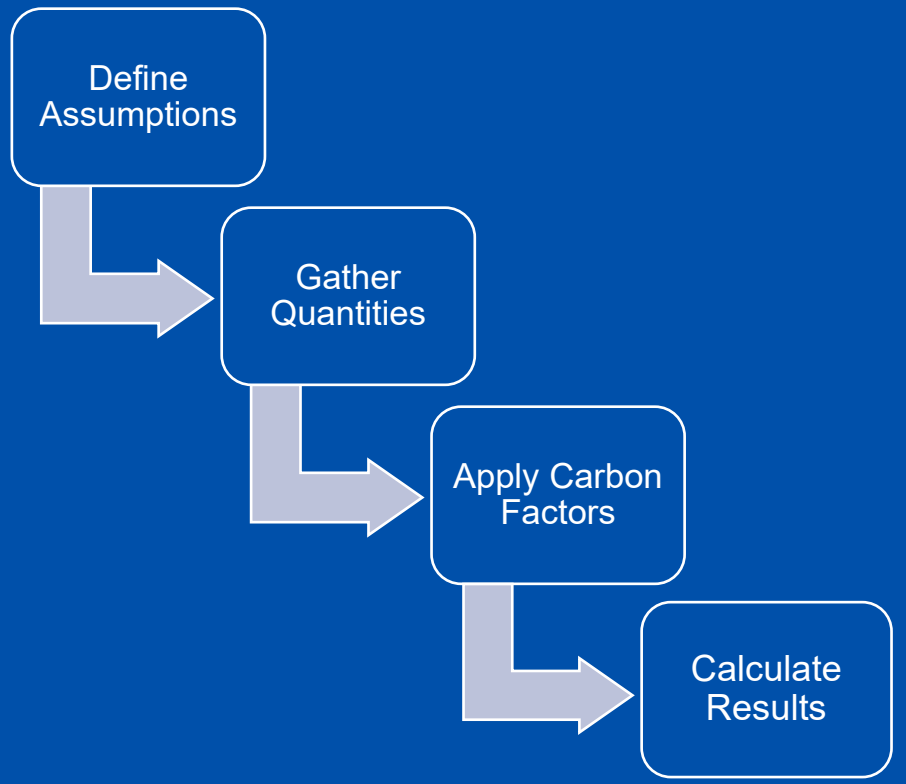
Receiving Pit –Slide Rail Pit

LCA Methodology



~55%

Step 1: Quantify - LCA Methodology



Step 1a: Quantify – Define Assumptions

- Distance materials travel?
  - Source to fabrication
  - Fabrication to the site
- Typical life span of materials?
- Will the materials end up as waste or be recycled?

Step 1b: Quantify – Gather Quantities

Conventional Steel

Material Type	Description	Volume [m^3] or Mass [kg]?	Material Quantity [m^3, kg]
Structural_Sections	(8) Soldier Pile (W14x120)	Mass [kg]	14,805
Plate	1-1/2" Steel Plate Lagging	Mass [kg]	42,674
Structural_Sections	(3) Rings Walers (W24x192) (56')	Mass [kg]	14,631
Structural_Sections	(12) Soldier Pile (W14x120)	Mass [kg]	18,492
Plate	1-1/2" Steel Plate Lagging	Mass [kg]	58,677
Structural_Sections	(5) Rings Walers (W24x192) (28')	Mass [kg]	12,192
Structural_Sections	(5) Rings Walers (W32x256) (56')	Mass [kg]	32,513
			193,984

Slide Rail

Material Type	Description	Volume [m^3] or Mass [kg]?	Material Quantity [m^3, kg]
Steel_EPD	GME Slide Rail	Mass [kg]	389,209

Mega Brace

Material Type	Description	Volume [m^3] or Mass [kg]?	Material Quantity [m^3, kg]
Galvanised_profiled_sheet	PZ-27 Sheets	Mass [kg]	66,281
Steel_EPD	(12) HYD, (6) 1M, (4) 5M	Mass [kg]	20,380
			86,661

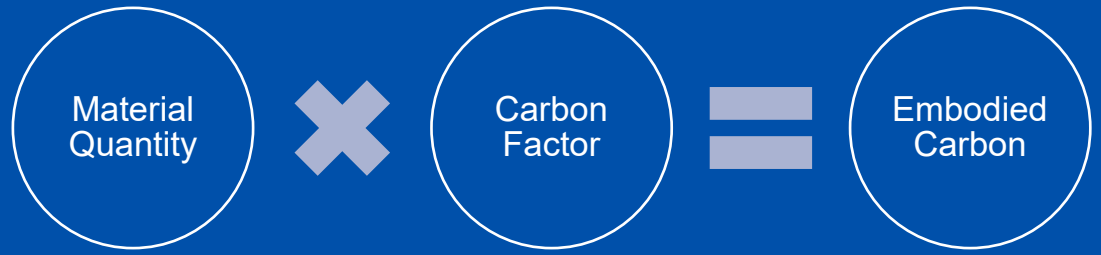
Step 1c: Quantify – Apply Carbon Factors

Steel Product	Unit	Emission Factor (kg CO_2e / t)	Source
Steel sheets	t	1,861.0	Ecoinvent v3.9
Steel rebars	t	1,838.0	Ecoinvent v3.9
Steel tubes	t	2,332.0	Ecoinvent v3.9
Steel wires	t	2,259.0	Ecoinvent v3.9

Mode / Vehicle Type	Unit	Emission Factor (kg CO_2e)	Source
Diesel	liter	3.20	Defra 2022 (v2)
Road – Rigid >3.5–7.5t	vkm	0.61	Defra 2022 (v2)
Road – Rigid >7.5–17t	vkm	0.74	Defra 2022 (v2)
Road – Rigid >17t	vkm	1.21	Defra 2022 (v2)
Road – Articulated >3.5–33t	vkm	0.95	Defra 2022 (v2)
Road – Articulated >33t	vkm	1.13	Defra 2022 (v2)
Maritime – General cargo	tkm	0.02	Defra 2022 (v2)
Maritime – Container	tkm	0.02	Defra 2022 (v2)

EEFC DFI Carbon Calculator Tool (2022).

Step 1d: Quantify – Calculate Results



$$EC_{A15} = EC_{A13} + \Sigma[Q_i(ECF_{A4} + ECF_{A53})] + EC_{A52}$$

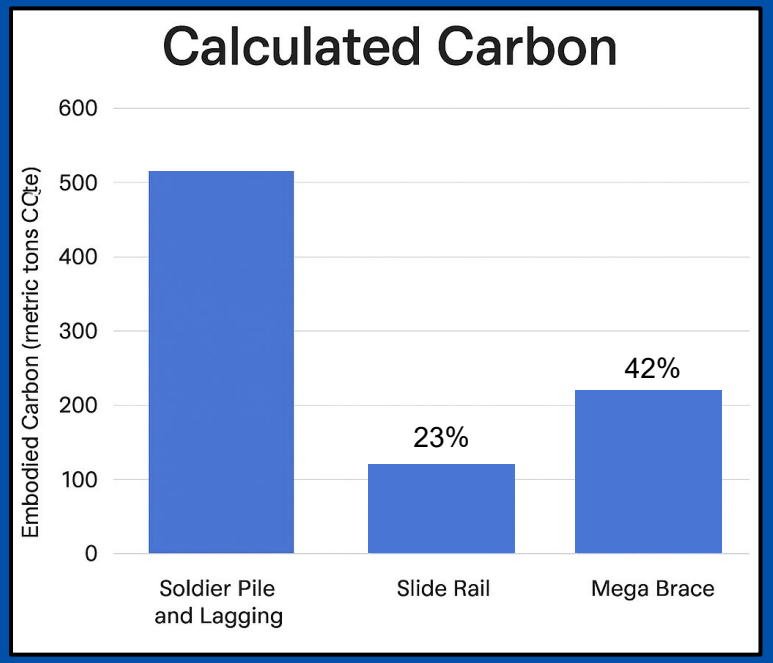
Product Stage
Cradle to Site

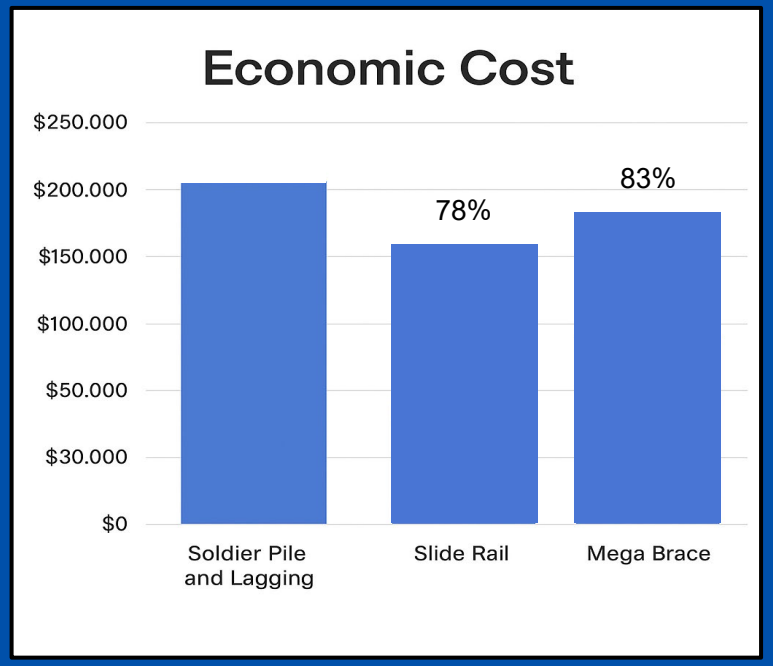
Transport Stage

Waste & Waste Management
1% for steel

Construction Activities
~small

Step 2: Compare





Step 2: Compare

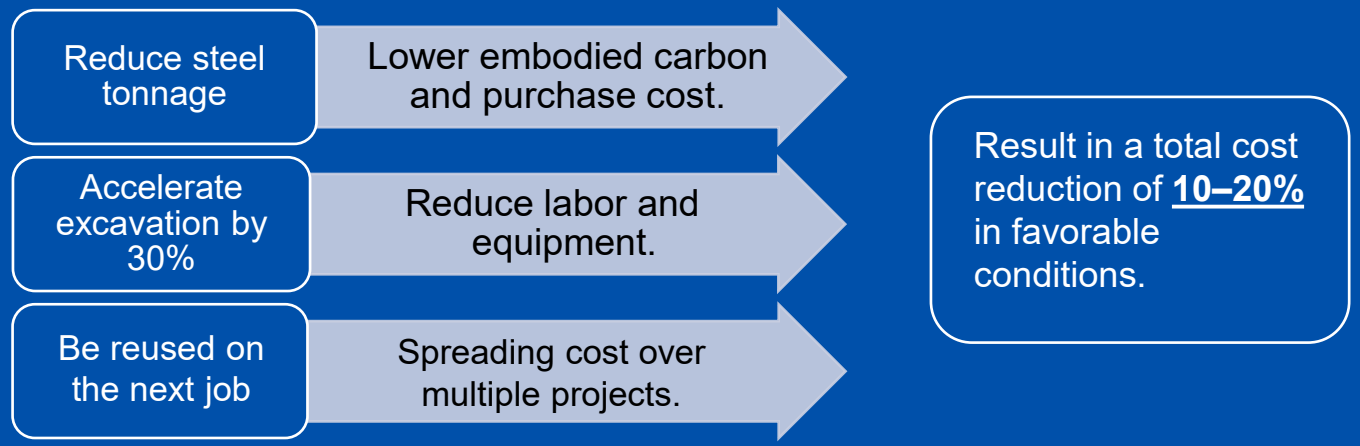
Economic Incentives to Cutting Carbon?

- Material efficiency = lower material costs
- Reduced transportation and handling costs
- Faster installation and removal = labor & equipment savings
- Reusability of materials
- Reduced overhead from schedule compression
- Potential incentives or market advantage

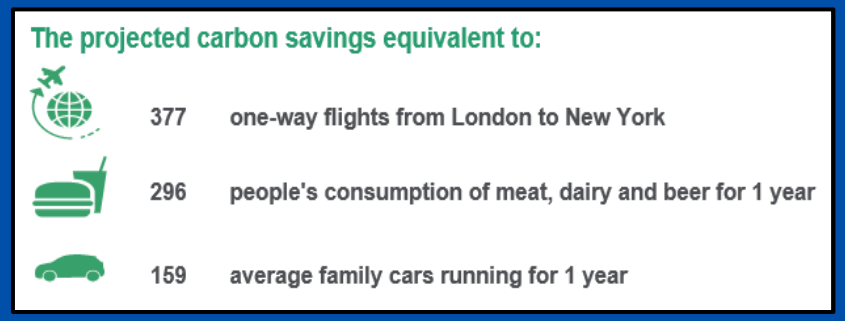


Step 2: Compare

Cost Savings Example



Step 3: Present





Slide Rail = 77% reduction
Mega Brace = 58% reduction

2,500 designs/year →
Potential 980,000+ tons CO2e saved

Questions?



Managing Risk in Critical Marine Infrastructure

O-Cell Bi-Directional Load Testing

DFI50 - MSSP

K. Vembu, P. Peralta, J. Sumputh, P. Bullock

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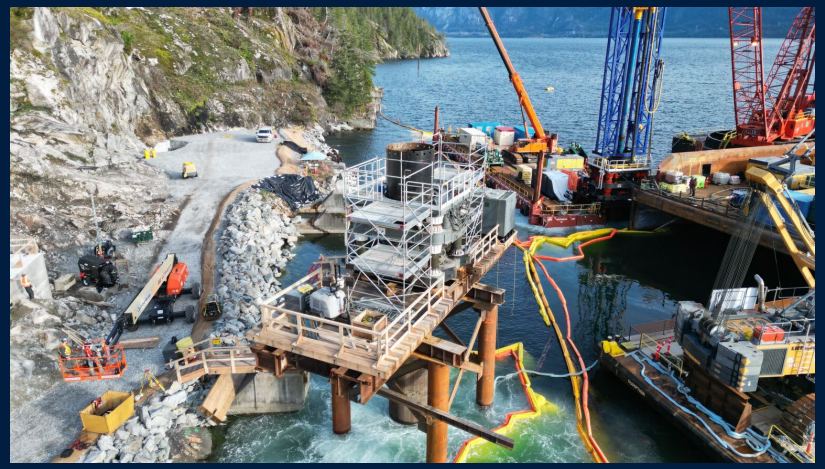
Overview of Contents



- Managing risk in geotechnical engineering
- Predictions and risk management is a process
- How pile testing enhances predictions
- Standards and safety factors for pile tests
- O-Cell® bi-directional load testing
  - Advantages
  - Applications
  - Offshore testing
  - Project experience

Managing Risk in Geotechnical Engineering

"Predictions have always played a significant role…Man apparently feels that if he can predict events correctly, he can act in his own best interests, and possibly alter the events he has predicted, i.e. *control his future*."

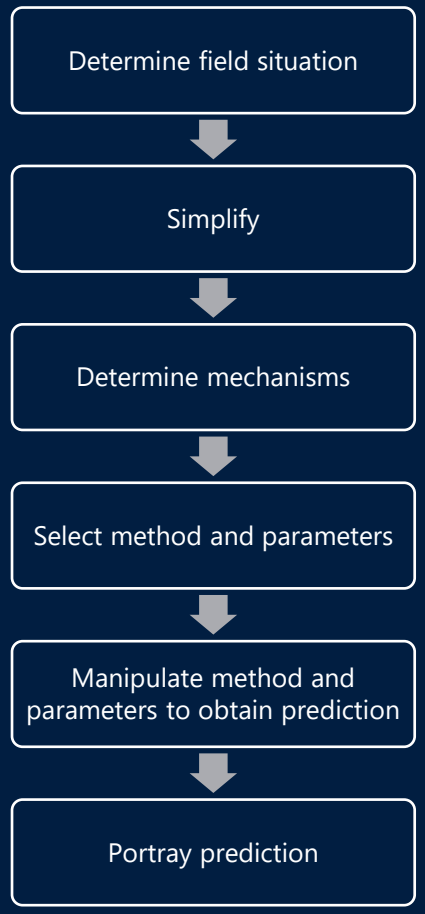
"Predicting constitutes an integral component – *the very heart* – of the practice of civil engineering. The successful engineer must not only predict but also make decisions and take actions on the basis of predictions. For example, the engineer must identify predictions which are critical to the safety, function, and economics of the project at hand, estimate the reliability of each prediction, employ predictions in design and construction, *assess the consequences of predictions*, especially erroneous predictions, and must select and execute appropriate actions based on comparisons of actual situations as they unfold and predictions."

"The *soil engineer may be the master predictor*. He must work with incomplete, widely scattering and varying data…"

T.W. Lambe, Rankine Lecture (1973)

Managing Risk in Geotechnical Engineering

Predictive process in geotechnical engineering (Lambe, 1973)

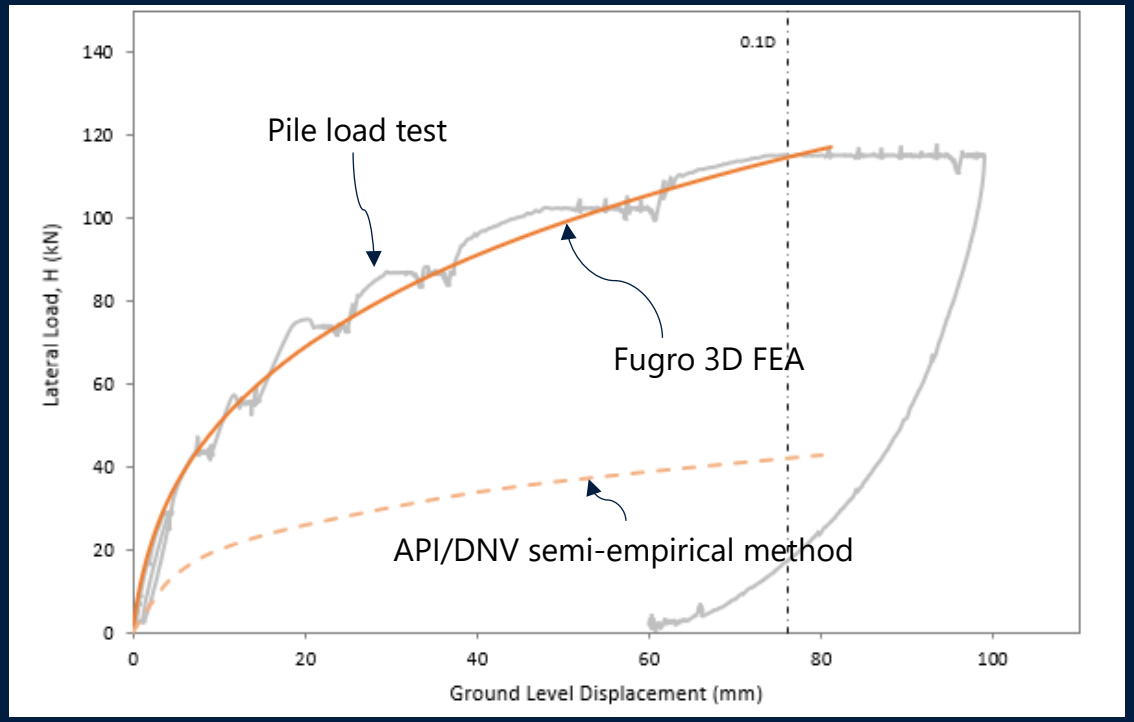


Prediction type	When prediction made	Results at time prediction made
A	Before event	-
B	During event	Not known
B1	During event	Known
C	After event	Not known
C1	During event	Known

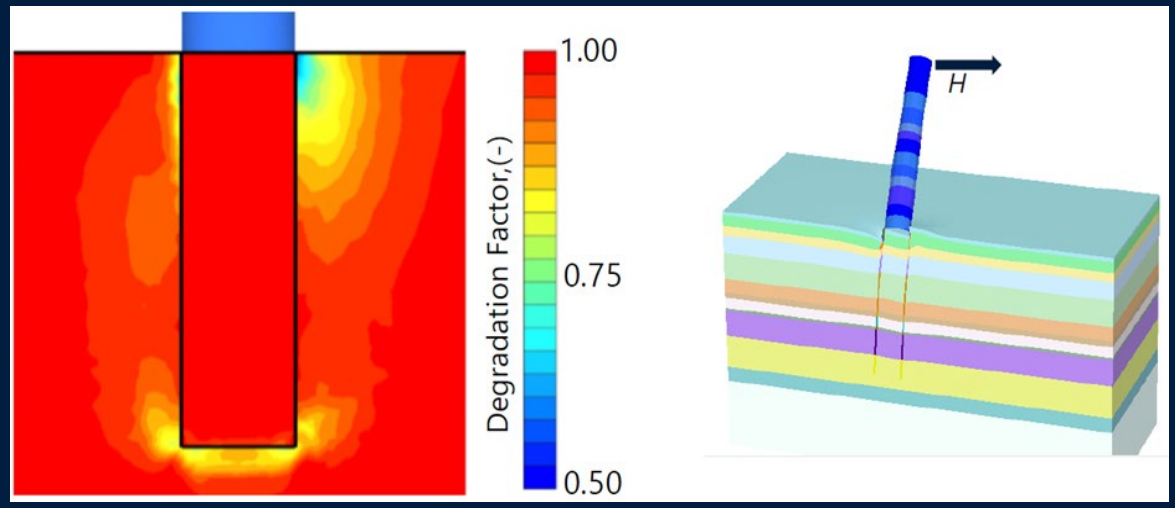
Pile Testing

Lateral pile load testing for monopiles for offshore windfarms (PISA)

(Byrne et al., 2017)







Pile Testing

- Full scale pile load testing is beneficial on projects when the potential cost benefits and/or risk reductions outweigh the cost of performing the tests itself, e.g.:
  - Reduction in total pile length due to higher shaft friction and/or end bearing resistance than anticipated
  - Risk reduction by refined input for the design and construction of the foundation
- Pile load tests are usually classified based on type of tests and procedures that offer varying levels of confidence in the pile's performance.
- The level of confidence is tied to whether the pile's ultimate capacity was confirmed by static load tests, which in turn influences the partial factor of safety used in the design

Pile Testing: Standards and Factor of Safety

- ASTM D8169 – Bi-directional static load tests
- ASTM D1143 – Static axial compression load testing
- ASTM D3689 – Static axial tension (uplift) load tests
- ASTM D3966 – Lateral load testing
- ASTM D4945 – High-strain dynamic testing

Global Factors of safety (Likins. G, 2004)

Code **Design loads**	**PDCA**	**AASHTO**	**IBC 2000** **>40T**	**Aus AS2159-95**	**ASCE for driven pile**			**ASCE (non-driven piles) >100 T**
					16 - 40 T	**40 - 100 T**	**>100 T**	
Static analysis (no test)	3.5	3.5	6.0	2.12 – 3.44	NA	NA	NA	NA
Static testing	1.8 – 2.0	2.0	2.0	1.53 – 1.93	1.5 – 1.8	1.6 – 1.9	1.8 – 2.2	2.3 – 3.2
Dynamic testing	1.9 – 2.1	2.25	2.0	1.72 – 2.12	1.6 – 2.0	1.7 – 2.0	2.0 – 2.4	2.6 – 3.6
Wave equation	2.5	2.75	NA	2.50 – 3.06	1.8 - 2.2	1.9 – 2.3	NR	NA
Notes: NR – Not recommended, NA – Not applicable								

Pile Testing: Standards and Factor of Safety

EM 1110-2-2906 US Army Corps of Engineers

Method of Determining Capacity	Loading Condition	Minimum Factor of Safety	
		Compression	Tension
Theoretical or empirical prediction verified by pile load test	Usual	2.0	2.0
	Unusual	1.5	1.5
	Extreme	1.15	1.15
Theoretical or empirical prediction verified by pile driving analyzer	Usual	2.5	3.0
	Unusual	1.9	2.25
	Extreme	1.4	1.7
Theoretical or empirical prediction not verified by load test	Usual	3.0	3.0
	Unusual	2.25	2.25
	Extreme	1.7	1.7

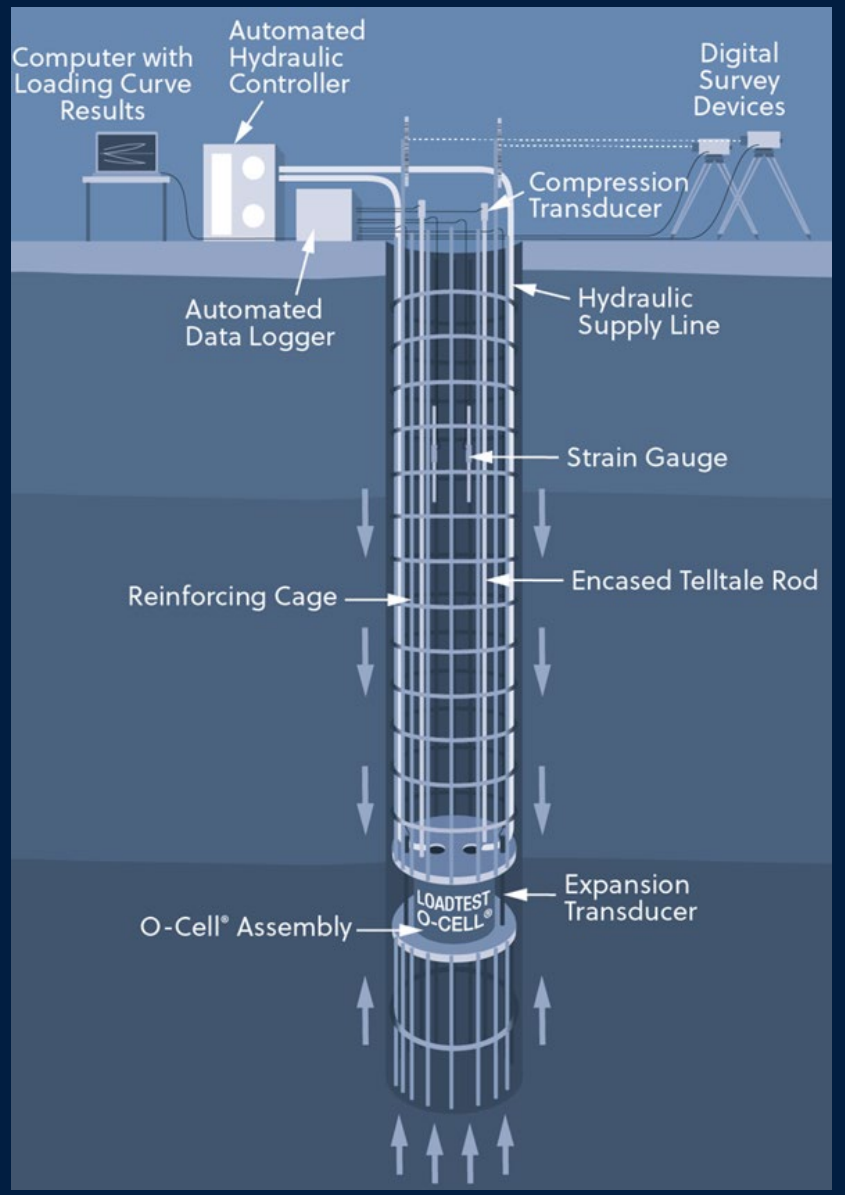
Eurocode 7, BS EN 1997-1

- Distinguishes tests by their function and how the results are used to calculate the final design resistance.
  - Trial pile load test
  - Working pile tests
- More tests lead to lower correlation factors and a less conservative design.

Feature	Trial pile tests (Class A Prediction)	Working pile tests (Class B Prediction)
Test piles	Conducted on sacrificial piles, installed specifically for testing before construction	Performed on working piles, part of final structure
Purpose	Establish ultimate capacity for design and verify design assumptions	To prove adequacy of final working piles and confirm quality
Loading protocol	Loaded 2 to 3 times of working load	Loaded to predetermined load (around 1.5 times the working load)
Acceptance criteria	Determine ultimate capacity which is then used to derive allowable working load with appropriate factor of safety	Analysis checks the pile's settlement remains within the acceptable limit at the proof load
Outcome for project	Results used to optimize design, potentially allowing reduced factor of safety and leading to a more economical foundation	Results confirm that the working piles are performing as expected. If fails decision to made to replace or undertake remedial action

O-Cell Pile Testing

- Alternative to the traditional static pile load testing
- Bi-directional O-cell load testing can be performed for different foundation types such as drilled shafts, driven piles, rock sockets, CFA/auger cast piles, barrettes
- Onshore and offshore
- Static and cyclic axial pile test
- Lateral static pile test
- Maximum axial load test to date up 360 MN



Schematic o-cell (axial) pile test

O-Cell Bi-Directional Pile Instrumentation



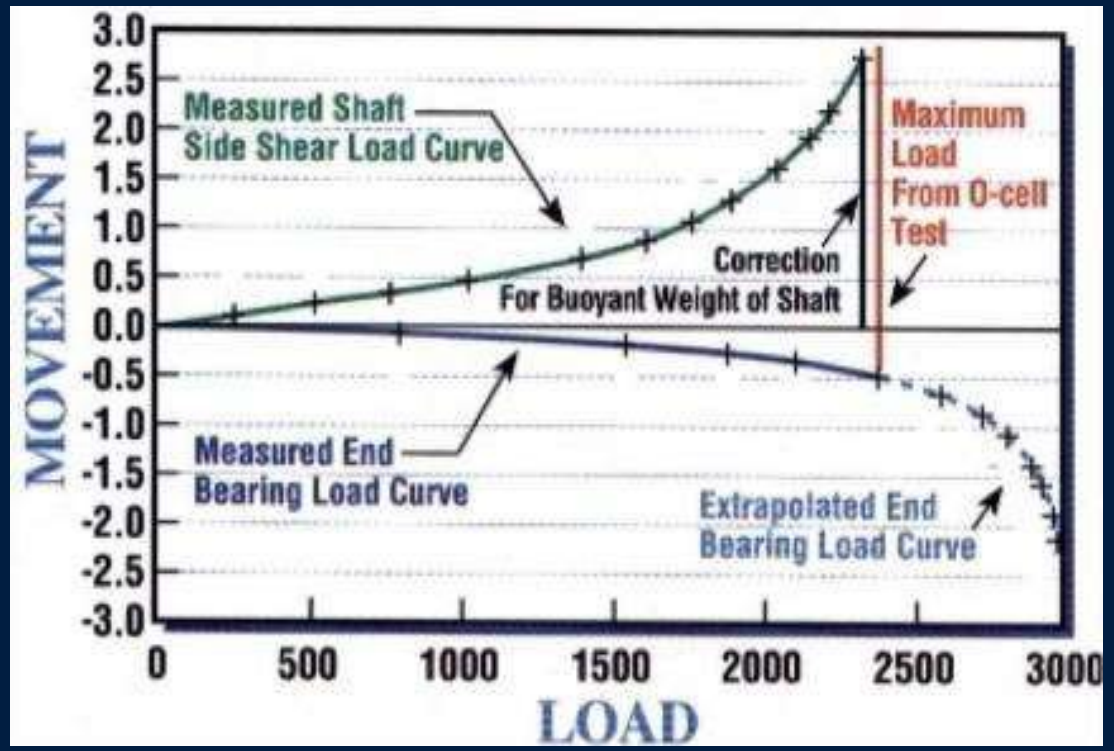


O-Cell Bi-Directional Pile Load Testing

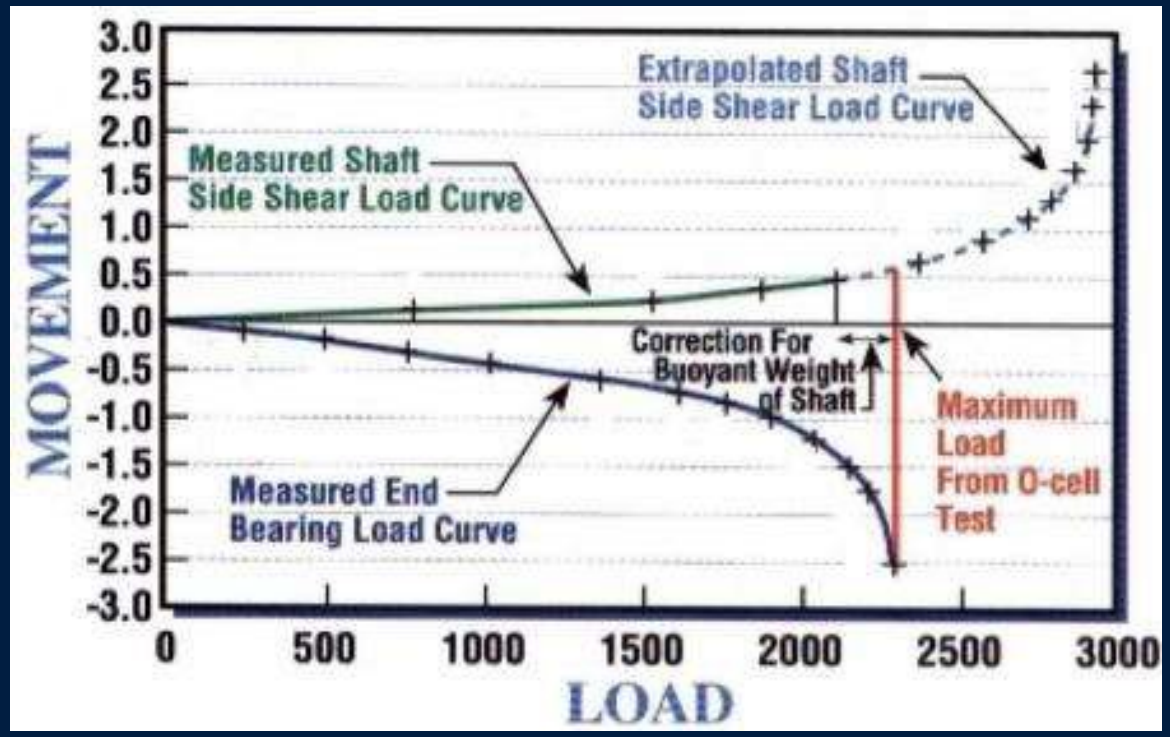




Axial Pile Load Displacement Curves (O-Cell Test)



A typical bi-directional load test where the ultimate side shear capacity was reached

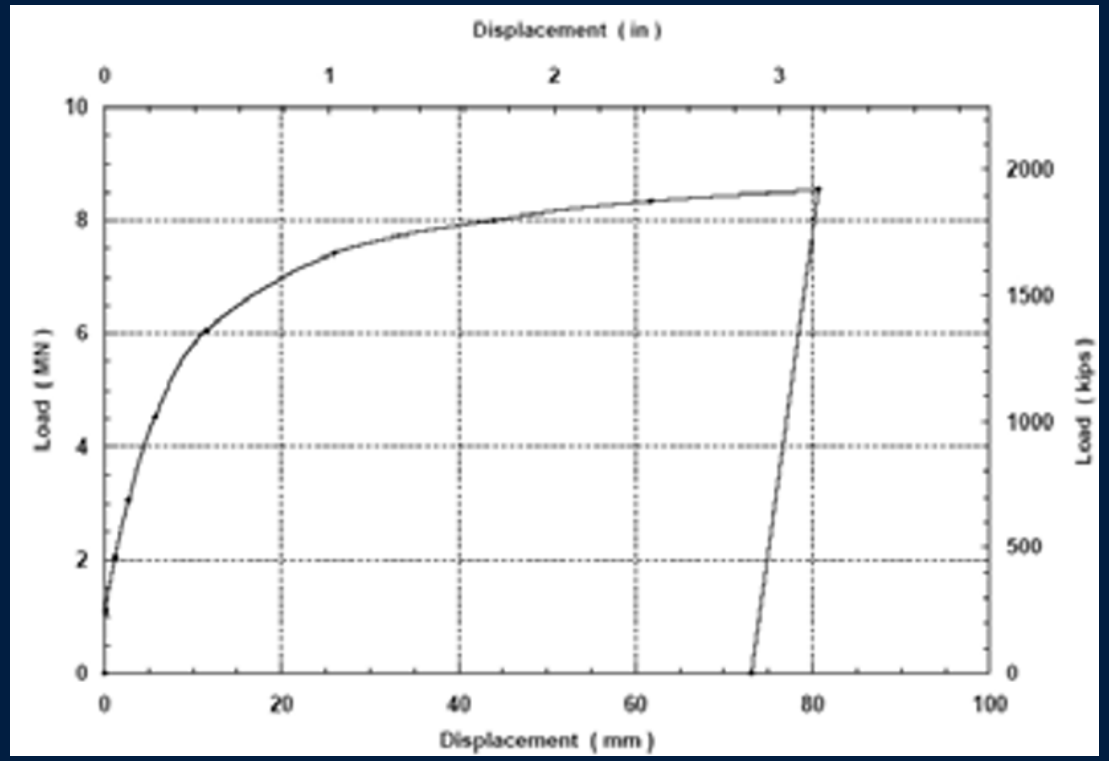


A typical bi-directional load test where the ultimate end bearing capacity was reached

O-Cell Split Lateral Pile Testing

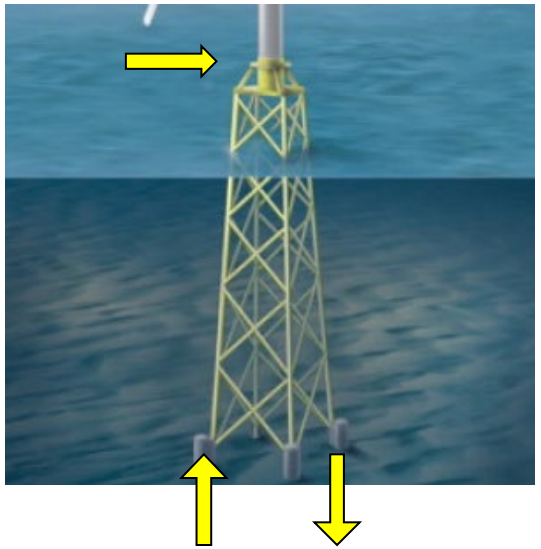
- Determines lateral stiffness of soil or rock formations at specific depths along a deep foundation
- O-Cell is assembled with the axis of the cell perpendicular to the shaft.
- The data gathered can be used to derive lateral soil resistance curves (i.e. p-y curves)



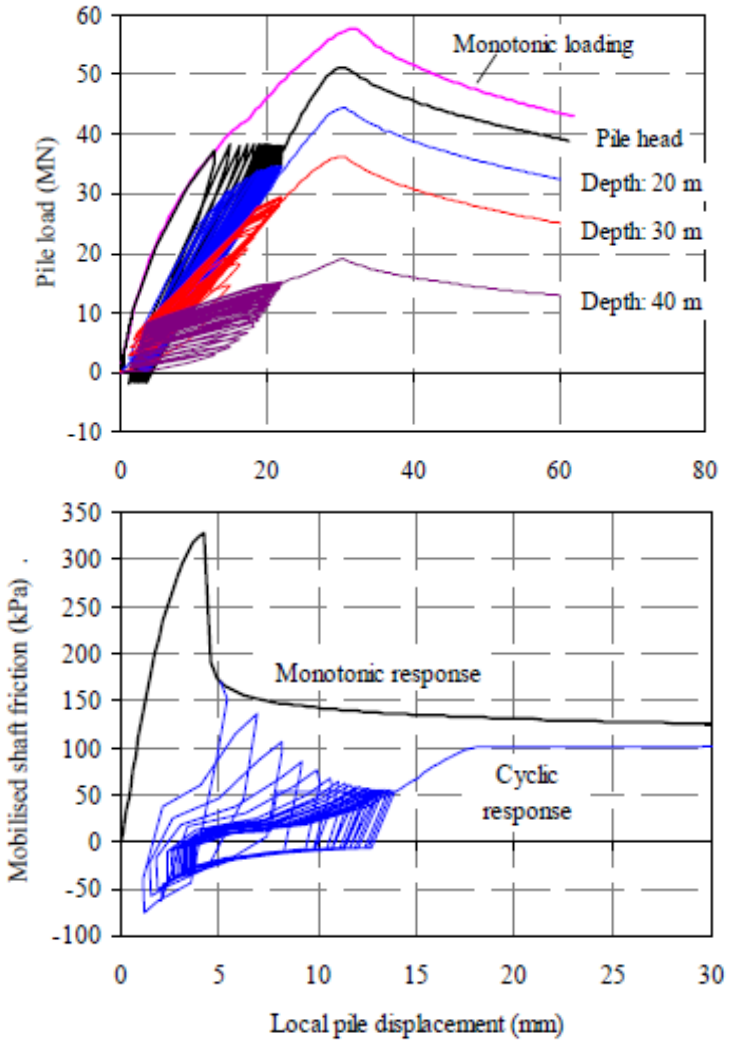


Project Highlight - Cyclic Axial Pile Testing

Offshore piled jacket foundation - Typical axial pile shaft friction response under storm (cyclic)



DFI 50th Anniversary Conference, Tennessee



Randolph, Cassidy, Gourvenec & Erbrich (2005)

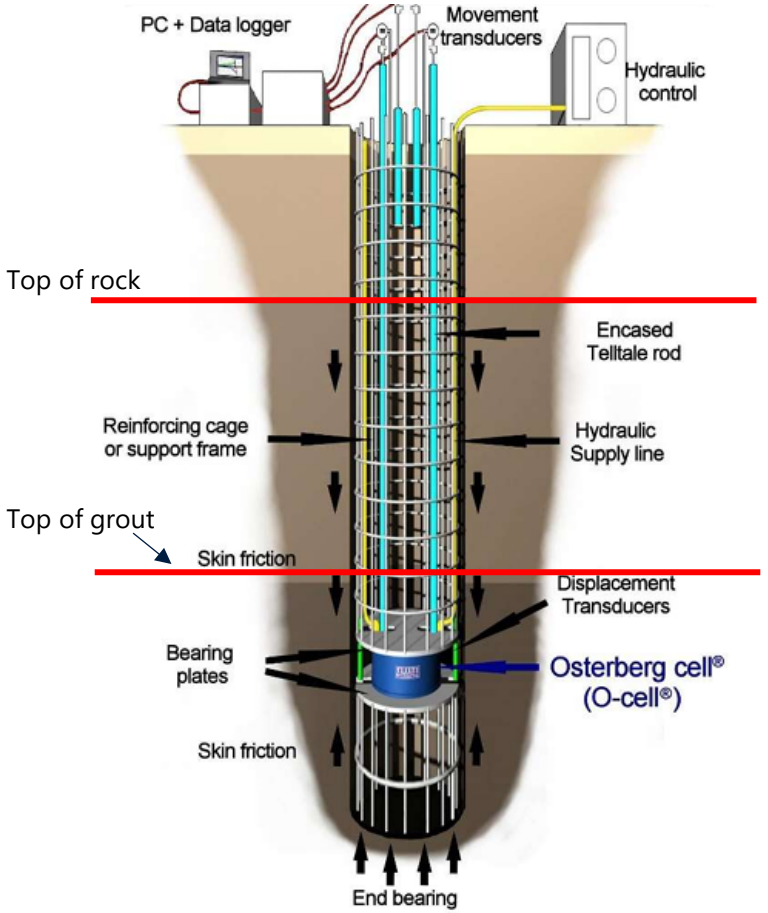
- Axial pile capacity and shaft friction degrades with applied load cycles
- Cyclic degradation of unit skin friction (t-z curves) and unit end bearing (Q-z curves) need to be considered

Project Highlight - Cyclic Axial Pile Testing

Offshore piled jacket foundation – Drilled and grouted pile

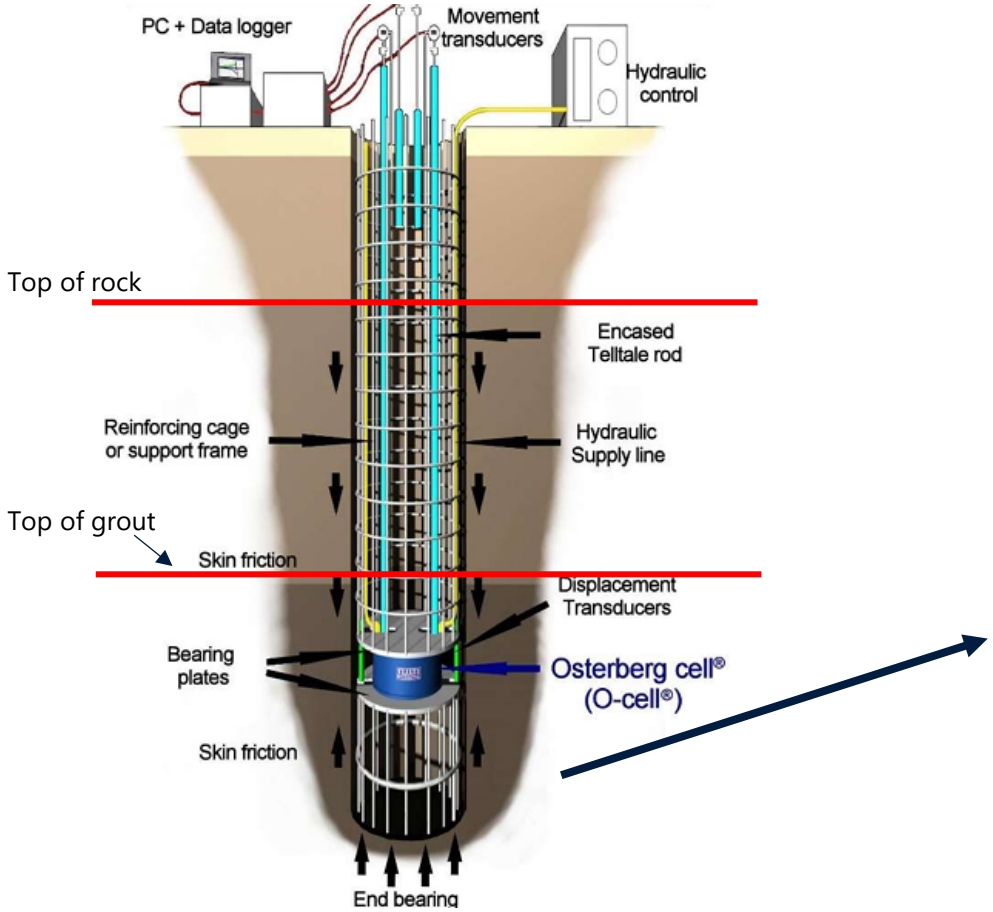
- Site conditions: very hard rock
- Diameter of grouted pile = 0.5 m
- Pile instrumented with linear vibrating wire displacement transducers (LVWDT) and fiber optics strain wires
- Applied static load greater than 30 MN
- Applied up 1000 load cycles up to 5 MN
- Period of each load cycle was 60 sec

Schematic



Project Highlight - Cyclic Axial Pile Testing

Schematic

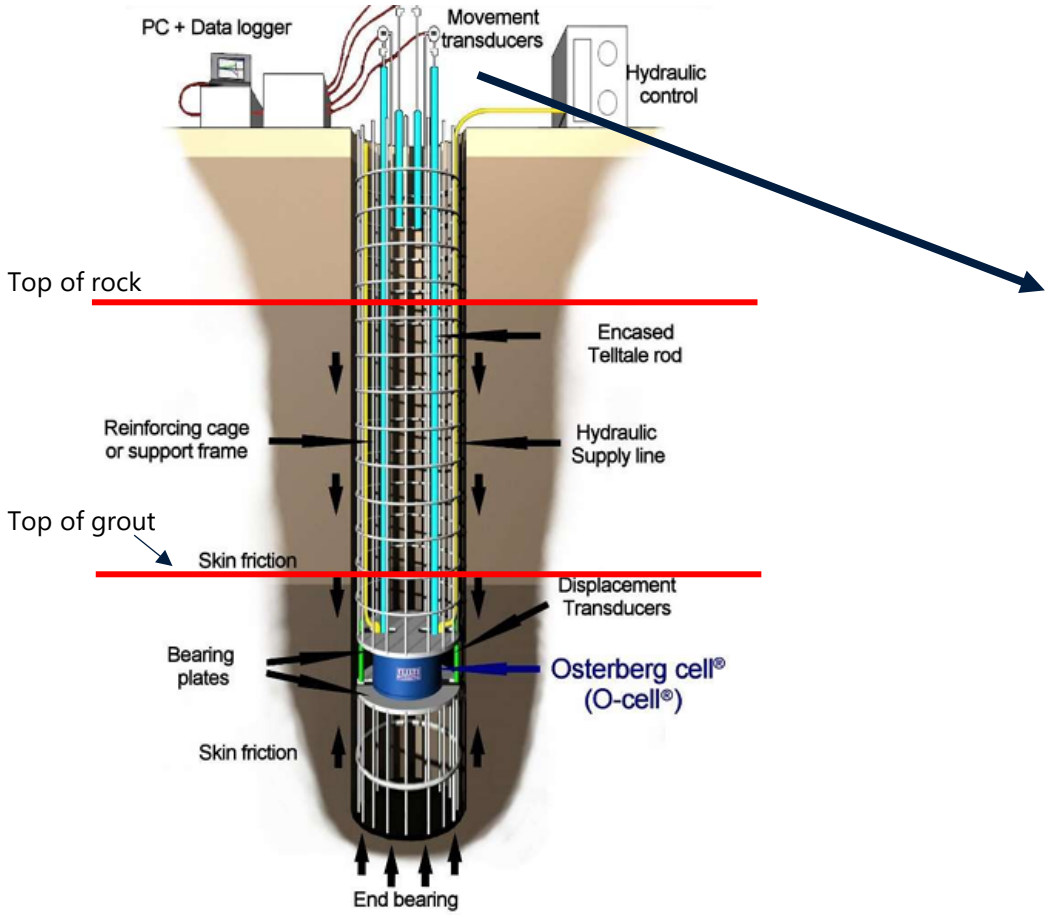


O-cell and transducers at base of grouted pile



Project Highlight - Cyclic Axial Pile Testing in Rock

Schematic

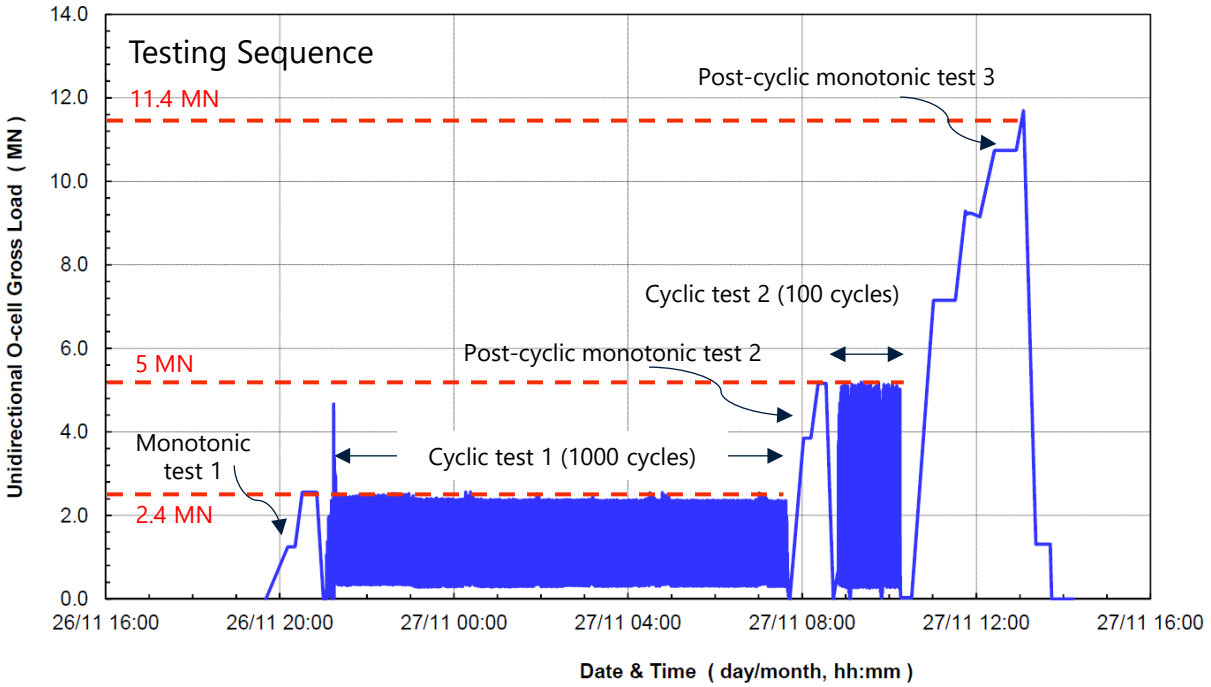


Transducers at pile head

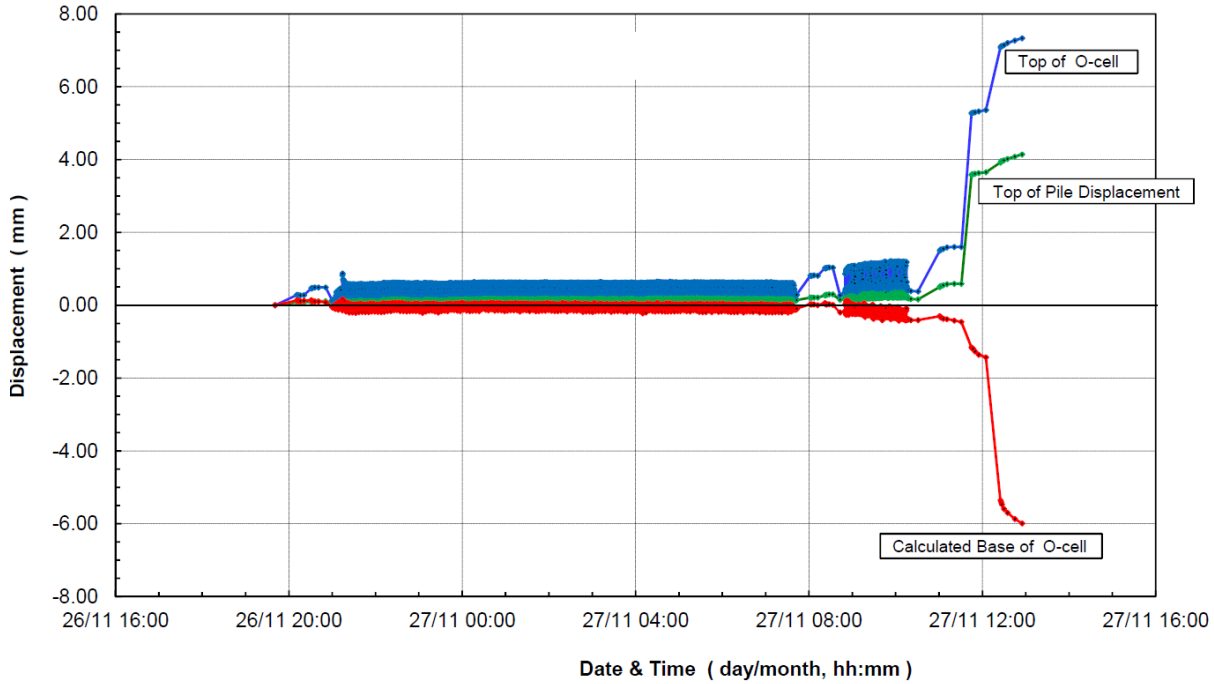


Project Highlight - Cyclic Axial Pile Testing

Load vs Time Plot

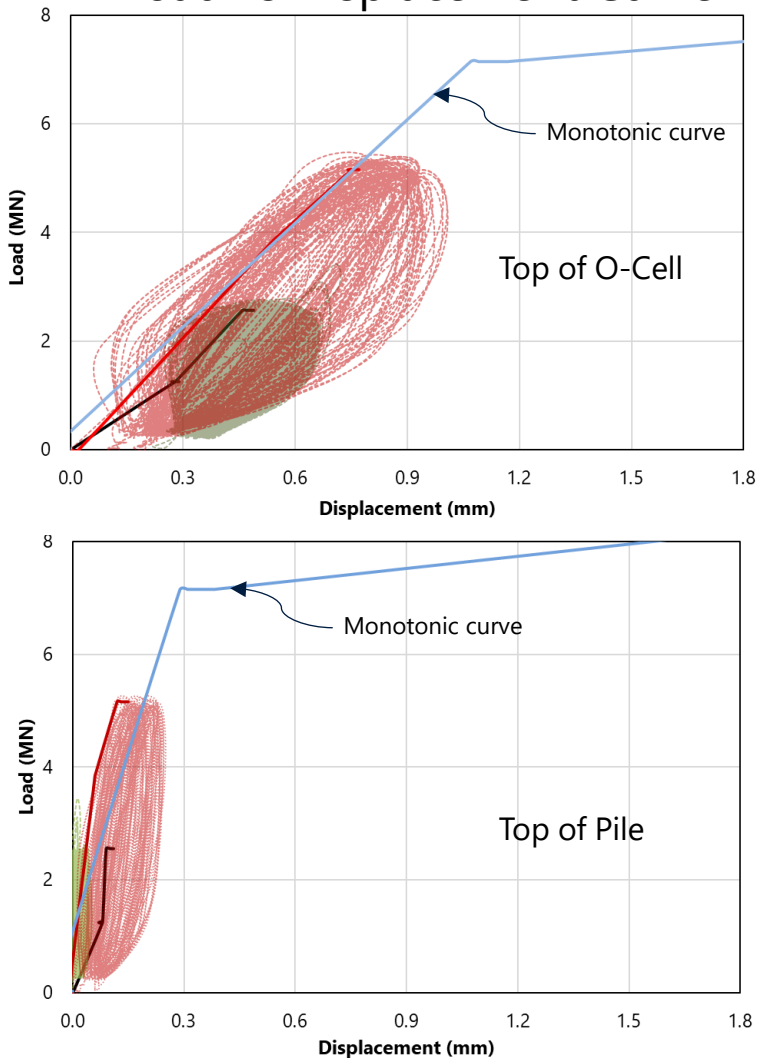


Displacement vs Time Plots

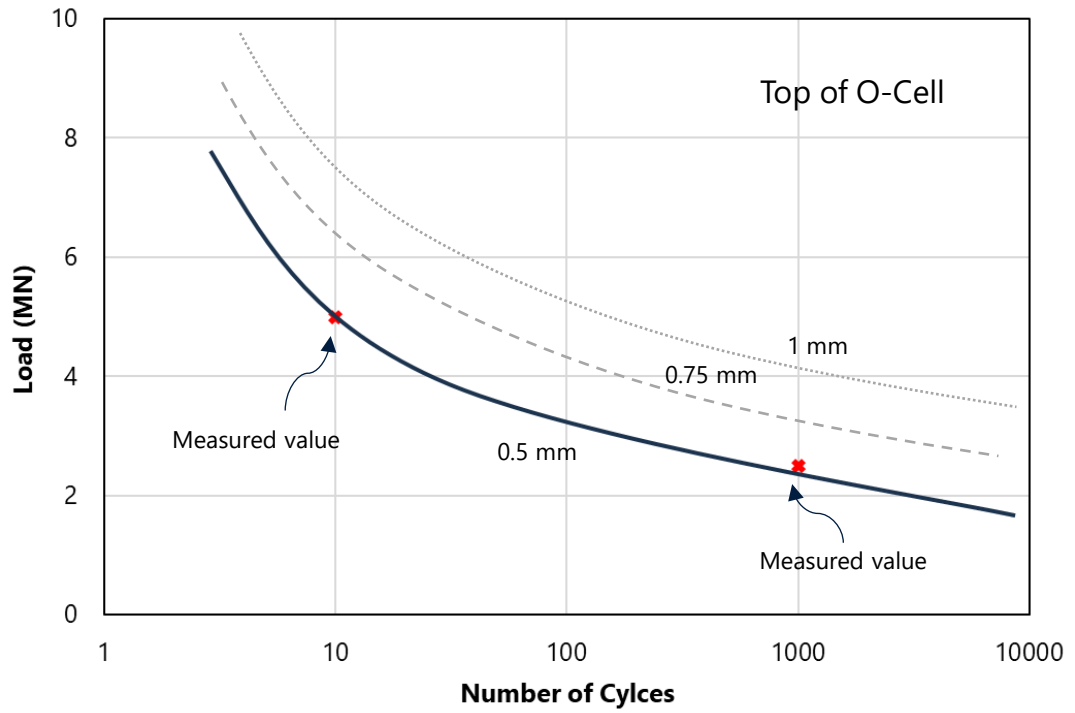


Project Highlight - Cyclic Axial Pile Testing

Load vs Displacement Curve



Interpreted Cyclic Degradation Curve



- Results show slight increase in stiffness of the monotonic curves after each cyclic loading stage
- Cyclic degradation for different displacement values can be plotted

Thoughts and Outlook

- Managing risk in geotechnical engineering problems means being able to accurately predict the outcome
- Pile testing is an integral part of the engineering predictive process
  - Increases reliability and confidence in the engineering methodology and predicted results
- Advancements in O-cell bi-directional pile testing allow us to model and understand pile-soil structure mechanisms and loading response in complex site conditions:
  - Pile testing in offshore and onshore environments
  - Axial pile testing with static and cyclic loading capabilities
  - Lateral pile load testing using the split pile with O-cells
- Our O-cell axial and lateral pile tests can be customized to fit any of your project needs
- Cost-effective with minimal overhead infrastructure and risk reduction with potential to fully optimize pile design

Thank You!

Don’t forget to meet and talk with our senior principal engineers and experts on O-cell testing and pile foundation design at our **Fugro Booth 208**

- ❑Joyshwin Sumputh (Senior Principal Engineer – Pile Testing Lead)
- ❑Paul Bullock (Chief Engineer/Senior Principal Engineer – Pile Testing Expert)
- ❑Proserpine Peralta (Senior Principal Engineer – Foundation Design Lead)

FUGRO

Unlocking **Insights**
from **Geo-data**

PLENARY SESSION 3

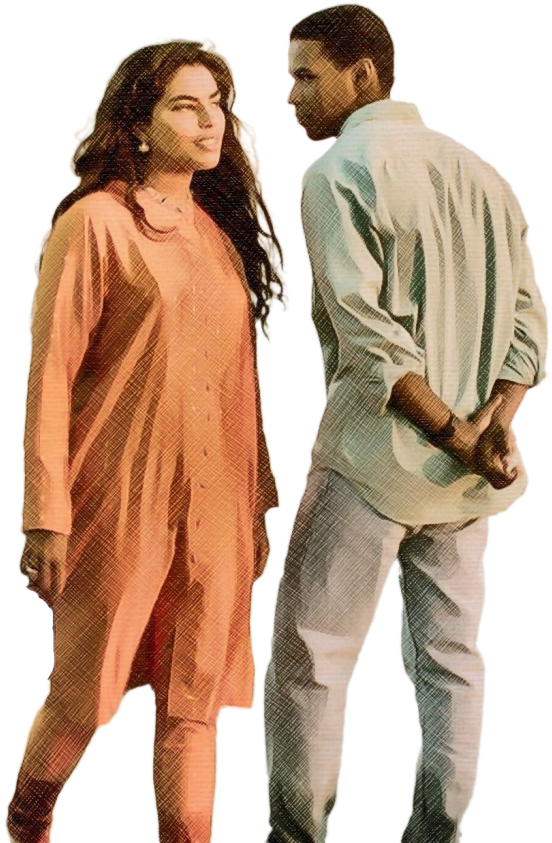
The Golden Rule

Monetary Masala

By: Anirban Basu
MPP, MA, JD, PHD
Sage Policy Group, Inc.

On Behalf of
Deep Foundations Institute

October 22, 2025



The Taking of Inflation 2.0



The Taking of Pelham 123 (2009)—Denzel Washington as Train Dispatcher

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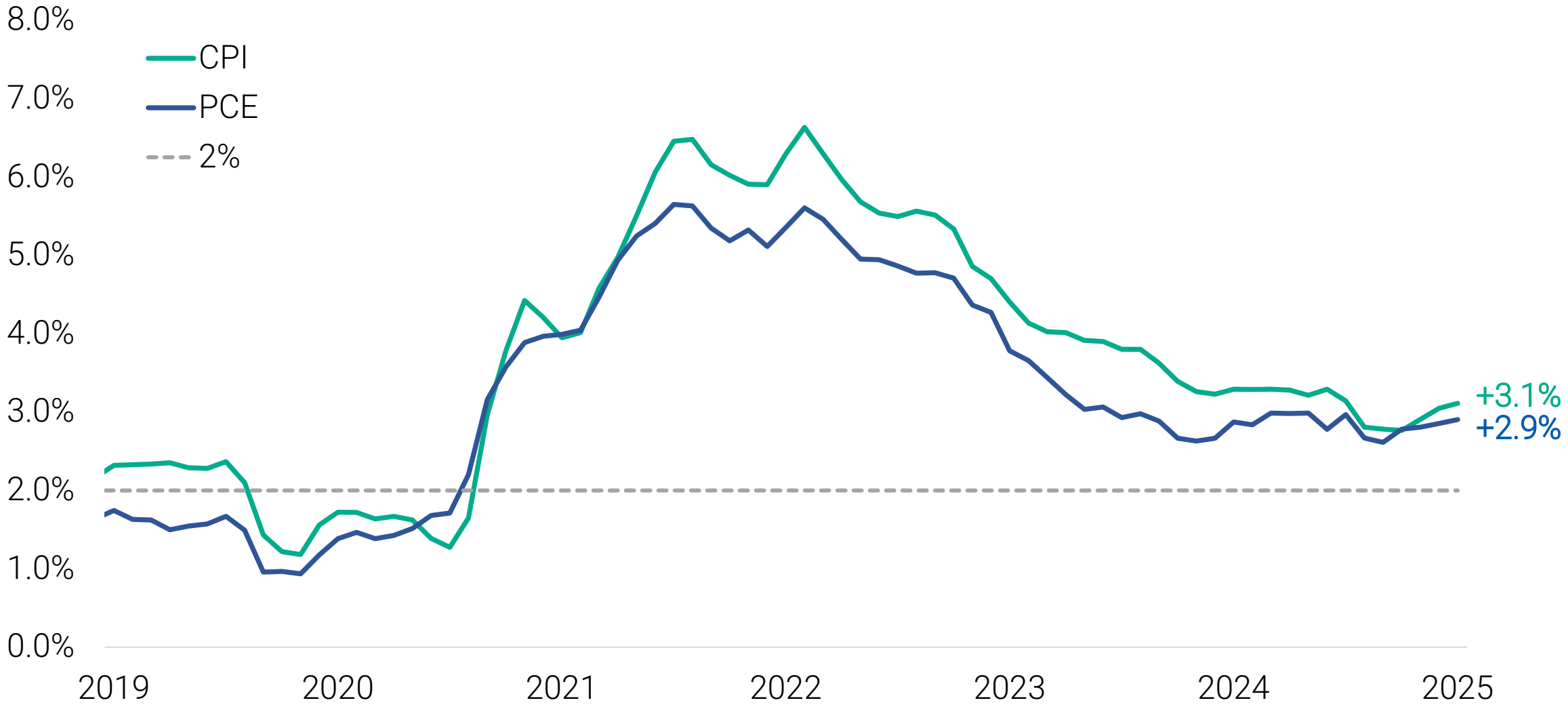
Core CPI & PCE, 12-Month % Change (SA)

2019 – August 2025



Source: U.S. Bureau of Labor Statistics; U.S. Bureau of Economic Analysis *Core: All items less food and energy

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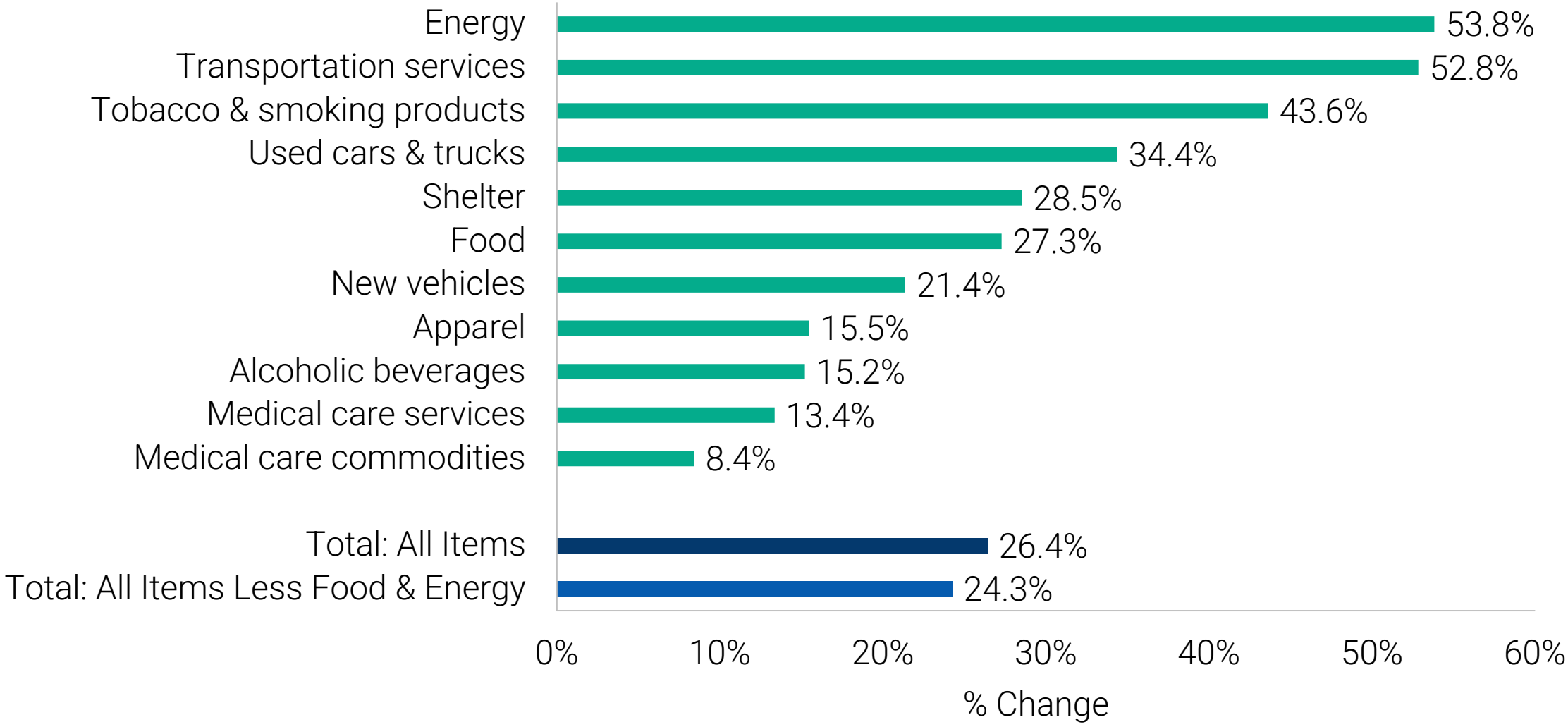
Consumer Price Index, Select Categories (SA)

% Change May 2020 v. August 2025



Source: U.S. Bureau of Labor Statistics

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The Manhattan Candidate

- 10% minimum tariffs on all countries; Canada (10-25%); China (10-125%); EU (15%); Mexico (25-30%); some of these delayed during negotiations.
- Tariffs on targeted goods: steel and aluminum (50%); copper (50%); automobiles (25%).
- Big Beautiful Bill: cuts taxes; raises debt ceiling; adds estimated $3.4 trillion to national debt over 10 years.
- Mass deportations: target of 3,000 ICE arrests per day, according to some sources.

*The Manchurian Candidate (2004)—Denzel Washington as Major Bennett Marco

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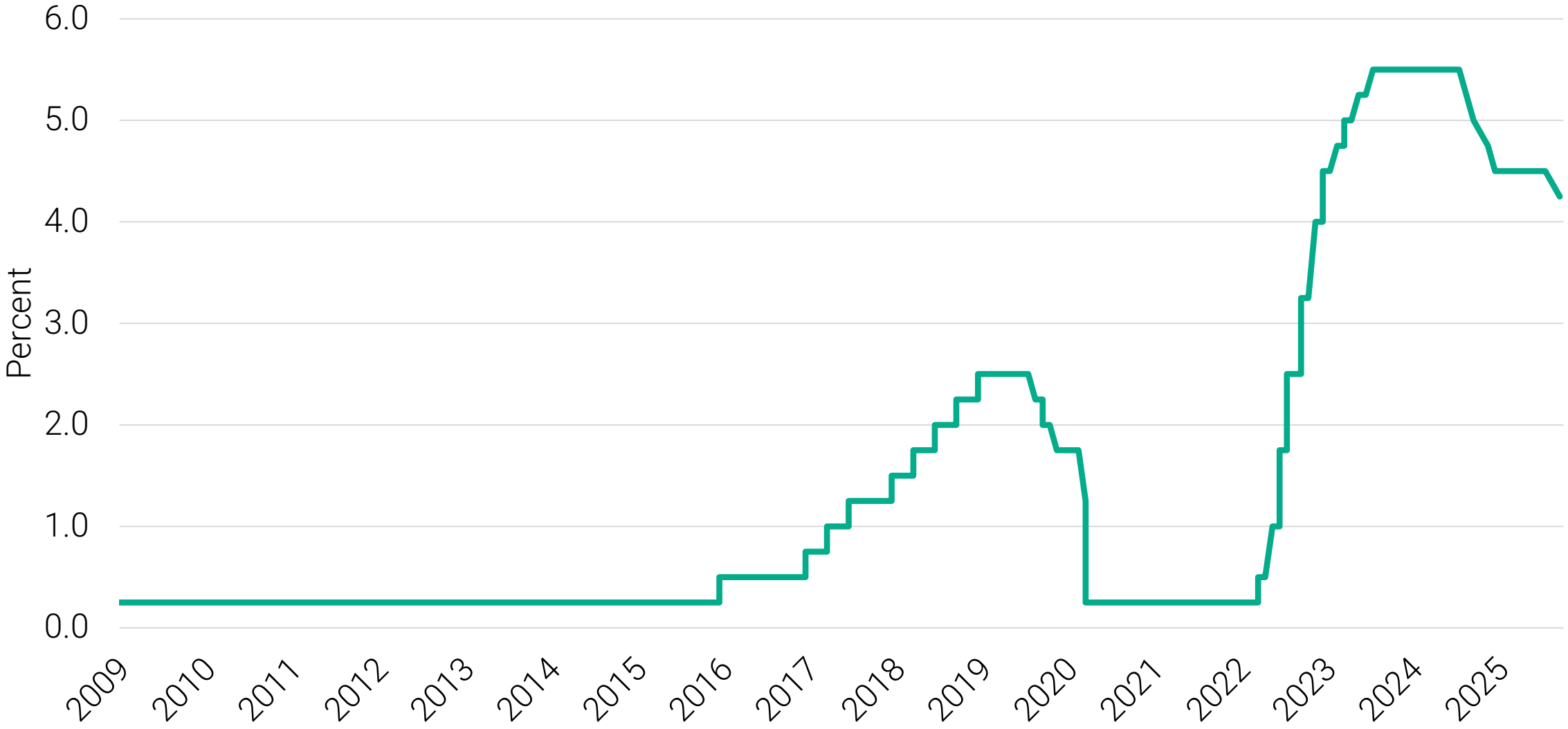
Federal Funds Rate, Target Rate Upper Limit, 2009 – 2025



Source: Board of Governors of the Federal Reserve System

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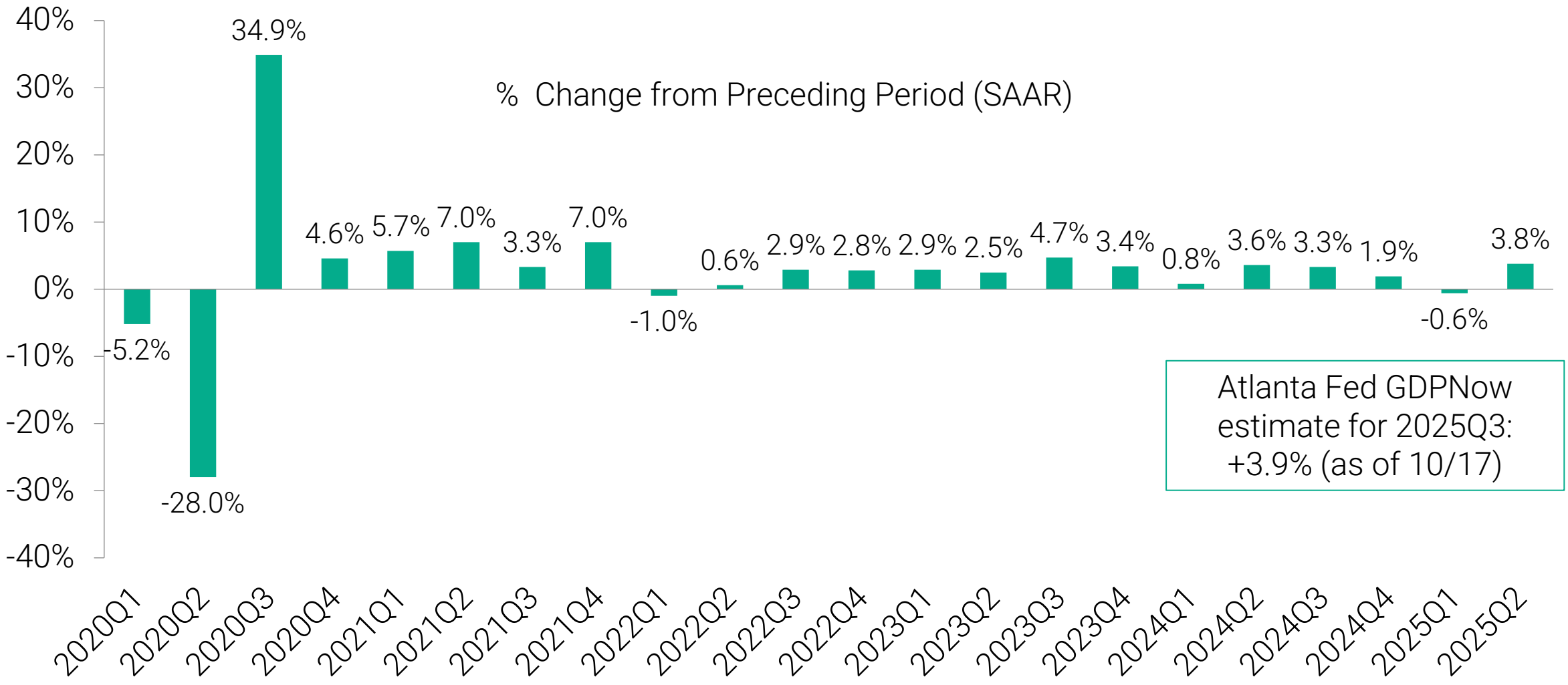
U.S. Gross Domestic Product Growth, 2020 – 2025Q2



Atlanta Fed GDPNow estimate for 2025Q3: +3.9% (as of 10/17)

Source: U.S. Bureau of Economic Analysis *2025Q2: 3rd (final) estimate

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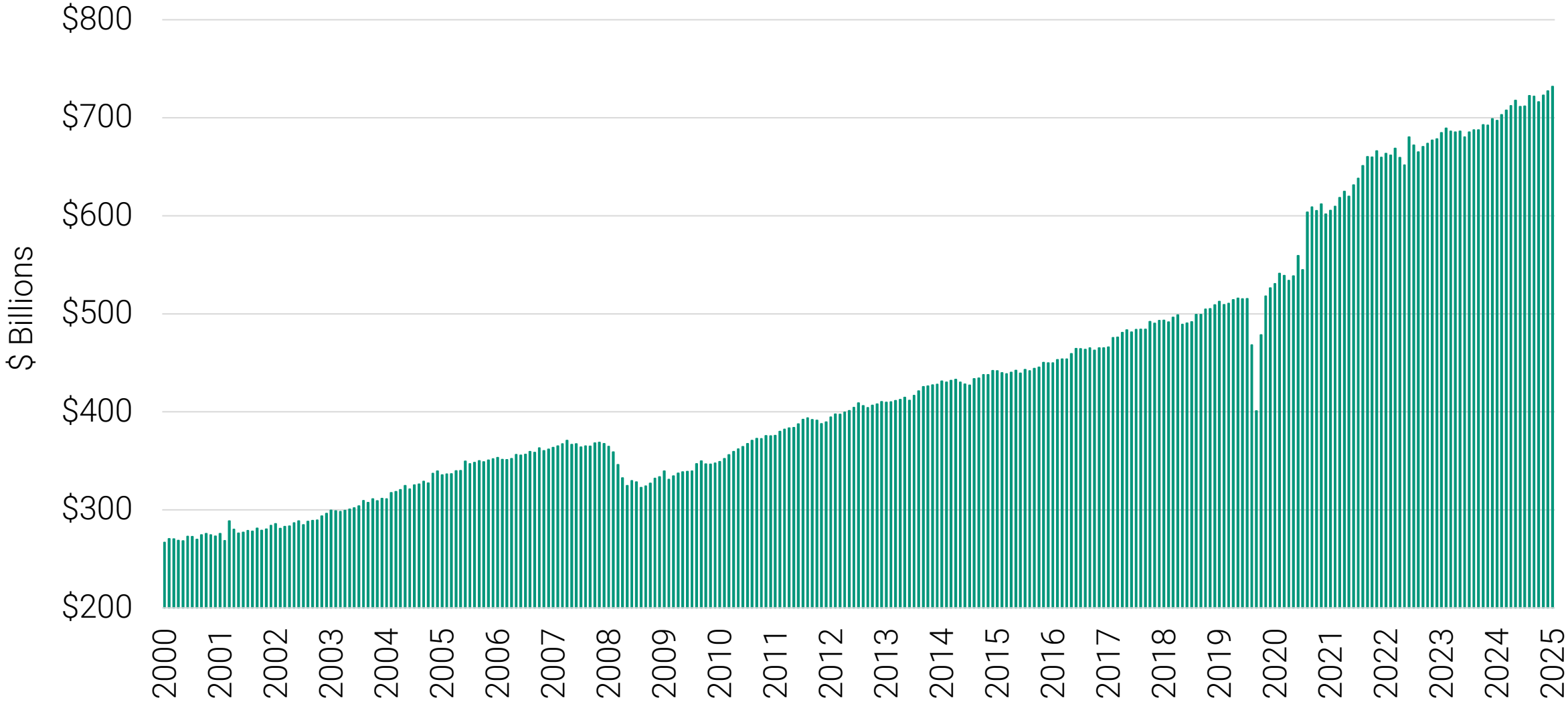
They Got Game

U.S. Retail Sales, 2000 – August 2025



Source: U.S. Census Bureau

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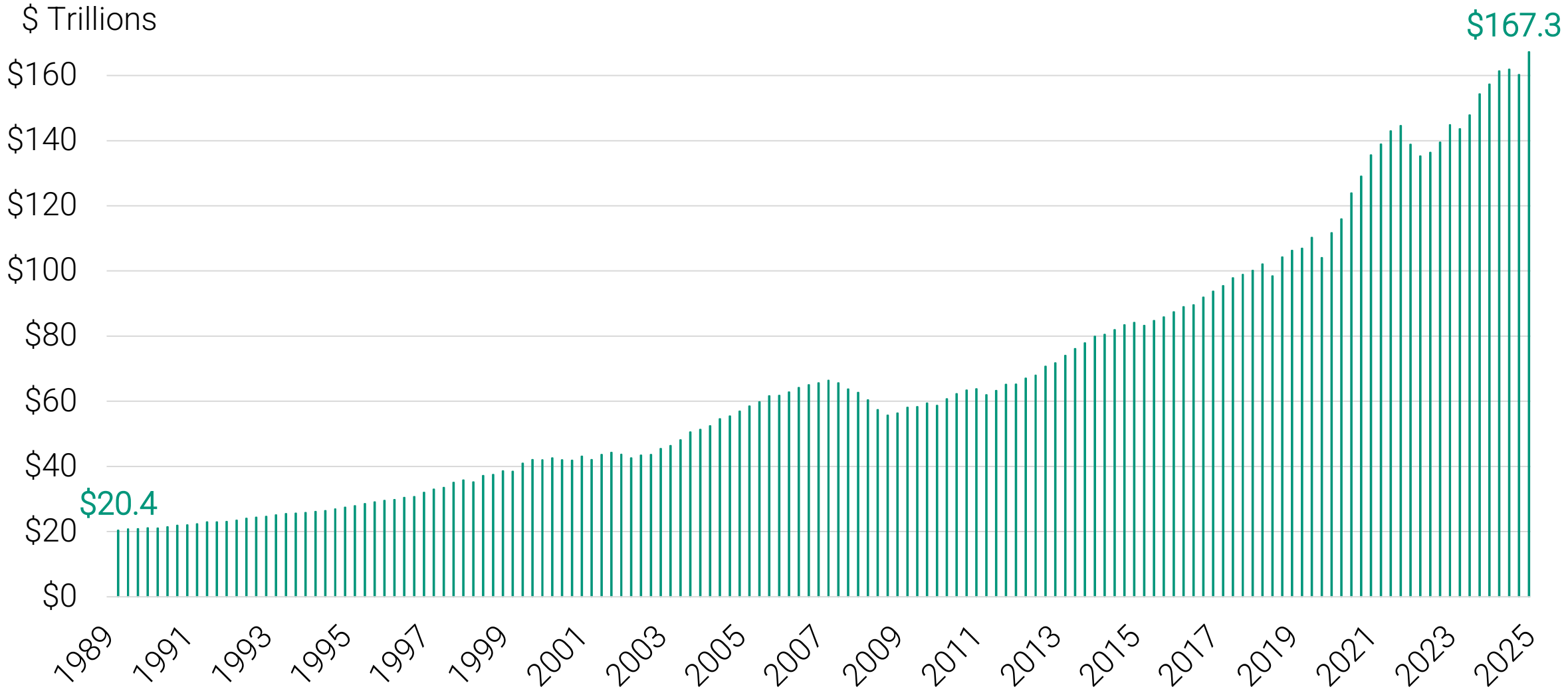
Total U.S. Household Wealth, 1989 – 2025



Source: Federal Reserve Board, Distributional Financial Accounts (DFAs)

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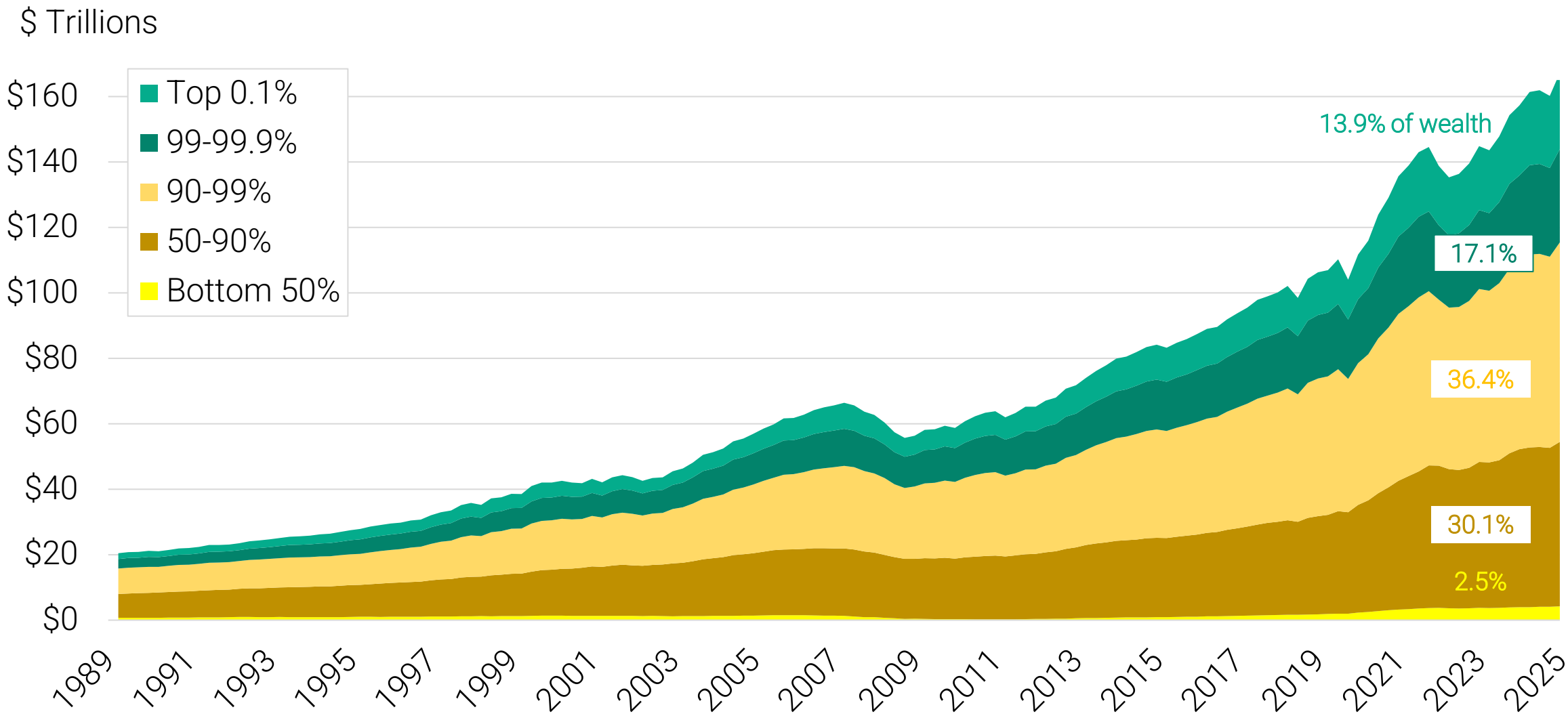
U.S. Household Wealth by Wealth Percentile Group, 1989 – 2025



Source: Federal Reserve Board, Distributional Financial Accounts (DFAs)

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The Equalizer



The Equalizer (2014, 2018, and 2023)—Denzel Washington as mystery man Robert McCall

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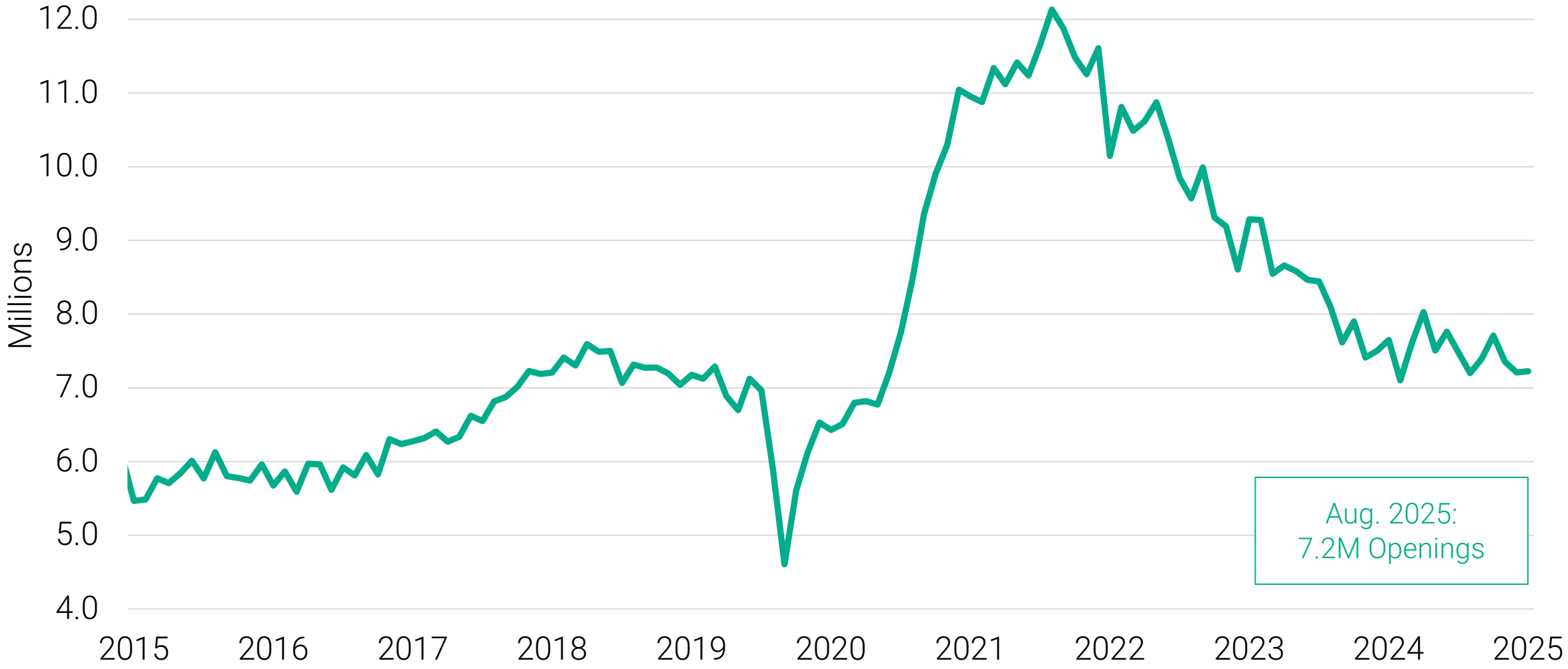
U.S. Job Openings, 2015 – August 2025



Source: U.S. Bureau of Labor Statistics

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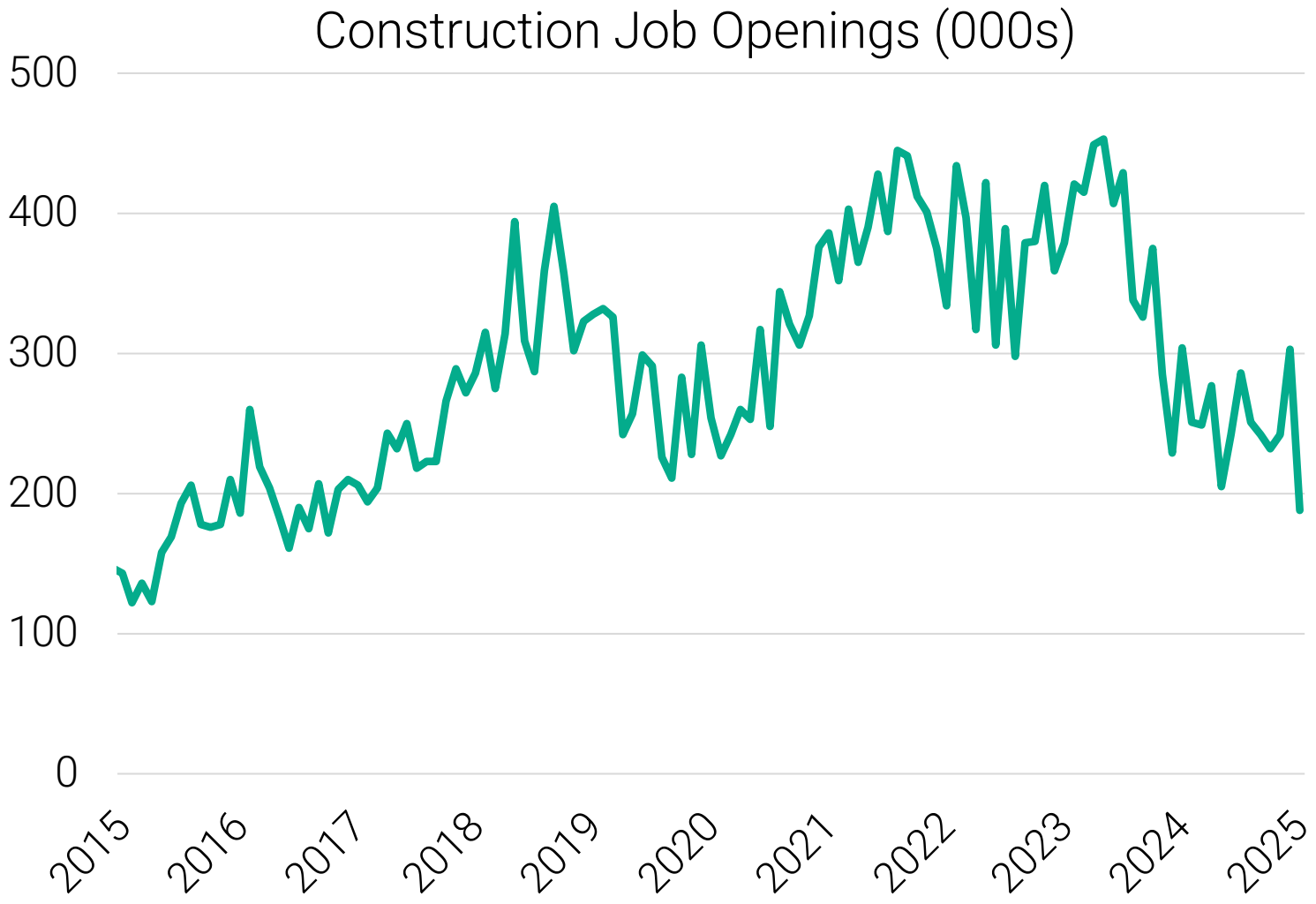
U.S. Construction Job Openings, 2015 – August 2025

- From 2015-2019 the number of unfilled U.S. construction jobs averaged 228,000.
- In 2024 there was an average of 306,000 unfilled construction jobs.
- That's 34% higher than the 2015-2019 average.



Source: U.S. Bureau of Labor Statistics

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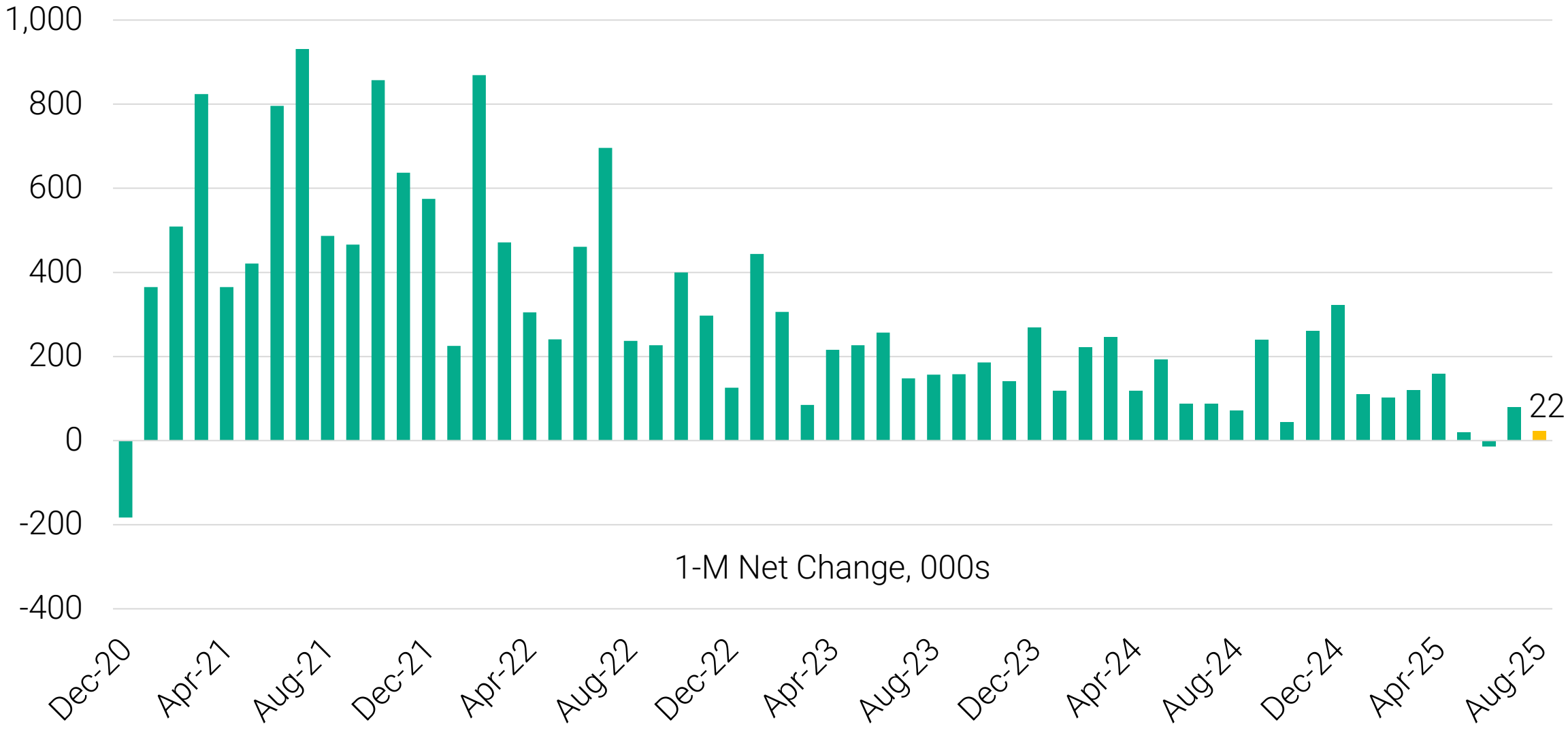
U.S. Job Growth, Monthly

December 2020 – August 2025



Source: U.S. Bureau of Labor Statistics

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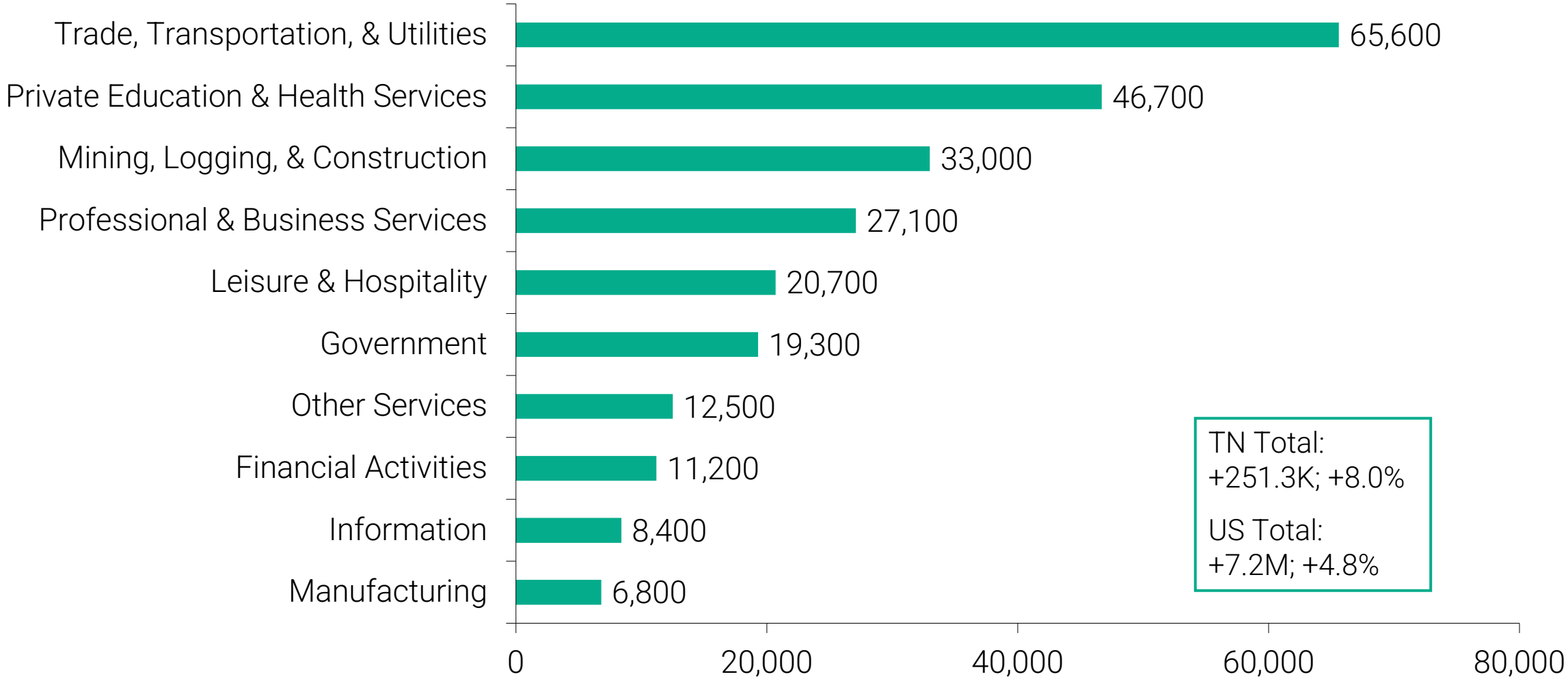
Tennessee Nonfarm Employment

February 2020 v. August 2025 Absolute Change



*According to the Local Area Unemployment Statistics (LAUS) program TN added 148,889 jobs between February 2020 and August 2025.

Source: U.S. Bureau of Labor Statistics
*Data are seasonally adjusted (SA)

Employment Growth, 25 Largest Metros

February 2020 v. August 2025 % Change

Rank	MSA	%
1	Austin-Round Rock-San Marcos, TX	19.5%
2	Dallas-Fort Worth-Arlington, TX	12.6%
3	Orlando-Kissimmee-Sanford, FL	11.8%
4	San Antonio-New Braunfels, TX	11.6%
5	Charlotte-Concord-Gastonia, NC-SC	11.3%
6	Tampa-St. Petersburg-Clearwater, FL	10.3%
7	Phoenix-Mesa-Chandler, AZ	9.5%
8	Atlanta-Sandy Springs-Roswell, GA	8.2%
9	Houston-Pasadena-The Woodlands, TX	8.1%
10	Miami-Fort Lauderdale-West Palm Beach, FL	7.5%
11	Riverside-San Bernardino-Ontario, CA	7.1%
12	Denver-Aurora-Centennial, CO	7.0%
13	Philadelphia-Camden-Wilm., PA-NJ-DE-MD	5.5%

Rank	MSA	%
14	New York-Newark-Jersey City, NY-NJ	4.1%
15	Chicago-Naperville-Elgin, IL-IN-WI	3.1%
16	Baltimore-Columbia-Towson, MD	3.0%
16	San Diego-Chula Vista-Carlsbad, CA	3.0%
18	Seattle-Tacoma-Bellevue, WA	2.6%
19	Detroit-Warren-Dearborn, MI	2.0%
20	Minneapolis-St. Paul-Bloomington, MN-WI	1.9%
20	Washington-Arlington-Alexandria, DC-VA-MD-WV	1.9%
22	St. Louis, MO-IL	1.1%
23	Boston-Cambridge-Newton, MA-NH	0.8%
24	Los Angeles-Long Beach-Anaheim, CA	-0.5%
25	San Francisco-Oakland-Fremont, CA	-3.5%

U.S. % Change 2/2020 v. 8/2025: +4.8%

Current Employment Statistics (CES) Survey. Note: data are not seasonally adjusted.

Construction Employment Growth, 25 Largest Metros

February 2020 v. August 2025 % Change

Rank	MSA	%
1	Minneapolis-St. Paul-Bloomington, MN-WI	34.1%
2	Phoenix-Mesa-Chandler, AZ	33.6%
3	Detroit-Warren-Dearborn, MI	32.4%
4	Austin-Round Rock-San Marcos, TX	27.2%
5	St. Louis, MO-IL	26.8%
6	San Antonio-New Braunfels, TX	24.8%
7	Chicago-Naperville-Elgin, IL-IN-WI	22.5%
8	Charlotte-Concord-Gastonia, NC-SC	21.0%
9	Dallas-Fort Worth-Arlington, TX	17.7%
10	Miami-Fort Lauderdale-West Palm Beach, FL	15.4%
11	Tampa-St. Petersburg-Clearwater, FL	15.0%
12	Washington-Arlington-Alexandria, DC-VA-MD-WV	13.4%

Rank	MSA	%
13	Orlando-Kissimmee-Sanford, FL	13.2%
14	Boston-Cambridge-Newton, MA-NH	12.4%
15	Atlanta-Sandy Springs-Roswell, GA	12.3%
16	Philadelphia-Camden-Wilm., PA-NJ-DE-MD	8.0%
17	San Diego-Chula Vista-Carlsbad, CA	7.6%
18	Denver-Aurora-Centennial, CO	4.3%
19	Baltimore-Columbia-Towson, MD	4.1%
20	Riverside-San Bernardino-Ontario, CA	1.7%
21	Houston-Pasadena-The Woodlands, TX	-0.1%
22	New York-Newark-Jersey City, NY-NJ	-2.0%
23	Seattle-Tacoma-Bellevue, WA	-3.5%
24	Los Angeles-Long Beach-Anaheim, CA	-4.2%
25	San Francisco-Oakland-Fremont, CA	-5.7%

U.S. % Change 2/2020 v. 8/2025: +9.0%

Current Employment Statistics (CES) Survey. Note: data are not seasonally adjusted.

Unemployment Rates, 25 Largest Metros, August 2025

Rank	MSA	%
1	Atlanta-Sandy Springs-Roswell, GA	3.5%
2	Denver-Aurora-Centennial, CO	3.7%
2	Miami-Fort Lauderdale-West Palm Beach, FL	3.7%
4	Austin-Round Rock-San Marcos, TX	3.9%
5	Charlotte-Concord-Gastonia, NC-SC	4.1%
5	Minneapolis-St. Paul-Bloomington, MN-WI	4.1%
7	Phoenix-Mesa-Chandler, AZ	4.2%
8	Baltimore-Columbia-Towson, MD	4.3%
8	Detroit-Warren-Dearborn, MI	4.3%
8	Orlando-Kissimmee-Sanford, FL	4.3%
8	Washington-Arlington-Alexandria, DC-VA-MD-WV	4.3%
12	Dallas-Fort Worth-Arlington, TX	4.4%
12	San Antonio-New Braunfels, TX	4.4%
12	Tampa-St. Petersburg-Clearwater, FL	4.4%
15	Boston-Cambridge-Newton, MA-NH	4.5%
15	St. Louis, MO-IL	4.5%
17	Chicago-Naperville-Elgin, IL-IN-WI	4.6%
17	Seattle-Tacoma-Bellevue, WA	4.6%
19	San Francisco-Oakland-Fremont, CA	4.8%
20	Houston-Pasadena-The Woodlands, TX	5.0%
20	San Diego-Chula Vista-Carlsbad, CA	5.0%
22	Philadelphia-Camden-Wilm., PA-NJ-DE-MD	5.1%
23	New York-Newark-Jersey City, NY-NJ	5.2%
24	Los Angeles-Long Beach-Anaheim, CA	5.9%
25	Riverside-San Bernardino-Ontario, CA	6.1%

U.S. Unemployment Rate: 4.3%

Local Area Unemployment Statistics (LAUS) program. Note: data are not seasonally adjusted

Devil in a Bunch of Data



Devil in a Blue Dress (1995)—Denzel Washington as Easy Rawlins

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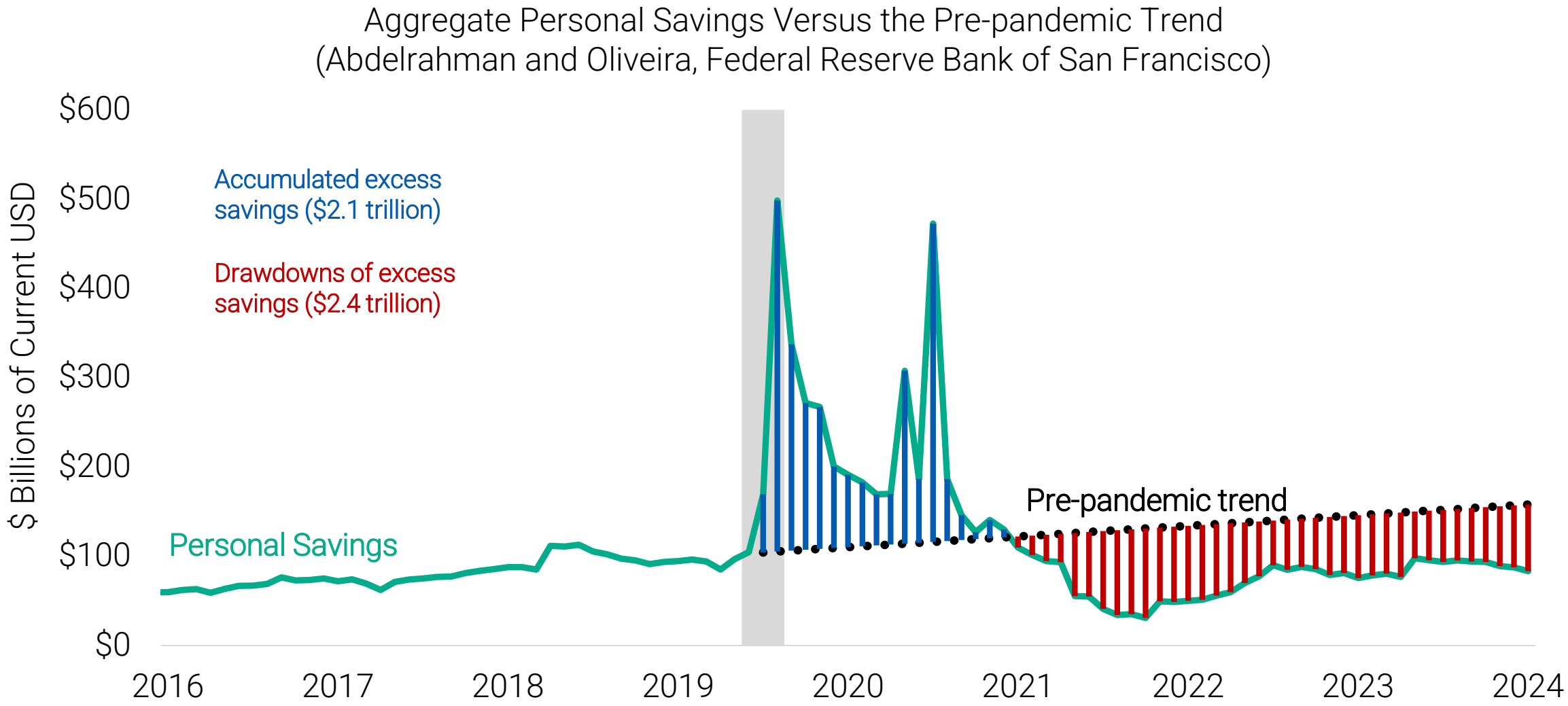
Drawdown of Pandemic Related Excess Savings



Source: Federal Reserve Bank of San Francisco

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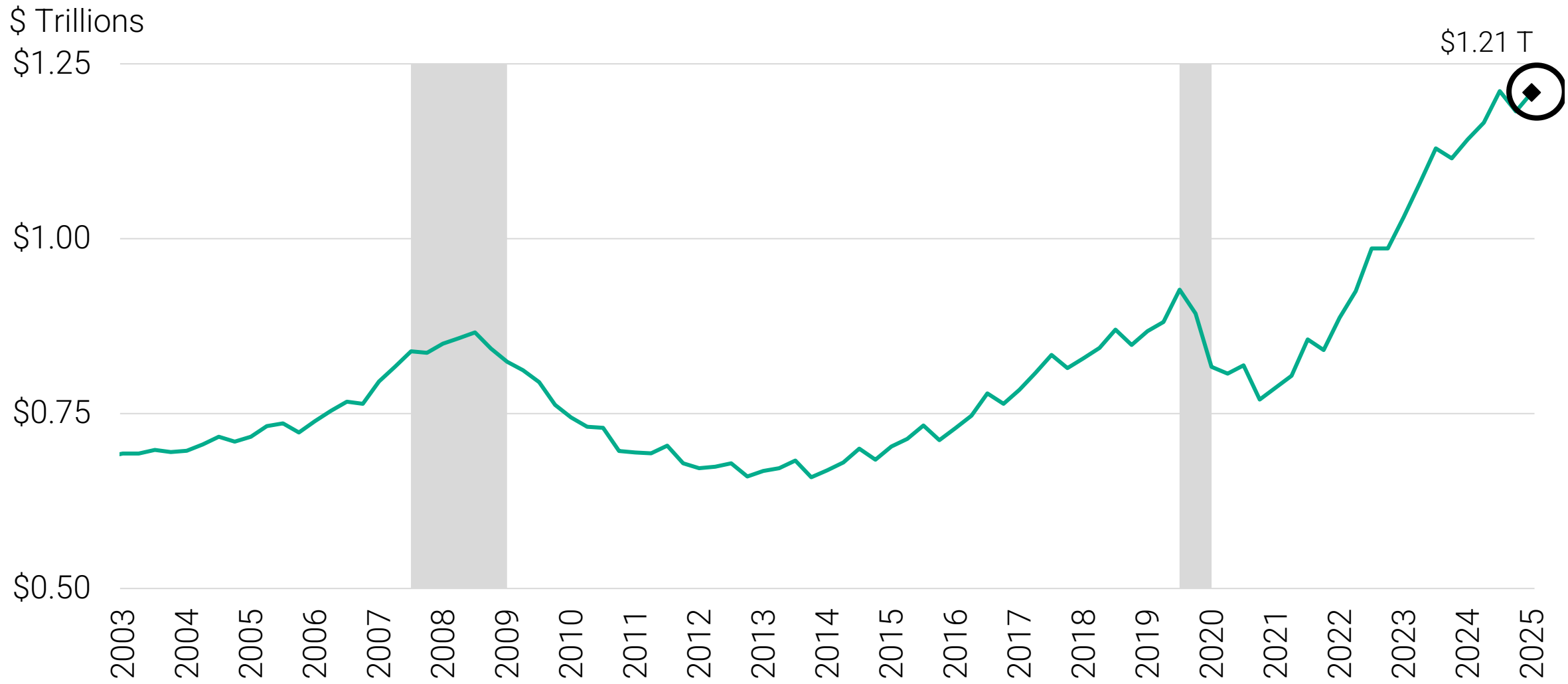
U.S. Credit Card Debt, 2003 – 2025



Shaded areas indicate U.S. recessions

Source: FRED; New York Fed Consumer Credit Panel/Equifax

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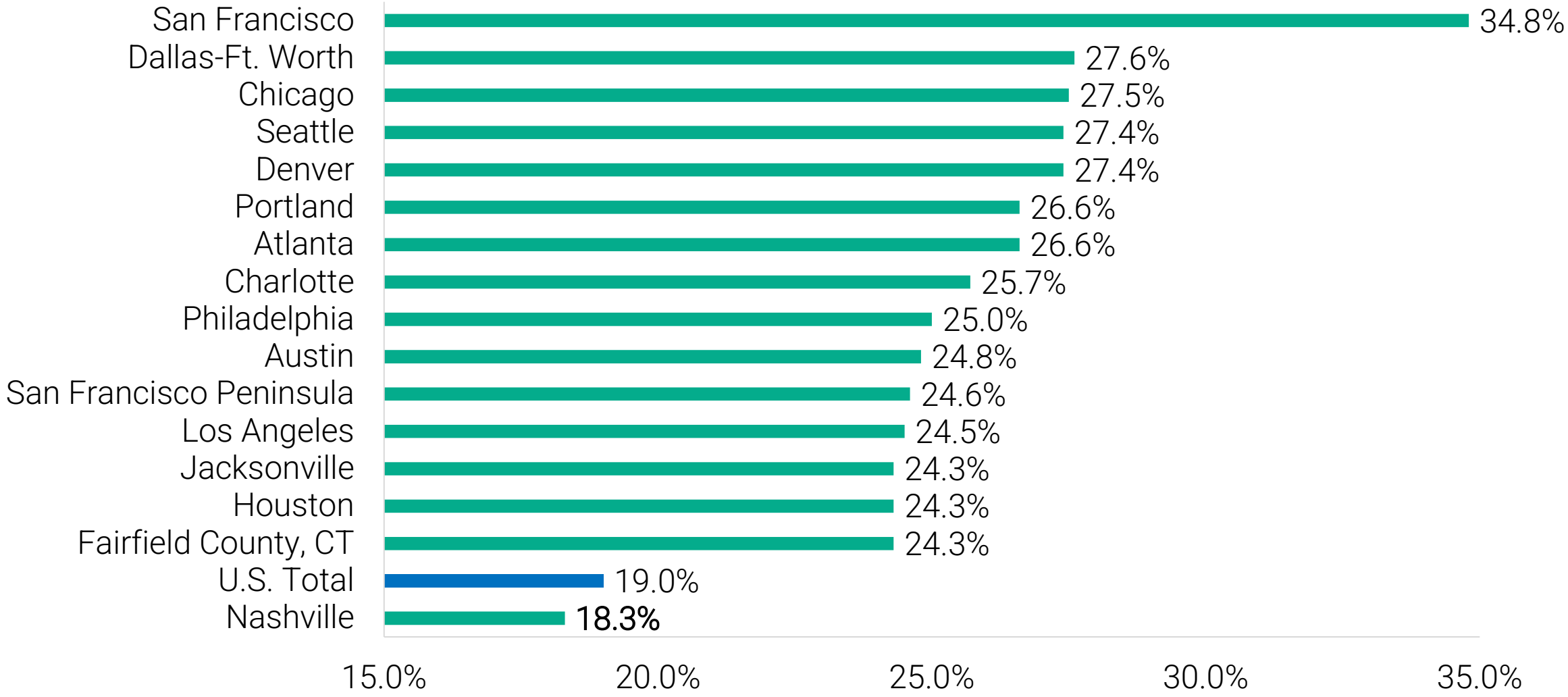
Highest Office Vacancy Rates, 2025Q2



Source: CBRE

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U.S. Cities Face Fiscal Stress

- With the city facing a projected $1.12 billion shortfall in 2026 Chicago Mayor Brandon Johnson said recently, “We have reached a point of no return”.
- Between December 2024 and April 2025, Chicago, Los Angeles, San Francisco, and Washington all experienced credit rating downgrades.
- According to research conducted by The Pew Charitable Trusts, at least 20 of the nation’s 25 most populous cities have reported budget gaps for fiscal year 2026—and often beyond.

Source: The Pew Charitable Trusts; CBSNews

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Fences



Fences (2016)—Denzel Washington as Troy Maxson, Pittsburgh sanitation worker

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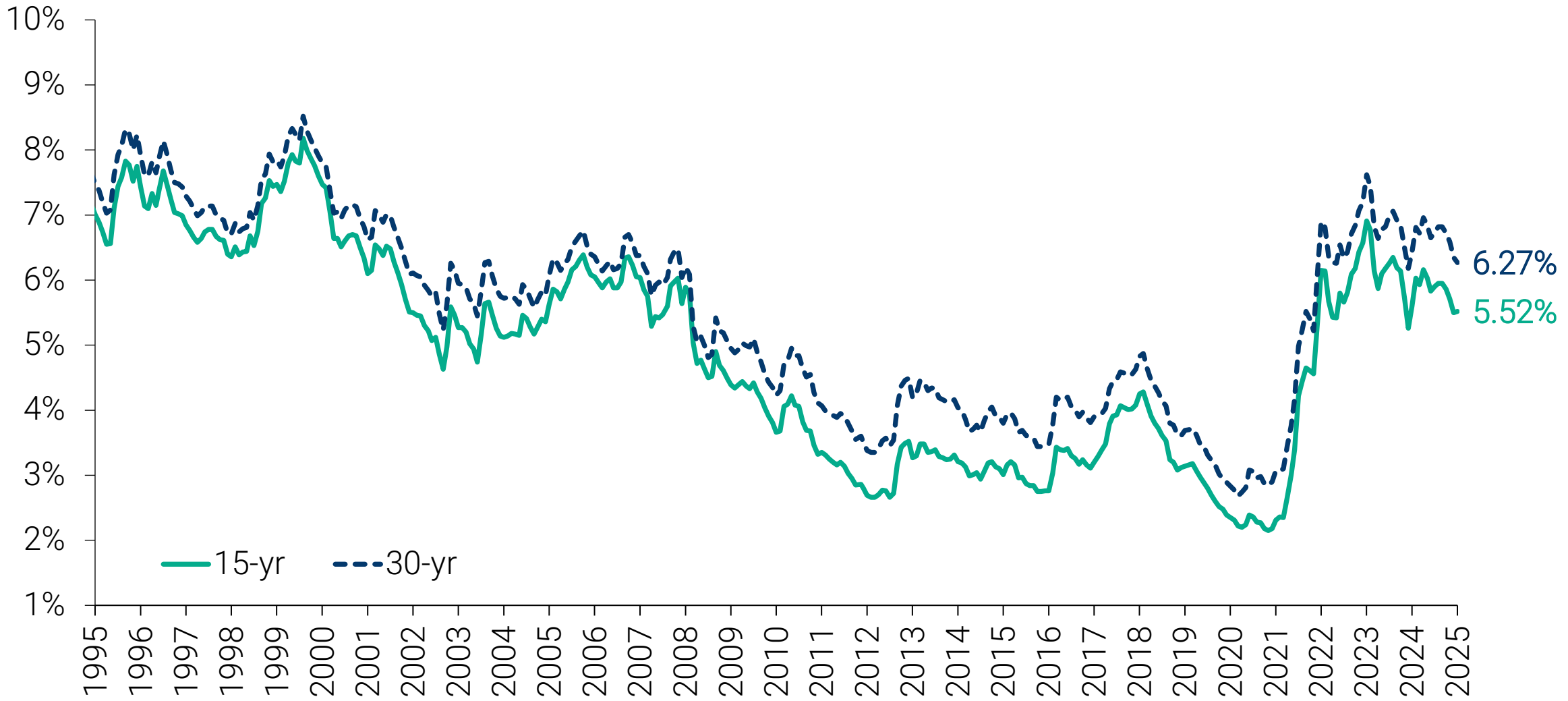
U.S. 15-Year & 30-Year Fixed Mortgage Rates

1995 – October 2025*



Source: Freddie Mac *Week ending 10/16/2025

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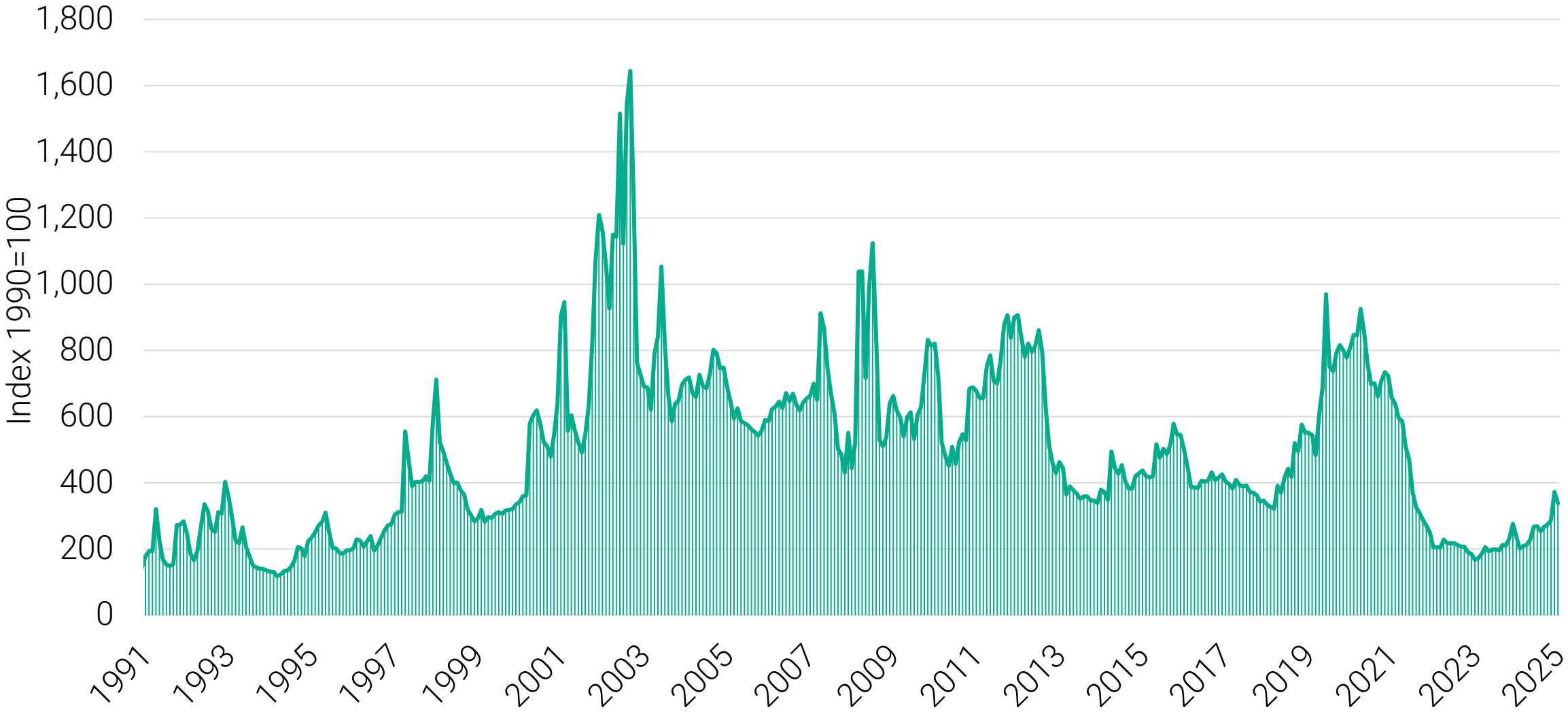
U.S. Mortgage Loan Applications Composite Index

1991 – October 2025



Source: Mortgage Bankers Association (MBA)

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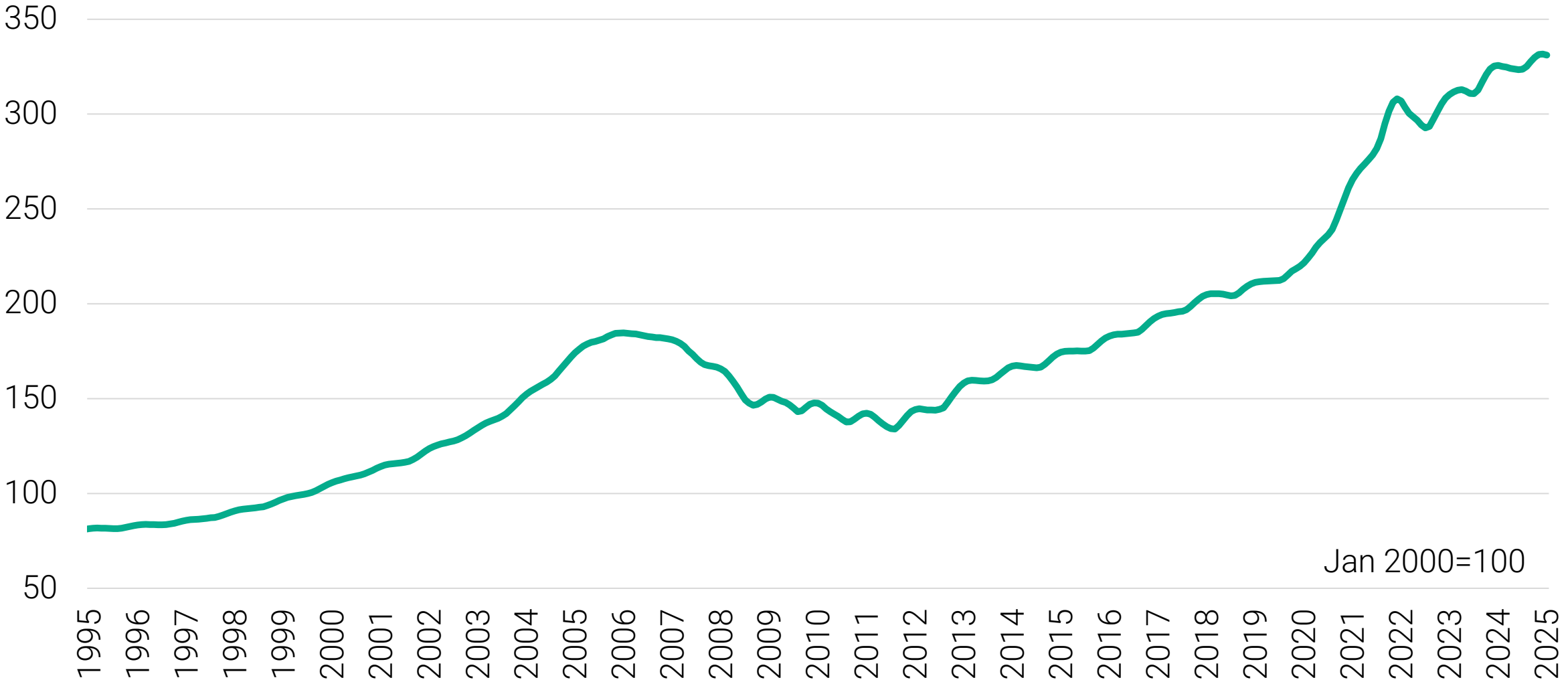
S&P Case-Shiller Home Price Index

1995 – July 2025



Source: Standard & Poor's

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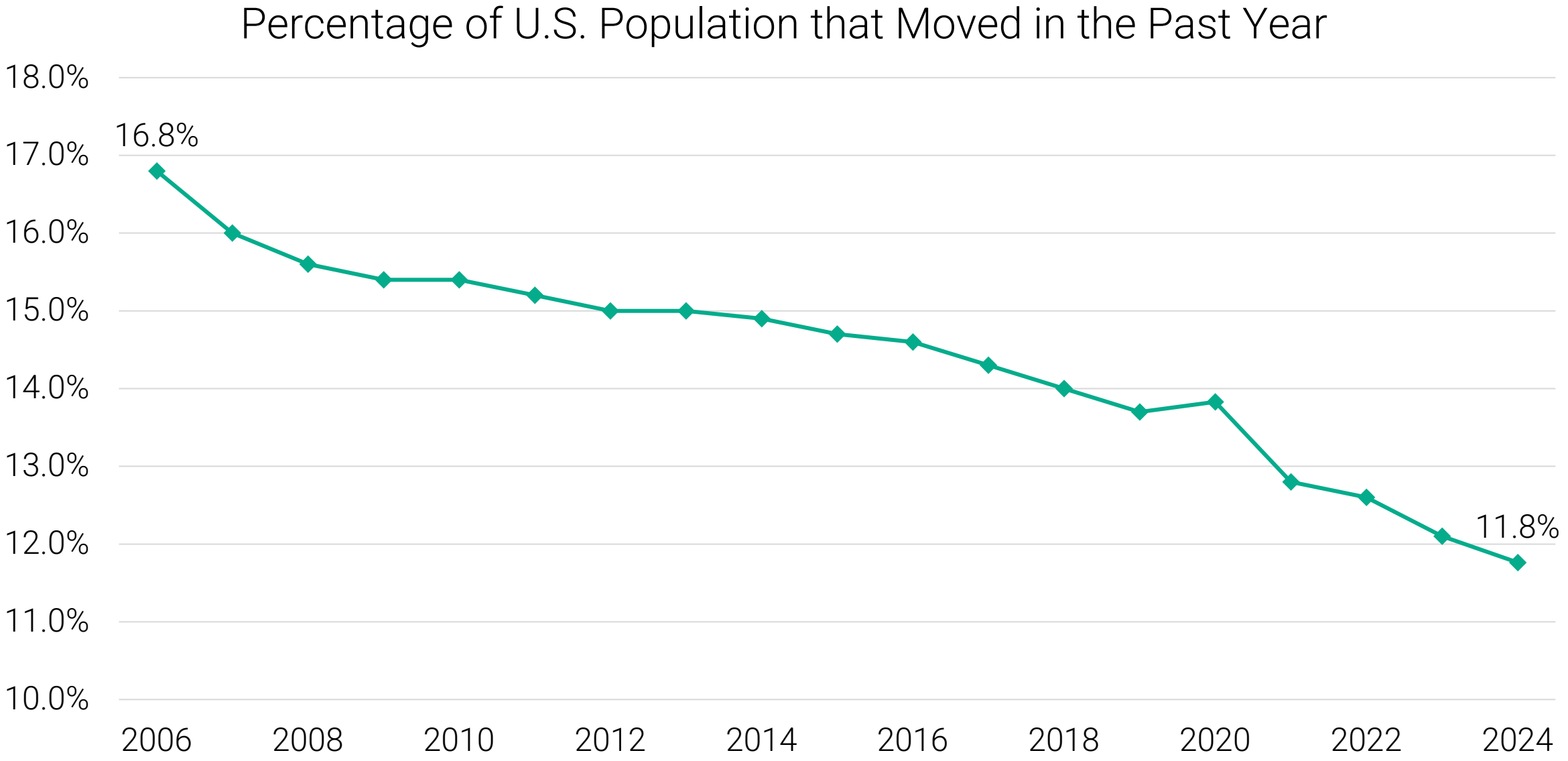
Americans are Increasingly Staying Put



Source: U.S. Census Bureau, American Community Survey

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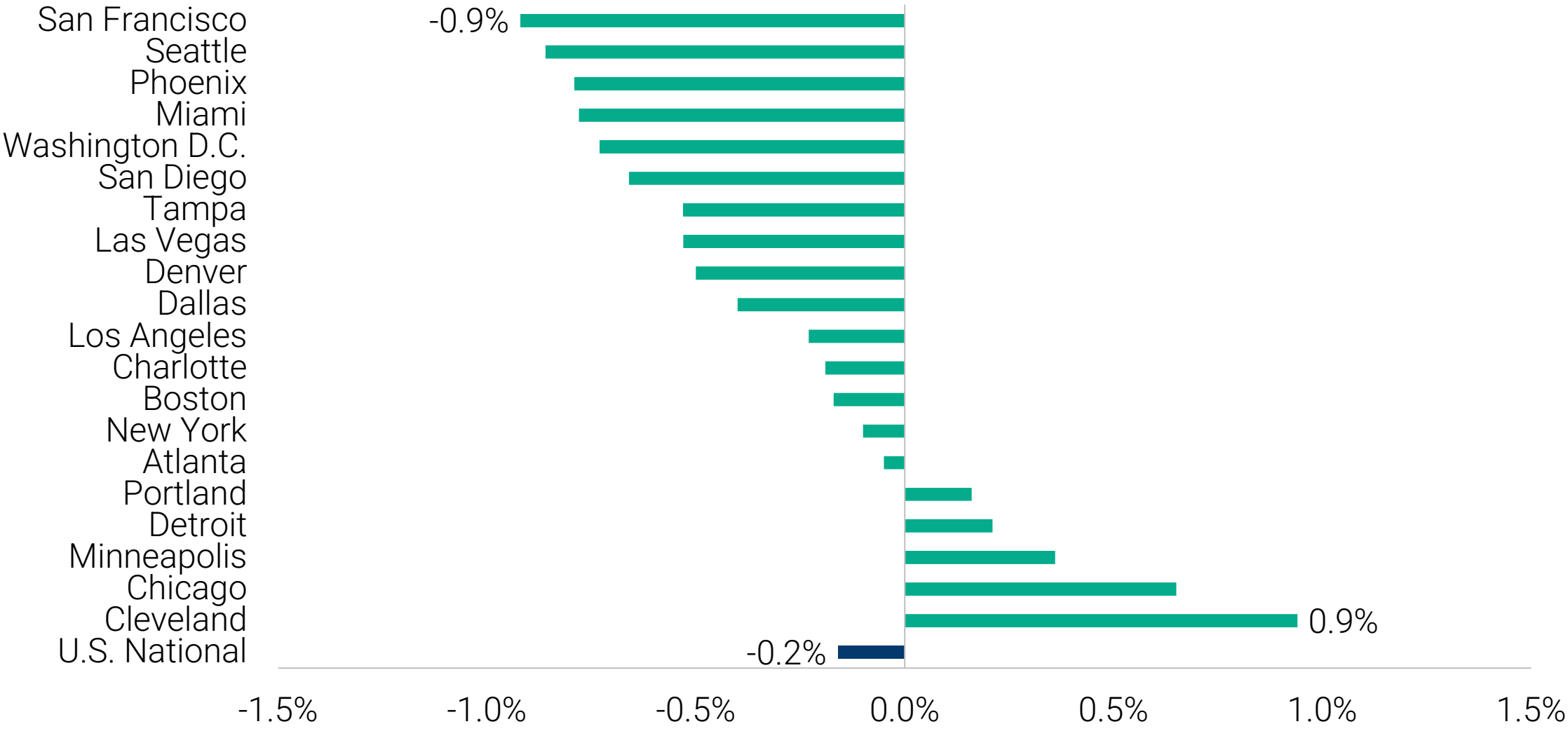
S&P Case-Shiller Home Price Index by Metro Area

1-Month % Change, July 2025



Source: Standard & Poor's

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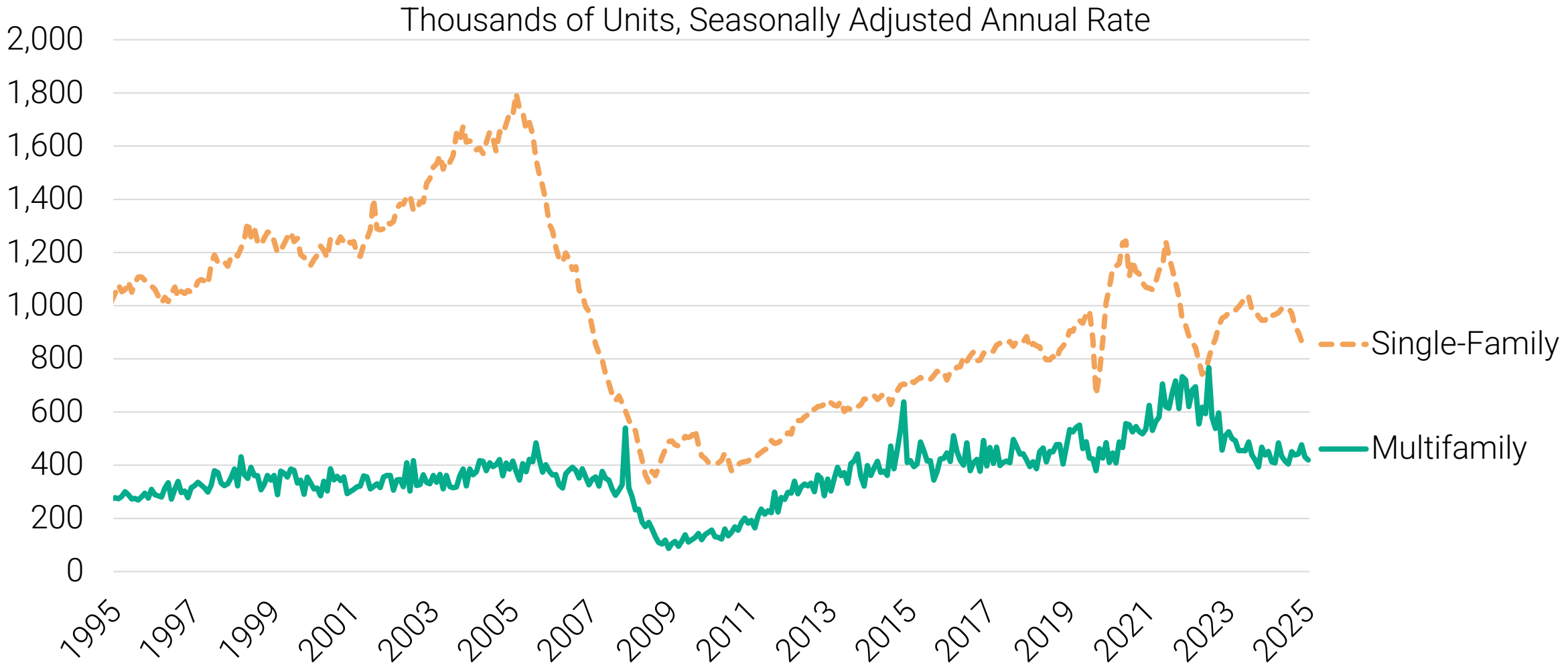
U.S. Residential Building Permits

1995 – August 2025



Source: U.S. Census Bureau

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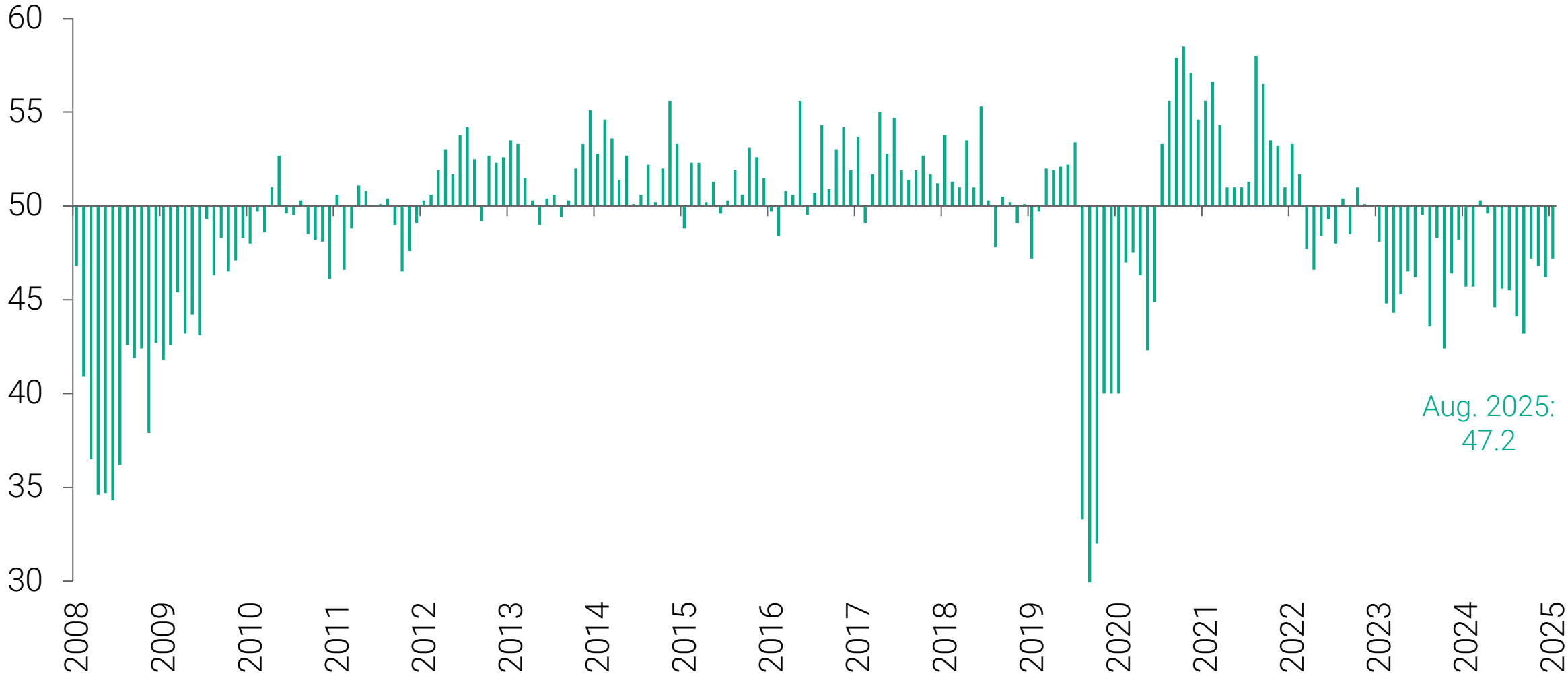
Architecture Billings Index

2008 – August 2025



Source: The American Institute of Architects

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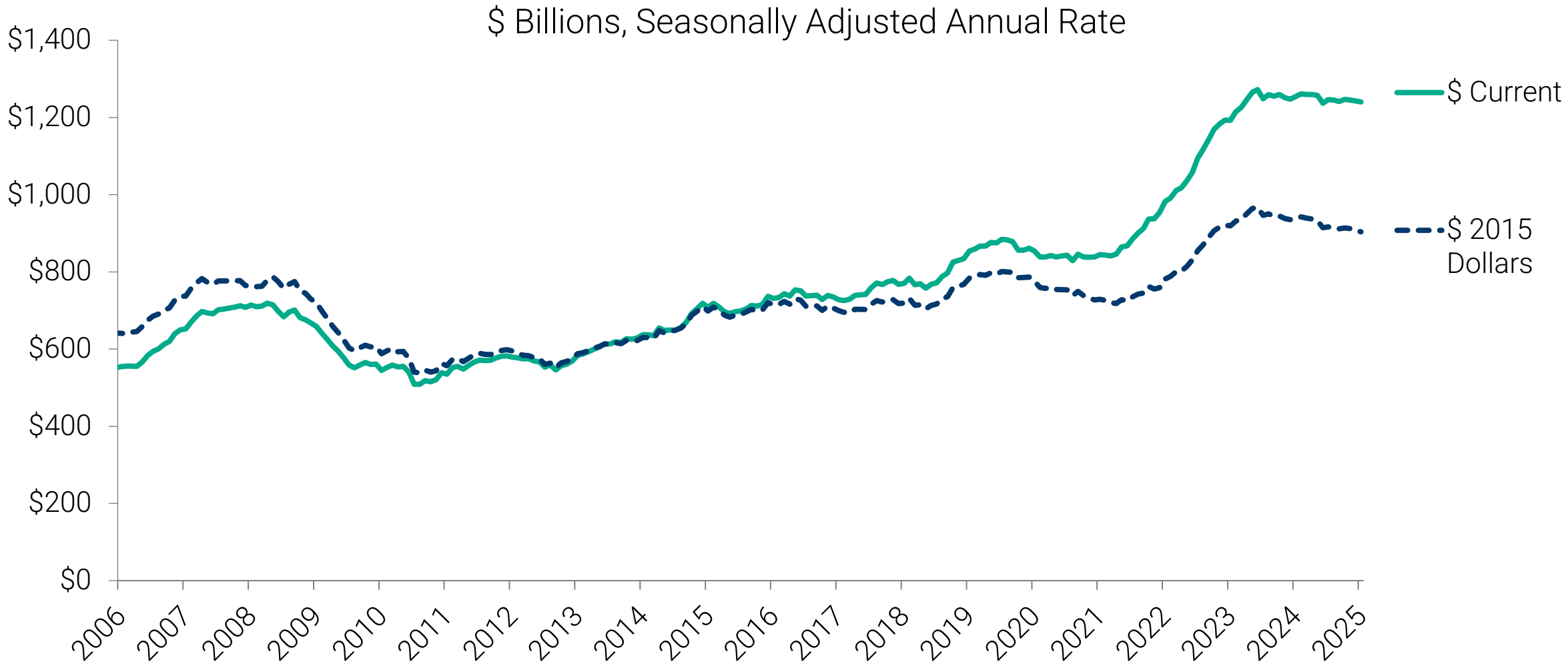
Total Nonresidential Construction Spending

Current/Constant Dollars, 2006 – July 2025



Source: U.S. Census Bureau; U.S. Bureau of Labor Statistics
*Adjusted with U.S. Bureau of Labor Statistics Consumer Price Index, All Urban Consumers, U.S. City Average

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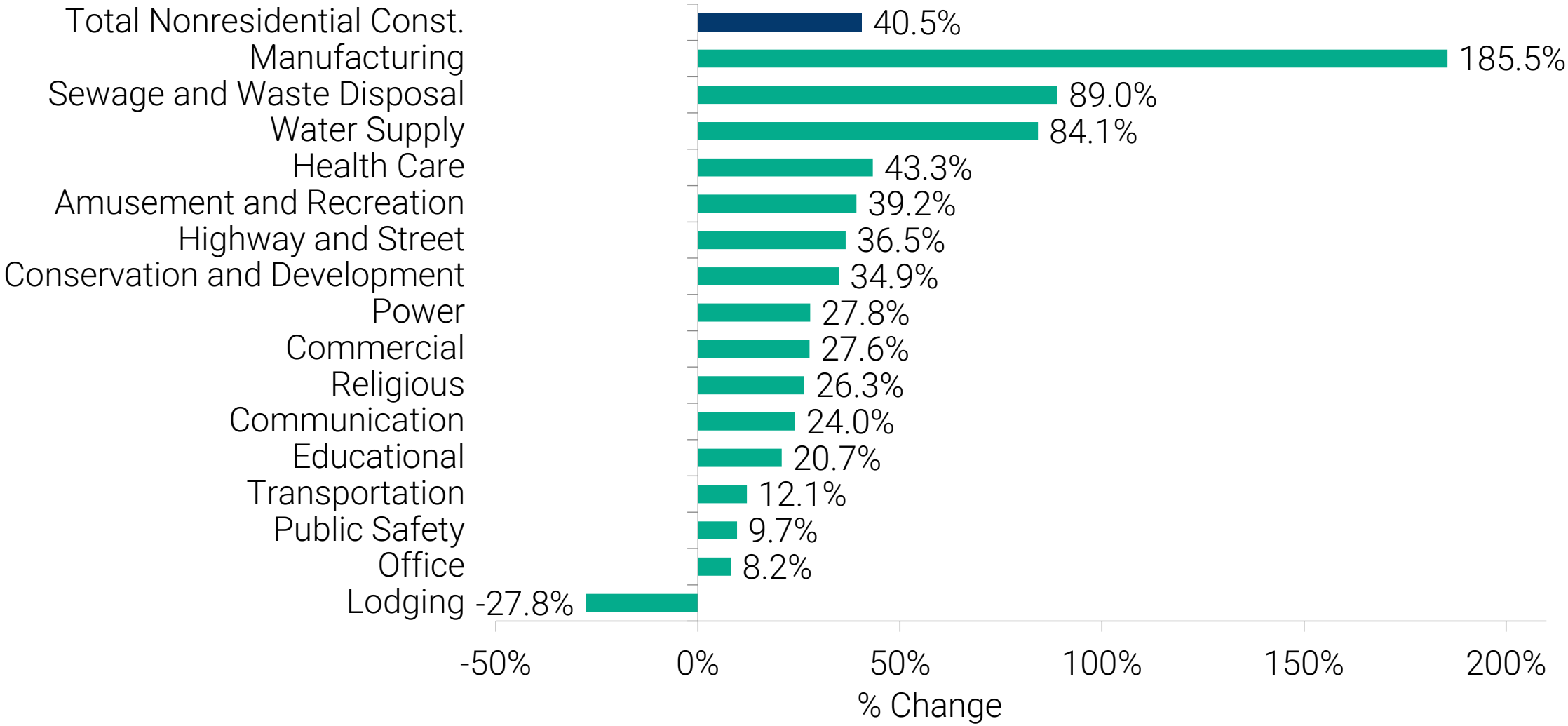
Nonresidential Construction Spending by Subsector

February 2020 v. July 2025 (Current $)



Source: U.S. Census Bureau

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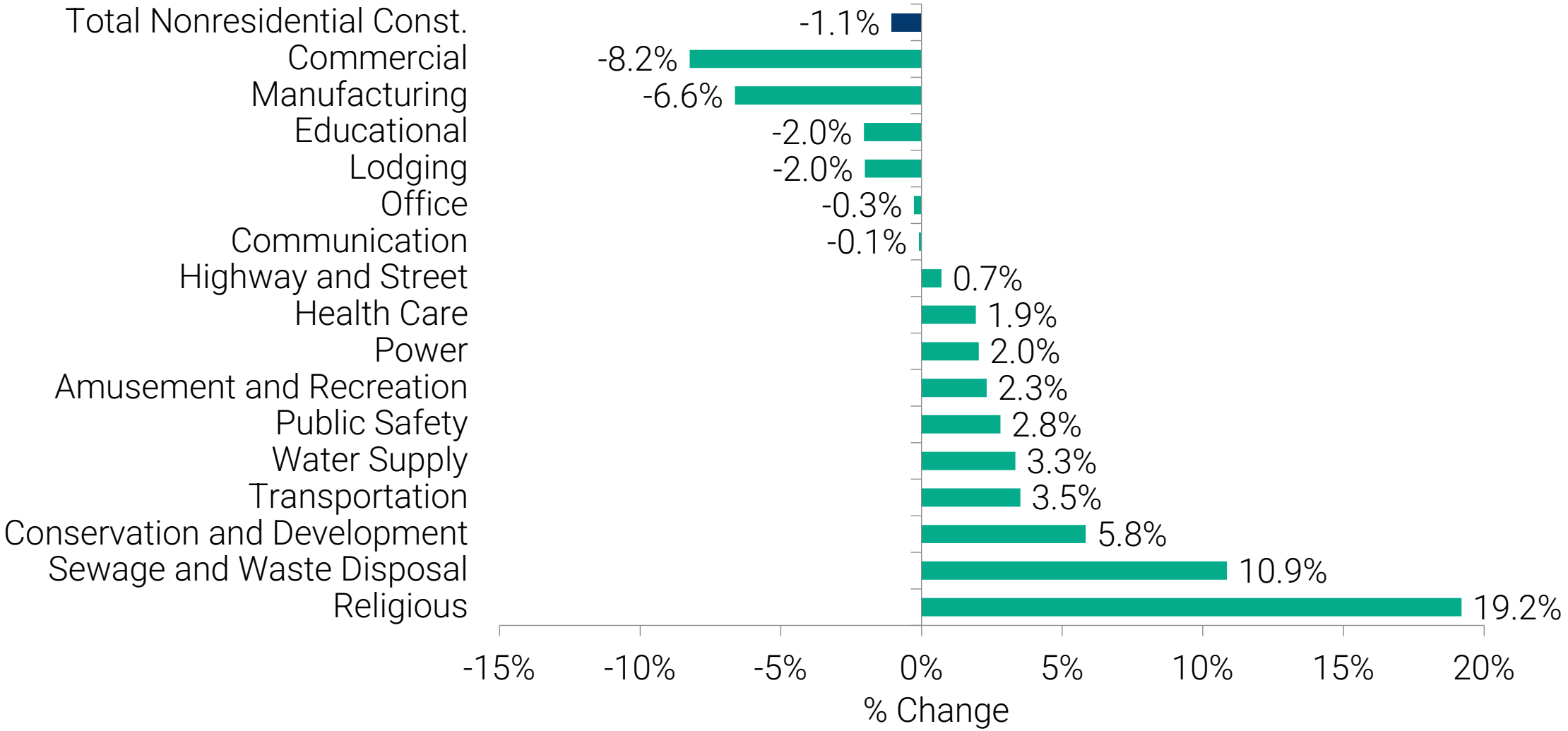
Nonresidential Construction Spending by Subsector

July 2024 v. July 2025 (Current $)



Source: U.S. Census Bureau

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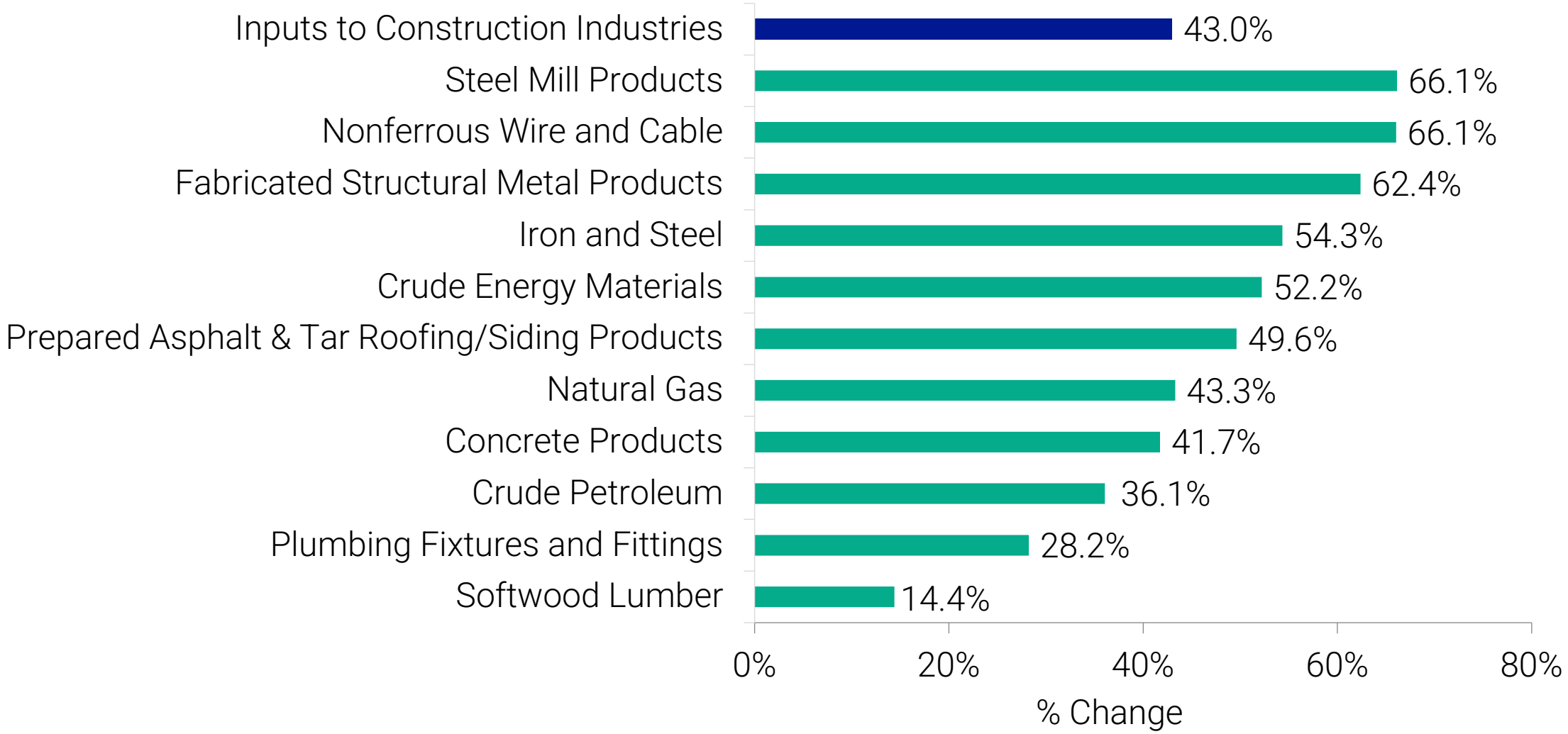
Construction Materials PPI (NSA)

February 2020 v. August 2025



Source: U.S. Bureau of Labor Statistics

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Glory

Glory (1989)—Denzel Washington as Trip, a member of the 54th Massachusetts infantry regiment

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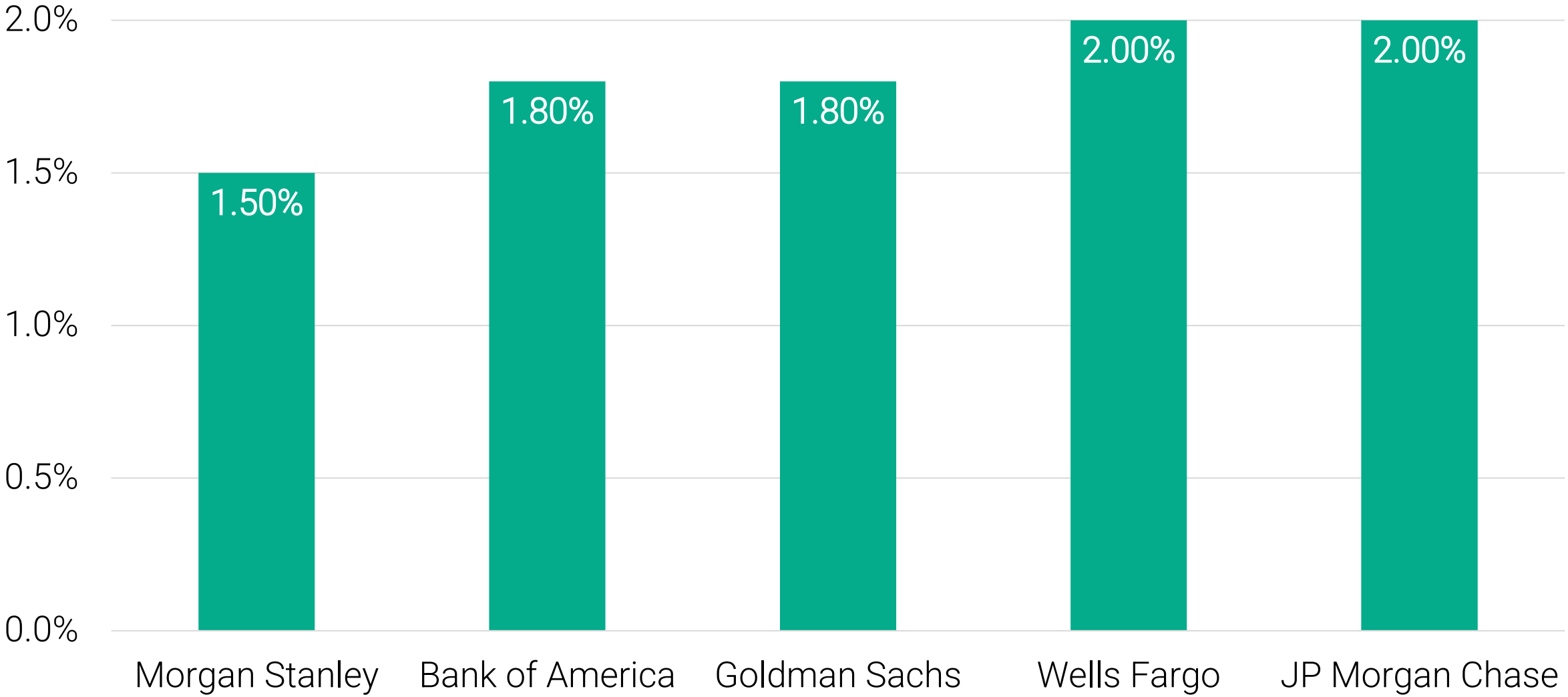
Major Banks' Forecasts for U.S. GDP Growth in 2025



Source: Brokerage and media reports

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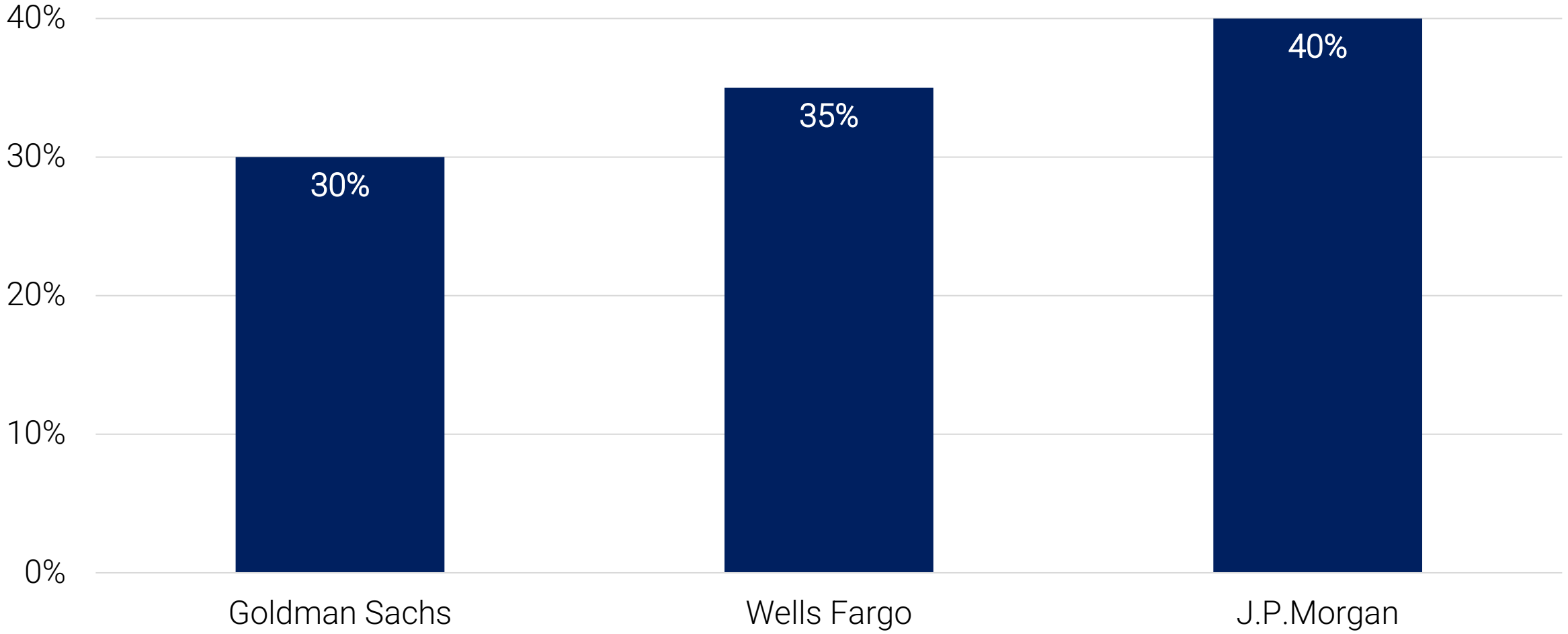
2025 Recession Probability, Major Brokerage Forecasts



Source: Reuters

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Out of Time

Could N. American Economy Surprise to the Downside?

- Inflation poised to stage a comeback – blame tariffs;
- Interest rates will be higher for longer – ditto;
- Many consumers now exhausted financially, and circumstances could worsen – right, because of tariffs;
- Are asset prices overextended? This may have something to do with tariffs;
- So forecast was for growth in 2025, but there are risks, including rising interest rates and falling asset prices, and that's because of . . . well you know.

*Out of Time (2003)—Denzel Washington as police chief Matt Lee Whitlock

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Thank You

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Q&A

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PARALLEL SESSION A3
En"cased" in Gold

APPLYING THE INCREMENTAL RIGIDITY METHOD TO STATIC LOAD TESTS PERFORMED ON ALL-STEEL PILES

Presented by:

Joseph (Joey) Mulcahey, E.I.T

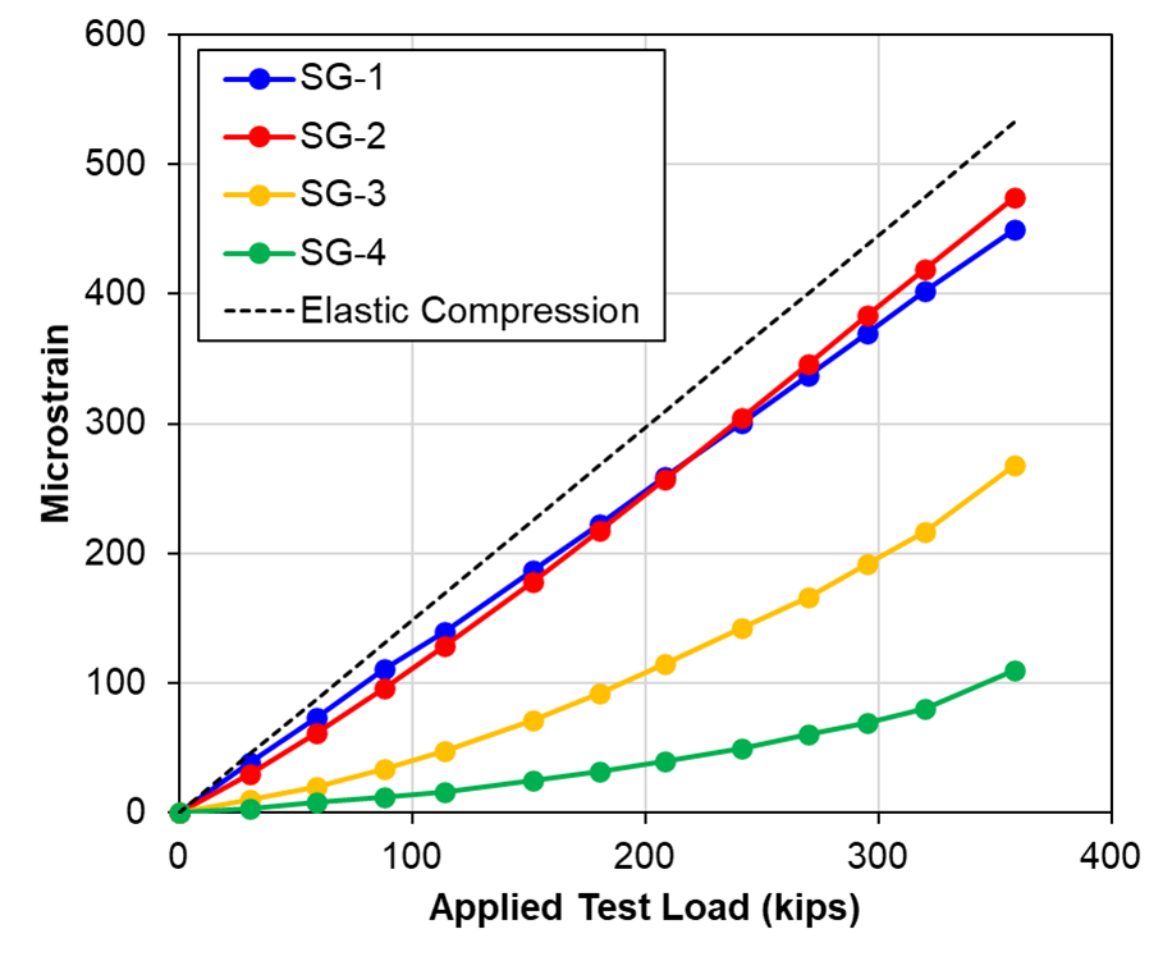
Geomechanical Engineer, RESPEC

The opinions expressed in this presentation and on the following slides are solely those of the presenter and not necessarily those of DFI. DFI does not guarantee the accuracy or reliability of the information nor endorse the recommendations offered by the presenter.

Outline

- Introduction – The Need for Something Better
- Technical Background
- Test Case Results
- Overall Results
- Summary

The Task



τ = Unit Shaft Resistance

q_t = Unit Toe Resistance

The Tool

$$F = EA\varepsilon$$

E = 29,000 ksi

A = Steel Manual Lookup

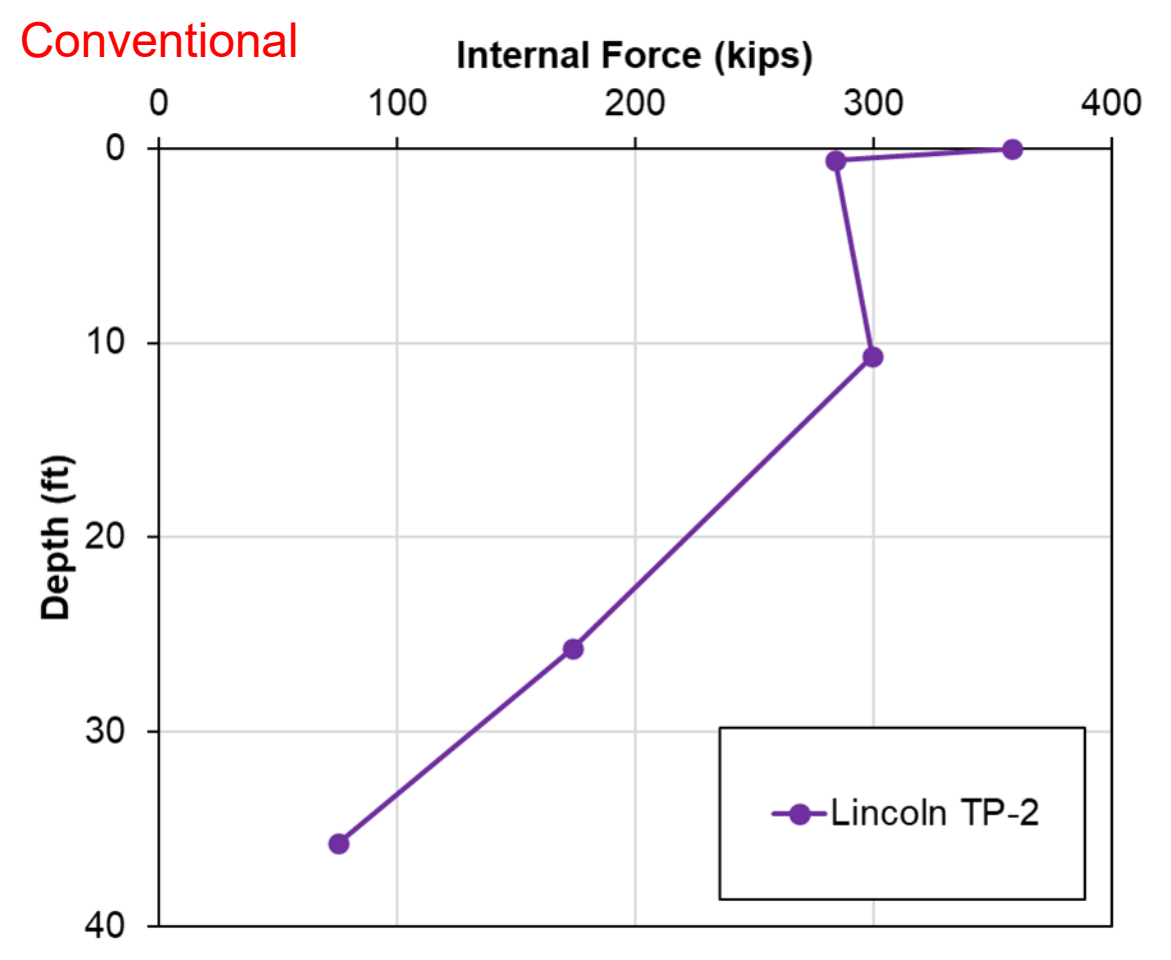
ε = measured strain



$$\tau = (F_2 - F_1) / (L_{1\text{-}2} \times P)$$

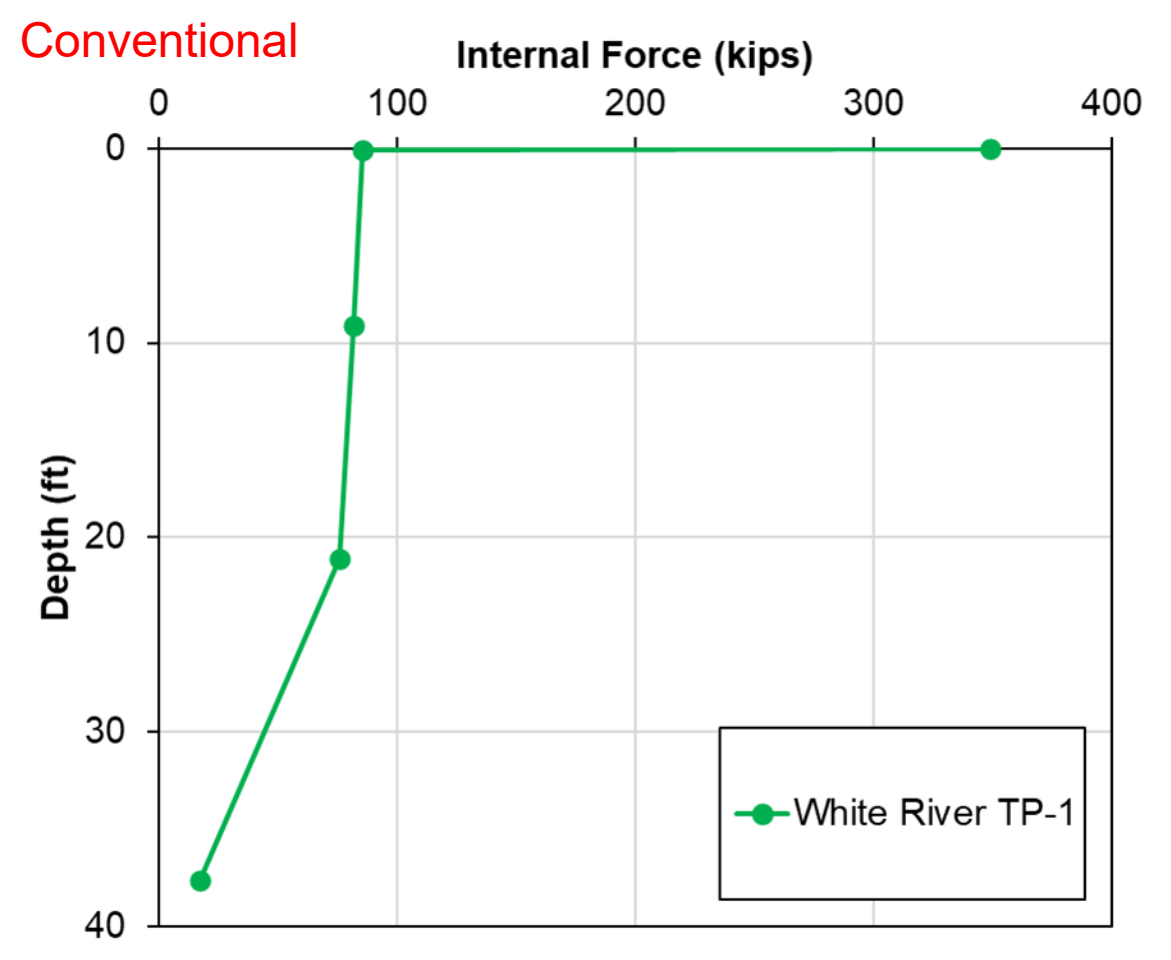
$$q_t = F_t / A$$

The Trouble



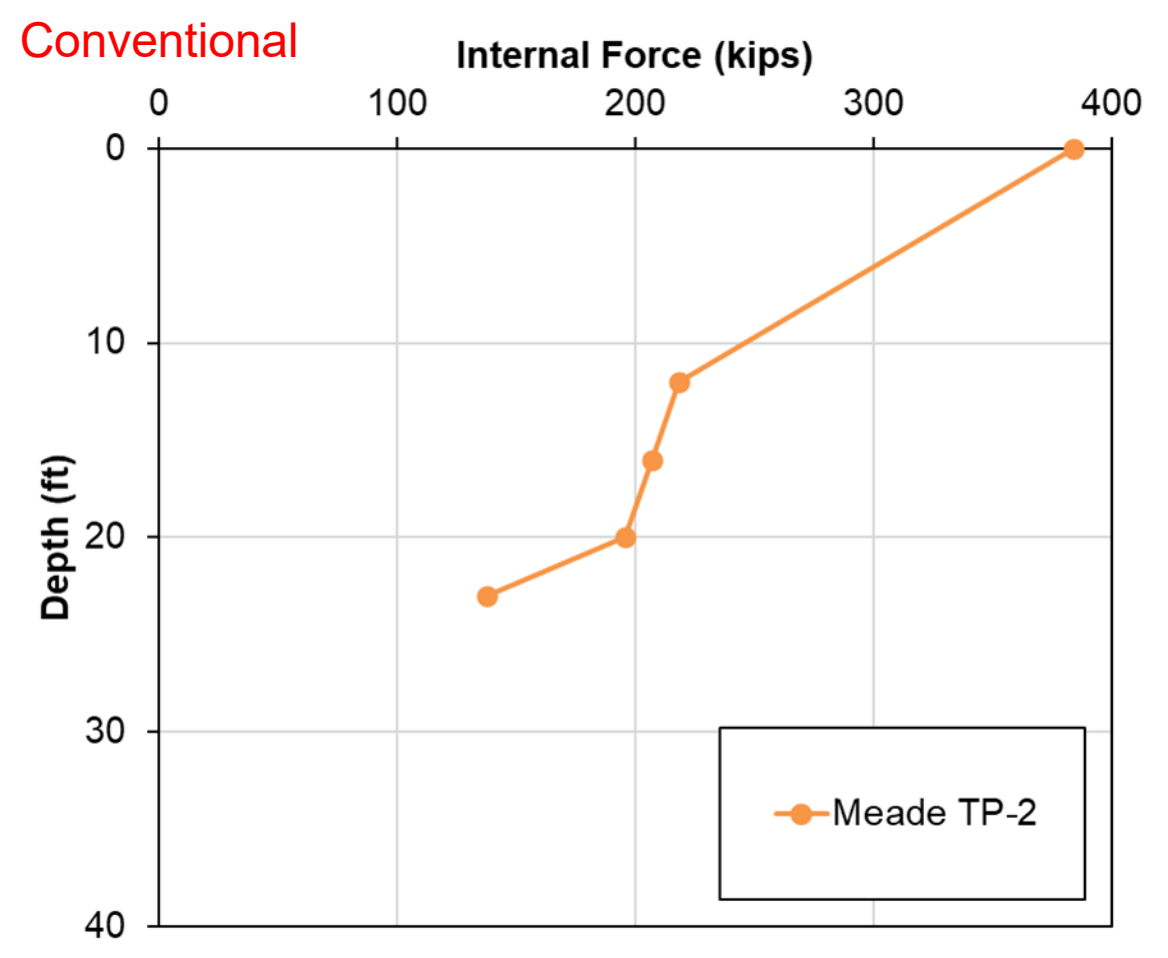
- Negative shaft resistance???

The Trouble



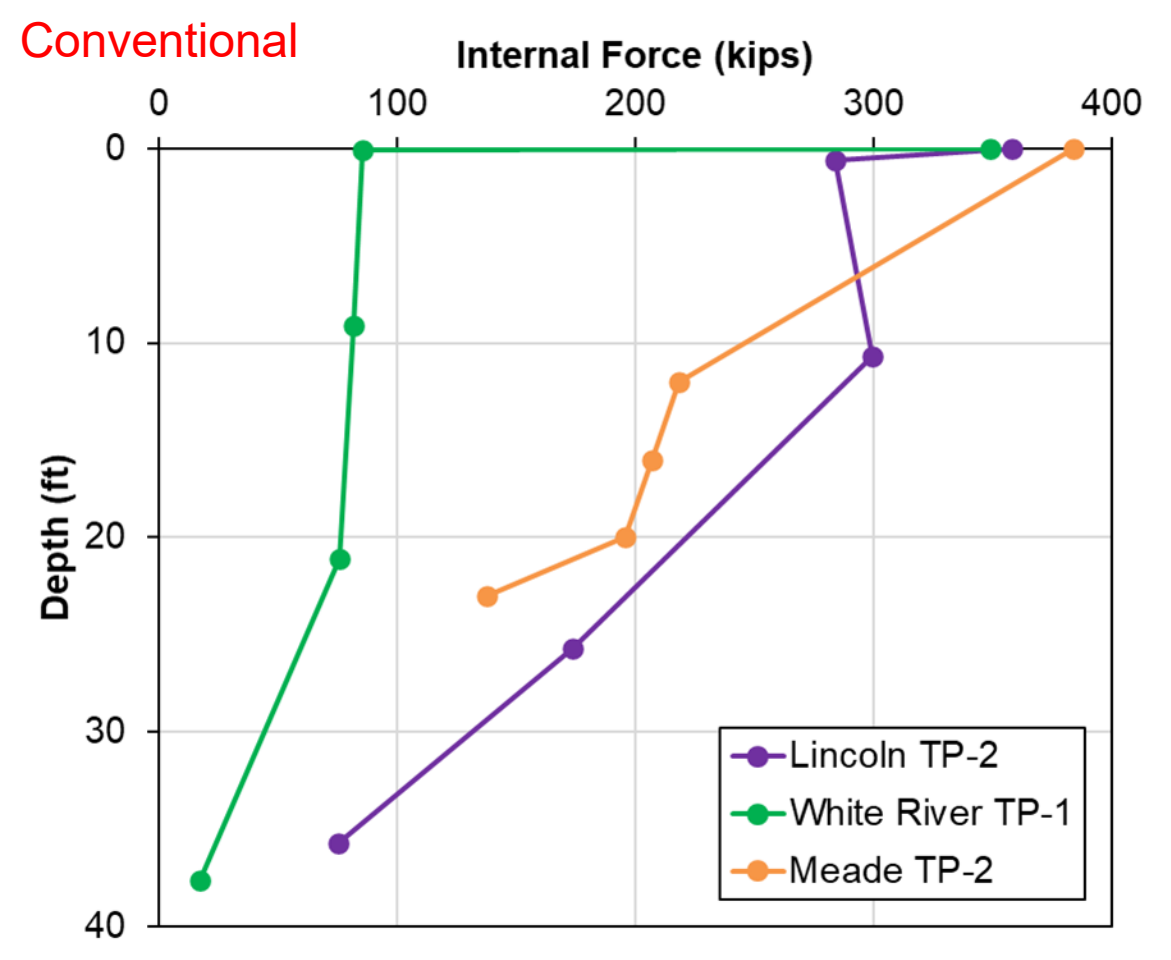
- Negative shaft resistance???
- 250 kips lost at the top???

The Trouble



- Negative shaft resistance???
- 250 kips lost at the top???
- High resistance in weak layer???

The Trouble

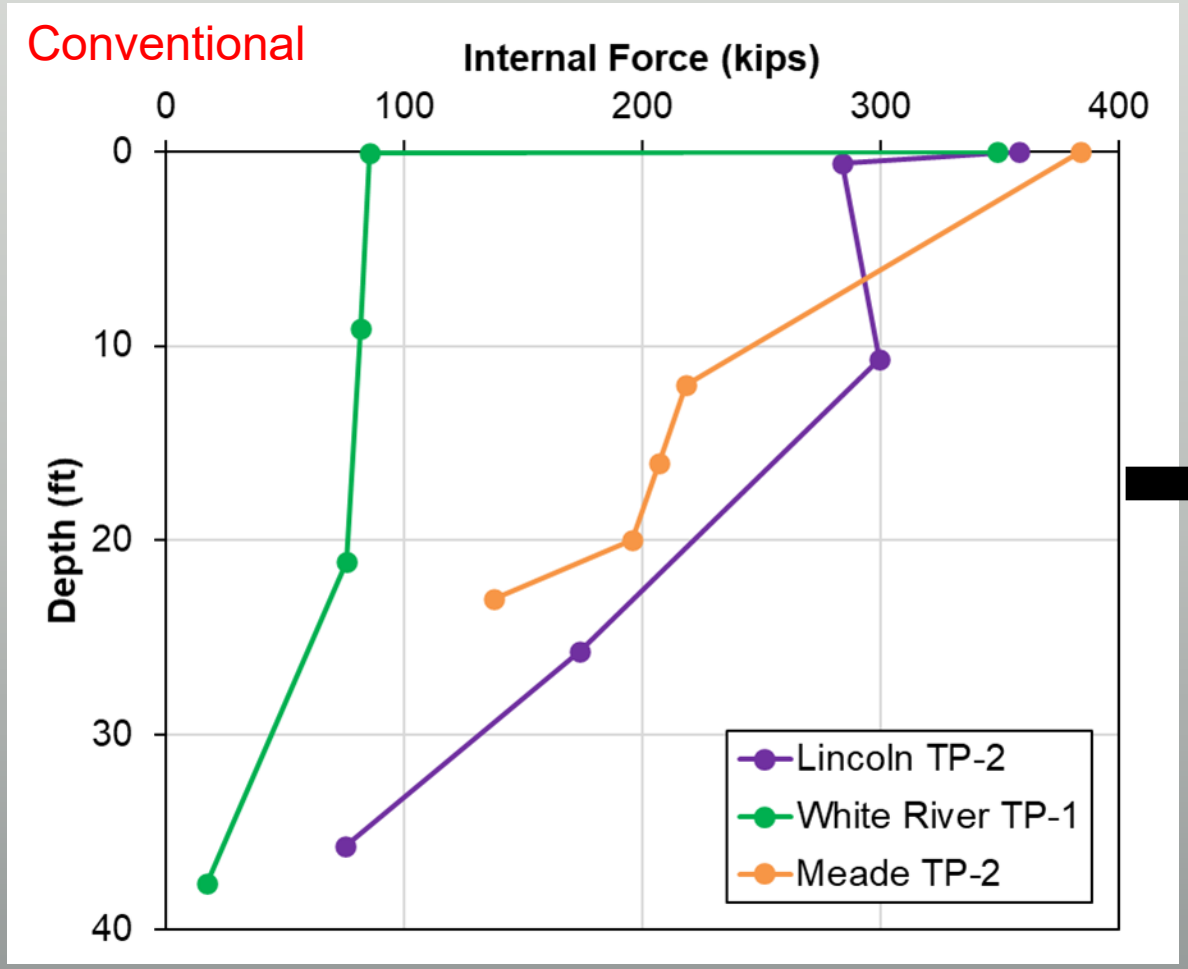


- Negative shaft resistance???
- 250 kips lost at the top???
- High resistance in weak layer???

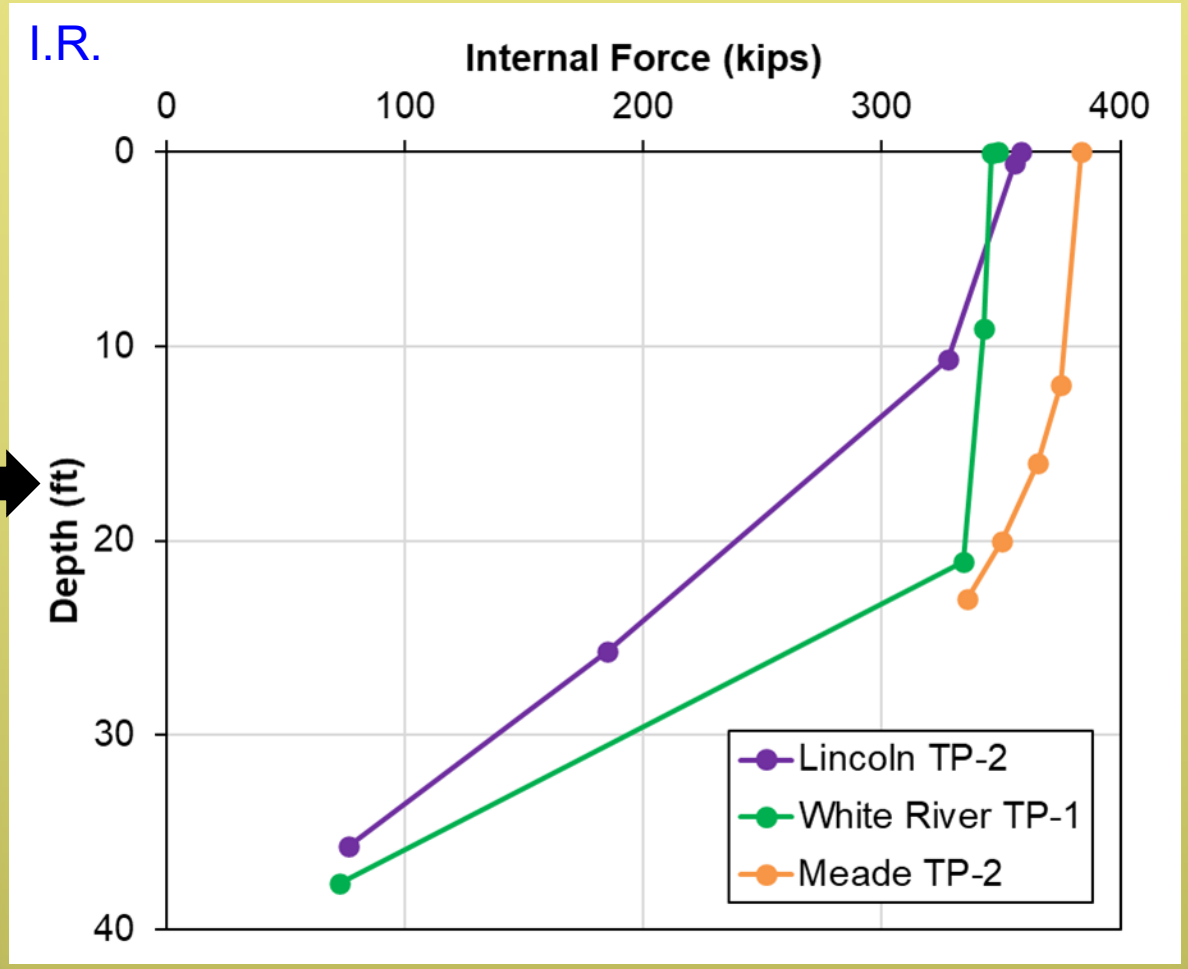
What do we do now?

The Turnaround

Conventional



I.R.



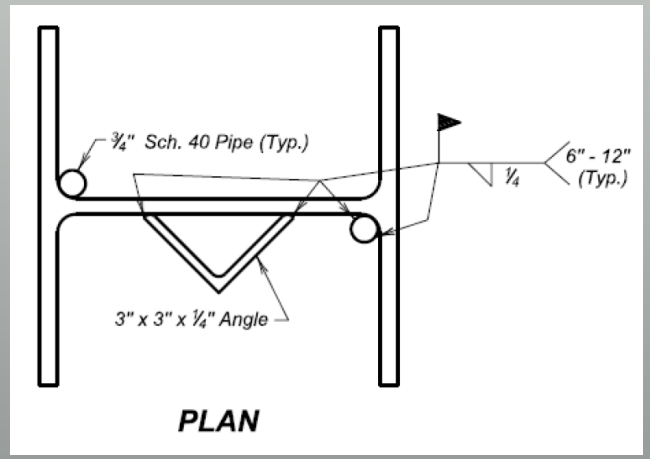
Can we reasonably assume E and A?

Variability in E

- Typically 29,000 – 30,000 ksi: ± 3.4%
- Differences usually related to metallurgy

Variability in A

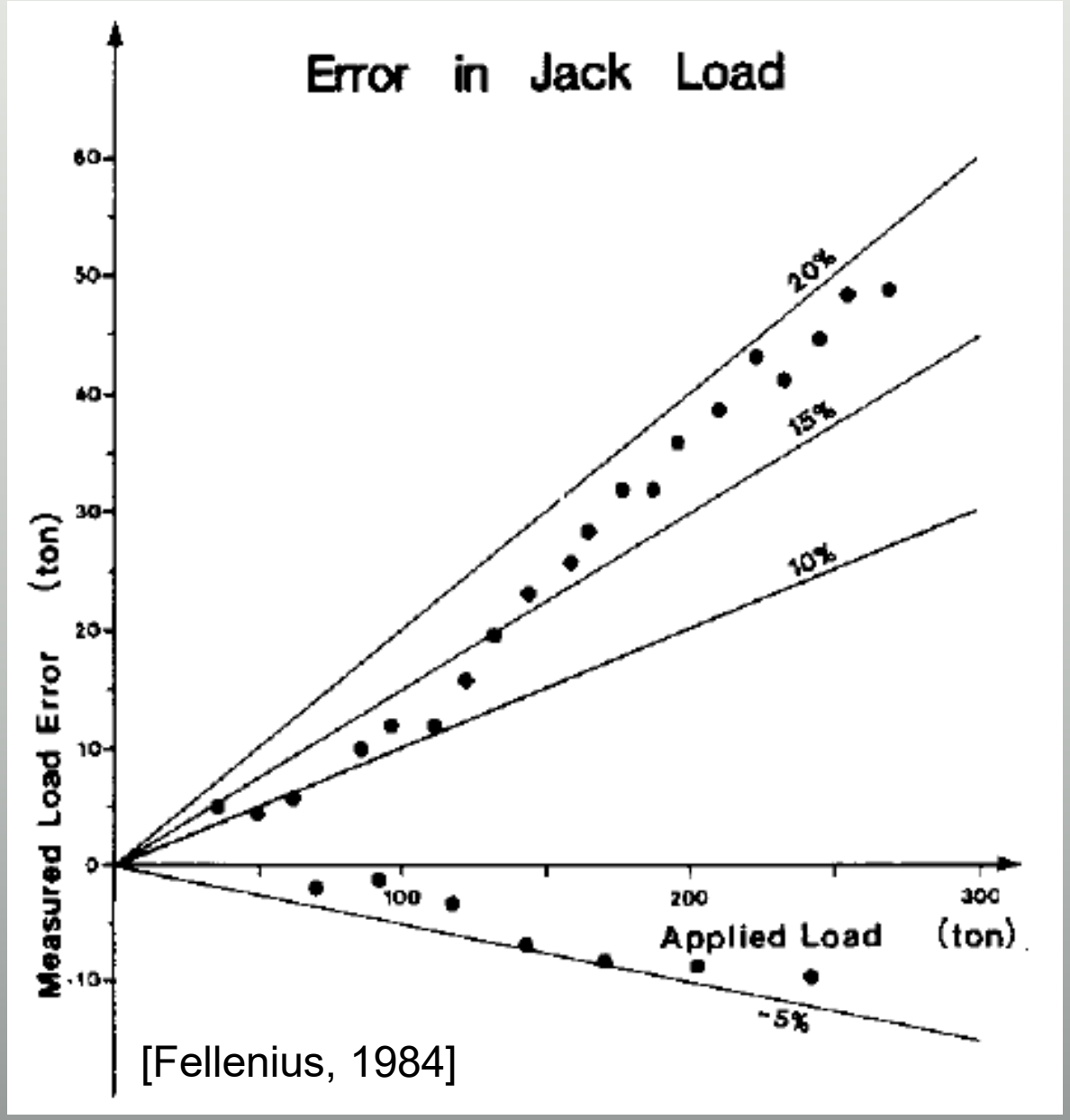
- AISC H-Section: ± 2.5%
- Area of protective channel: + 6.7-11.6%



Other Possible Sources of Error

- Load cell error (right)
- Strain gauge calibration
- Uncertain local soil conditions
- Impacts of pile driving (e.g., batter)





[Fellenius, 1984]

I.R. Equation Modified for Steel

Theoretical I.R. Relationship

if a ≠ 0 (e.g., concrete pile)

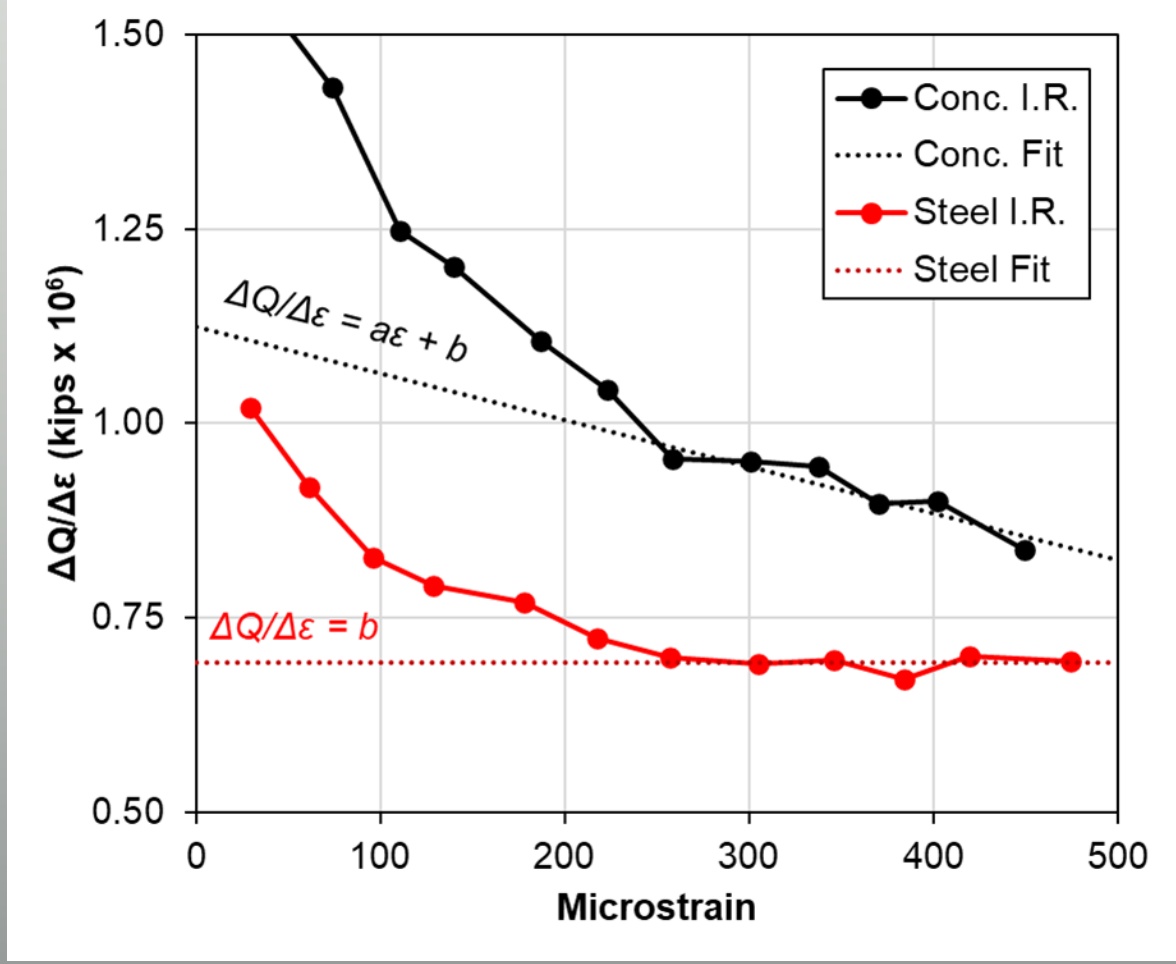
$\Delta Q/\Delta\varepsilon = a\varepsilon + b$

$\boldsymbol{F = 0.5a\varepsilon^2 + b\varepsilon}$

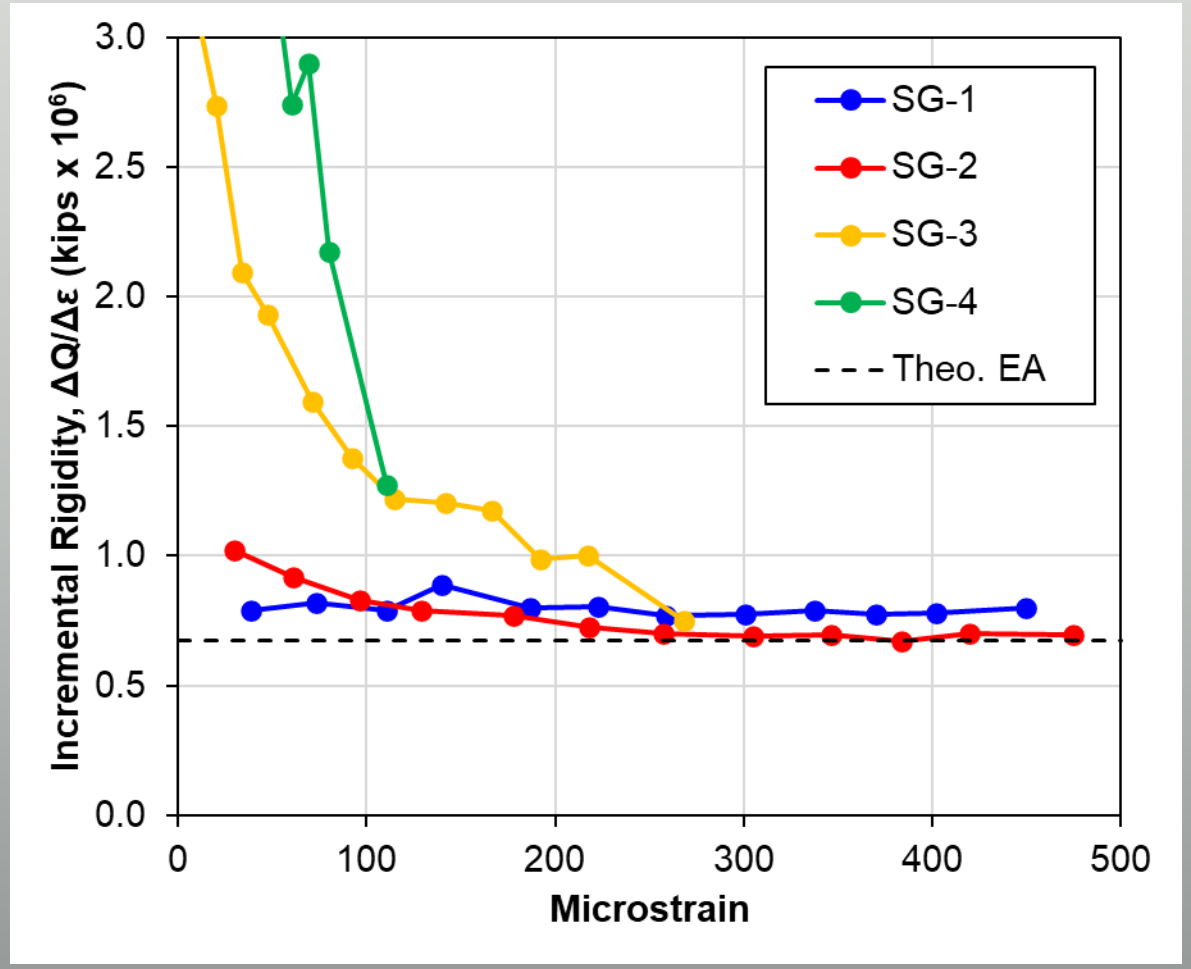
if a = 0 (e.g., all-steel pile)

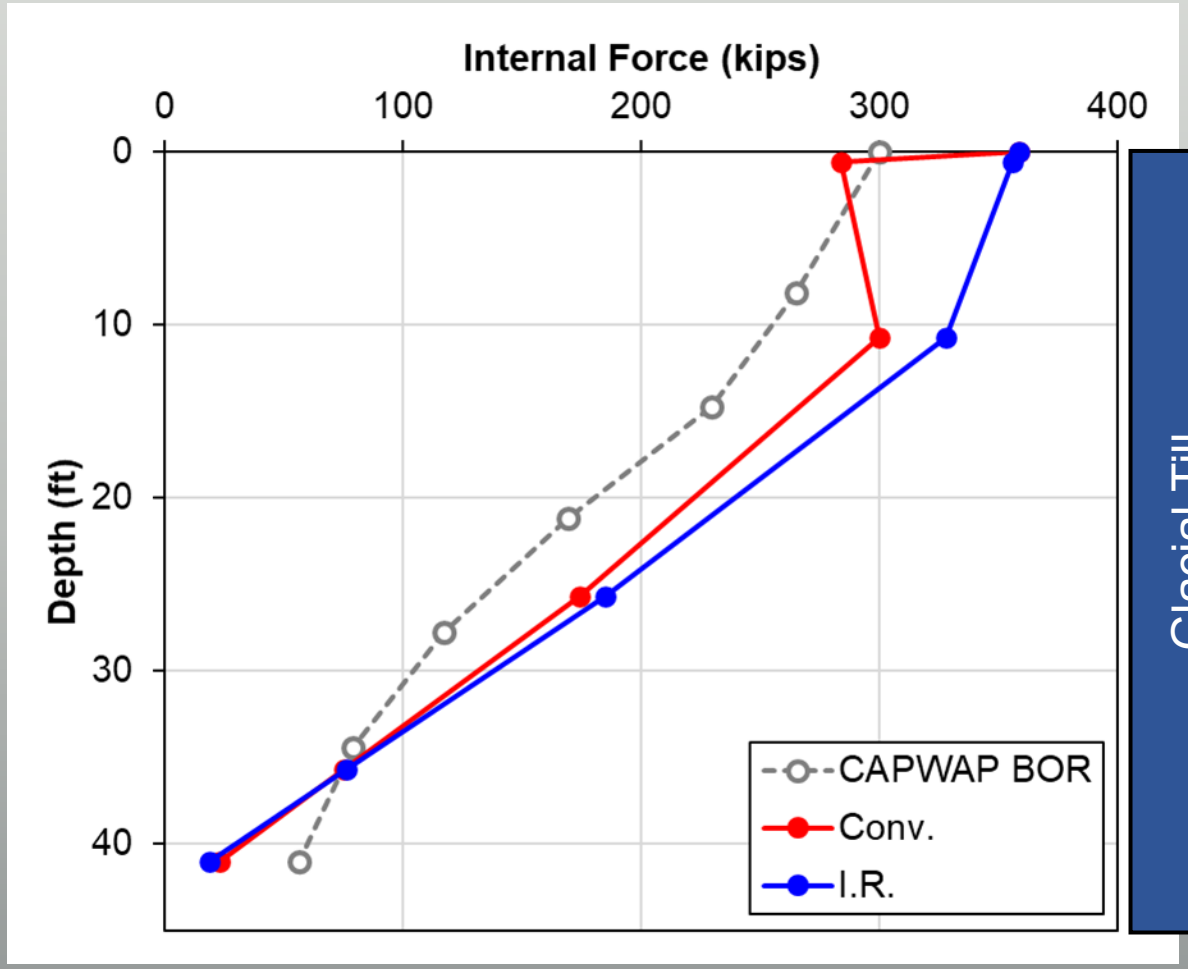
$\Delta Q/\Delta\varepsilon = b$

$\boldsymbol{F = b\varepsilon}$

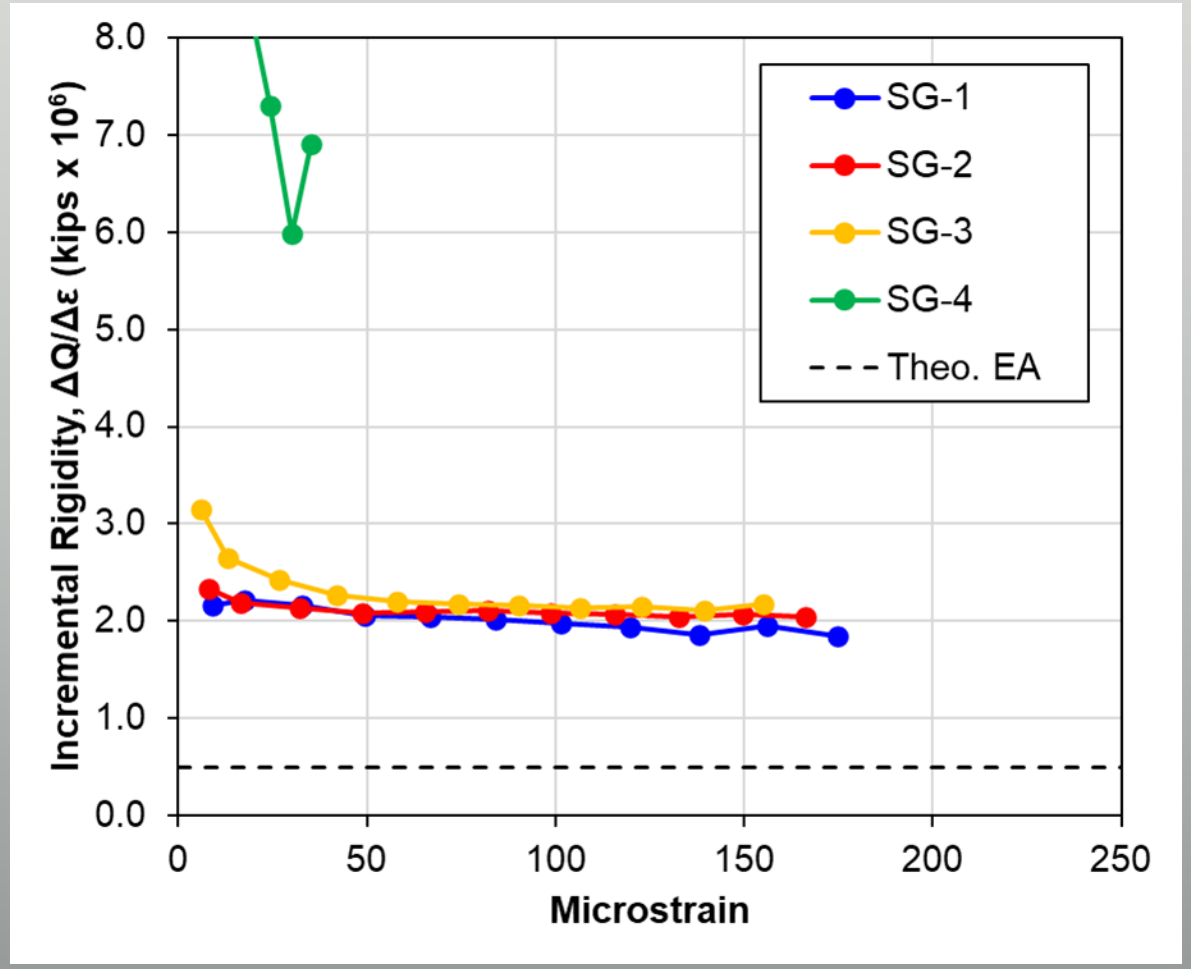


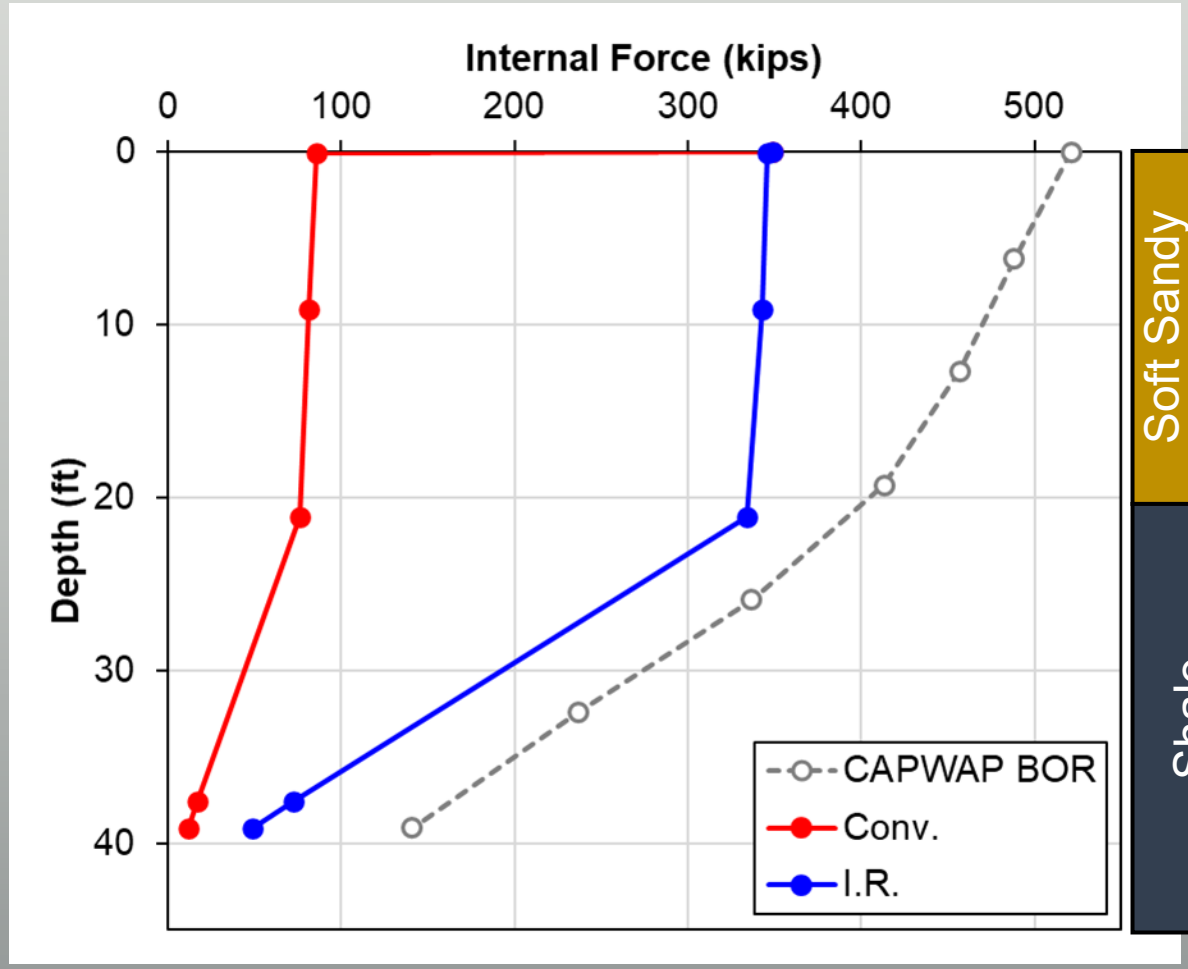
Case #1 – Lincoln TP-2



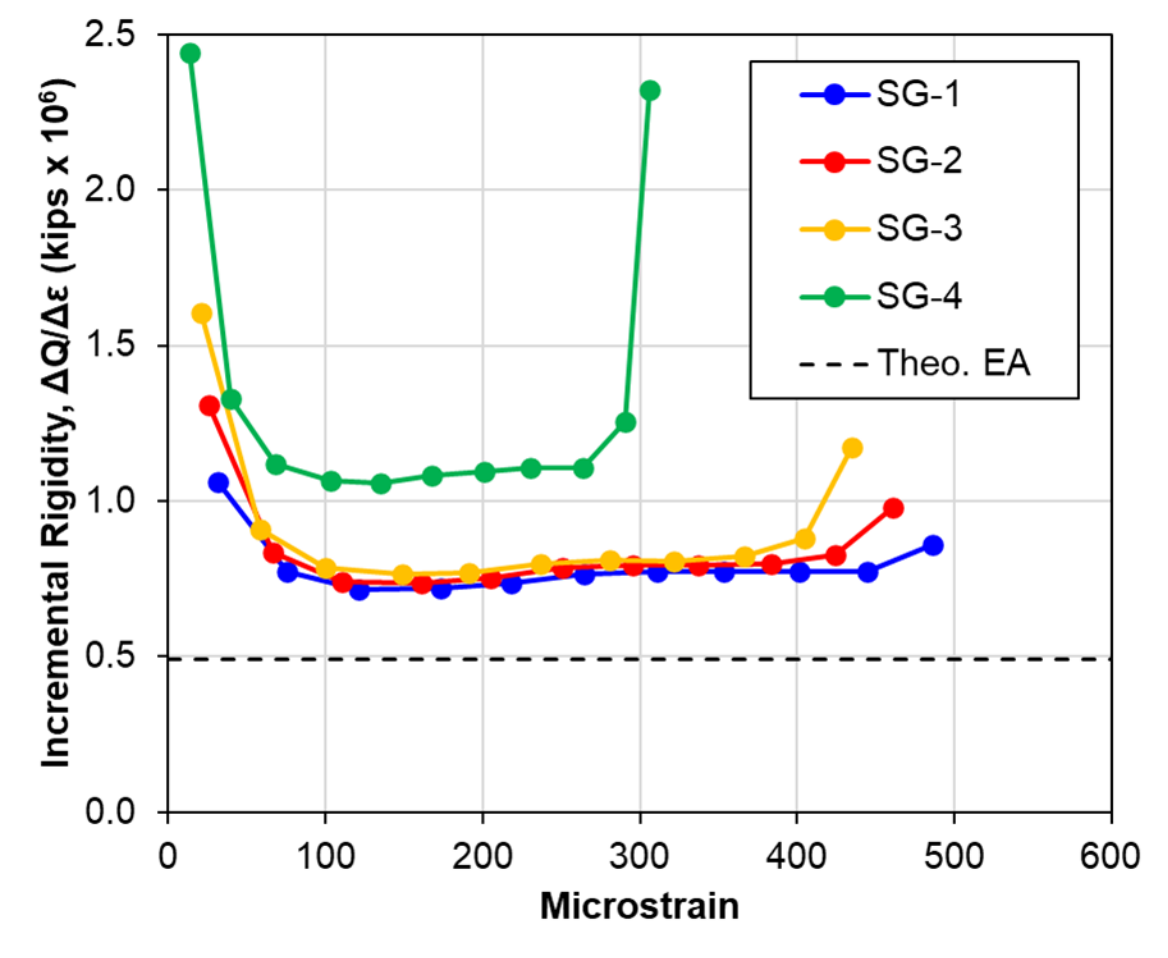


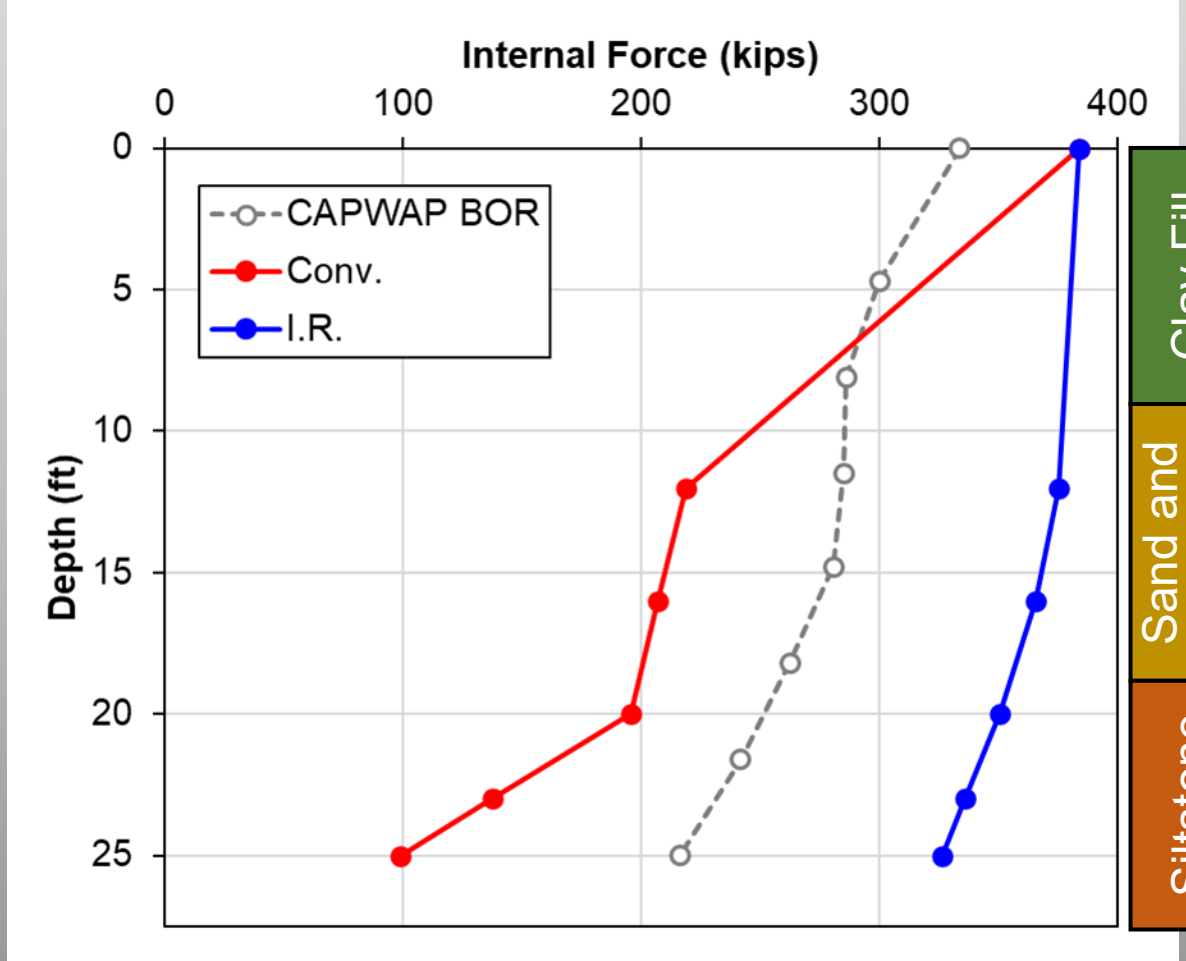
Case #2 – White River TP-1





Case #3 – Meade TP-2

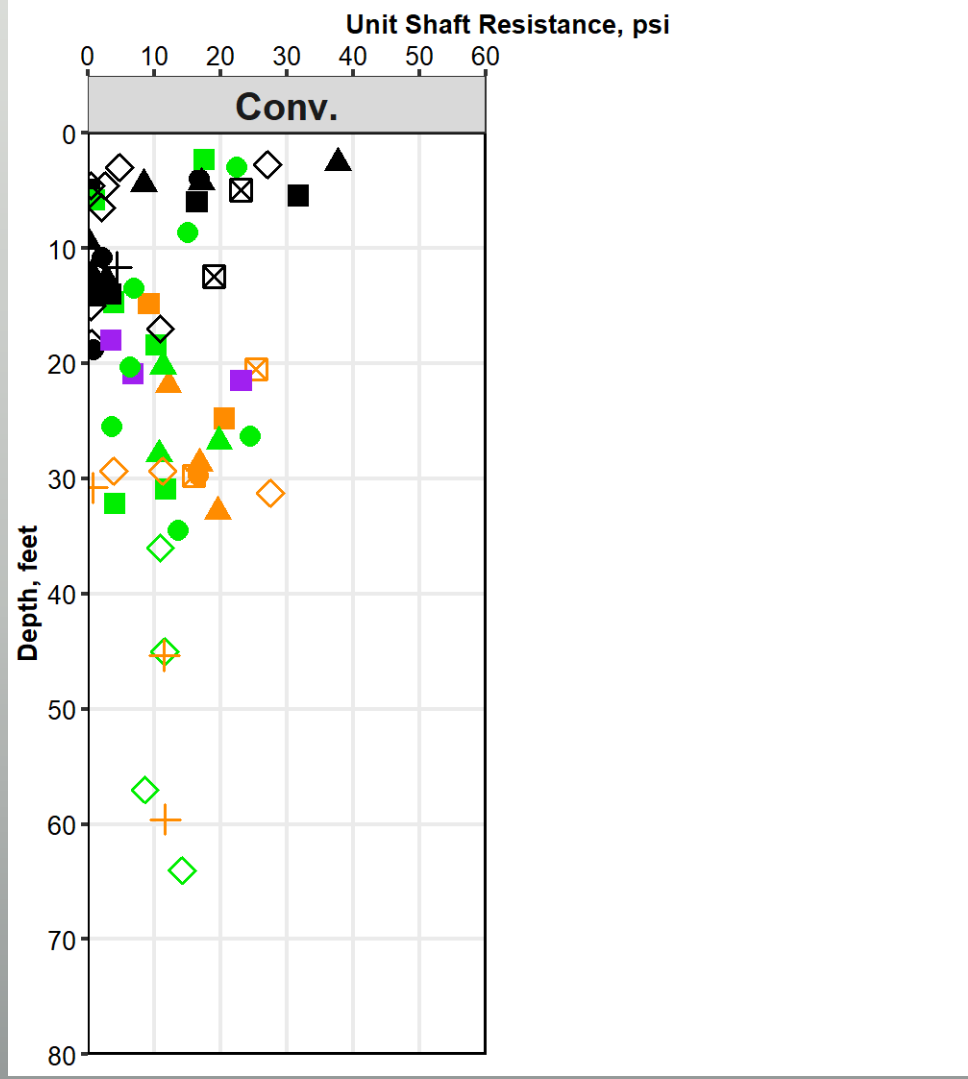


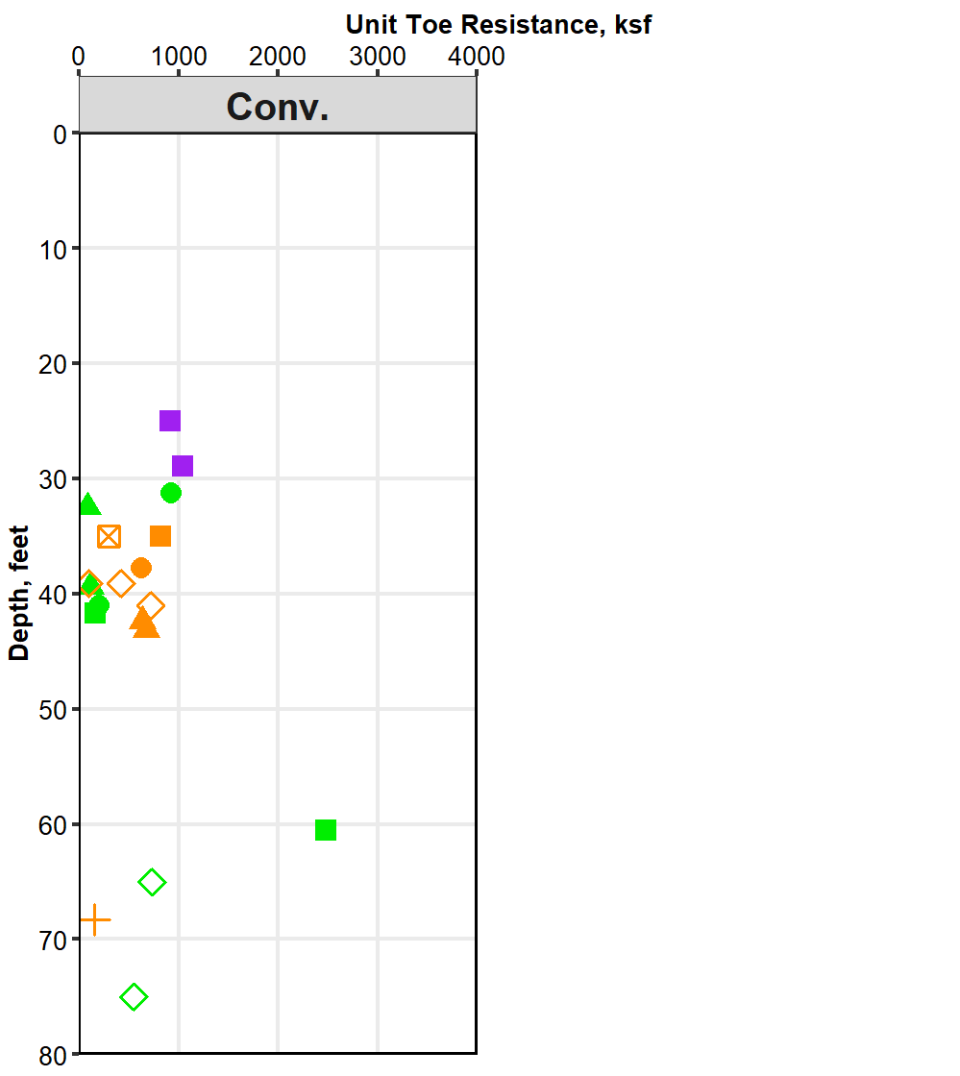


Unit Resistance Visual Comparisons

■ = Overburden ■ = Glacial Till
■ = Shale ■ = Siltstone

Corson
Lincoln
Fall River
Spink
Minnehaha
Buffalo
Brookings
Meade
White River
Harding
Keldron





Unit Resistance Visual Comparisons

= Overburden
= Shale
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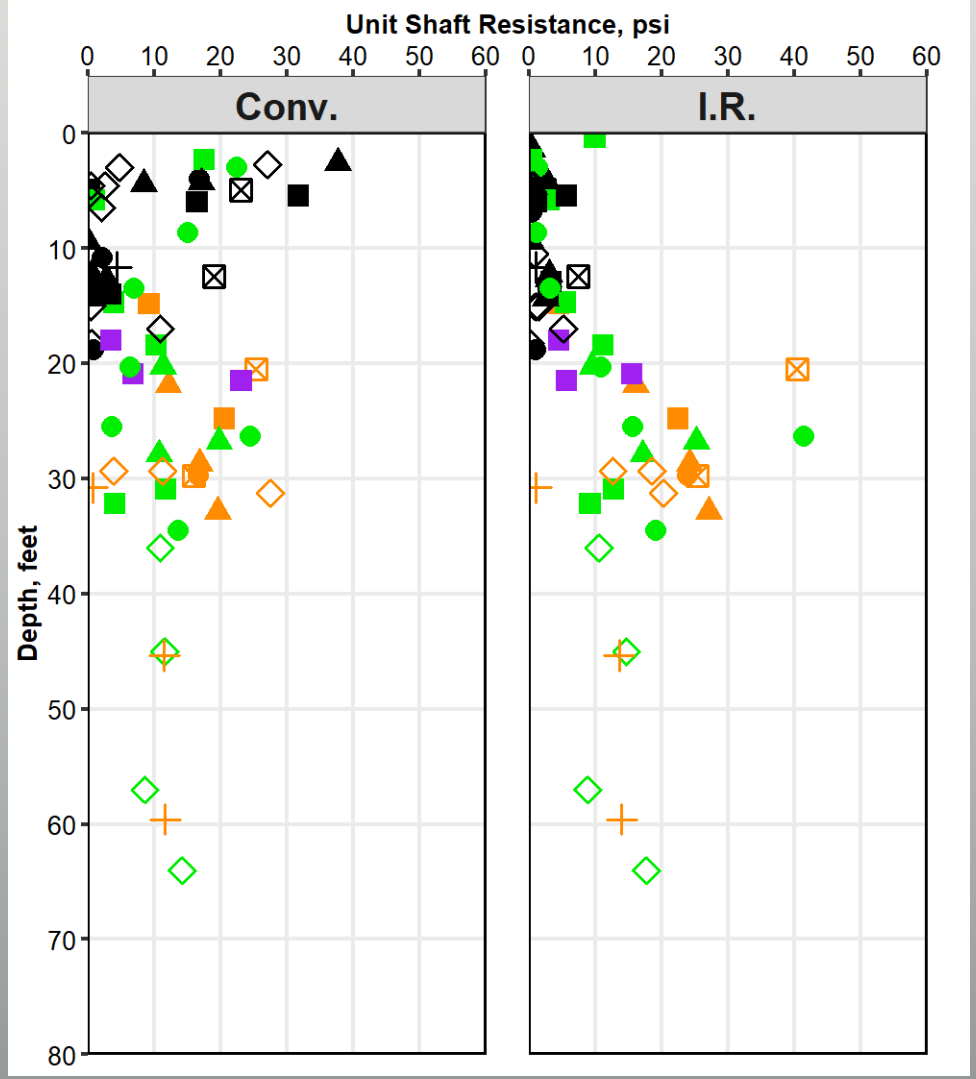
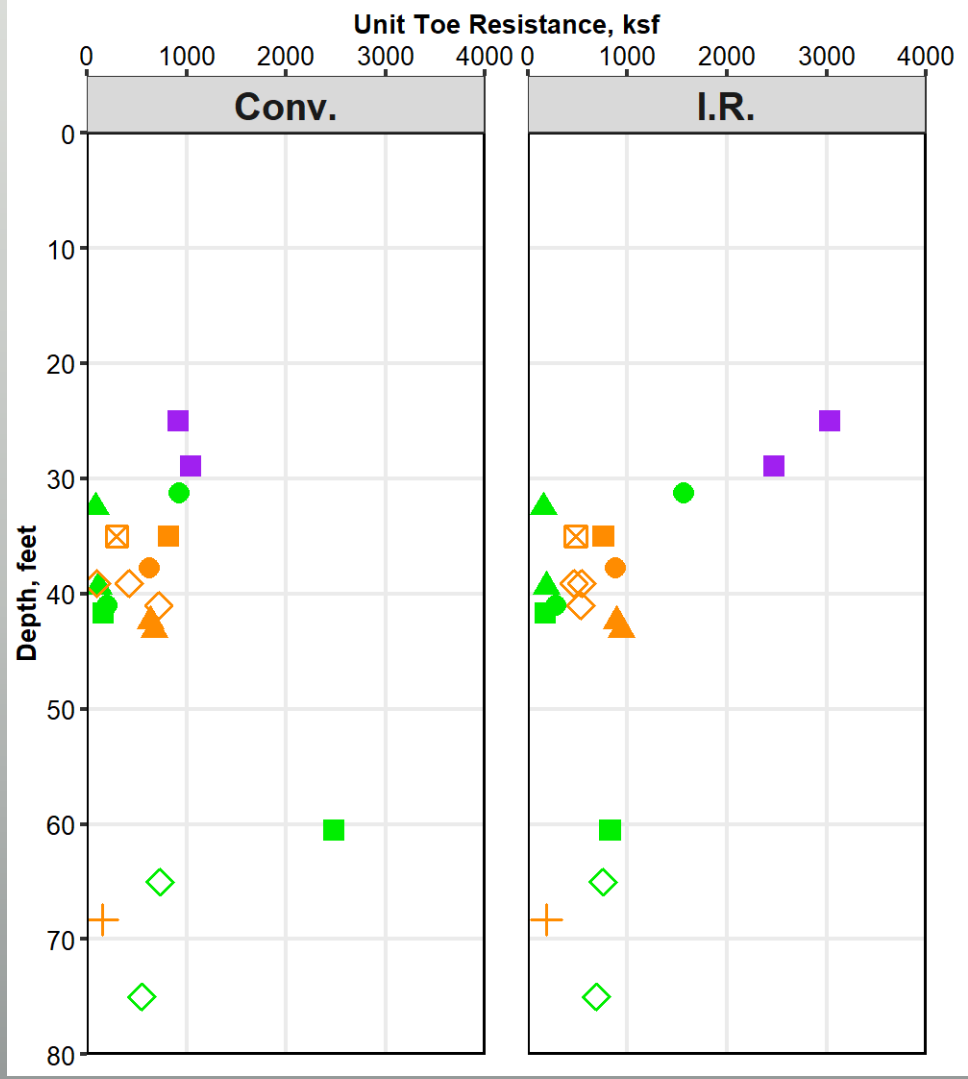




Unit Resistance Statistical Comparisons

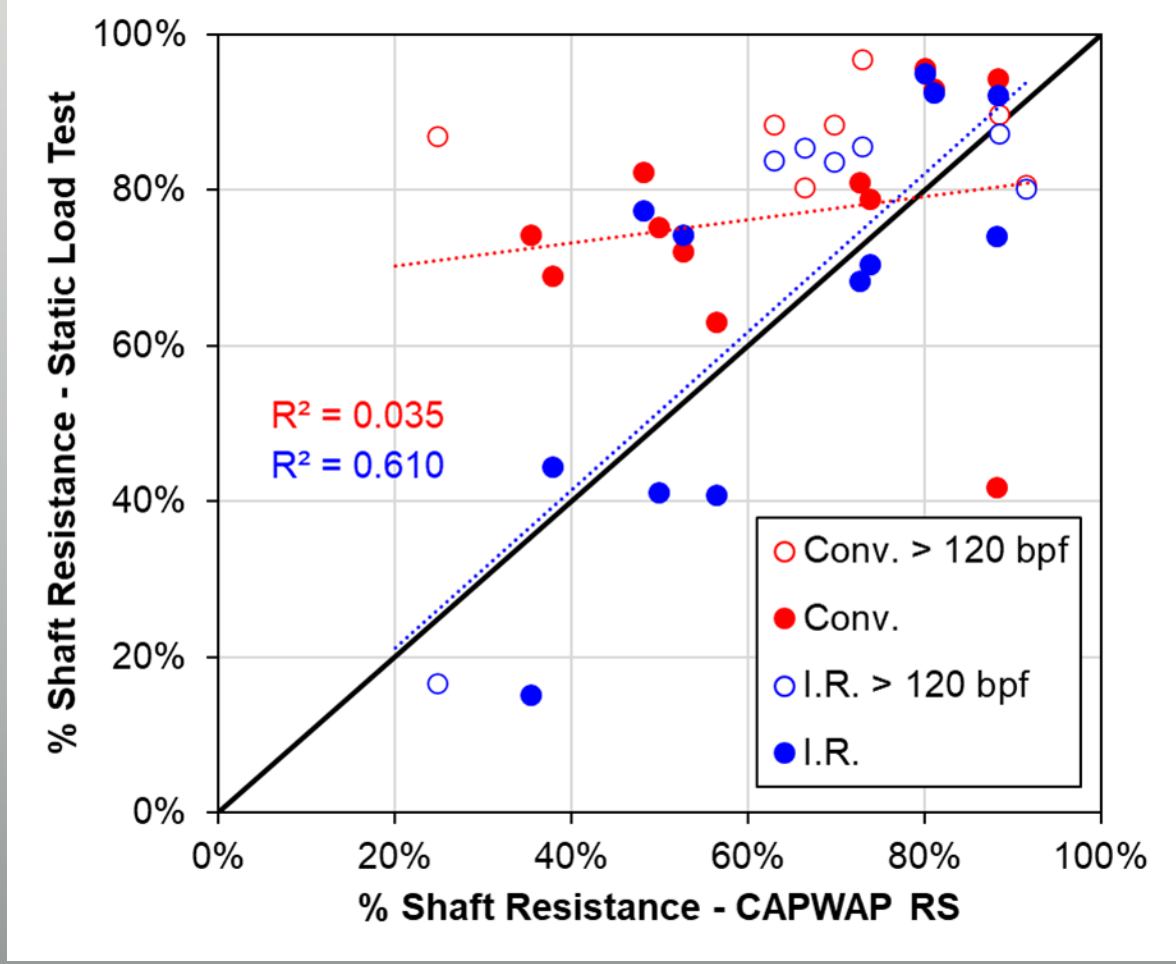
- Observations
  - CV generally decreased (except for shale shaft resistance)
- Takeaways
  - I.R. gave us a higher confidence in characterizing unit resistance parameters

Parameter	Metric	Weak Overburden		Shales Only		Tills Only	
		Conv.	I.R.	Conv.	I.R.	Conv.	I.R.
# of Tests (Shaft Segments)		19 (27)		9 (14)		8 (21)	
Unit Shaft Resistance (psi)	AVG	10.6	1.8	14.5	18.9	16.2	11.9
	SD	15.8	1.9	7.5	9.8	22.9	9.4
	CV	**1.49**	**1.02**	**0.52**	**0.52**	**1.41**	**0.79**
Unit Toe Resistance (ksf)	AVG	-	-	497	630	658	580
	SD	-	-	261	252	800	485
	CV	-	-	**0.53**	**0.40**	**1.21**	**0.84**

CAPWAP Comparison

- Observations
  - I.R. shows better correlation to % shaft resistance from CAPWAP RS
  - Conventional method can overestimate shaft resistance
- Takeaways
  - I.R. more accurately delineates shaft and toe resistance distribution than the conventional method

Note: data points not included in the fit where RS resistance > 120 bpf



Summary

- Conclusions
  - The I.R. method produced internal force results that agreed better with other available data compared to simply assuming E and A
  - The I.R. method allowed us to salvage strain gauge data that would have otherwise been unusable
- Recommendations
  - The favorable results justify the use of the I.R. method for steel piles, despite the extra analysis effort and apparent subjectivity
  - The quality of the analysis is believed to be enhanced with scrutinizing site- and test-specific data to validate I.R. results

Thank you for attending!
Questions?

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Geomechanical Engineer, RESPEC
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(605) 646-4571





The impact of RIs on the design of footings and slabs

By: Sonia Sorabella Swift, PE and Jules Petrella

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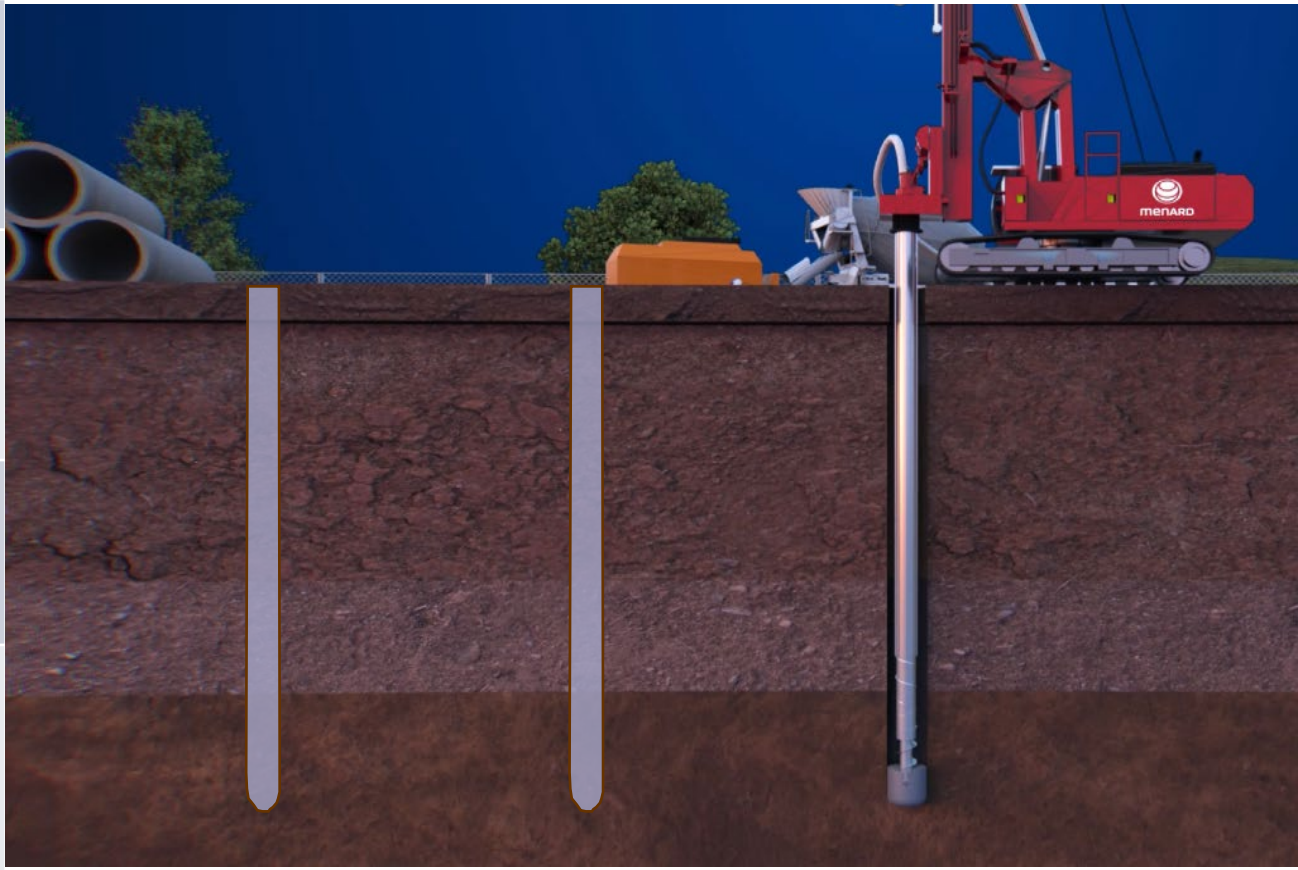
Outline

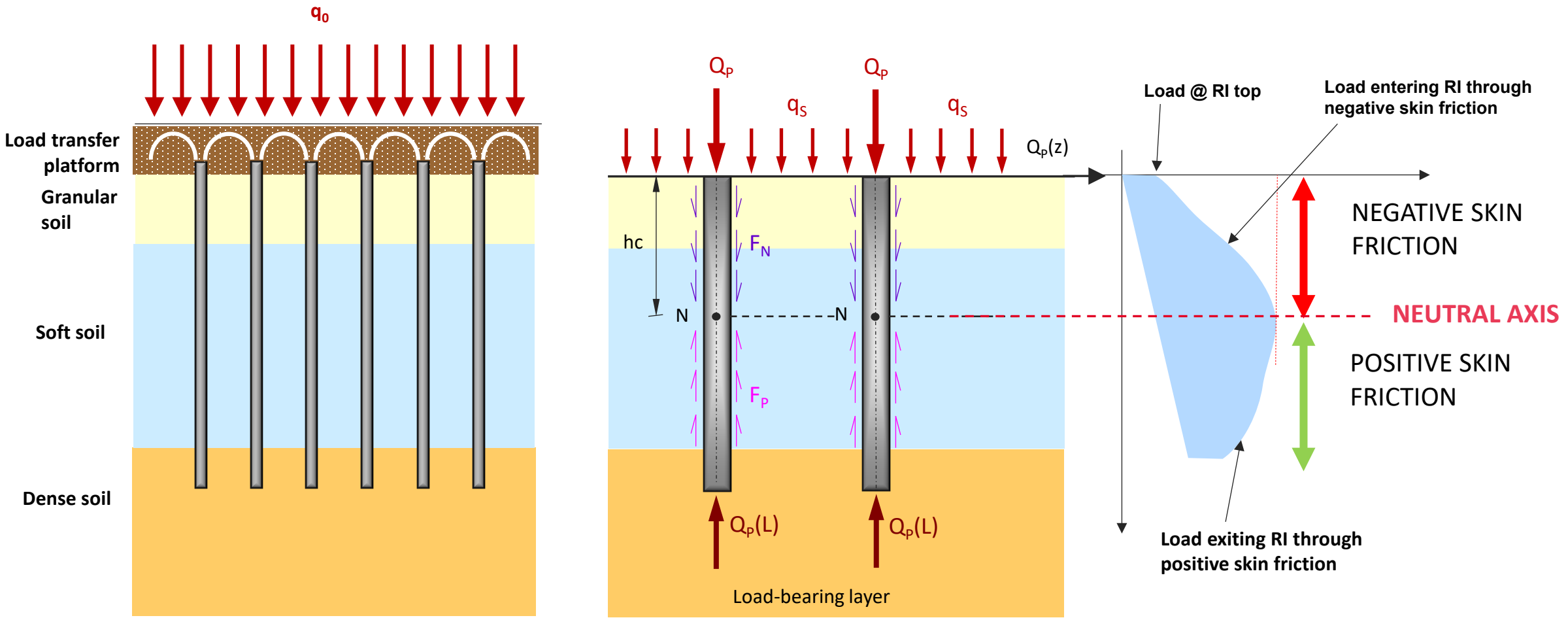
1. Intro to RIs
2. RI interaction with slabs
3. RI interaction with footings
4. Recommendations

Rigid Inclusion (RI) Basics

Use	Typical Composition
Bridge soft, compressible, or variable soils	Grout or cement
Accommodate high loads with stiff, fully-grouted element	Unreinforced
Ideal for contaminated sites (low/no spoils)	Displacement elements (not mixed)
Flexible system optimized to meet project conditions	10 to 18 inches in diameter
Applicable for seismic conditions	Reinforced as needed to provide adequate ductility



RIs: Load Sharing Principles



Design Methodology



Ground Improvement Methodology

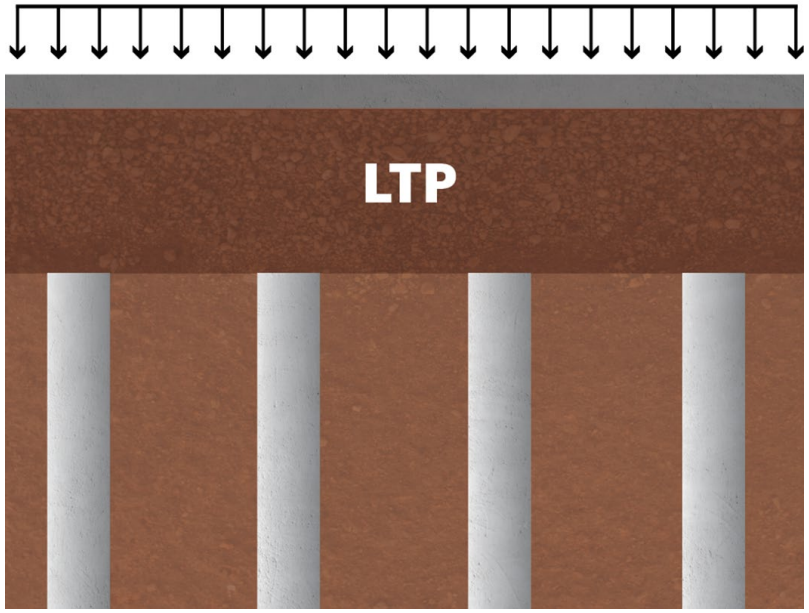
- Modest improvement needed
- Primarily settlement mitigation
- Load is shared between the soil and the RI
- Redundant system provided
- Stresses on foundations- relatively uniform
- Independent design of foundations and RI system

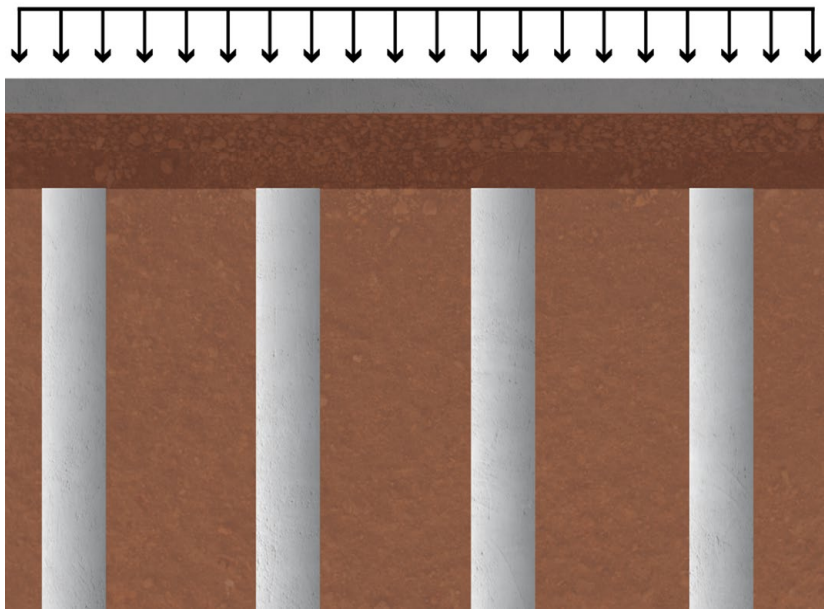
Deep Foundation Methodology

- Major improvement needed
- RIs providing bearing capacity
- Load is primarily resisted by the RI (low load sharing)
- Concentrated stresses on foundations
- Design of foundations and RI system may require coordination

Slabs and LTPs

- Slabs are designed as unreinforced concrete that doesn't crack. (Steel for T&S only)
- Most slabs-on-grade are not designed but rather based on experience.
- Slab-on-grade designed for uniform subgrade – RIs may result in non-uniform stresses
- LTP distributes load so that the stresses at the bottom of the slab are fairly uniform
- Reducing the thickness of the LTP increases the impact of the RI on the slab

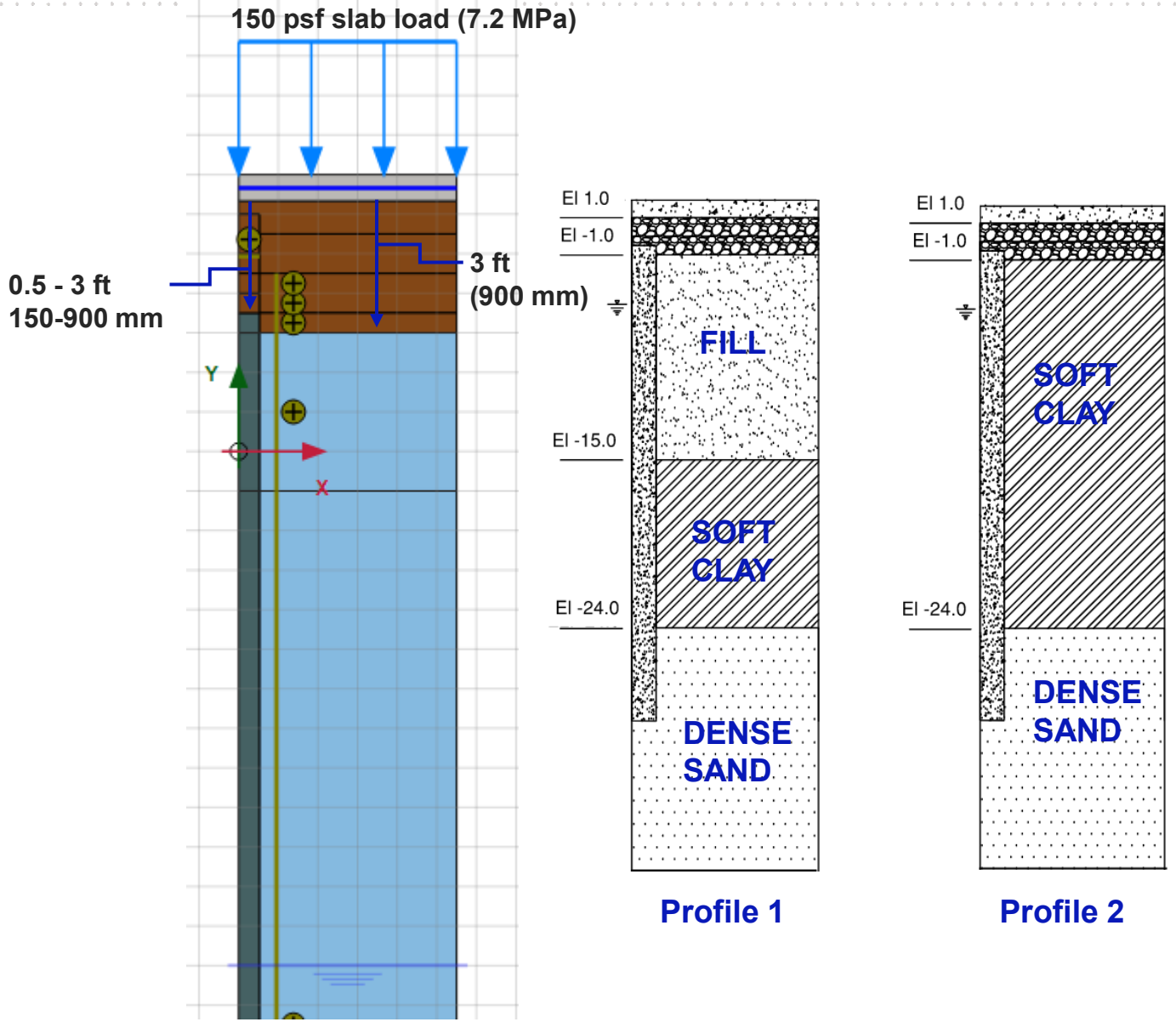




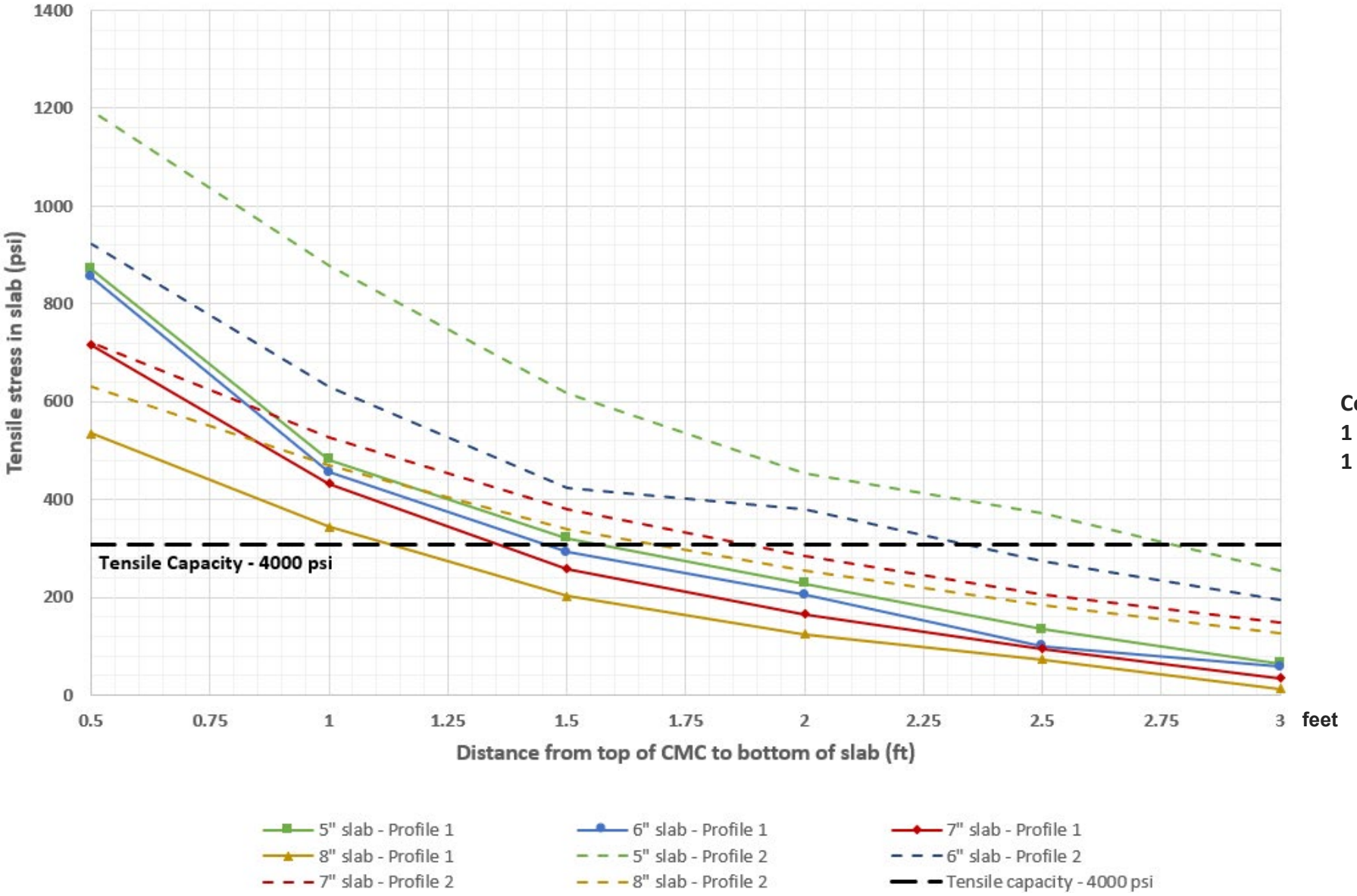


Evaluating the impact

- Modeled RIs supporting a slab in Plaxis (axisymmetric model)
- Used two different soil profiles
- Varied the thickness of the LTP and the slab to evaluate the tensile stress in the slab.
- Compared the tensile stress to the modulus of rupture of the slab.



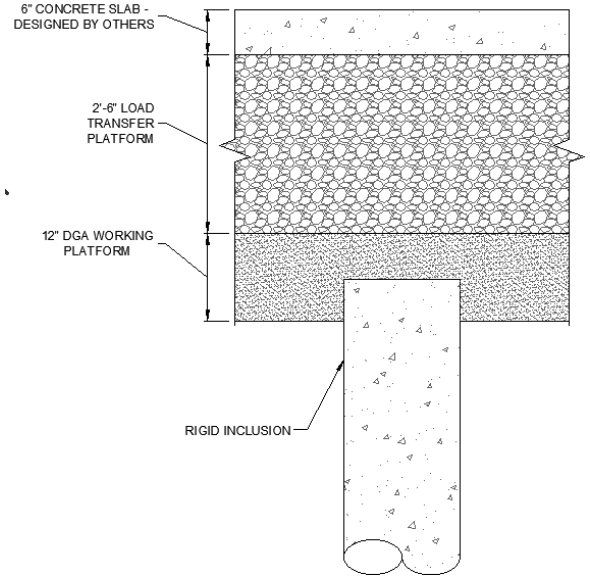
Modeling Results

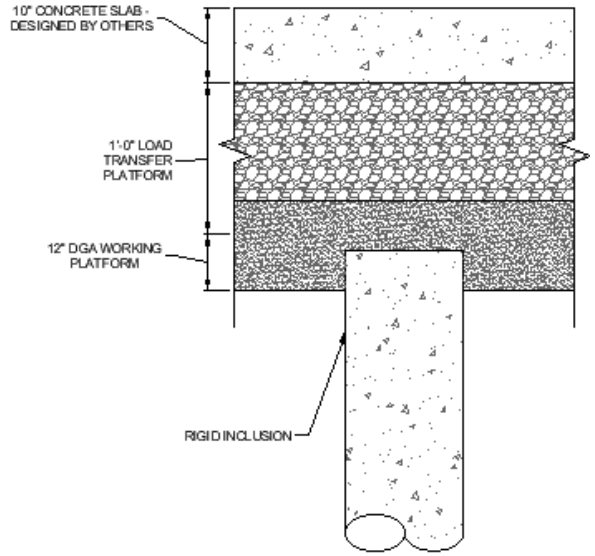


Conversion:
1 ft = 0.3 m
1 psi = 1.4 MPa

Design Recommendations (Slabs)

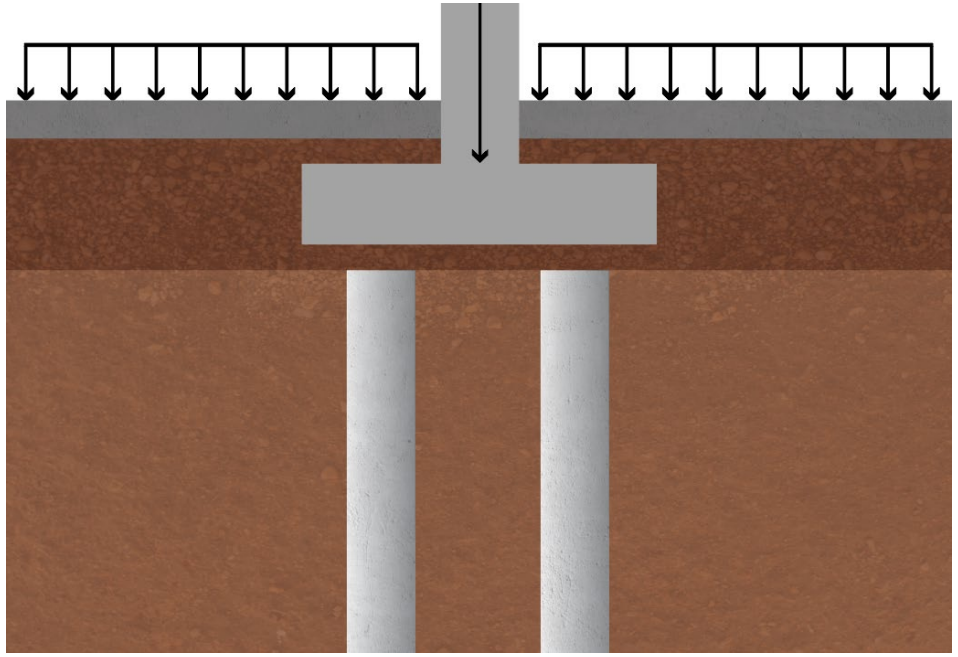
- Carefully consider slab stresses for LTP thicknesses less than:
  - 2 ft (60 mm) for competent soil profiles (Profile 1)
  - 3 ft (90 mm) for soft soil profiles (Profile 2)
- If the tensile stress is too high:
  - Thicken the LTP
  - Coordinate with the structural engineer to increase the moment capacity of the slab
- You can find a solution for almost any situation; it's about finding the right solution for your project.
  - Thin slab-on-grade, thick LTP
  - Thin LTPs, thick slab-on-grade
  - No LTP, structural slab
  - …and many combinations in between.



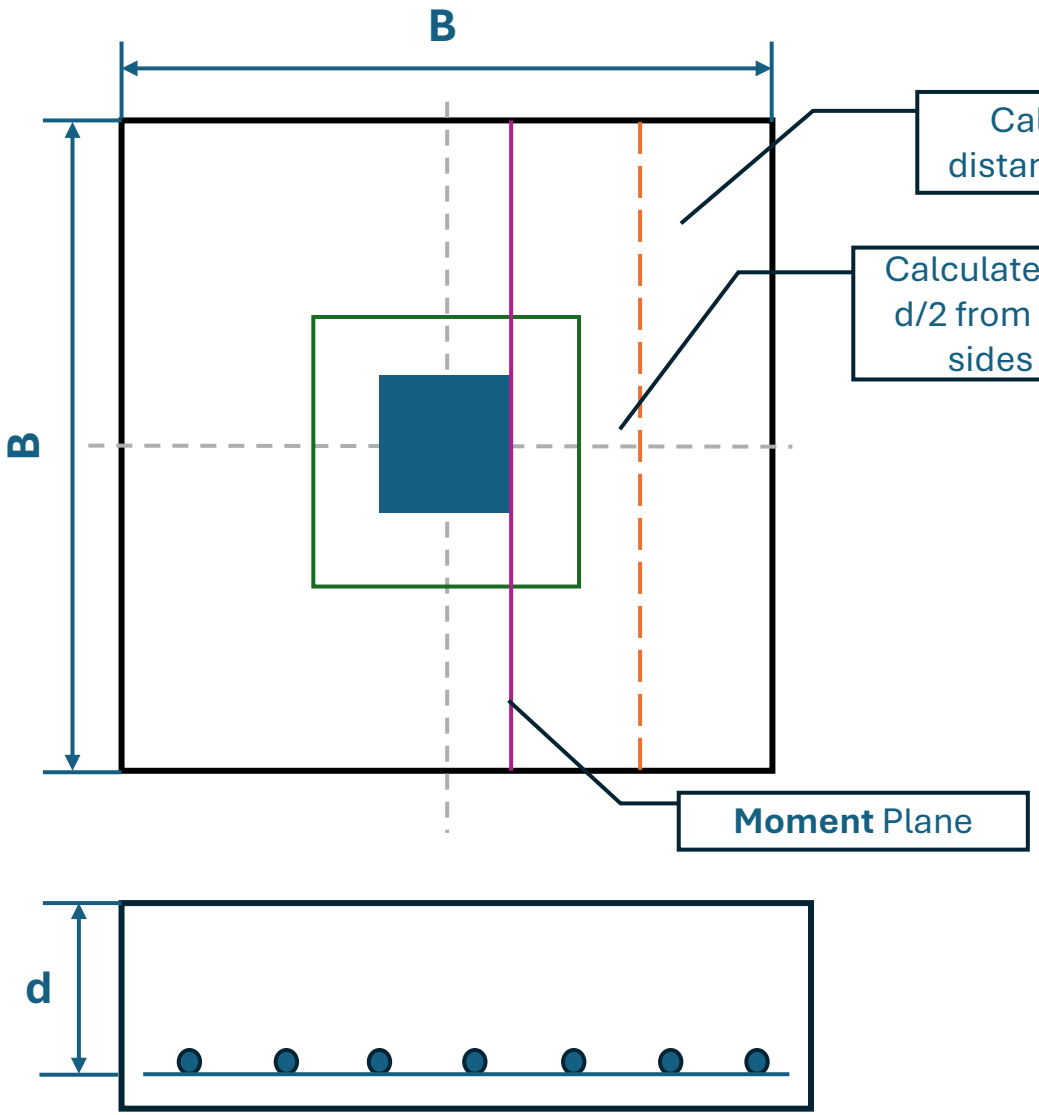


Footing Interaction

- Footings are designed for uniform bearing
- CMCs **DO** create stress concentrations at the bottom of footings. LTP helps to distribute those stresses.
- The point loads themselves don't matter, it's their impact on the footings.
- Load sharing helps! The lower the loads in the CMCs, the smaller the impact from the CMCs.



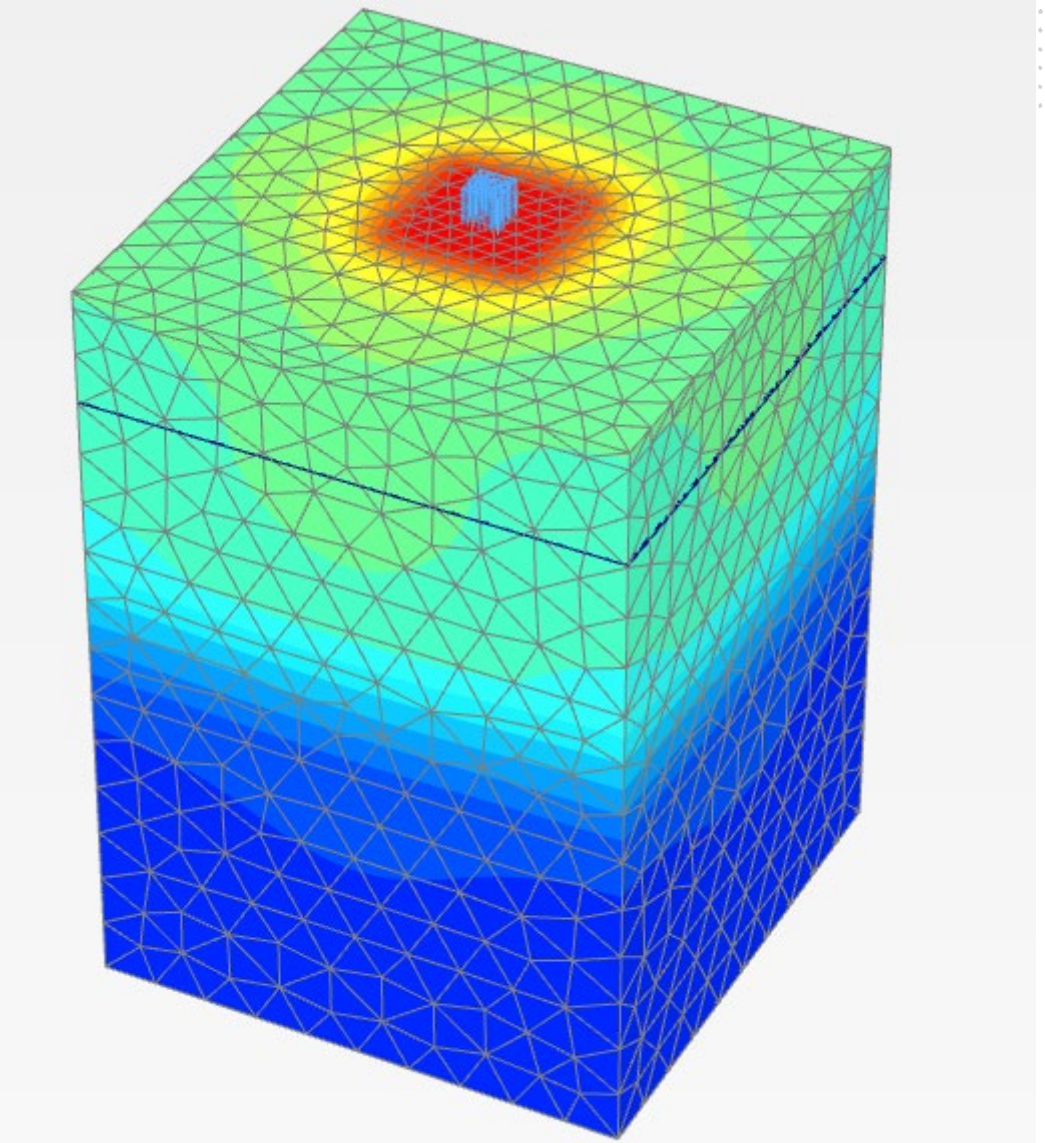
Structural Forces



- Uniform bearing also results in moments and shear forces in the footing.
- Our checks are typically against uniform bearing, but footing capacity may be greater than required.

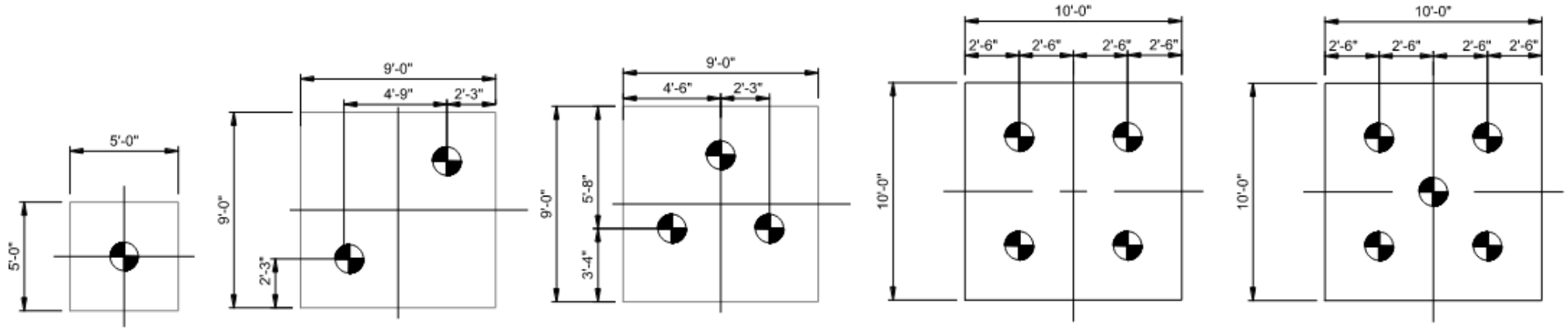
Evaluating the impact

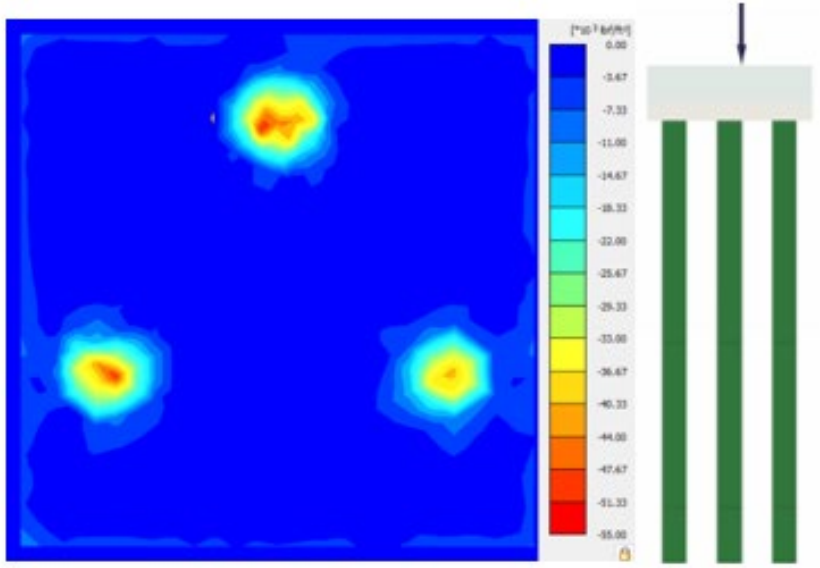
- Modeled RIs supporting a footing in Plaxis (3D model)
- Modeled an equivalent composite case (linear-elastic block of soil with similar deformation characteristics to RI support)
- Used two different soil profiles
- Compared the moment and shear forces in the footing



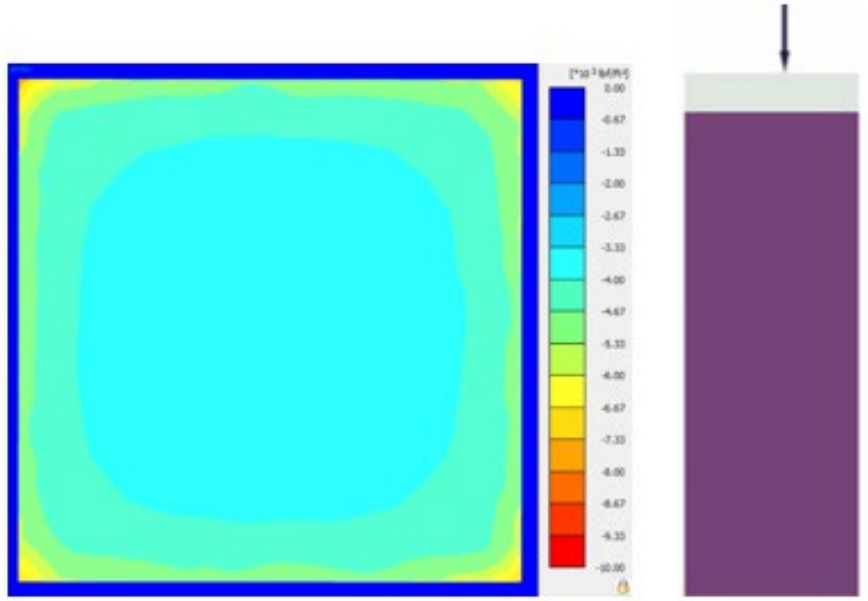
Total displacements u_z (scaled up 20.0 times) (Time 3650 day)

Cases Analyzed



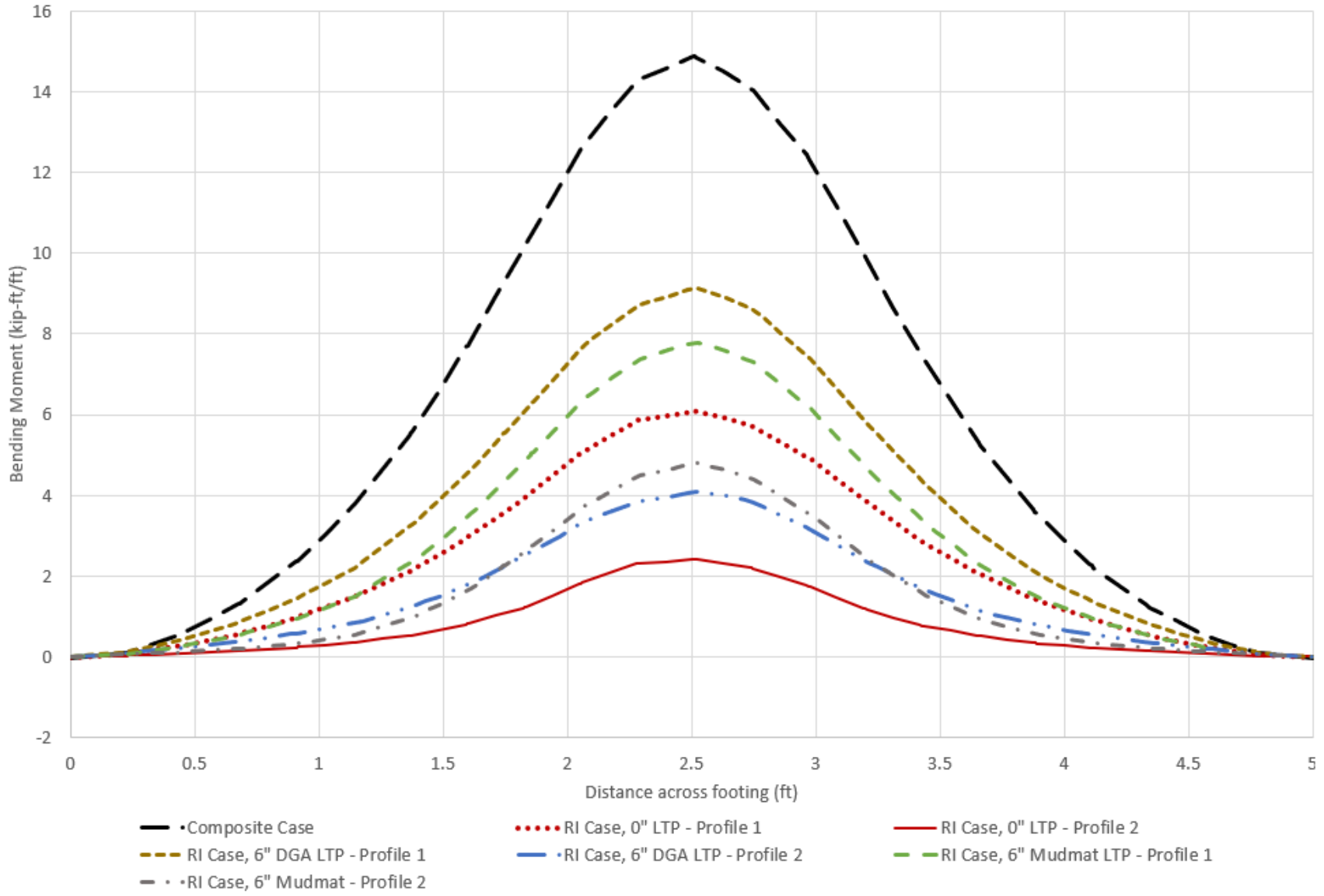


Vertical stress at the bottom of the footing



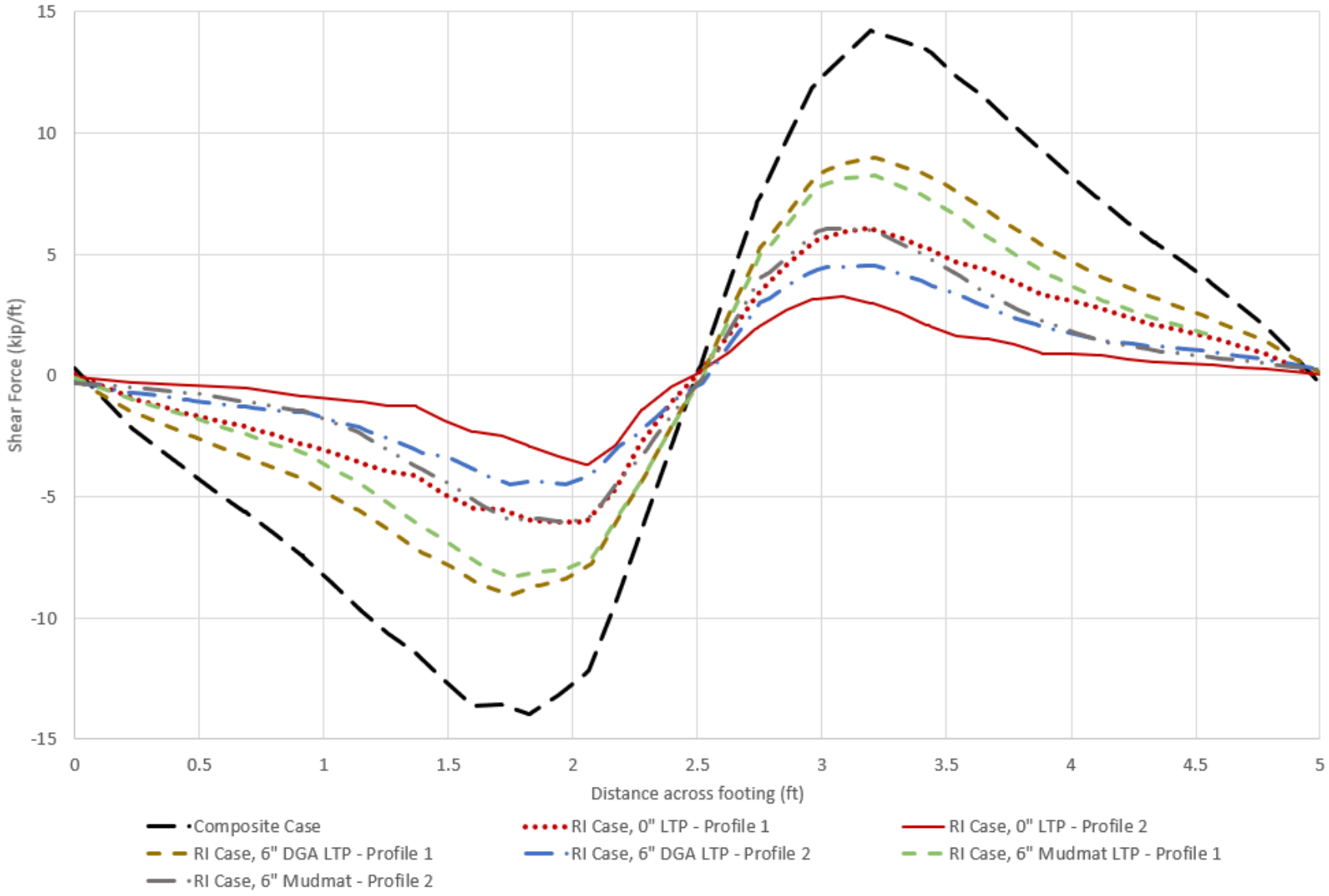
Vertical stress at the bottom of the footing

Moments for 5 x 5 ftg supported by 1 CMC



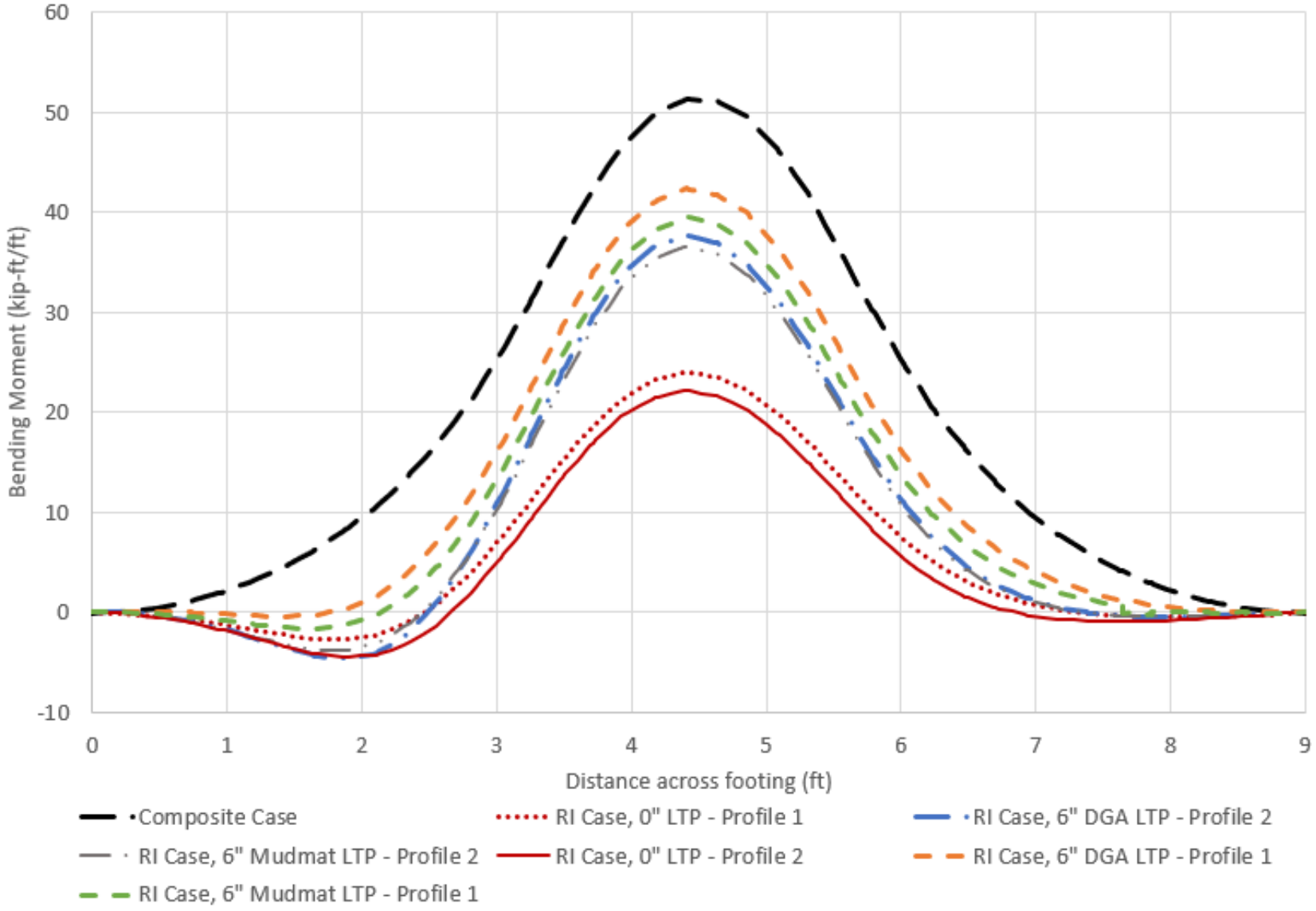
Conversion:
1 ft = 0.3 m
1 kip-ft = 1.4 kN-m

One-way shear for 5 x 5 ftg supported by 1 CMC



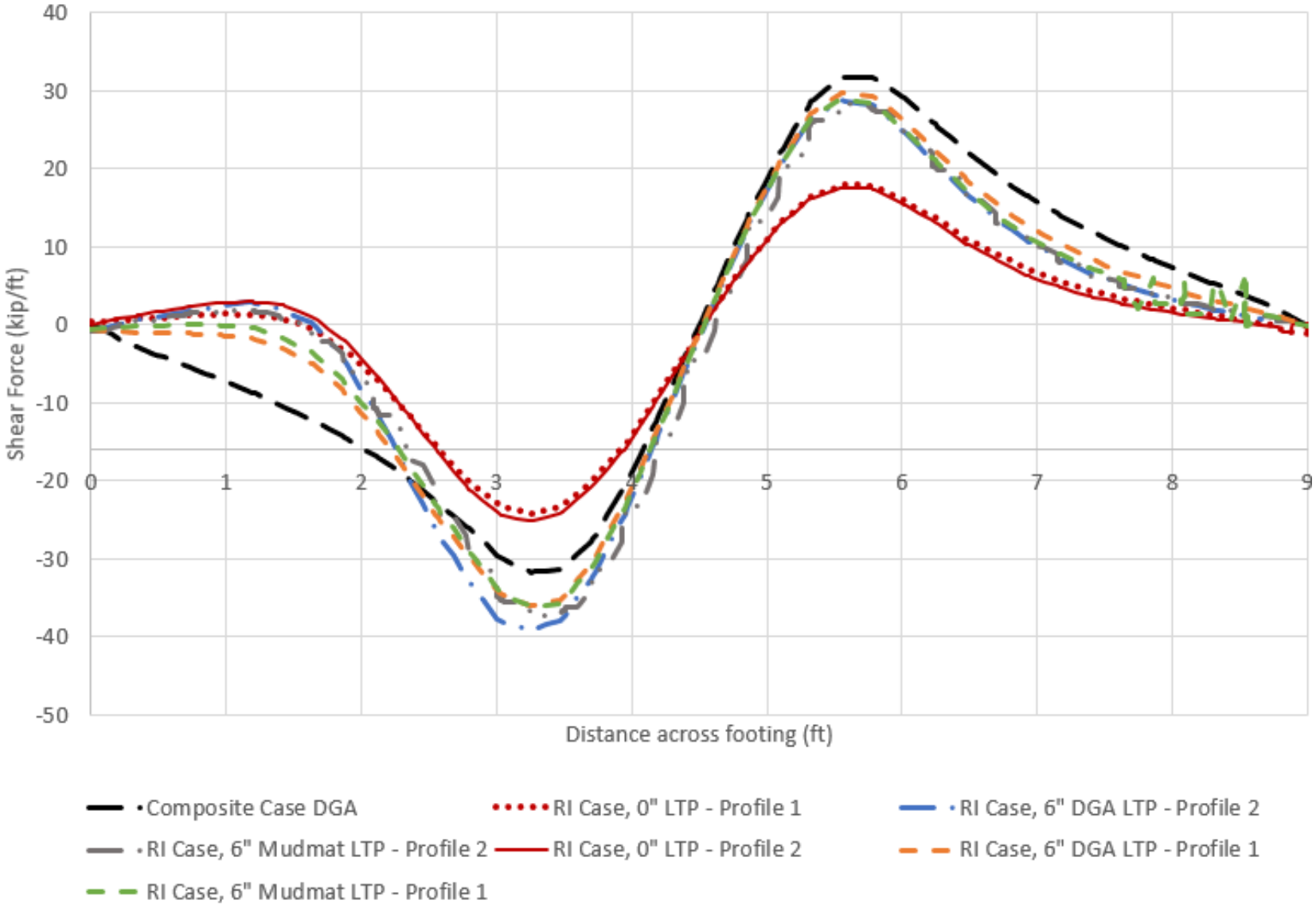
Conversion:
1 ft = 0.3 m
1 kip-ft = 1.4 kN-m

Moments for 9 x 9 ftg supported by 3 CMCs



Conversion:
1 ft = 0.3 m
1 kip-ft = 1.4 kN-m

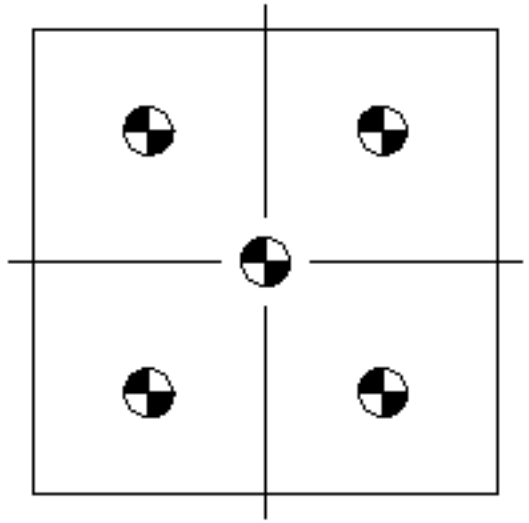
One-way shear for 9 x 9 ftg supported by 3 CMCs



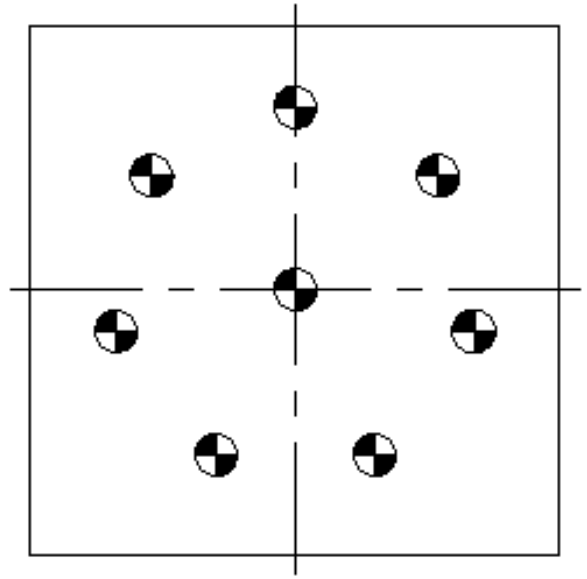
Conversion:
1 ft = 0.3 m
1 kip-ft = 1.4 kN-m

Design Recommendations

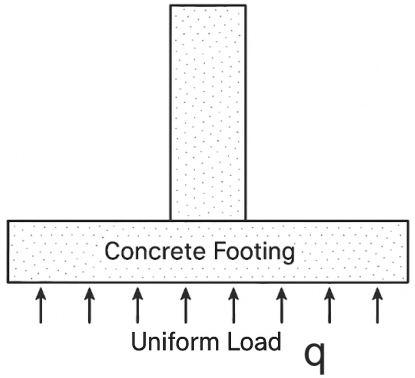
- Use circular patterns whenever possible
- Add a CMC in the center of the footing
- Keep the CMCs within B/4 of the center of the footing



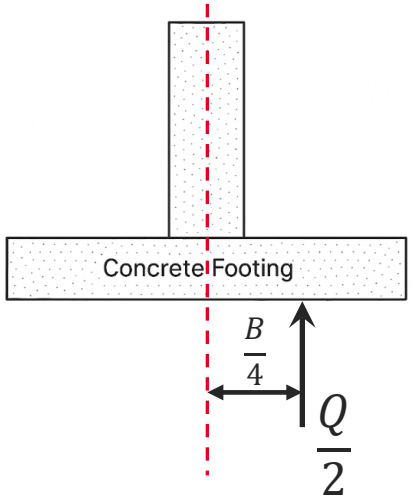
- Design the footing for uniform bearing – impact due to RIs will be less than or equal to moments and forces from uniform bearing.



Behind the Math



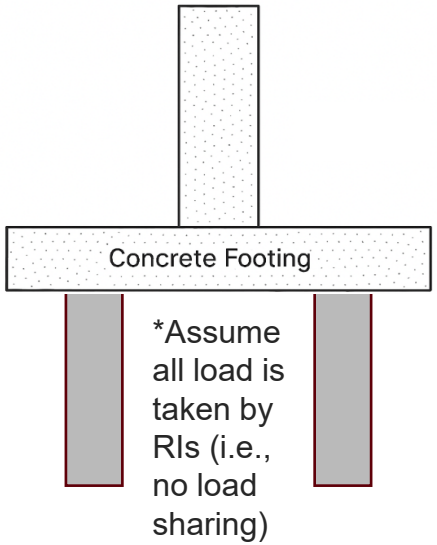
Total ftg load $Q = qB^2$

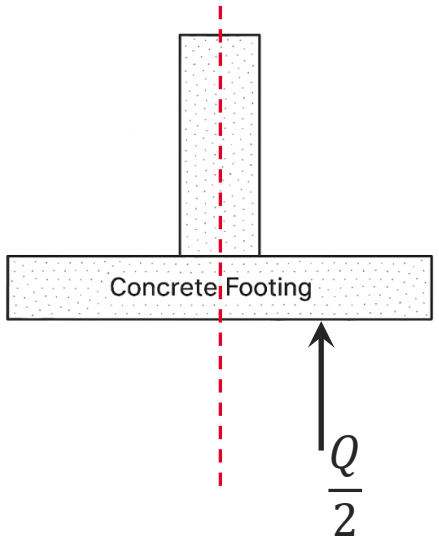


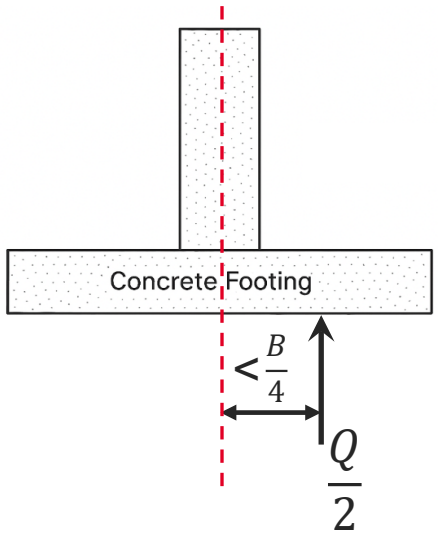
(Resultant of Uniform Load)

Uniform Soil Support

The moment caused by the uniform load is equal to the magnitude of the resultant of the uniform load times the distance from the center of the footing (or allowable distance).







RI Support

The moment caused by the RI is equal to the RI load times the distance from the center of the footing (or allowable distance).

If the distance to the RI is ≤ B/4 from the center of the footing, then the moment generated by the RIs will be ≤ the moment caused by uniform support.

Closing Thoughts



- Carefully consider risk
- Use Ground Improvement Methodology when appropriate
- Interact within project teams – RI designers and structural engineers should be working together
- Relish the unique conditions on each site

Application of Auger Cast-in-Place (ACIP) Piles in Urban Infrastructure: A Case Study of Philadelphia's Samuel S. Baxter Water Treatment Plant

Presented by:

Michael Whelan, P.E., Asst. Manager of Facilities Design (Phila. Water Dept.: Philadelphia, PA)

Abhijit Sheth, P.E., Principal Geotechnical Engineer (GFT, Inc.: Valley Forge, PA)

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October 22, 2025

Ingenuity That Shapes Lives™

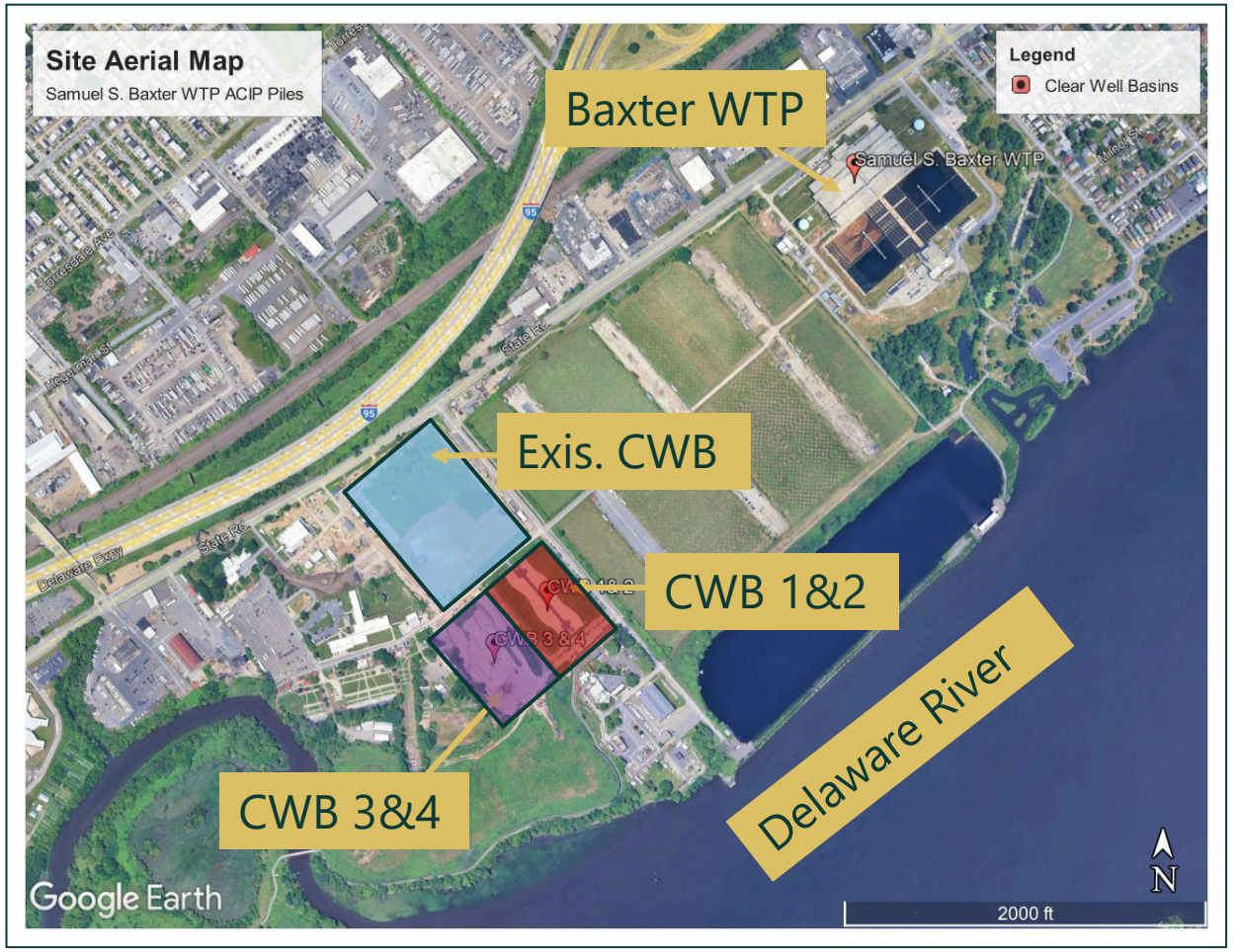
Presentation Overview

- Baxter WTP Plant Background
- Clear Well Basins 3 and 4 Site Overview
- Subsurface Investigation
- Foundation design
- Indicator Piles and Field Load Tests
- Auger Cast-In-Place (ACIP) Pile Installation Criteria
- Construction Challenges and Resolutions
- Pile Installation Summary
- Conclusions
- Construction Progress Photos

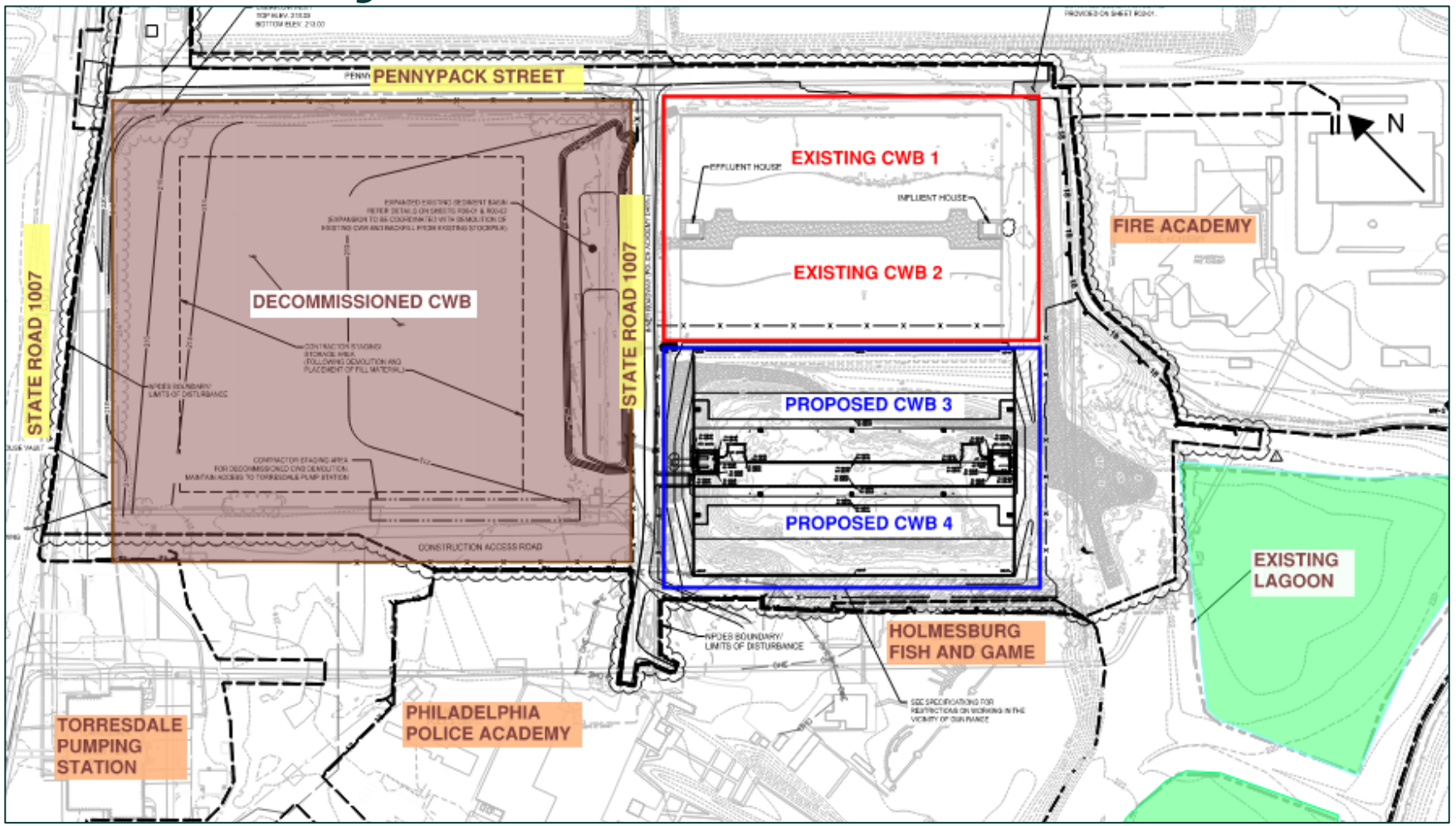
The opinions expressed in this presentation and on the following slides are solely those of the presenters and not necessarily those of DFI. DFI does not guarantee the accuracy or reliability of the information nor endorse the recommendations offered by the presenters.

Site and Baxter WTP Plant Background

- CWB 3 & 4 site located in city of Philadelphia along Delaware River
- Philadelphia Water Department (PWD) has 3 water treatment facilities
  - Baxter Plant provides 60% of city's water and has capacity to supply 100% if other 2 plants go offline to supply a total 226 MGD
- Existing clear well basin(CWB) 120+ yrs old (underground, 20 MGD capacity) needed replacement

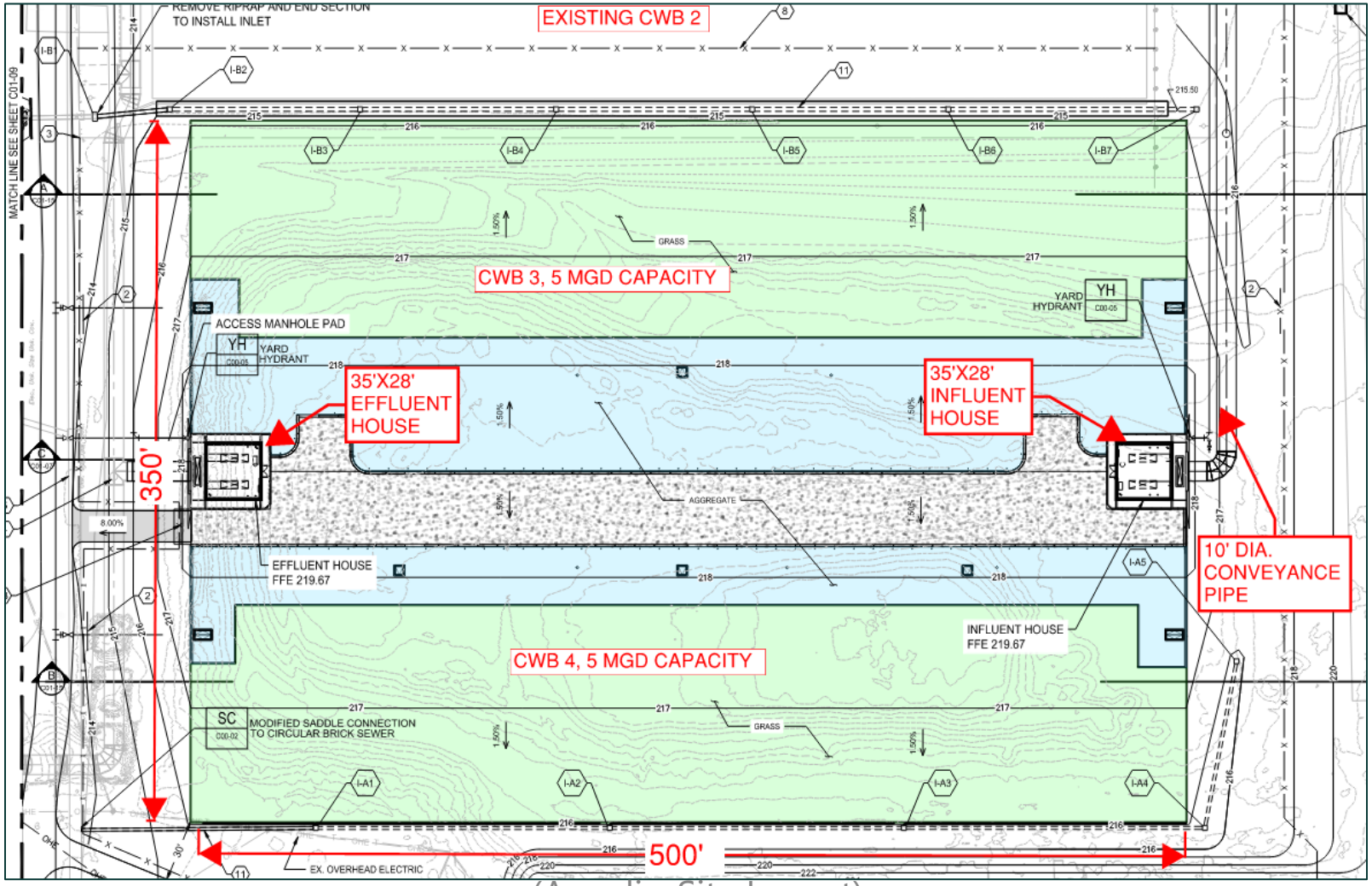


Overall Site Layout



(Arcadis, Site Layout)

Site Layout– CWB 3 & 4



(Arcadis, Site Layout)

- Proposed grade: EL 218
- Basin Ht. ~22'
- Basin mat foundation pressure: 2,500 to 3,000 psf
- Influent/Effluent House mat foundation pressure: 4,500 psf

Construction Sequence – CWB 3 & 4

- Demo existing CWB
- Excavate from EL 264.0 to EL 216 & relocate stockpile
- Install dewatering points
- Excavate from EL 216 to EL 195.0 (mat. EL)
- Install deep foundations
- Construct tank foundation
- Connect to existing pipe from CWB 1 & 2
- Disinfect, test and startup



(DJK, June 2021, Drone shot looking SW)

CWB 3 & 4 Stockpile



(DJK, June 2021, Drone shot looking SW)

Demolition of Existing CWB



(DJK, April 2024, Drone Looking North)



(DJK, April 2024 Looking NW)

Subsurface Investigation

Site EL: ~ 264.0 to 240’



GFT, December 2018, Aerial Drone Survey

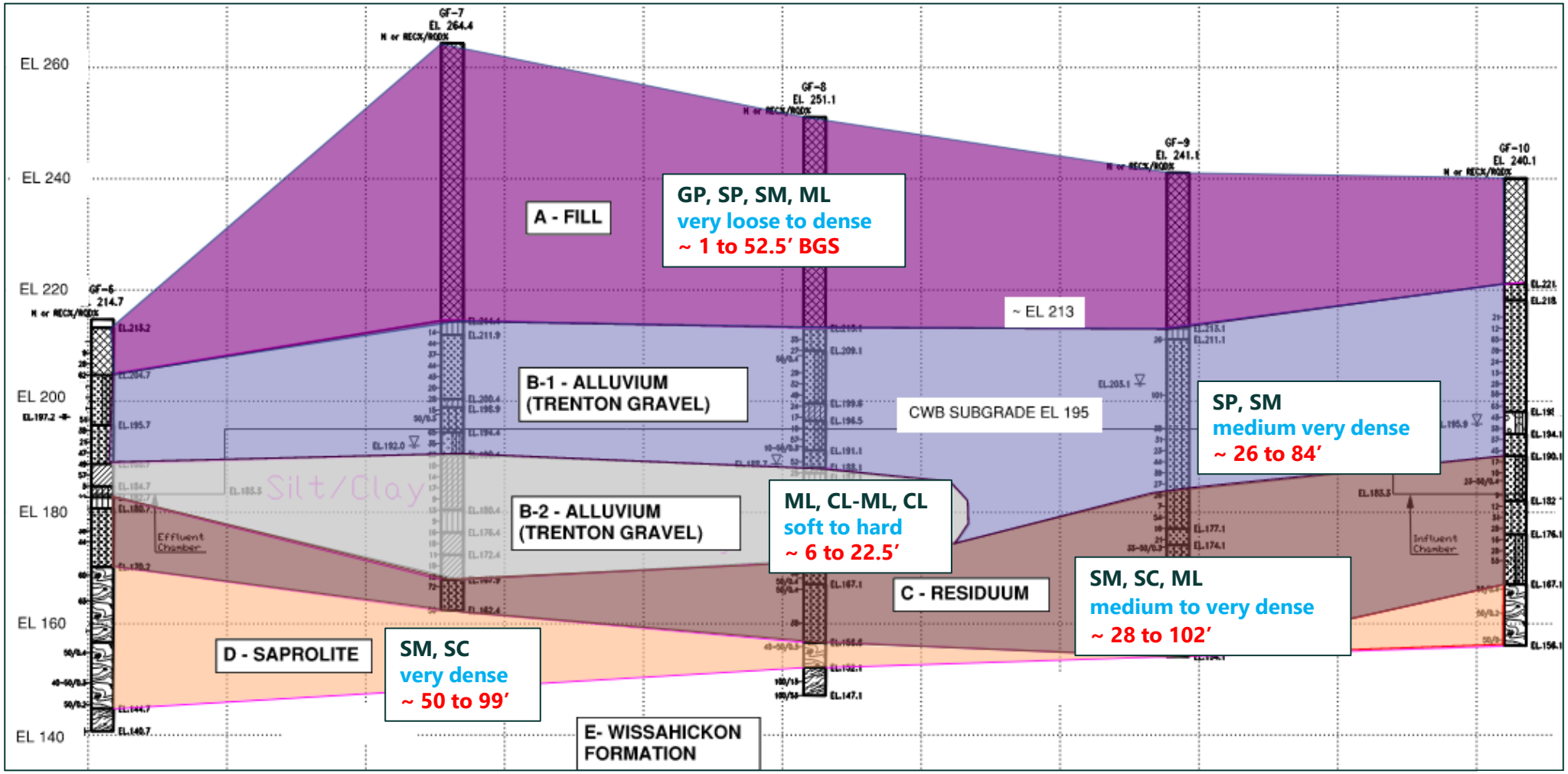
Boring Program Highlights:

- **23 borings and 7 DMTs**
- **Test Boring Depth**: 18’ to 104’ BGS
- **Auger Refusal Depth**: 28’ to 100’ (El. 191 to El. 107)
- **Groundwater EL**: El. 203 to El. 195

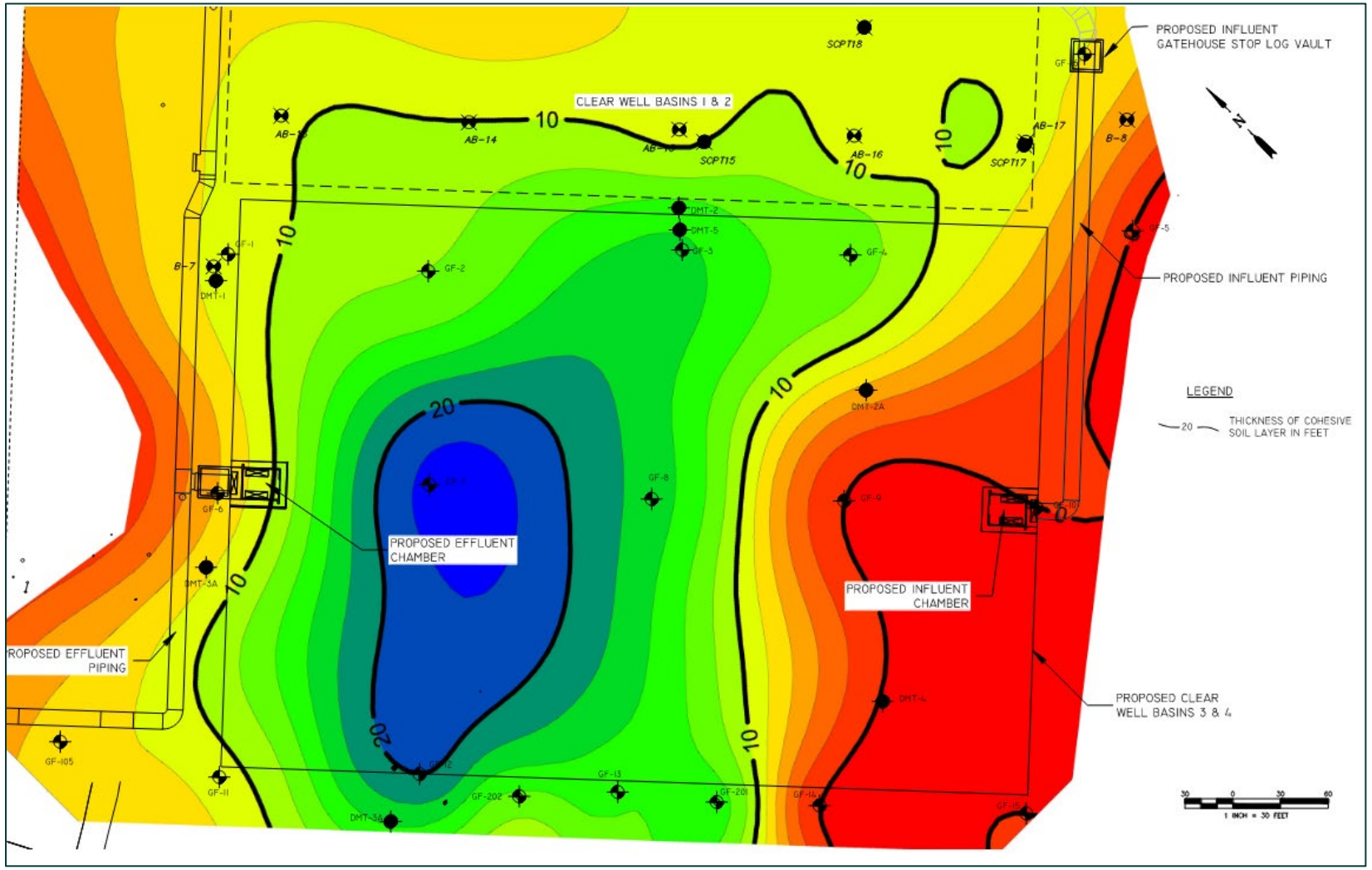
Site Geology:

- Trenton Gravel Alluvium
- Wissahickon Formation (varying weathering profile)

Typical Subsurface Profile



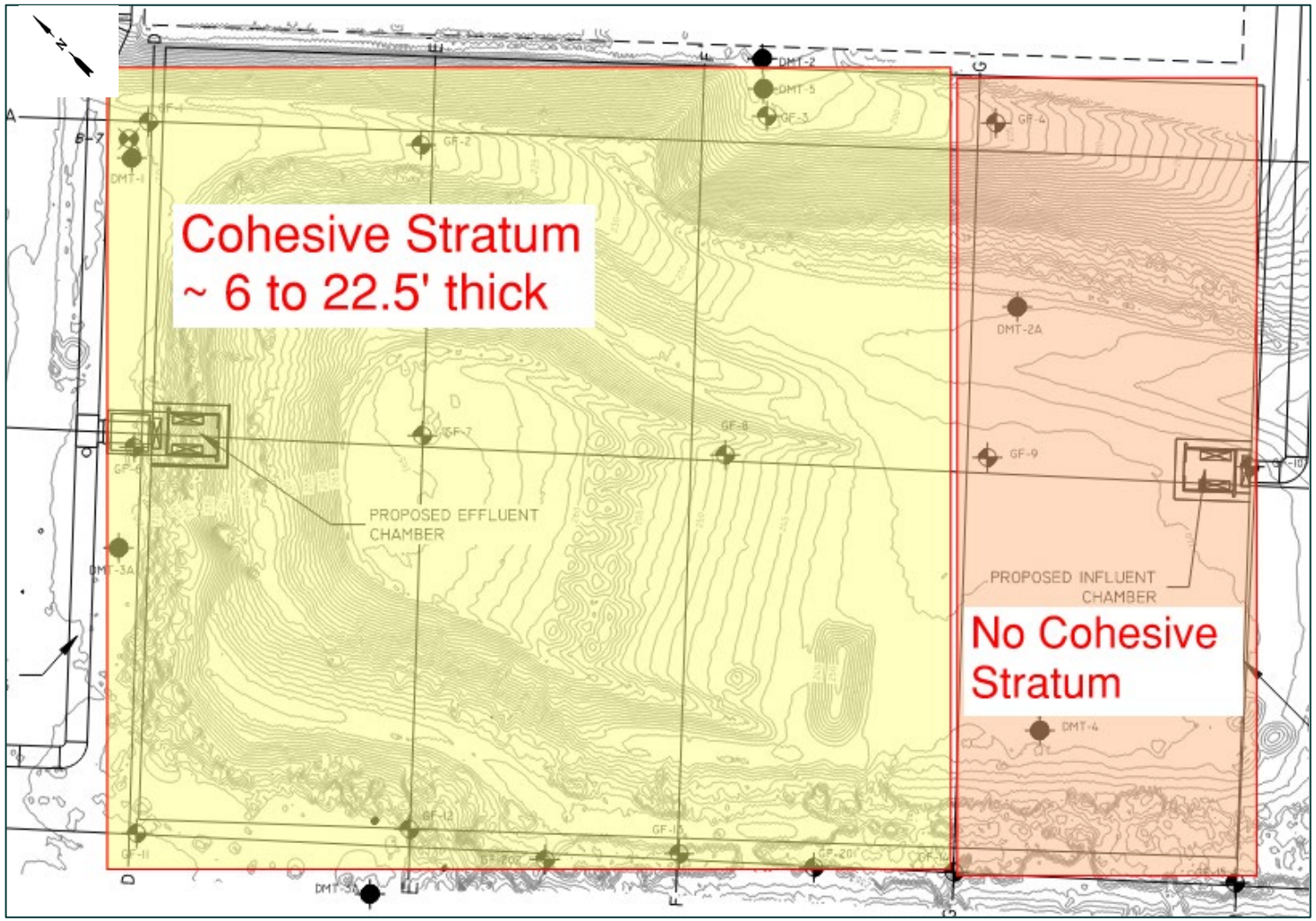
Cohesive Stratum - Contour Map



Cohesive Stratum:

- **Critical aspect of project**
- Encountered in 13 out of 18 borings in the vicinity of tank
- Thickness: ~ 6’ to 22.5’
- PP: ~ 0.5 tsf to 4 tsf

Cohesive Stratum



(Site Layout - Cohesive Stratum Coverage)

Foundation Design - Constraints

- **Variable subsurface conditions:** varying depth to bedrock, weathering profile of saprolite, scattered cobbles/boulders
- **Cohesive Trenton Gravel:** encountered in 13 borings, variable thickness (6–22.5 ft), strength 0.5–4.0 tsf,
- **Shallow groundwater (~El. 205 > BFE El. 195):** uplift pressures; dewatering required
- **Adjacent Structures:** Proximity to CWB 1 & 2 foundations; vibration and dewatering impacts
- **Site logistics:** staging of multiple construction activities
- **Strict QA/QC requirements :** for specialty foundations
- **Large volume of soil cuttings & spoils management**

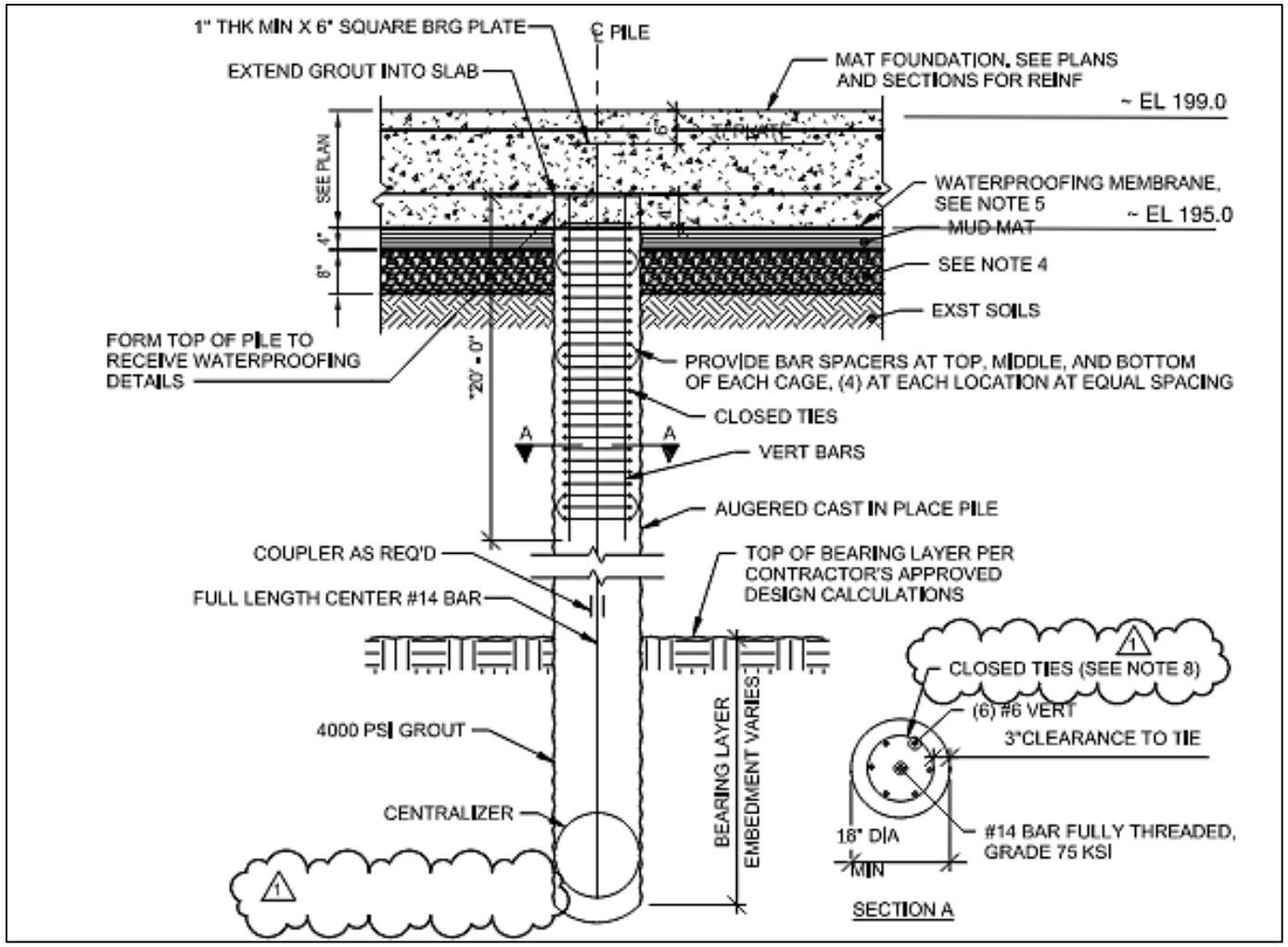
Foundation Design: Alternatives Analysis

- **Mat Foundation on Alluvial Soils:** Possible in some areas but in other areas ruled out due to predicted settlement of 4.8" within the cohesive stratum. Overexcavation and replacement would be cost prohibitive.
- **Mat foundation w/ground improvement:**
  - **Aggregate piers:** Feasible but ruled out because of vibration risk to CWB 1 & 2 foundations plus need of tiedown anchors to resist uplift forces
  - **Driven piles:** Ruled out because of extensive vibrations
  - **Drilled shafts:** Feasible but costly; would've required temporary casing and rock sockets to handle uplift forces
  - **Auger cast-in-place (ACIP) piles**: Feasible; low vibrations, high load capacity

Foundation Design: ACIP Piles

- **ACIP piles were chosen because of offered several advantages**:
  - Minimal construction vibrations and noise impact
  - Provide high load-bearing capacity under the tank's dynamic loads
  - Provide uplift resistance w/o rock socket
  - Reduced spoil generation
  - Adaptable to challenging subsurface conditions
  - Rapid installation
  - Enhanced durability in high groundwater table environment

Contract ACIP Pile Design

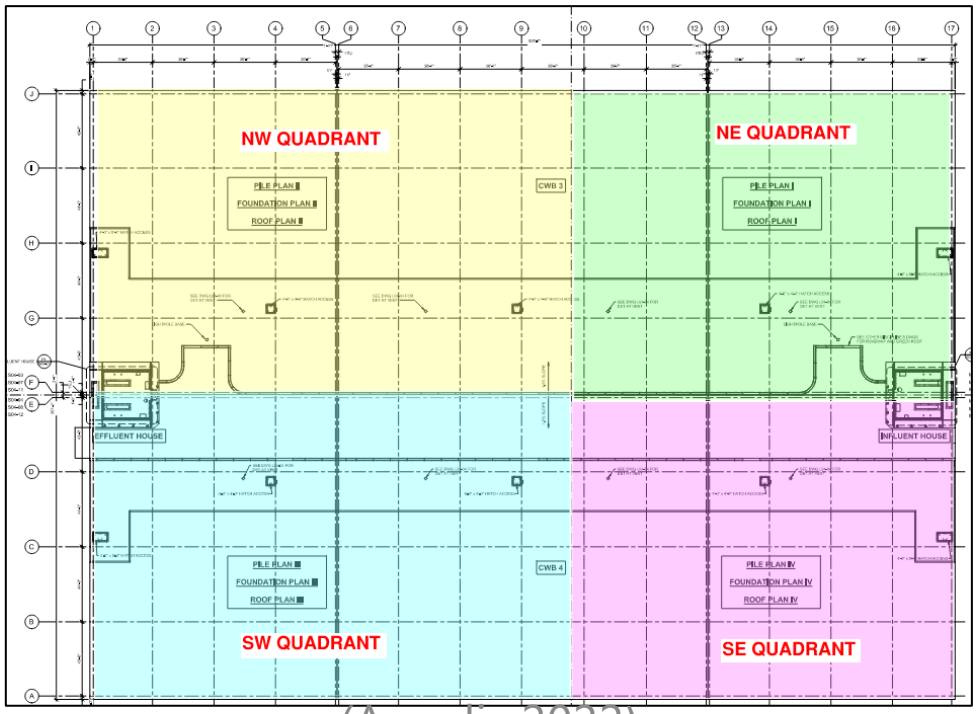


(GFT/Arcadis 2022)

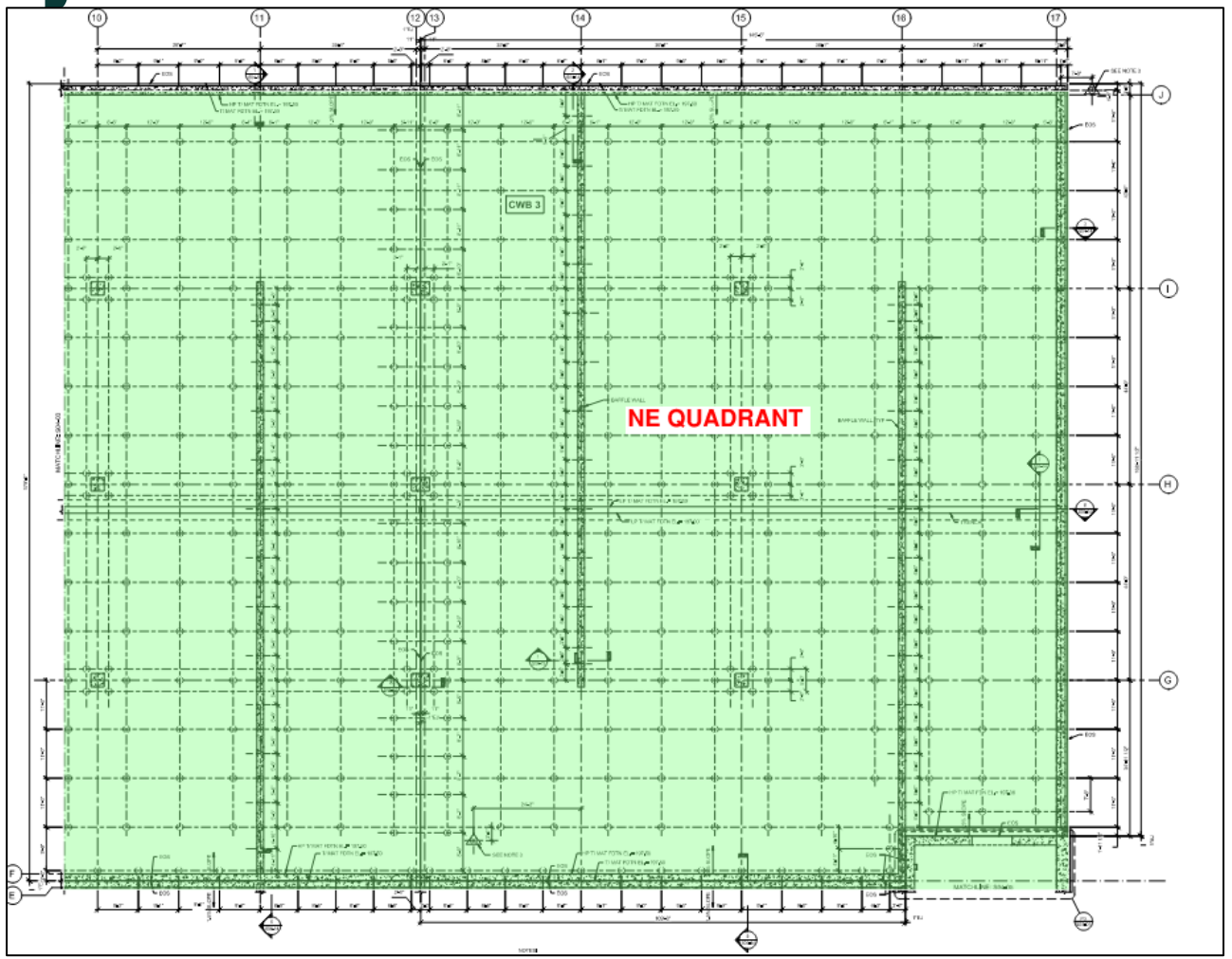
GFT & Arcadis drafted prelim. ACIP pile design

- Mat foundation EL 195.0
- 18″ pile diameter (socketed 20′-30′ into residuum/saprolite, ~30-60′ below BFE)
- Pile center-to-center spacing: 12′
- No. 14 (75 ksi) full-length bar
- Min. 20′ rebar cage with (6) #6 bars and #3 spiral ties
- Max. allowable compressive load: 100 tons (200 kips)
- Uplift capacity: 45 tons (90 kips); Lateral capacity: 9.5 tons(19 kips)

Contract ACIP Pile Layout



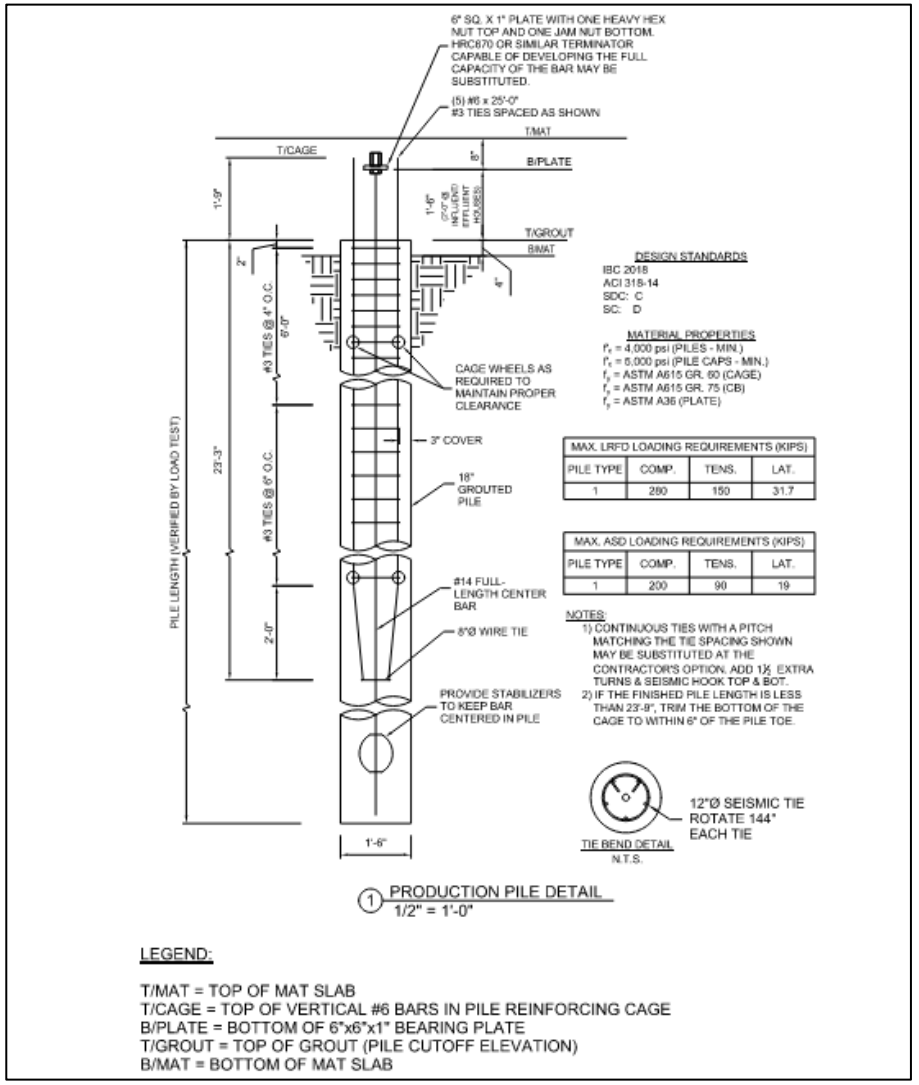
(Arcadis 2022)



(Arcadis 2022)

Finalized ACIP Pile Design

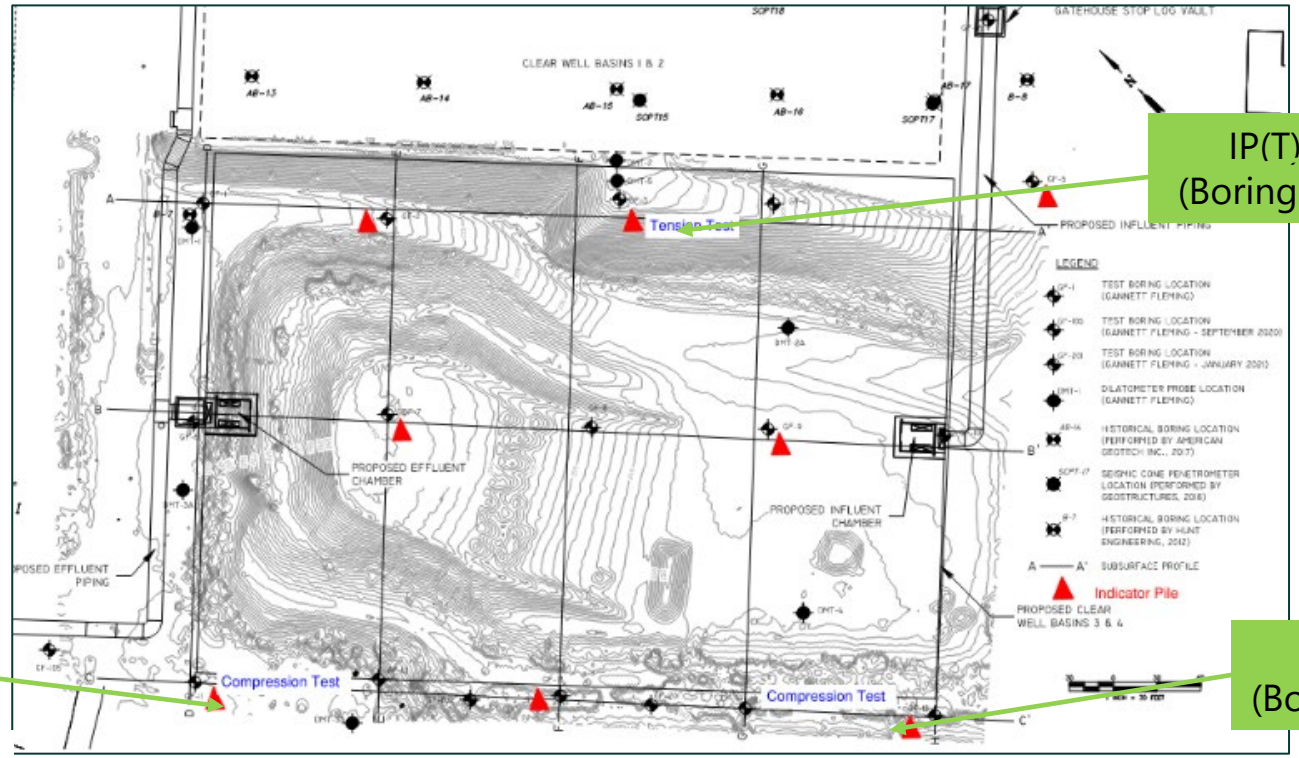
- Contract required delegated design for ACIP piles
- Berkel was awarded pile installation
- Final ACIP pile configuration:
  - 18” pile diameter, 4,000 psi grout
  - No. 14 (75 ksi) full-length centering bar
  - 23’-3” cage with (5) #6 bars and spiral #3 ties
  - 2,098 piles, totaling est. 96,000’
- GFT provided full-time inspection of ACIP pile installation



(Berkel 2024)

Indicator Piles & Field Load Tests

- Contract required 8 indicator piles to evaluate/refine procedures
- 3 of the 8 indicator piles load-tested (2 compressive, 1 tensile)
  - Loaded to 250% of design load; Instrumented w/5 ε gauges



Test Pile Setup Photos

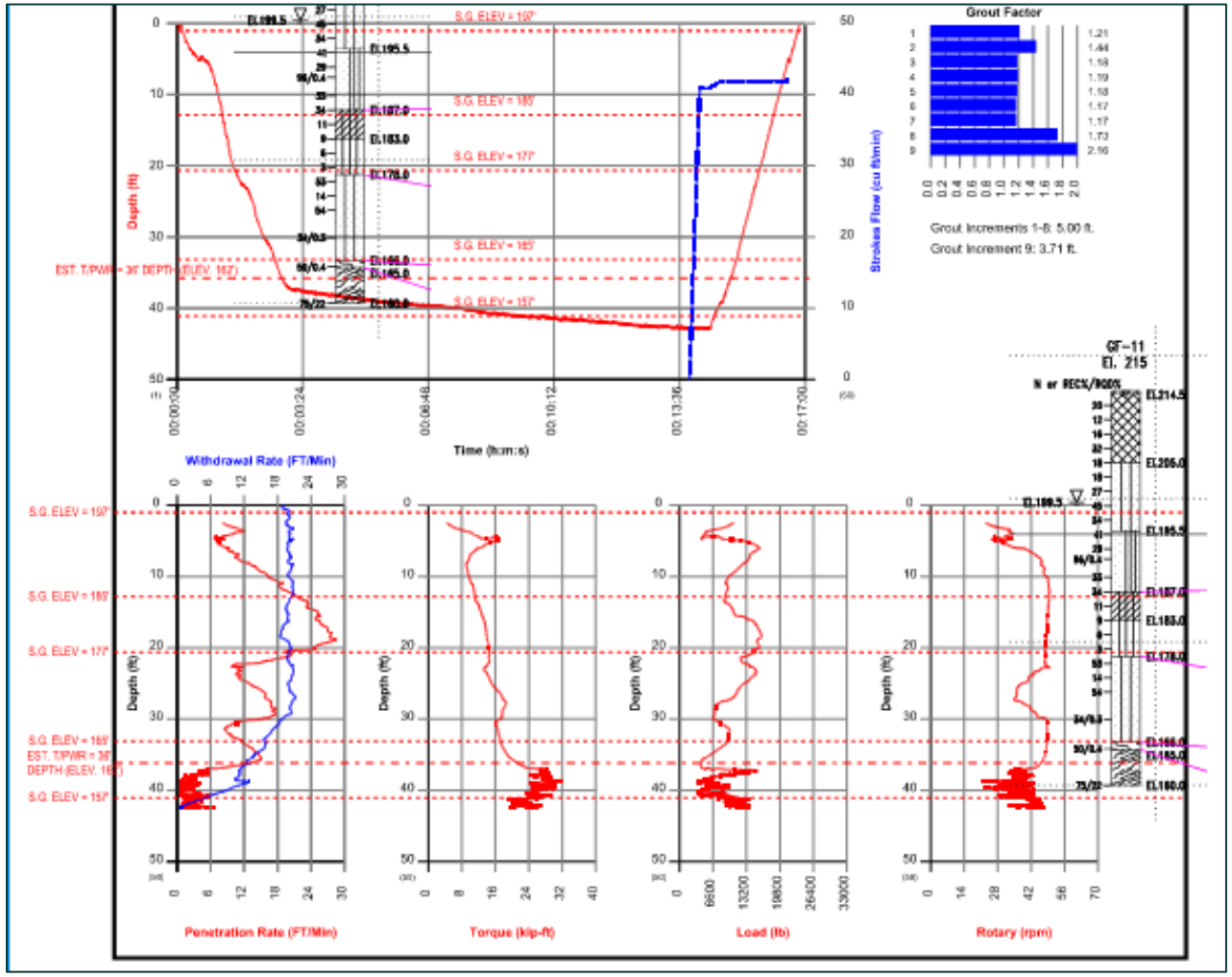


(Indicator Pile Drill Setup, November 2024)



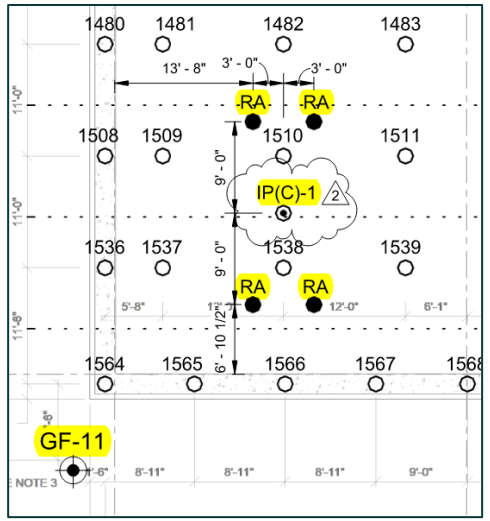
(Load Test Setup, November 2024)

Field Load Test – IP(C)-1A AME Log

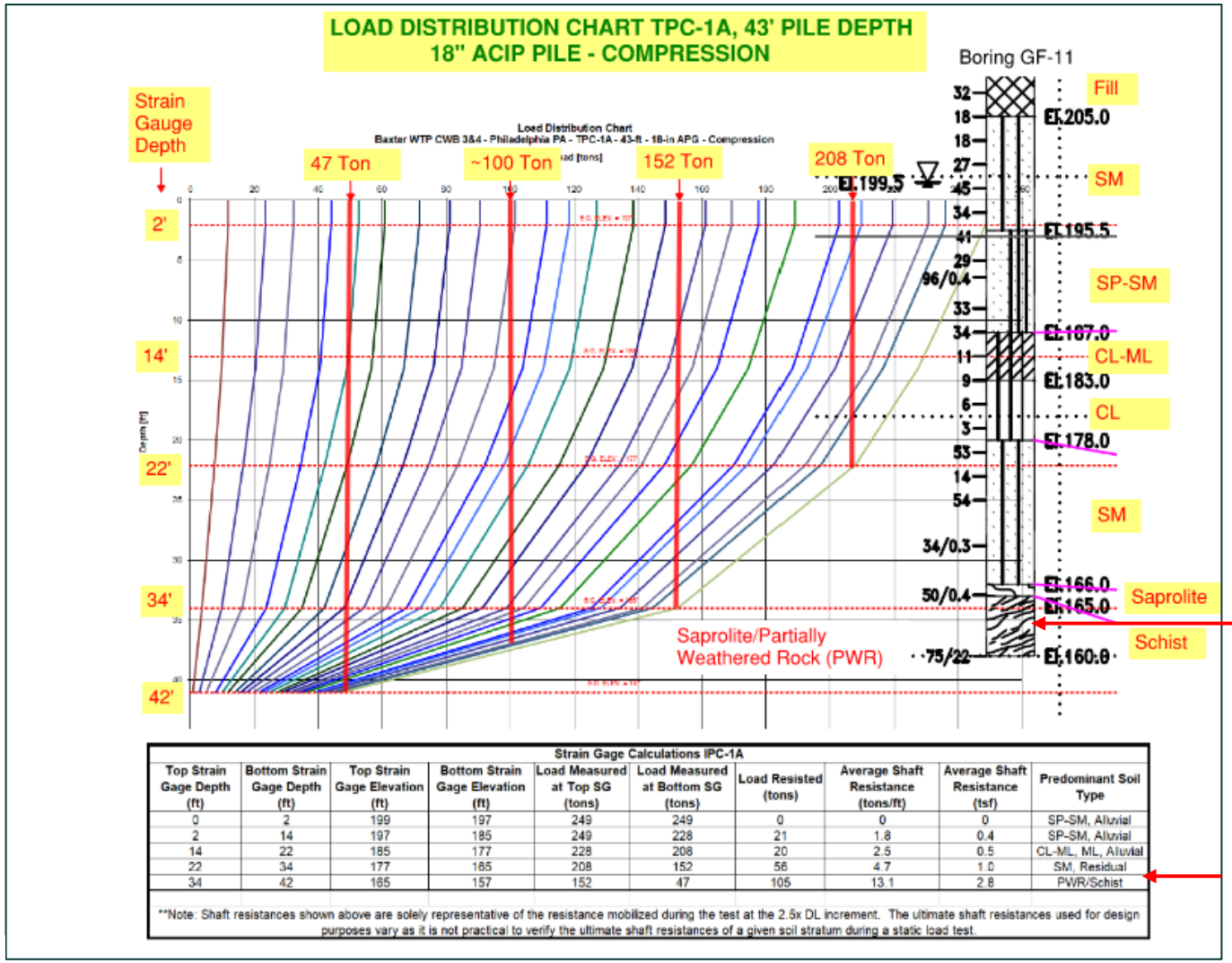


(Berkel 2024)

- Nearby Boring – SW Quadrant, GF-11
- Estimated PWR EL in GF-11 – **EL 165.0**
- Test Pile drilled to 43′ (EL 155.0)
- Estimated PWR EL per AME - **EL 162.0**
- Total embedment in PWR: ~7′
- Penetration Rate in PWR: ~ 6 Ft/Min



Field Load Test Results – IP(C)-1A



Strain Gage Calculations IPC-1A

Top Strain Gage Depth (ft)	Bottom Strain Gage Depth (ft)	Top Strain Gage Elevation (ft)	Bottom Strain Gage Elevation (ft)	Load Measured at Top SG (tons)	Load Measured at Bottom SG (tons)	Load Resisted (tons)	Average Shaft Resistance (tons/ft)	Average Shaft Resistance (tsf)	Predominant Soil Type
0	2	199	197	249	249	0	0	0	SP-SM, Alluvial
2	14	197	185	249	228	21	1.8	0.4	SP-SM, Alluvial
14	22	185	177	228	208	20	2.5	0.5	CL-ML, ML, Alluvial
22	34	177	165	208	152	56	4.7	1.0	SM, Residual
34	42	165	157	152	47	105	13.1	2.8	PWR/Schist

**Note: Shaft resistances shown above are solely representative of the resistance mobilized during the test at the 2.5x DL increment. The ultimate shaft resistances used for design purposes vary as it is not practical to verify the ultimate shaft resistances of a given soil stratum during a static load test.

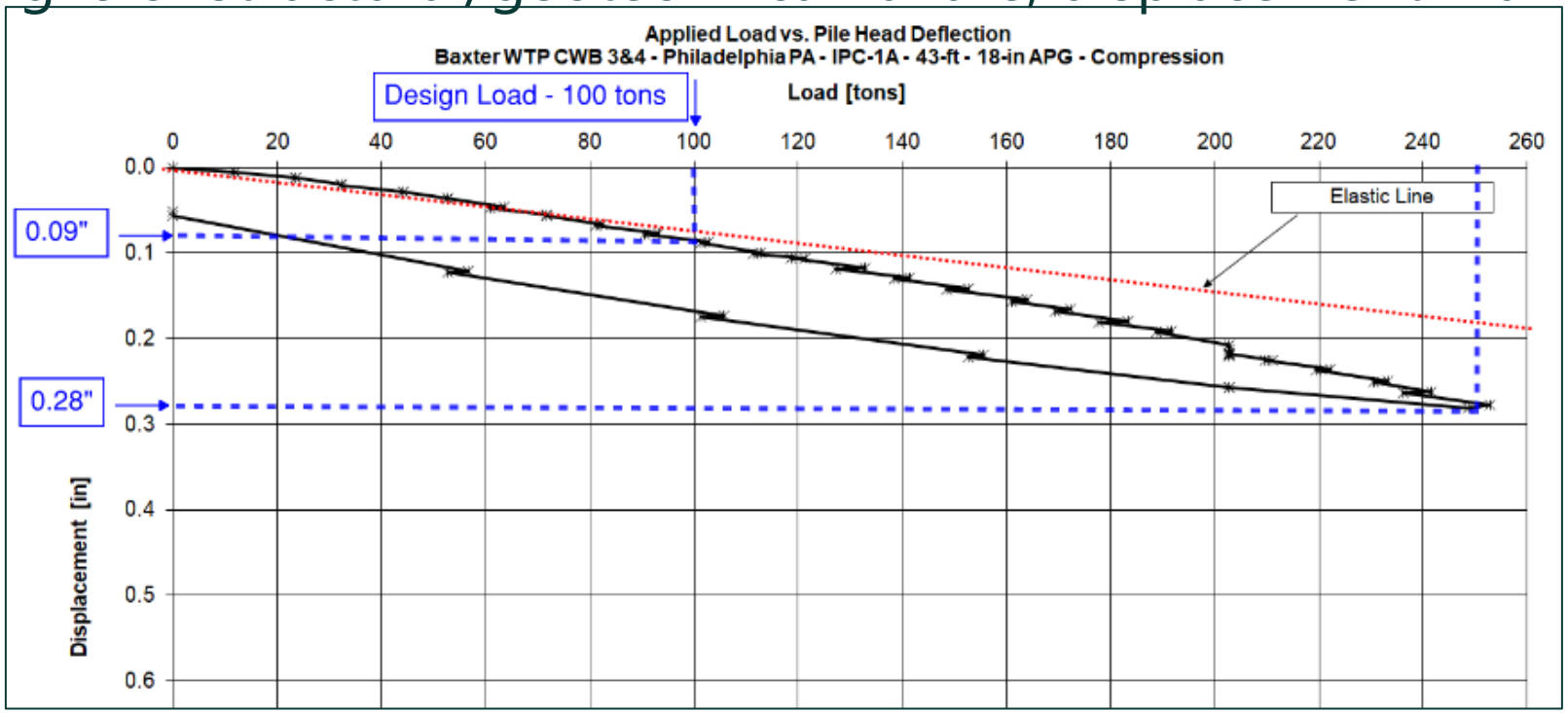
Pile embedment into Saprolite/PWR, ~7'

Most resistance from Saprolite/PWR, ~105 T

(Berkel 2024)

Field Load Test Results – IP(C)-1A

- Compressive test pile – drilled to 43’ (SW Quad., Boring GF-11)
- Displacement: 0.09” @ 100 tons (design load), 0.28” @ 250 tons
  - No signs of structural/geotechnical failure; displacement mainly elastic



(Berkel 2024)

Pile Installation Criteria

- Variable depths to saprolite = need for nuanced termination criteria
- Criteria based on auger refusal (≤ 1′/min.) and partially weathered rock (PWR):
  - 22′-30′ long: Refusal and 5′ PWR
  - 30′-35′ long: Refusal and 3′ PWR
  - ≥ 35′ long – One of following: 1) Refusal, 2) 15′ PWR, 3) Pile length of 65′
- Depth criteria modified for influent/effluent houses (different BFE)



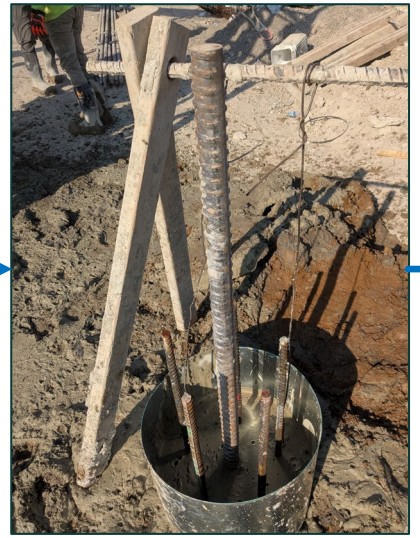
Pile Installation













Pile Installation

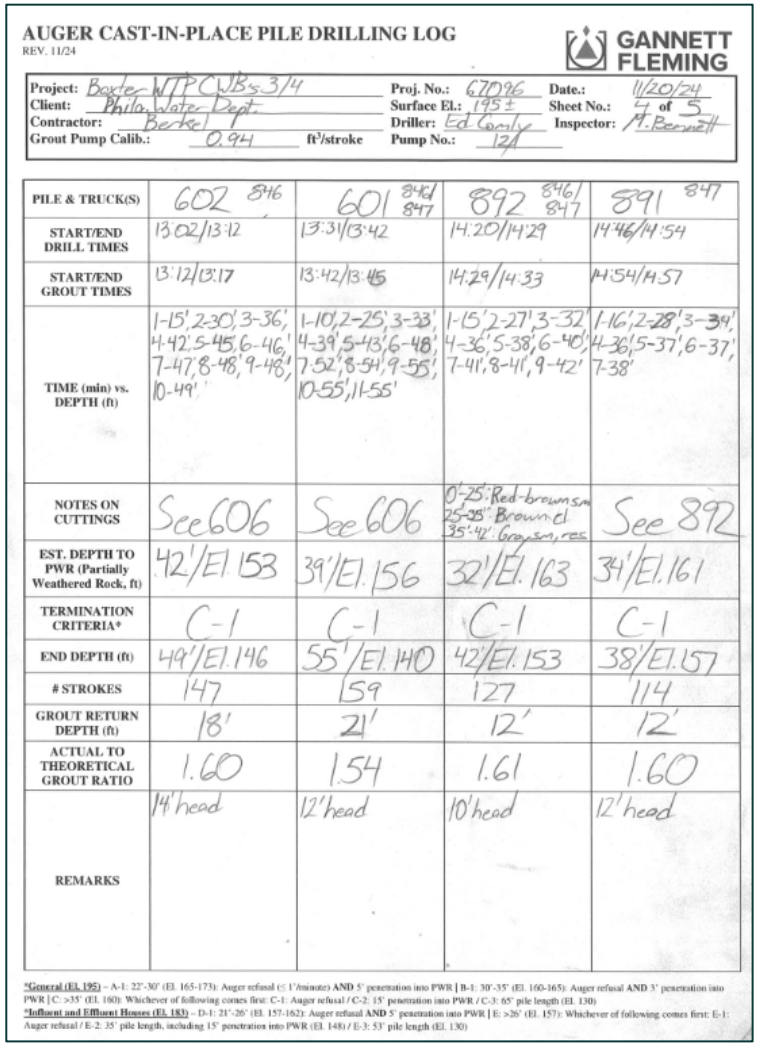


(GFT, Pile Installation, Jan 2025,)



(GFT, Mud Mat Installation, Feb 2025)

Field Logs vs. AME Log



AUGER CAST-IN-PLACE PILE DRILLING LOG

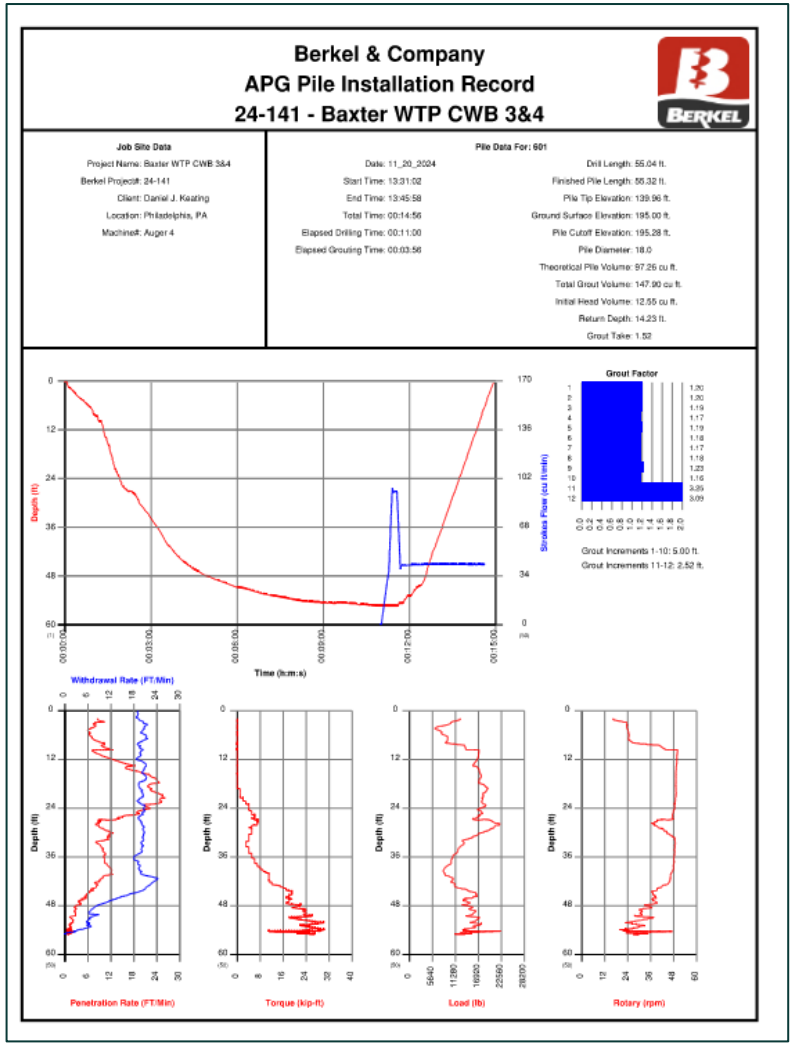
REV. 11/24

GANNETT FLEMING

Project: Baxter WTP CWBs 3/4 | Proj. No.: 67096 | Date: 11/20/24
Client: Philadelphia Water Dept. | Surface El.: 195± | Sheet No.: 4 of 5
Contractor: Berkel | Driller: Ed Comly | Inspector: M. Bennett
Grout Pump Calib.: 0.94 ft³/stroke | Pump No.: 121

PILE & TRUCK(S)	602 846	601 846/847	892 846/847	891 847
START/END DRILL TIMES	13:02/13:12	13:31/13:42	14:20/14:29	14:46/14:54
START/END GROUT TIMES	13:12/13:17	13:42/13:45	14:29/14:33	14:54/14:57
TIME (min) vs. DEPTH (ft)	1-15', 2-30', 3-36', 4-42', 5-45', 6-46', 7-47', 8-48', 9-48', 10-49'	1-10', 2-25', 3-33', 4-39', 5-43', 6-48', 7-52', 8-54', 9-55', 10-55', 11-55'	1-15', 2-27', 3-32', 4-36', 5-38', 6-40', 7-41', 8-41', 9-42'	1-16', 2-28', 3-34', 4-36', 5-37', 6-37', 7-38'
NOTES ON CUTTINGS	See 606	See 606	0'-25': Red-brown sm 25'-35': Brown cl 35'-42': Gray sm, res	See 892
EST. DEPTH TO PWR (Partially Weathered Rock, ft)	42'/El. 153	39'/El. 156	32'/El. 163	34'/El. 161
TERMINATION CRITERIA*	C-1	C-1	C-1	C-1
END DEPTH (ft)	49'/El. 146	55'/El. 140	42'/El. 153	38'/El. 157
# STROKES	147	159	127	114
GROUT RETURN DEPTH (ft)	18'	21'	12'	12'
ACTUAL TO THEORETICAL GROUT RATIO	1.60	1.54	1.61	1.60
REMARKS	14' head	12' head	10' head	12' head

*General (El. 195) – A-1: 22'-30' (El. 165-173): Auger refusal (≤ 1'/minute) AND 5' penetration into PWR | B-1: 30'-35' (El. 160-165): Auger refusal AND 3' penetration into PWR | C: >35' (El. 160): Whichever of following comes first: C-1: Auger refusal / C-2: 15' penetration into PWR / C-3: 65' pile length (El. 130)
*Influent and Effluent Houses (El. 183) – D-1: 21'-26' (El. 157-162): Auger refusal AND 5' penetration into PWR | E: >26' (El. 157): Whichever of following comes first: E-1: Auger refusal / E-2: 35' pile length, including 15' penetration into PWR (El. 148) / E-3: 53' pile length (El. 130)



Berkel & Company
APG Pile Installation Record
24-141 - Baxter WTP CWB 3&4

Job Site Data
Project Name: Baxter WTP CWB 3&4
Berkel Project#: 24-141
Client: Daniel J. Keating
Location: Philadelphia, PA
Machine#: Auger 4

Pile Data For: 601
Date: 11_20_2024
Start Time: 13:31:02
End Time: 13:45:58
Total Time: 00:14:56
Elapsed Drilling Time: 00:11:00
Elapsed Grouting Time: 00:03:56

Drill Length: 55.04 ft.
Finished Pile Length: 55.32 ft.
Pile Tip Elevation: 139.96 ft.
Ground Surface Elevation: 195.00 ft.
Pile Cutoff Elevation: 195.28 ft.
Pile Diameter: 18.0
Theoretical Pile Volume: 97.26 cu ft.
Total Grout Volume: 147.90 cu ft.
Initial Head Volume: 12.55 cu ft.
Return Depth: 14.23 ft.
Grout Take: 1.52

Grout Factor
Grout Increments 1-10: 5.00 ft.
Grout Increments 11-12: 2.52 ft.

Depth (ft)
Stroke Flow (cu ft/min)
Time (h:m:s)
Withdrawal Rate (FT/Min)
Penetration Rate (FT/Min)
Torque (kip-ft)
Load (lb)
Rotary (rpm)

Construction Challenges and Solutions

Like any construction project, team had to navigate some initial challenges related to ACIP pile installation

1) Discrepancy in PWR elevations between field logs and AME logs.
   - Auger Penetration rate – 6′/min. to start, adjusted based on encountered stratigraphy and surrounding piles
   - Appearance of cuttings from rig (i.e. Rock dust from auger)
   - Rig chatter during pile drilling

2) Grout cover over rebar cage: Contract requirement min. 3″ ± 0.5″
   - Redistribute the shear demand to remaining piles in the mat
   - Implementation of specialty centering devices – 5″ vs. 3″ dia. spacers
   - Strengthened QA/QC safeguards – dedicated QA/QC team on site

Construction Challenges and Solutions (Ctd.)

3) Bent central bar/rebar cage
   - Rebars bent < 30 deg. – manually bent back into position
   - Rebars bent > 30 deg. - new section installed via a mechanical coupler

4) Effluent and Influent house piles
   - 9’ grade differential, piles needed to be exposed and chipped down
   - Constraints – high groundwater table and concurrent const. activities

Bent Bar vs. Bars fixed with Mechanical Coupler





Effluent House Piles



(Influent House Exposeed ACIP Piles, December 2024)



(Influent House ACIP Piles Chipped to Slab Grade, March 2024)

ACIP Pile Installation Summary

- **Total No. of Production Piles** – 2,104
- **Start/Finish Date**: October 23, 2024, to August 01, 2025
- **Total duration** – 282 days (~ 10 months)
- **LF of Piles Installed** – 112,500 LF
- **Average LF/Pile** – 53.5 LF
- **Production rate/day** – 10 piles/day
- **Grout Quantity** – 11,100 CY

Conclusions

- **Capability of ACIP Piles**
  - Supported the dynamic nature of design loads for Clear Well Basins 3 & 4
  - Penetrated complex geologic conditions with minimal vibrations
  - Achieved required load capacity, primarily from unit side shear in residuum and PWR

- **Importance of Indicator Pile and Load Test Program**
  - Validated ACIP pile performance from a design and constructability standpoint
  - Notably, tension test, though less common, is recommended for structures with uplift loads, as in this project

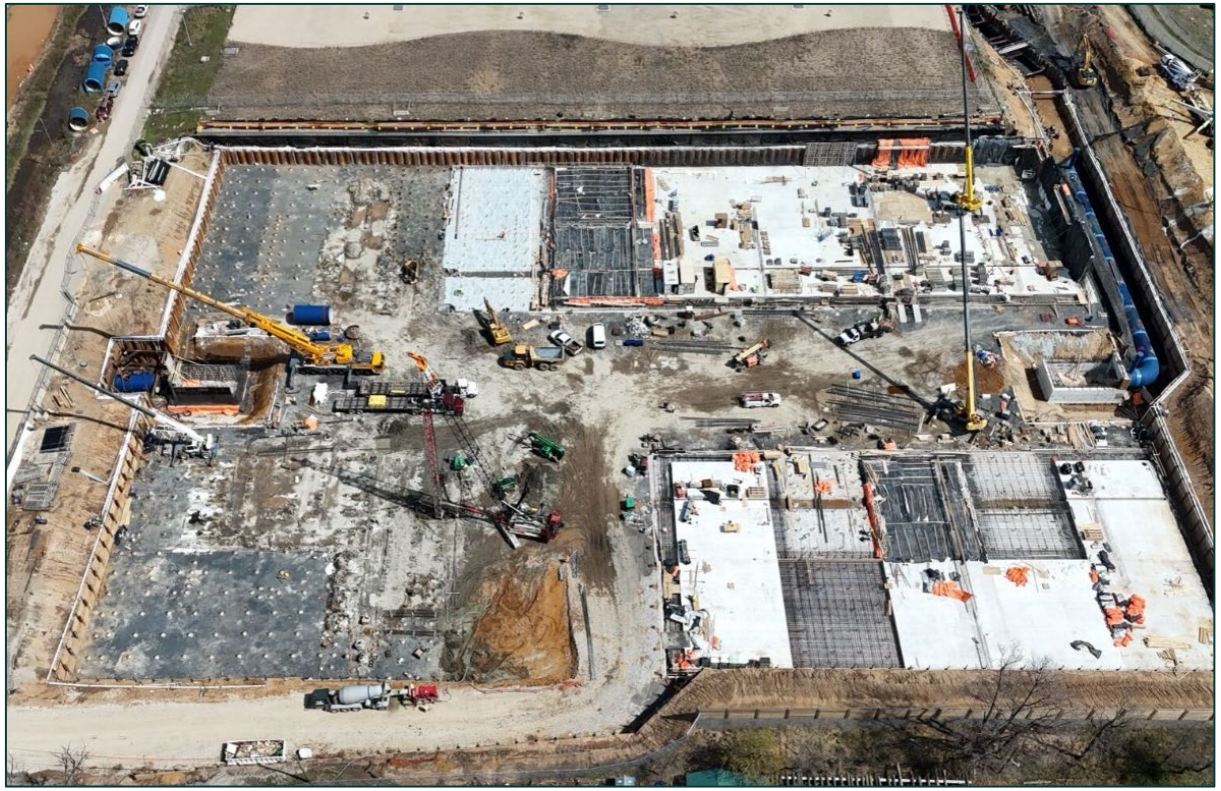
Conclusions (cont'd)

- **QA/QC**
  - One of the most important aspects for successful implementation of a specialty foundation system, as in this project
  - A dedicated QA/QC team on-site helped to effectively navigate through any constructability issues, design refinements and/or enhancements
- **Teamwork and Collaboration**
  - Owner, design professionals, contractors, and testing agencies collaborated seamlessly to meet critical infrastructure demand
  - Open communication and rapid issue resolution overcame initial ACIP pile installation setbacks
  - Regular coordination meetings kept all teams informed and responsive

Construction Progress Photos



(DJK, February 2025, Drone Shot Looking W)



(DJK, April 2025, Drone Shot Looking N)

Construction Progress Photos



(DJK, Sept. 2025, Drone Shot Looking NW)



(PWD CWB 3 & 4 Construction Time-Lapse Video)

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Acknowledgments

- Philadelphia Water Department (PWD)
- Project team: Arcadis, Berkel & Co., Daniel J. Keating Co., GeoStructures (PA), D.W. Kozera, Inc. (PIT Testing) In-Situ Soil Testing, Keller, Navarro & Wright, Pennoni, SANO Drilling, Schnabel Foundation

GFT

Thank You!

Questions?

Ingenuity That Shapes Lives™

Sacramento River East Levee Contract 4

Seepage Cutoff Wall Construction Challenges and Solutions

Tim Williams, PE, GE – Kleinfelder

Nagesh Malyala – Stantec; Kimberly Brown, PE – Kleinfelder; Chris Hamilton – USACE SPK

U.S. Army Corps of Engineers
Sacramento District

Kleinfelder - Stantec
USACE | A JOINT VENTURE



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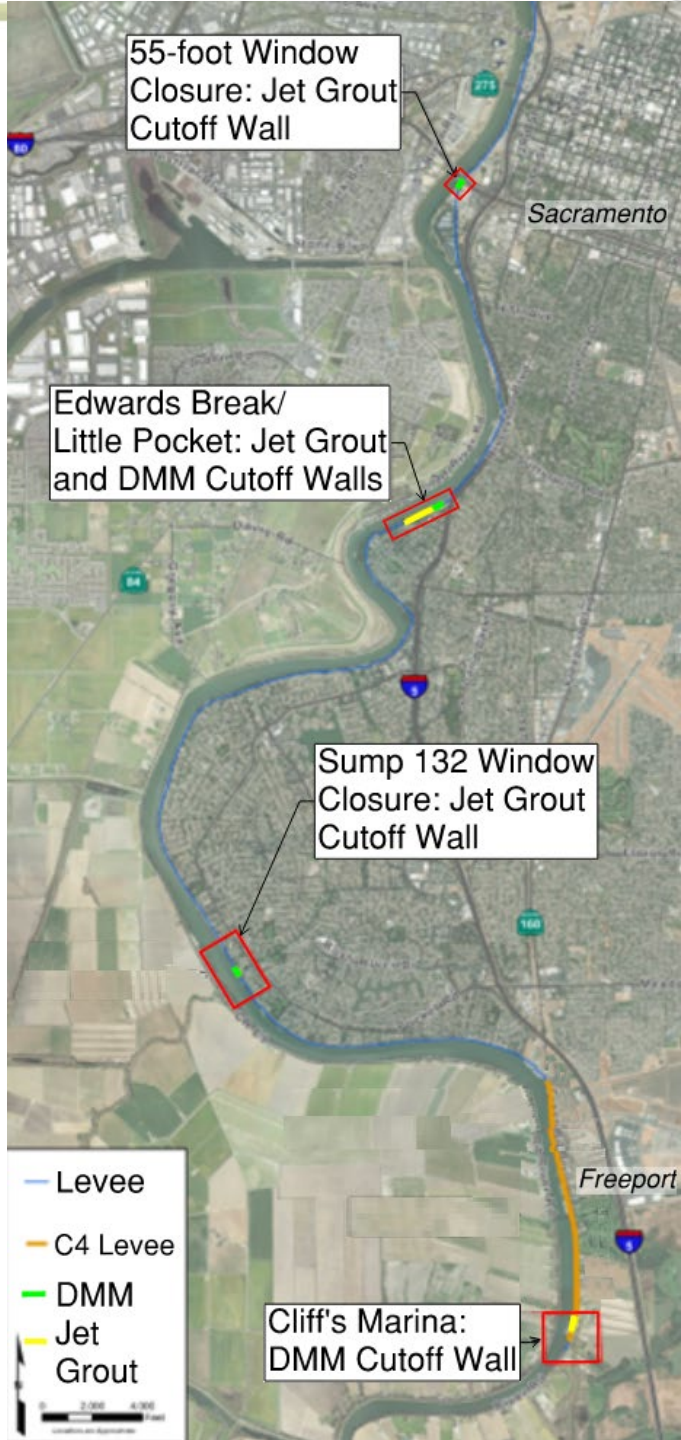
Presentation Narrative

- Project Background & Challenges
- Design Philosophy & Solutions
- Construction Challenges
  - DMM Wall
  - Jet Grout Wall
  - Jet Grout Data Processing
- Closing Summary

Project Background & Challenges

PROJECT CHALLENGES

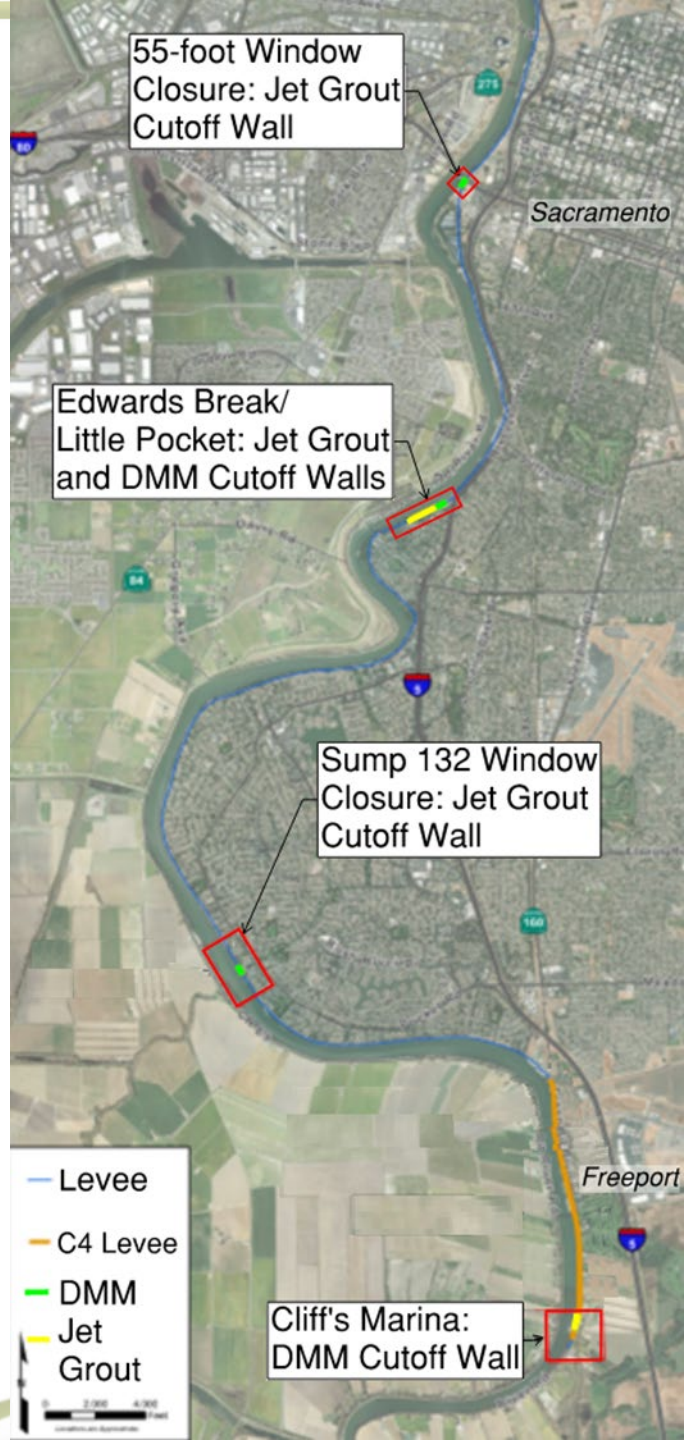
- Close the window in an existing cutoff wall over a conflicting utility.
- Address Edwards Break historical breach site and seepage risk in the Little Pocket area.
- Deepen existing DMM cutoff wall at municipal storm drain Sump 132.
- Address potential levee breach remediation from south of the SREL/Morrison Creek Beach Lake Levee intersection (the project end).



Design Philosophy & Solutions

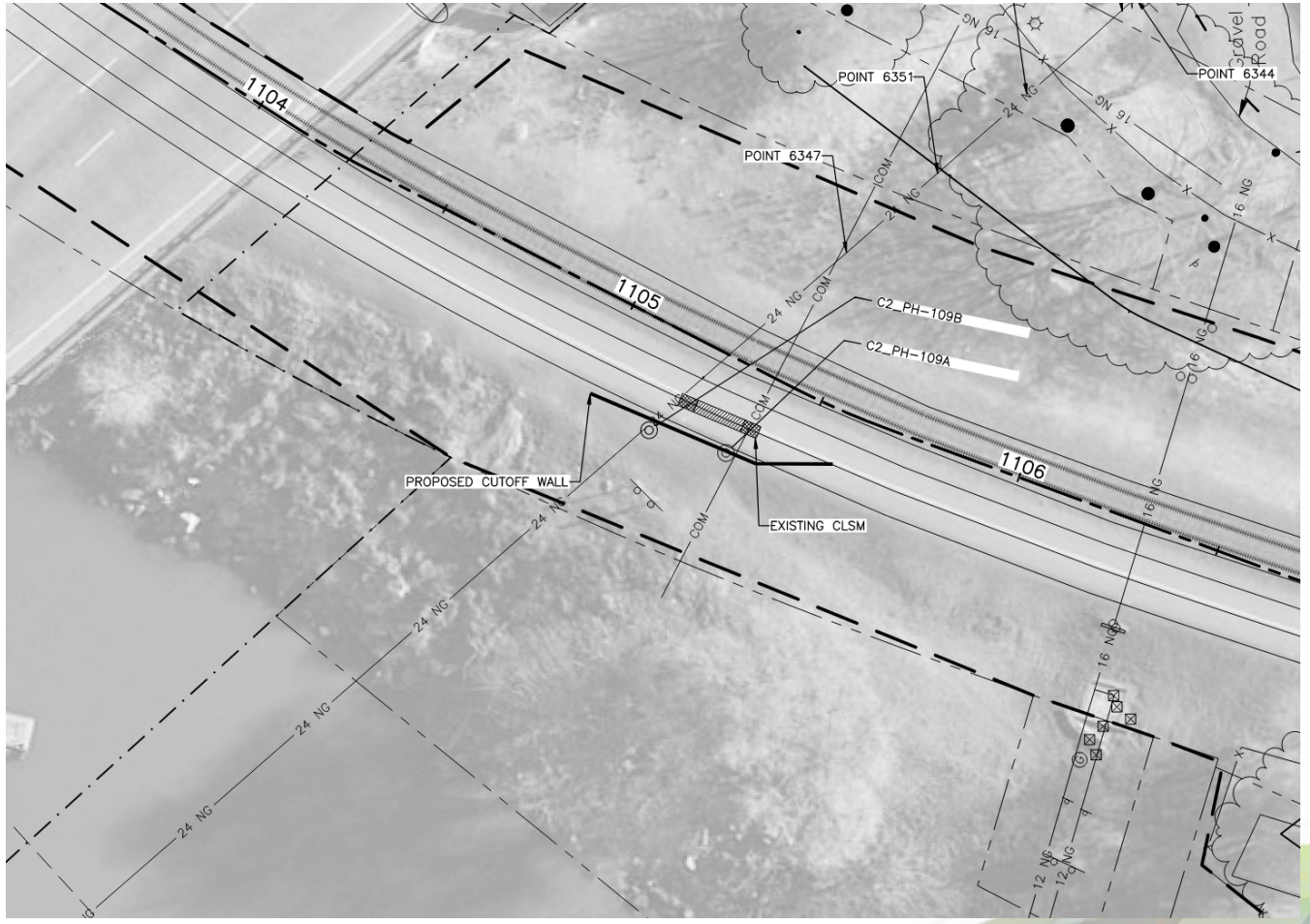
DESIGN SOLUTIONS

- Closing a 55-foot cutoff wall window near the U.S. Route 50 bridge over the Sacramento River.
- Constructing a 1,427-foot cutoff wall at the Edwards Break site and in the Little Pocket area.
- Deepening an existing 200-foot-long deep mix method (DMM) cutoff wall at municipal Sump 132.
- A cutoff wall and seepage berm extending south of the SREL/Morrison Creek Beach Levee intersection (the project end) for a potential levee breach remediation.



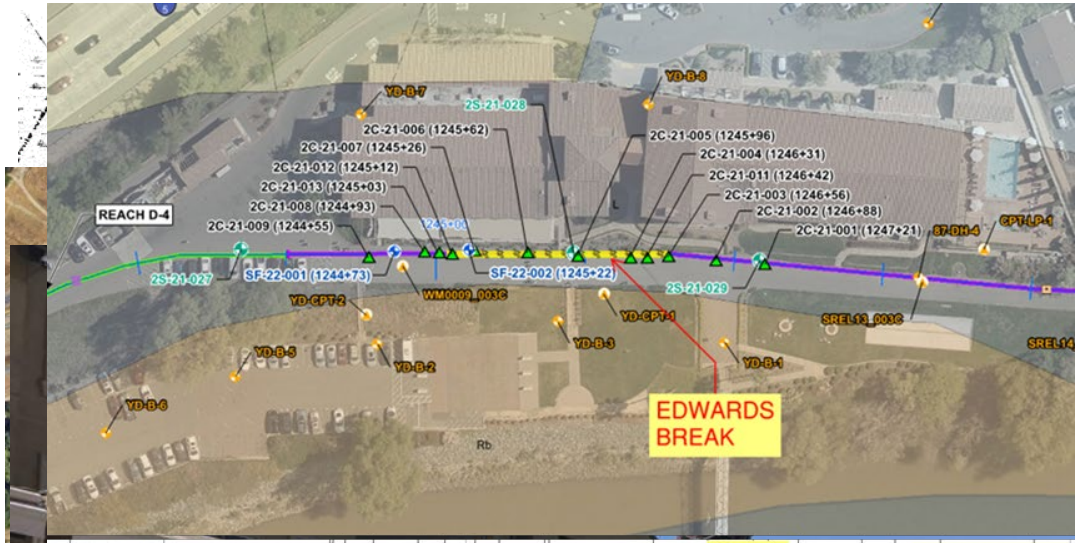
Broadway

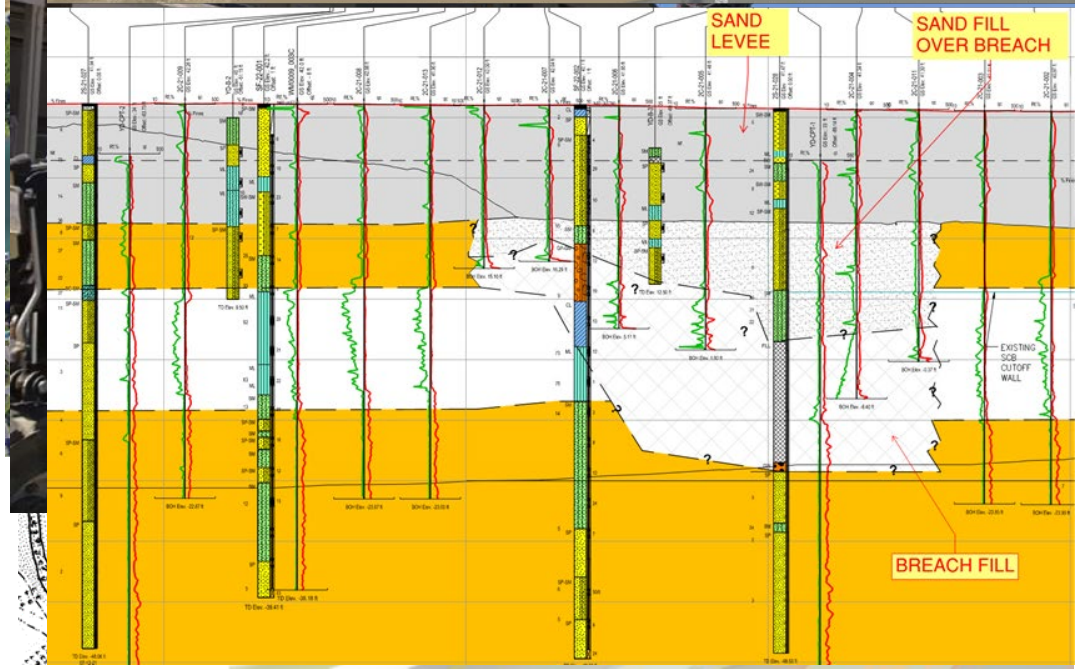
- 55-foot window in a previous jet grout cutoff wall contract due to a buried 24-inch diameter high pressure natural gas line installed by HDD.
- Owner would not allow work to proceed.
- Extensive potholing was done to locate the pipe and obtain owner agreement.



Edwards Break

- Edwards Break was a levee breach that occurred in 1904.
- Break resulted in scour hole and scour pond that were filled with debris.
- Berm placed behind the break.
- Area downstream needed remediation and cutoff wall selected.
- Cutoff wall extended through debris fill to reduce risk of seepage around end of cutoff wall.
- Jet grout selected through breach fill.





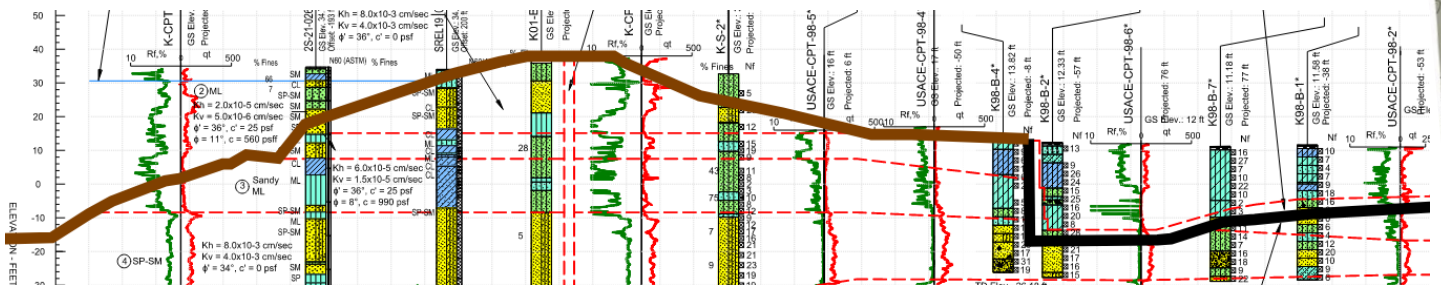
Little Pocket

- 1,427 feet of cutoff wall in urban environment.
- Tie into Edwards Break cutoff wall and downstream DMM cutoff wall installed in 2003.
- Limited access and room for batch plant or ponds.
- Contractor given option of DMM or open slurry trench, soil bentonite cutoff wall.



Sump 132

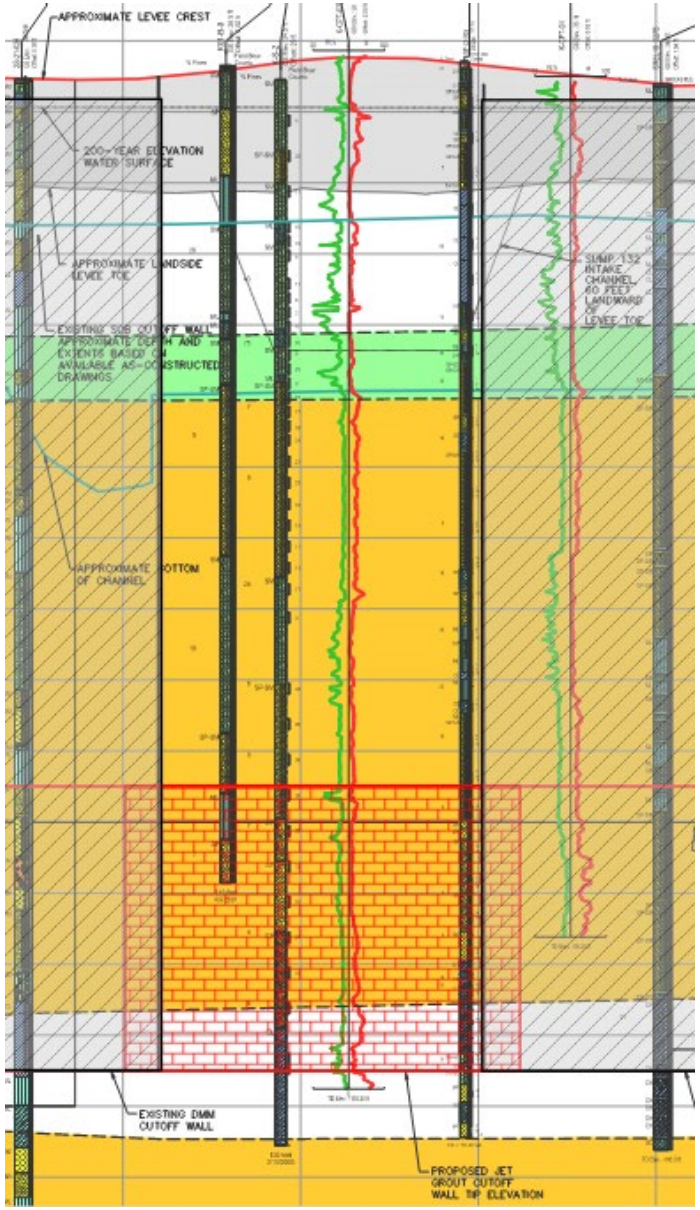
- Sump 132 pump station drains a large suburban area of Sacramento.
- Pile-supported, 8 pumps, inlet channel about Elevation -3, wet well at about Elevation -15.
- Boils observed in inlet channel in 1986 and heavy seepage in 1997 flood. Repairs by City.

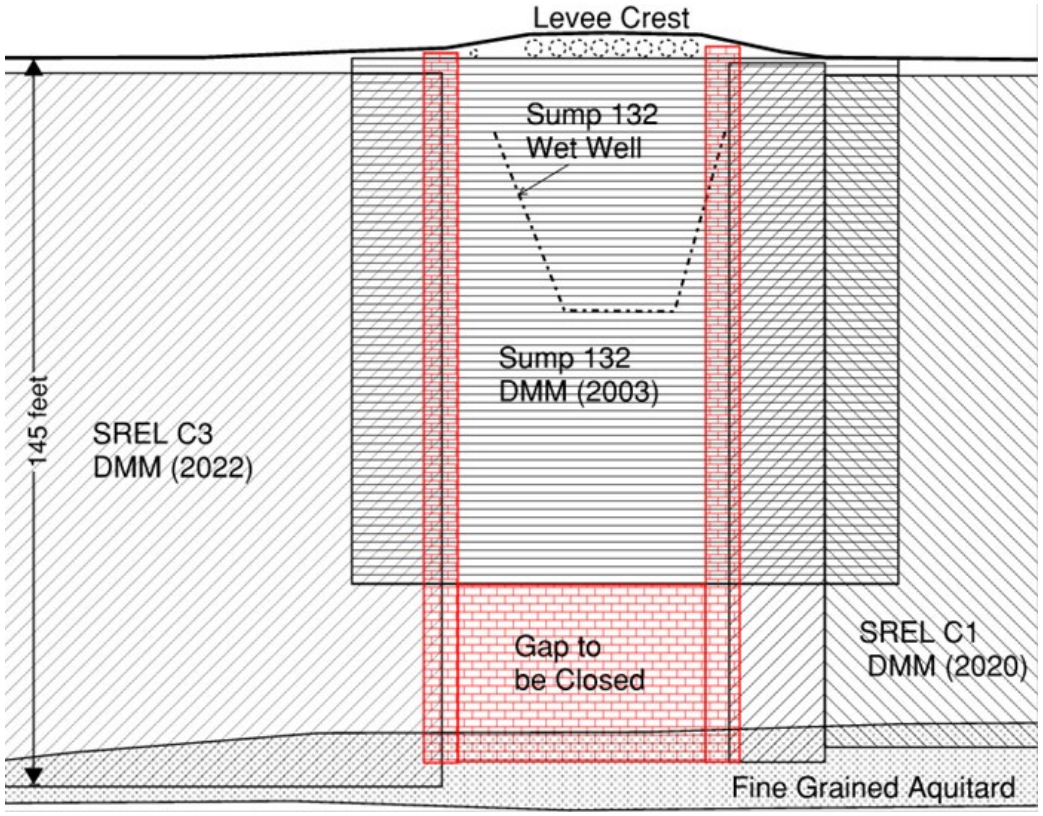


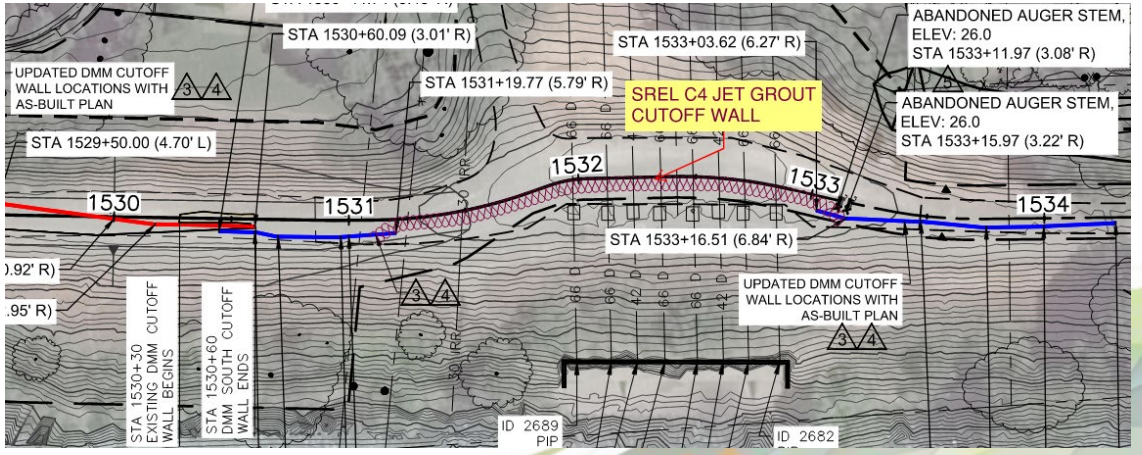


Sump 132

- 2003 SAFCA installed cutoff wall and relief wells.
- SREL C1 and C3 installed cutoff walls upstream and downstream.
- SREL C4 closed gap under cutoff wall.

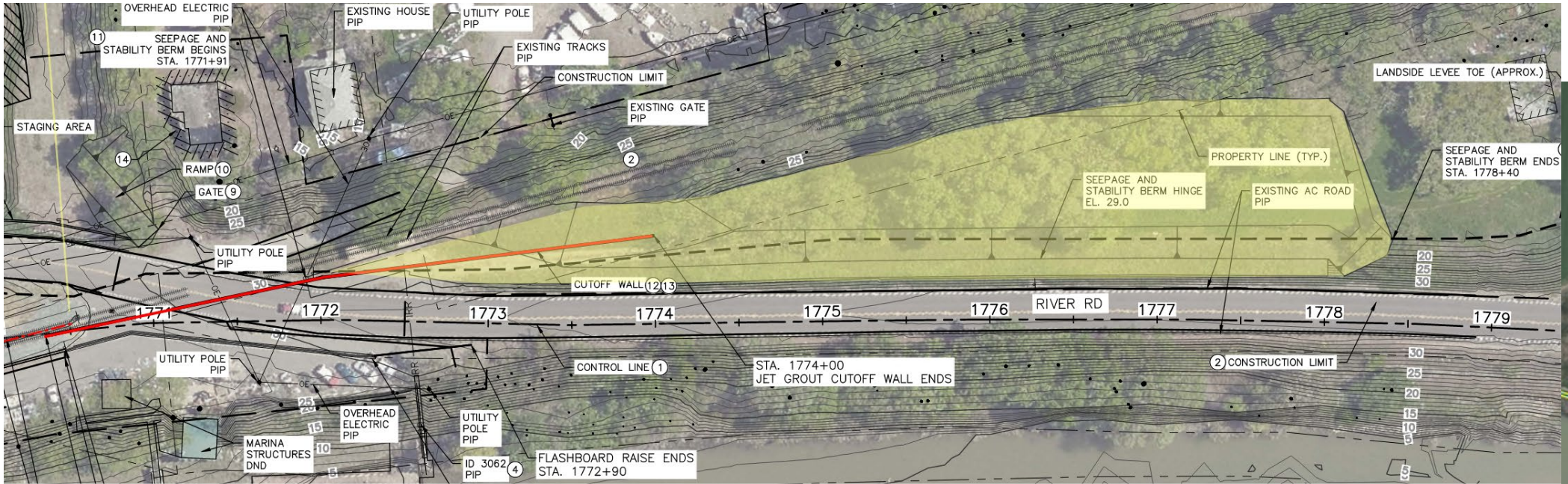


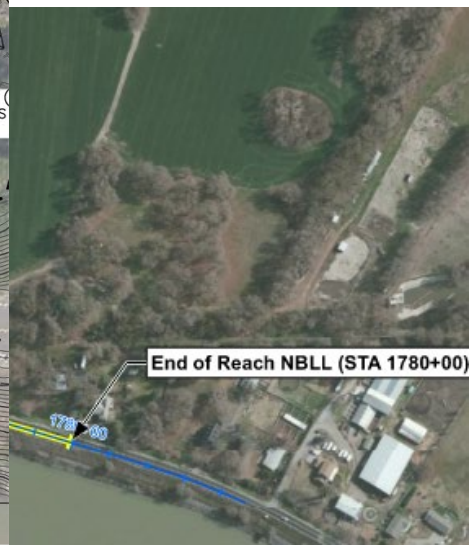




Cliffs Marina

- Intersection of the SREL with the Morrison Creek Beach Lake levee. End of SREL Project.
- Levee downstream does not meet Urban Levee Design Criteria or EM 1110-2-1913 (Design and Construction of Levees).
- Breach risk just downstream of intersection.
- SREL improvements extended 750 feet to address breach risk with seepage cutoff wall and seepage berm.





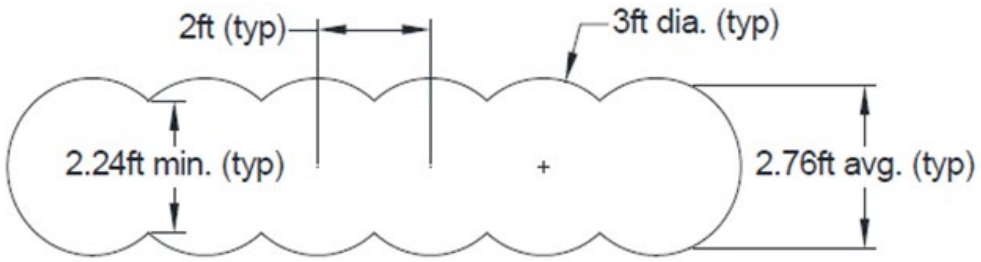
Construction Challenges & Solutions
DMM Cutoff Walls

DMM Cutoff Walls

- DMM was used to construct cutoff walls at two locations where conditions were too restrictive for open trench jet grout methods.
  - **Little Pocket**: 1,427 linear feet at Edwards Break and Little Pocket. Proposed option for jet grout and open trench SB cutoff walls.
  - **Cliff's Marina**: 388 linear feet to provide cutoff wall proposed as a VE option for jet grout crossing of a state highway.

Little Pocket

- A six-auger DMM rig was used to construct 1,107 linear feet of 80-feet deep SCB cutoff wall (from degraded levee elevation).
- The contractor had the choice of open-trench SB wall or DMM and chose DMM because the 6-auger rig allowed for faster production.



Typical Six-Auger Element



Little Pocket

- **Elimination of bench scale test section**
  - The contract required a test section of the contractor's proposed method and mix design to evidence ability to provide a DMM cutoff wall meeting the specifications.
  - This requirement was waived in favor of the contractor's experience in the area in similar conditions with similar methods.
  - The condition that a verification coring and in-situ water test would be performed every 250 linear feet of cutoff wall (for a total of six coring locations).
    - **A trade of additional testing for time savings.**

Little Pocket

- **Difficulty coring after covering the top of wall**
  - The results of all but one coring location met the Specification requirements. **One corehole had a rising head test hydraulic conductivity of 4.60E-05 cm/sec, while the requirement is a hydraulic conductivity of no greater than 1x10-6 cm/sec.**
  - The failing test was attributed to excessive horizontal corehole deviation, leading to the wall being too thin to properly test. The top of wall had been covered, making it difficult to precisely locate the wall, leading to rod punch-out.
  - Core sample laboratory testing for unconfined compressive strength and permeability were required in place of the in-situ water testing.
    - **The core sample tests had satisfactory results, so the project delivery team accepted the DMM cutoff wall.**

Little Pocket

- **Cavities in the exposed top of wall**
  - After the DMM wall was constructed and prior to the levee rebuild, the uppermost foot of the DMM wall was removed and the top of the wall was visually inspected and probed. **During this inspection, several cavities within the treatment were observed.**
  - The project delivery team required that each cavity be excavated and vacuumed out and backfilled with approved CLSM mixture.





Cliff's Marina

- VE recommendation to replaced proposed jet grout cutoff wall with DMM.
- Raito used 3-auger configuration for 47-foot deep, 388-foot-long cutoff wall across paved parking lot and SR160.
- Crossed SR160 utilized single lane closures to keep highway open.
- Used trench plates to protect cutoff wall in SR160 and parking lot.
  - ❖ VE saved significant schedule and cost.







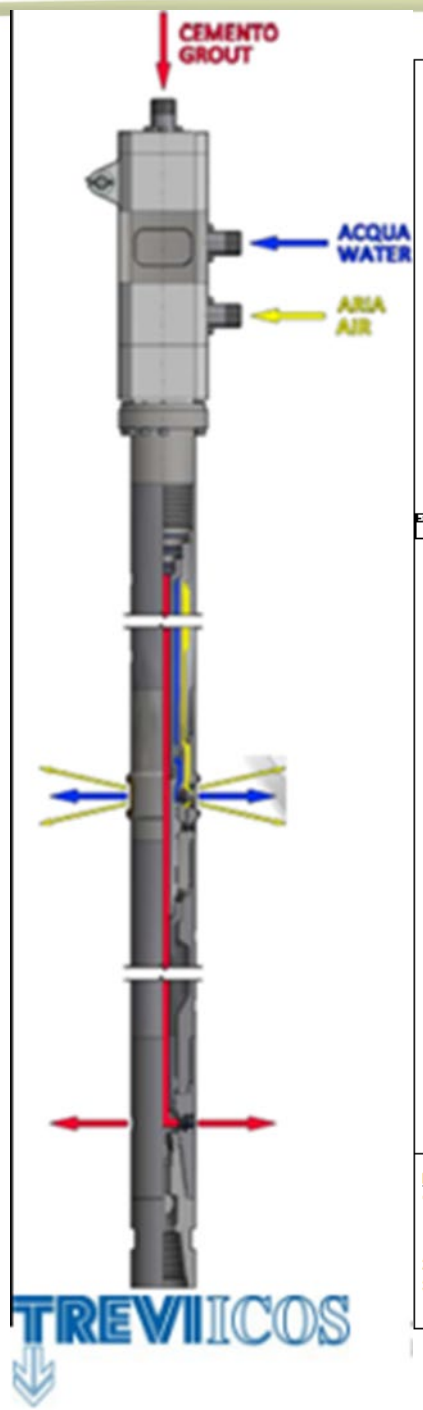
Construction Challenges & Solutions
Jet Grout Cutoff Walls

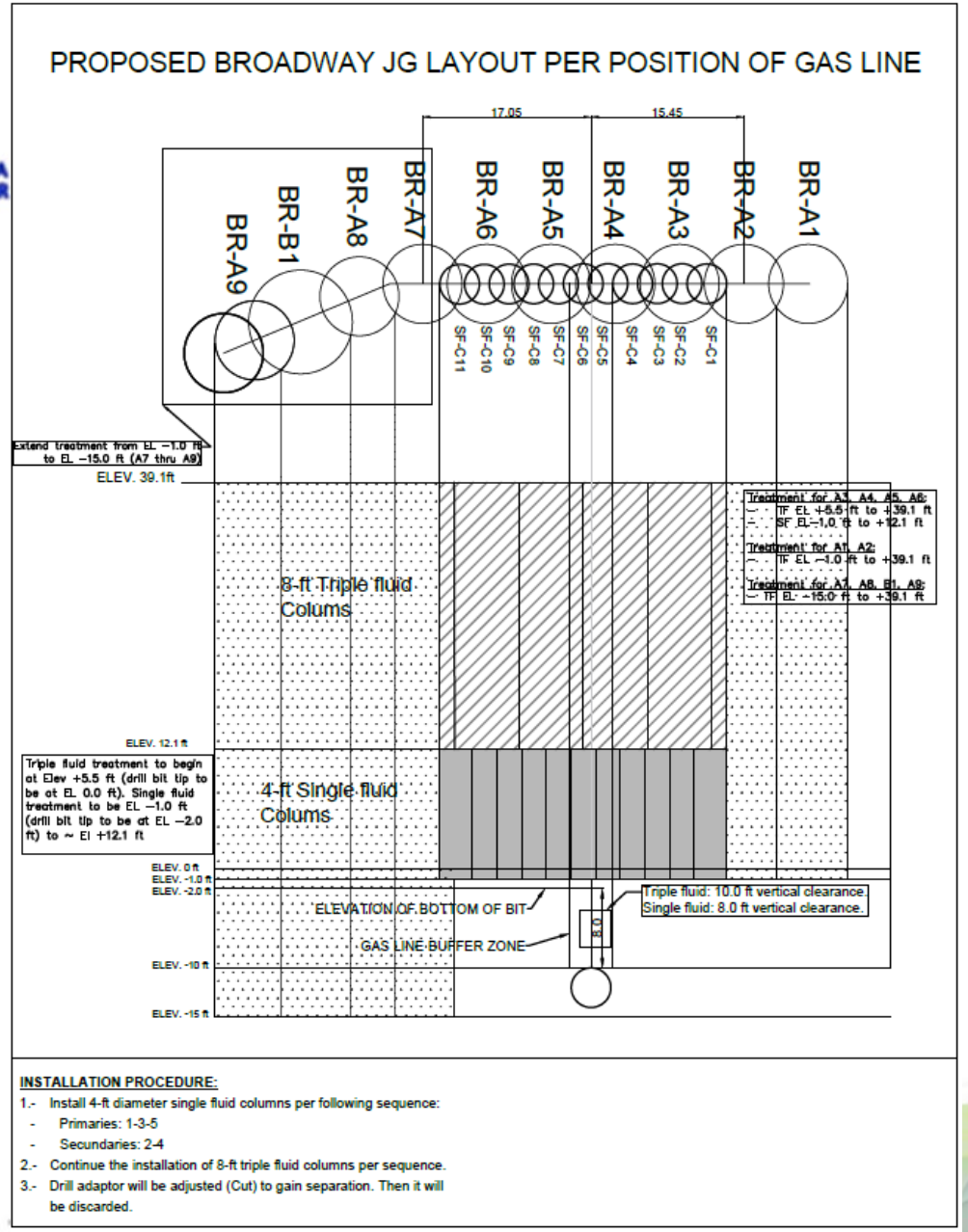
Jet Grout Cutoff Walls

- Triple fluid jet grouting was used to construct cutoff walls in three locations where conditions were too restrictive for open trench or DMM methods.
  - **Broadway**: 55 linear feet next to a railroad and above a 24-inch diameter high pressure gas line.
  - **Edwards Break**: 292 linear feet to provide cutoff wall through an historical 1904 levee breach filled with random fill, adjacent to two commercial buildings.
  - **Sump 132**: 228 linear feet around up to 60-inch diameter municipal sump pipes, filling in a window between existing DMM cutoff wall.
- Single fluid jet grout was used as needed, for stability at Edwards Break and additional clearance at Broadway.

Broadway Heading

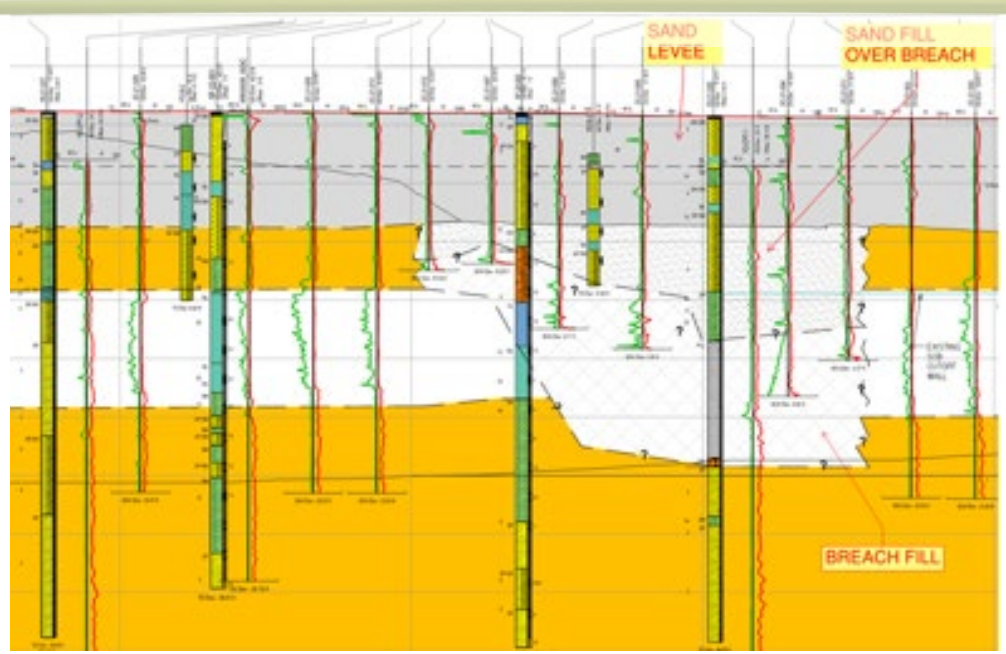
- Triple fluid jet grout monitor extends below cutoff wall bottom for water/air jets to cut soil.
- The cutoff wall was constructed to the design elevation above the pipe, while maximum vertical clearance was obtained using a modified single fluid monitor, which has a shorter distance between the grout jet and drill bit compared to a triple fluid monitor.
  - **Construction solution adopted to conditions without compromising on the design cutoff depth.**





Edwards Break Heading

- Sinkholes developed during construction of the pre-production test section of 3 pairs of overlapping columns (one pair inside the historical breach with air, one inside the historical breach without air, and one outside the historical breach with air).
- The potential for sinkhole development during production of the rest of the wall needed to be addressed.
- Sinkholes posed a threat to rig stability, personnel safety, and the integrity of the adjacent buildings' foundation (rammed aggregate pier).

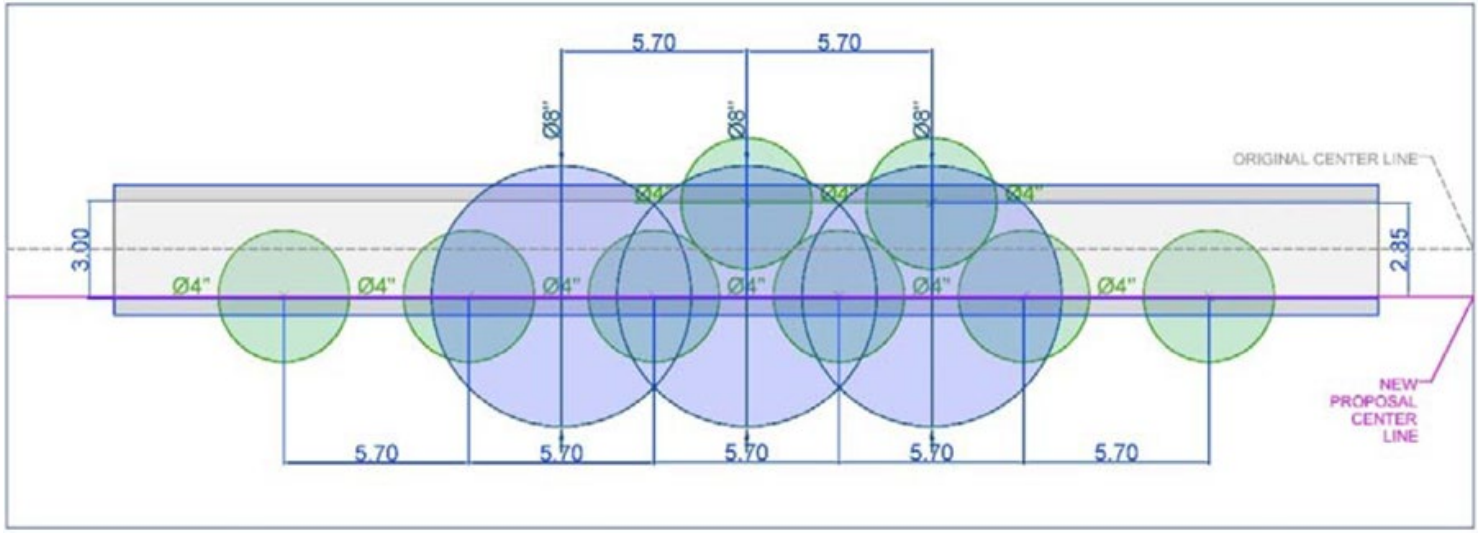


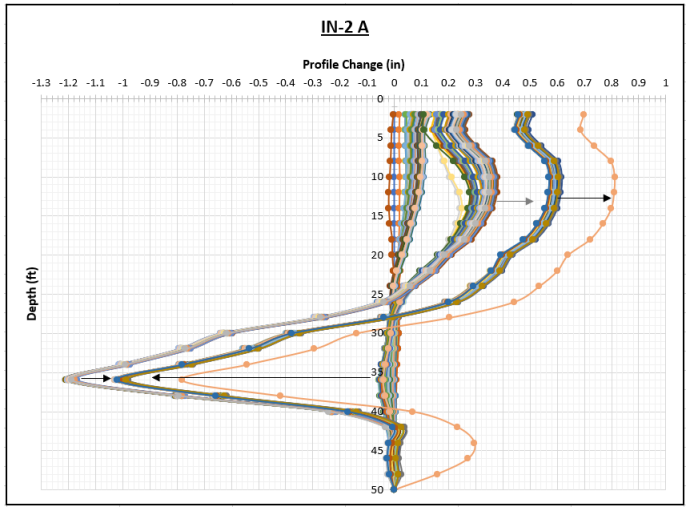


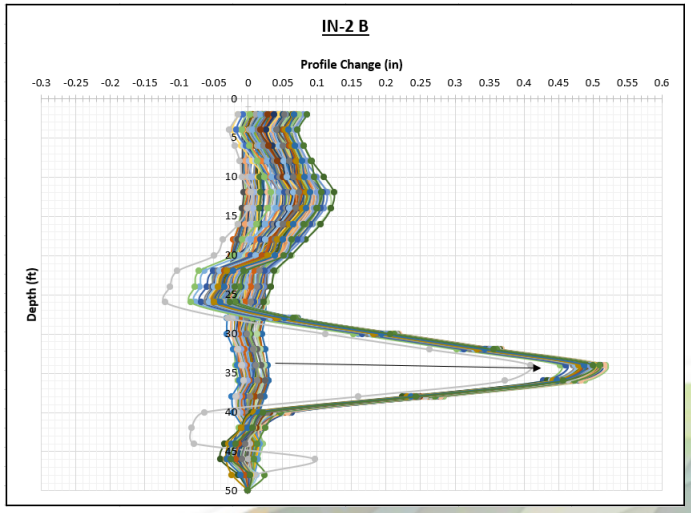


Edwards Break Heading

- The contractor proposed a stabilization method that used 20-foot-deep single fluid columns in a triangular pattern on the landside of the alignment and overlapping of triple fluid columns.
- Three inclinometer casings were installed to monitor for signs of effect to the adjacent buildings' foundation. Inclinometer casings within 50 feet of drilling and jet grouting were measured three times per day.







Edwards Break Heading

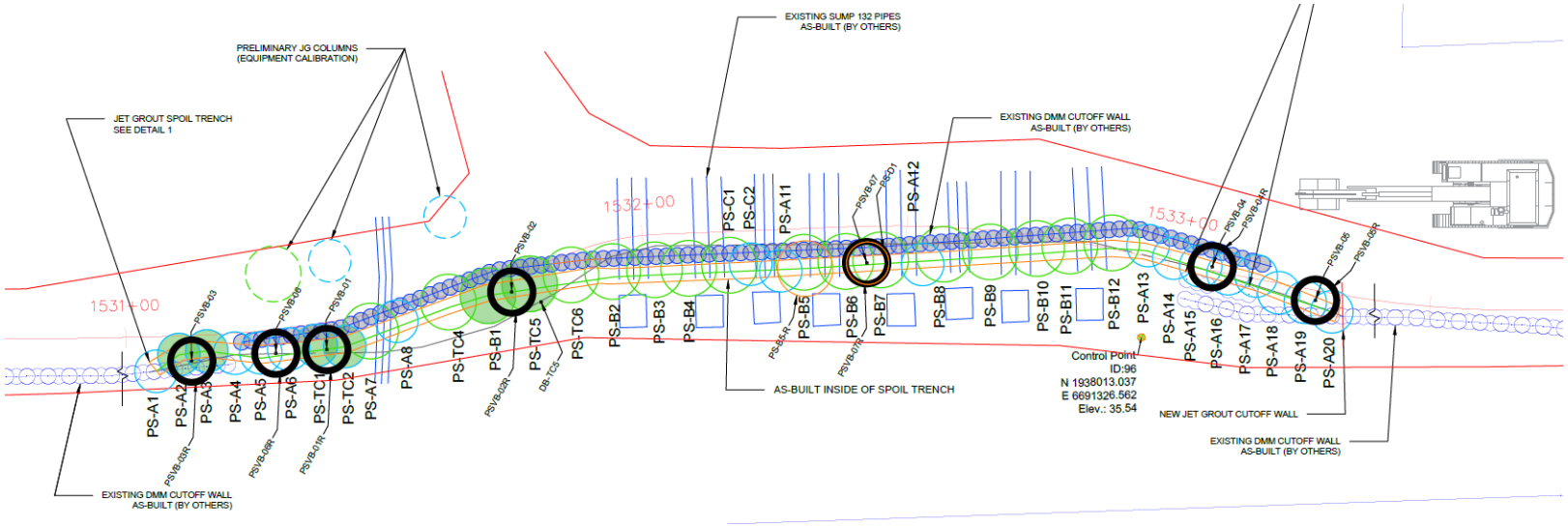
- Sinkholes continued to develop despite the single fluid stabilization columns and the contractor's efforts to prevent them.
- Alternate method to install the stage-based grouting on remaining columns to a depth below the potential breach. Reduction in formation of sinkholes.
- Woody debris from buried trees, as well as boulders from the historical breach fill, posed additional challenge.
  - ❖ Columns broken up into staged construction over several days, causing delay
  - ❖ Voids and corehole collapse leading to fluid loss and the need to pump a much greater volume of grout





Sump 132 Heading

- The existing DMM cutoff wall “mushroomed” at the top, having a geometry wider than 3-feet at the top before narrowing to 3-feet.
- The jet grout alignment’s offset from the existing DMM wall could not be moved far out enough to avoid the “mushroom” top, since a connection between the walls had to be made for proper seepage cutoff.

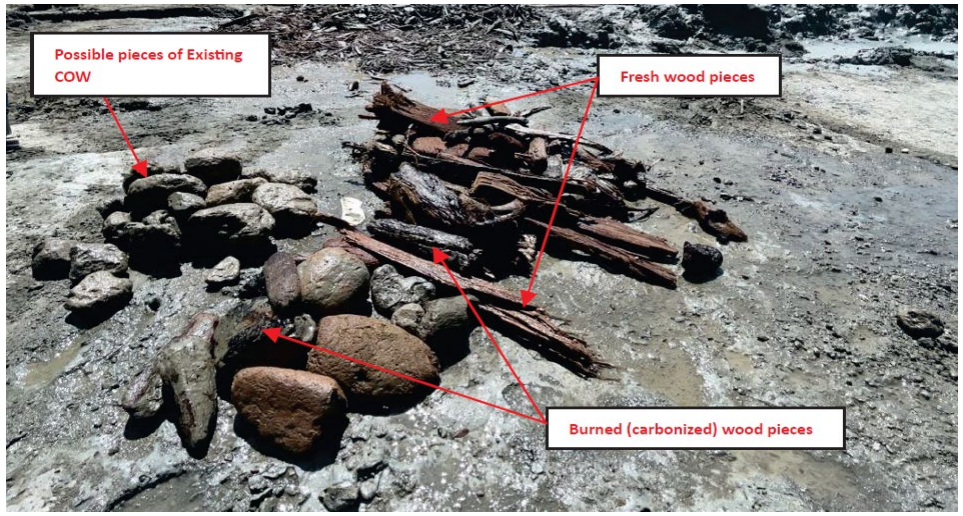




Sump 132 Heading

- Dense gravel layers, cobble, and buried trees/wood provided difficult drilling conditions.
  - ❖ Difficult drilling conditions lead to excessive drilling deviation. Several attempts to drill deviated too greatly and had to be backfilled with grout and redone.
  - ❖ Wood debris clogged spoils pumps.
  - ❖ This also slowed down the production process, making it necessary to stage column construction over several days.







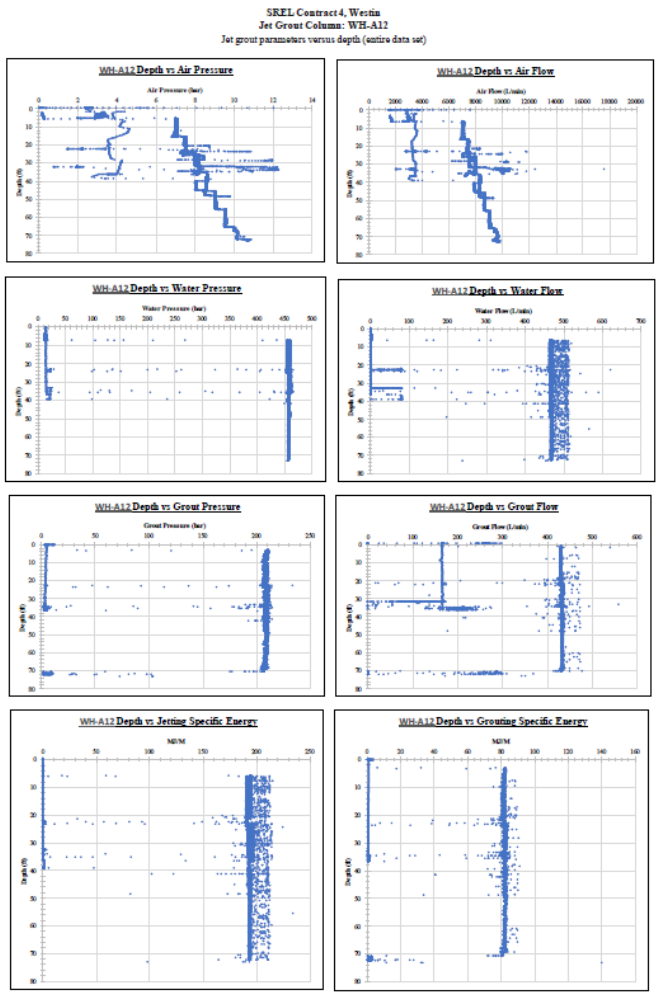
Construction Challenges & Solutions
Jet Grout Wall – Data Processing

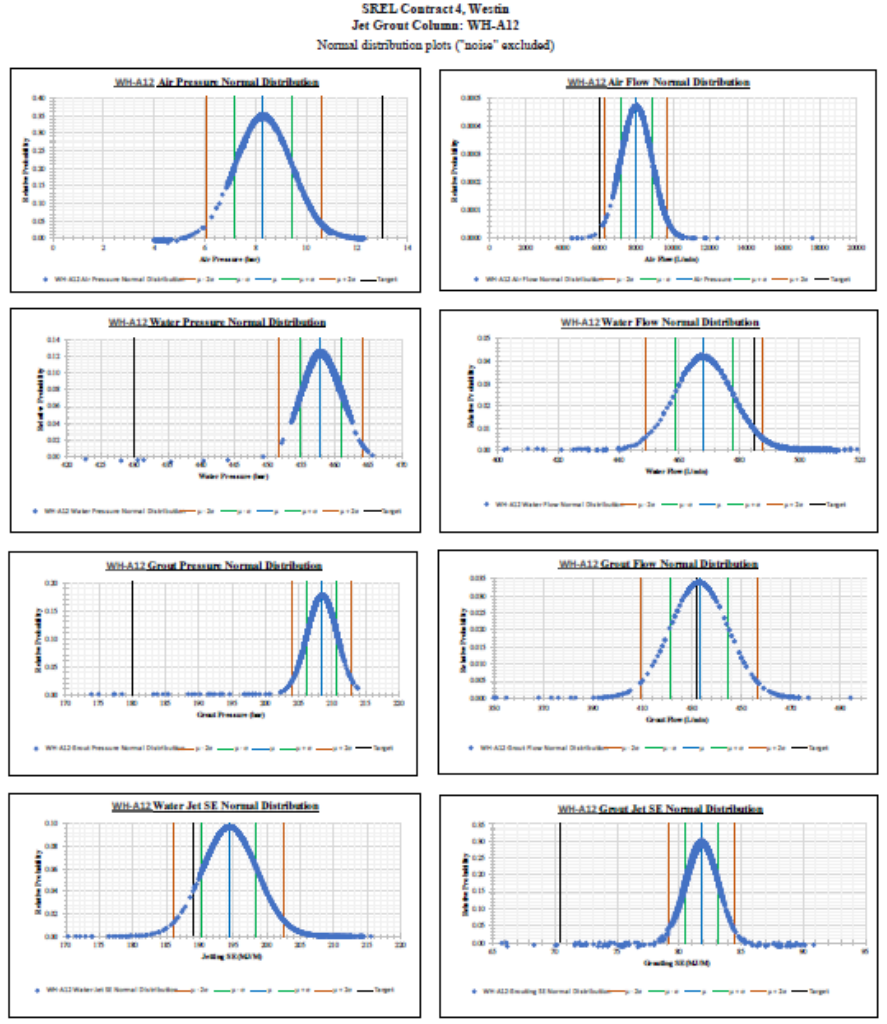
JET GROUT RAW DATA PROCESSING

- Acceptance criteria for the jet grout cutoff walls include depth, thickness, hydraulic conductivity, compressive strength, continuity, and homogeneity requirements.
- Acceptance criteria also includes the evaluation of jetting and grouting parameters used to produce a compliant soil-cement material based on parameters chosen by the Contractor during the preproduction phase. More importantly, **the specific energy of jetting and grouting, must not be less than 90% of the design values selected during preproduction**.
- The jetting and grouting specific energies are considered critical parameters that can be correlated to the geometry and homogeneity of the cutoff elements.
  - ❖ **A statistical method was developed to verify the construction parameters were met. Comparison between normal distributions informed decisions on coring locations.**

JET GROUT RAW DATA PROCESSING

Analyze all data → Remove the outliers → Analyze the realistic pool





WH-A12	Parameter							
	Air Pressure	Air Flow	Water Pressure	Water Flow	Grout Pressure	Grout Flow	Jetting SE	Grouting SE
Unit	bar	L/min	bar	L/min	bar	L/min	MJ/M	MJ/M
Target	13	6000	430	485	180	432	189	70
Min	4.00	4509.00	323.19	378.00	136.90	342.00	148.99	53.23
Median	8.21	7903.20	457.71	467.00	208.62	433.00	193.79	81.88
Mean	8.30	7997.55	457.90	468.22	208.45	432.88	194.37	81.79
Stdev.P	1.13	845.01	3.12	9.57	2.25	11.80	4.10	1.33
Max	12.27	17569.20	465.55	619.00	232.73	1486.00	234.24	139.71

WH-A12	Parameter							
	Air Pressure	Air Flow	Water Pressure	Water Flow	Grout Pressure	Grout Flow	Jetting SE	Grouting SE
Unit	bar	L/min	bar	L/min	bar	L/min	MJ/M	MJ/M
Stdev.P (σ)	1.13	845.01	3.12	9.57	2.25	11.80	4.10	1.33
μ - 2σ	6.03	6307.53	451.65	449.08	203.94	409.28	186.17	79.13
μ - σ	7.17	7152.54	454.77	458.65	206.19	421.08	190.27	80.46
Mean (μ)	8.30	7997.55	457.90	468.22	208.45	432.88	194.37	81.79
μ + σ	9.44	8842.56	461.02	477.79	210.70	444.67	198.47	83.12
μ + 2σ	10.57	9687.57	464.15	487.36	212.95	456.47	202.57	84.45

WH-A12	Air Pressure	Air Flow	Water Pressure	Water Flow	Grout Pressure	Grout Flow	Jetting SE	Grouting SE
Unit	bar	L/min	bar	L/min	bar	L/min	MJ/M	MJ/M
Target	13	6000	430	485	180	432	189	70
Total Count	9300	9188	9100	9096	9366	9368	9093	9359
Target ± 5%	0.00%	0.09%	0.10%	98.52%	0.12%	99.05%	92.37%	0.18%
Target ± 10%	1.43%	0.97%	99.86%	1.12%	0.19%	0.80%	4.92%	0.37%
Target ± 15%	0.32%	0.49%	0.02%	0.24%	12.56%	0.07%	2.67%	10.93%
Target ± 20%	0.92%	19.59%	0.01%	0.08%	87.10%	0.03%	0.02%	87.84%
Target ± 25%	6.46%	18.79%	0.01%	0.03%	0.02%	0.01%	0.02%	0.43%
Target ± 100%	1.27%	60.96%	0.00%	0.01%	0.01%	0.02%	0.00%	0.25%
Target ± 300%	0.00%	0.02%	0.00%	0.00%	0.00%	0.01%	0.00%	0.00%

WH-A12	Air Pressure	Air Flow	Water Pressure	Water Flow	Grout Pressure	Grout Flow	Jetting SE	Grouting SE
Unit	bar	L/min	bar	L/min	bar	L/min	MJ/M	MJ/M
Target	13	6000	430	485	180	432	189	70
Total Count	9300	9188	9100	9096	9366	9368	9093	9359
95% to 100%	0.00%	0.01%	0.03%	93.39%	0.05%	41.86%	0.32%	0.03%
90% to 95%	1.43%	0.02%	0.03%	0.41%	0.01%	0.26%	0.19%	0.04%
85% to 90%	0.32%	0.00%	0.02%	0.22%	0.01%	0.05%	0.10%	0.00%
80% to 85%	0.92%	0.01%	0.01%	0.07%	0.01%	0.03%	0.01%	0.04%
75% to 80%	6.46%	0.04%	0.01%	0.02%	0.02%	0.01%	0.01%	0.01%
50% to 75%	89.59%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
0% to 50%	1.27%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
ntal Below	100.00%	0.09%	0.11%	94.11%	0.11%	42.21%	0.63%	0.13%

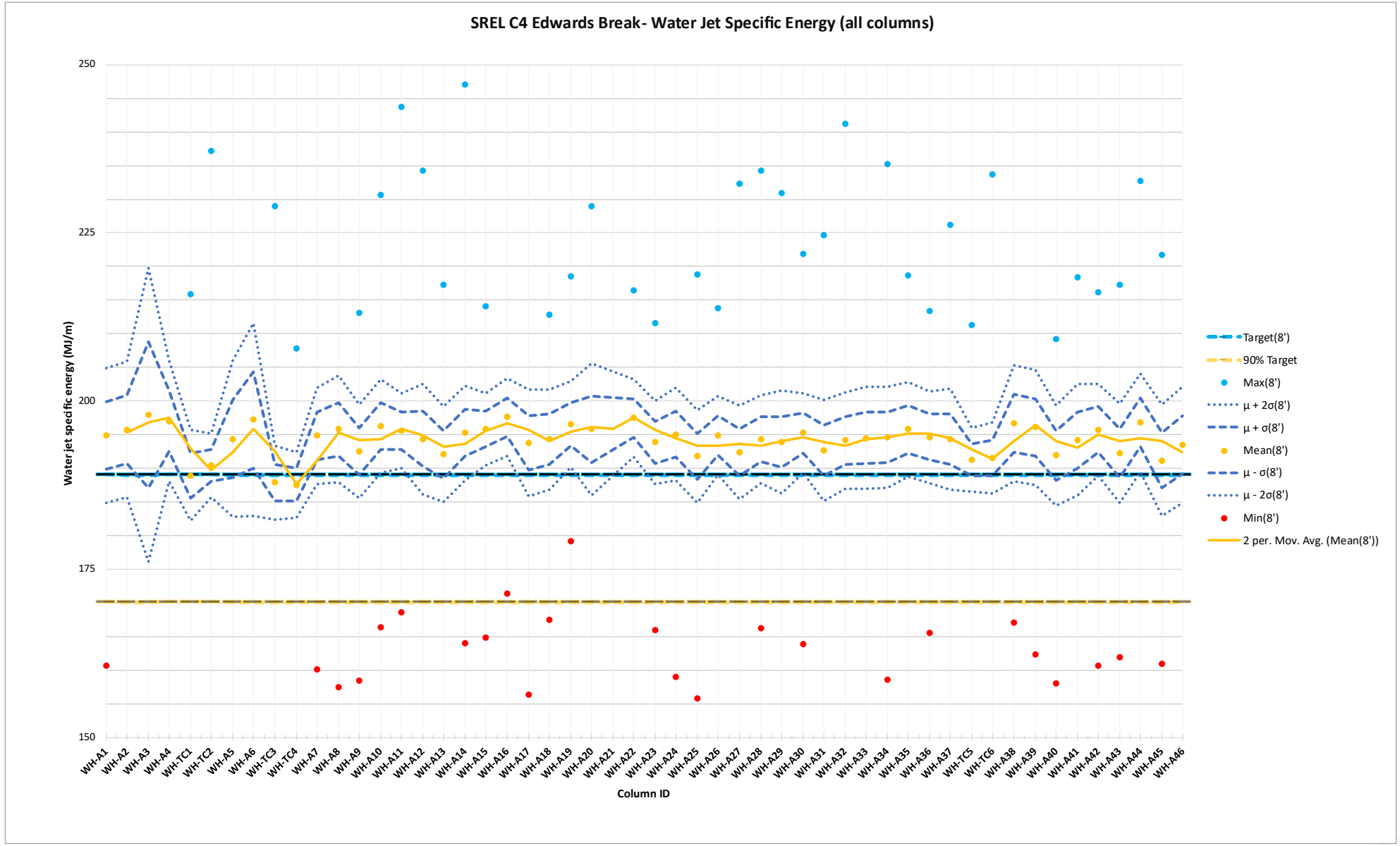
WH-A12	Air Pressure	Air Flow	Water Pressure	Water Flow	Grout Pressure	Grout Flow	Jetting SE	Grouting SE
Unit	bar	L/min	bar	L/min	bar	L/min	MJ/M	MJ/M
Target	13	6000	430	485	180	432	189	70
Total Count	9300	9188	9100	9096	9366	9368	9093	9359
100% to 105%	0.00%	0.08%	0.07%	5.12%	0.06%	57.19%	92.05%	0.15%
105% to 110%	0.00%	0.04%	99.82%	0.71%	0.18%	0.54%	4.73%	0.33%
110% to 115%	0.00%	0.49%	0.00%	0.02%	12.55%	0.02%	2.57%	10.93%
115% to 120%	0.00%	19.58%	0.00%	0.01%	87.09%	0.00%	0.01%	87.80%
120% to 125%	0.00%	18.74%	0.00%	0.01%	0.00%	0.00%	0.01%	0.42%
125% to 200%	0.00%	60.96%	0.00%	0.01%	0.01%	0.02%	0.00%	0.25%
200% to 400%	0.00%	0.02%	0.00%	0.00%	0.00%	0.01%	0.00%	0.00%
400% to 600%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
Total Above	0.00%	99.91%	99.89%	5.89%	99.89%	57.79%	99.37%	99.87%

WH-A20	Parameter							
	Air Pressure	Air Flow	Water Pressure	Water Flow	Grout Pressure	Grout Flow	Jetting SE	Grouting SE
Unit	bar	L/min	bar	L/min	bar	L/min	MJ/M	MJ/M
Target	13	6000	430	485	180	432	189	70
Min	4.00	4567.20	333.32	396.00	135.01	339.00	144.83	53.62
Median	8.39	7687.20	456.73	467.00	208.38	431.00	193.71	81.50
Mean	8.51	7758.64	456.04	473.78	208.29	432.44	195.80	81.54
Stdev.P	1.53	896.82	4.06	13.91	2.04	14.86	4.92	1.54
Max	14.38	14757.00	495.43	728.00	217.29	1847.00	228.94	175.89

WH-A20	Air Pressure	Air Flow	Water Pressure	Water Flow	Grout Pressure	Grout Flow	Jetting SE	Grouting SE
Unit	bar	L/min	bar	L/min	bar	L/min	MJ/M	MJ/M
Target	13	6000	430	485	180	432	189	70
Total Count	12385	11953	11742	11750	12151	12307	11741	12153
95% to 100%	0.37%	0.15%	0.04%	78.88%	0.03%	67.07%	0.26%	0.04%
90% to 95%	0.73%	0.01%	0.05%	0.31%	0.03%	0.24%	0.17%	0.04%
85% to 90%	4.03%	0.00%	0.03%	0.14%	0.01%	0.09%	0.06%	0.03%
80% to 85%	1.33%	0.03%	0.03%	0.05%	0.02%	0.00%	0.03%	0.03%
75% to 80%	15.33%	0.05%	0.03%	0.00%	0.02%	0.02%	0.02%	0.04%
50% to 75%	74.08%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
0% to 50%	3.54%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
Total Below	99.42%	0.23%	0.17%	79.37%	0.11%	67.42%	0.53%	0.19%

WH-A20	Parameter							
	Air Pressure	Air Flow	Water Pressure	Water Flow	Grout Pressure	Grout Flow	Jetting SE	Grouting SE
Unit	bar	L/min	bar	L/min	bar	L/min	MJ/M	MJ/M
Stdev.P (σ)	1.53	896.82	4.06	13.91	2.04	14.86	4.92	1.54
μ - 2σ	5.46	5965.00	447.92	445.96	204.21	402.73	185.95	78.46
μ - σ	6.98	6861.82	451.98	459.87	206.25	417.58	190.88	80.00
Mean (μ)	8.51	7758.64	456.04	473.78	208.29	432.44	195.80	81.54
μ + σ	10.03	8655.47	460.11	487.69	210.33	447.30	200.73	83.08
μ + 2σ	11.56	9552.29	464.17	501.60	212.37	462.15	205.65	84.62

WH-A20	Air Pressure	Air Flow	Water Pressure	Water Flow	Grout Pressure	Grout Flow	Jetting SE	Grouting SE
Unit	bar	L/min	bar	L/min	bar	L/min	MJ/M	MJ/M
Target	13	6000	430	485	180	432	189	70
Total Count	12385	11953	11742	11750	12151	12307	11741	12153
100% to 105%	0.23%	0.38%	4.45%	19.40%	0.03%	30.72%	76.84%	0.08%
105% to 110%	0.20%	4.34%	95.15%	0.99%	0.06%	1.36%	19.94%	0.12%
110% to 115%	0.15%	12.40%	0.20%	0.09%	12.97%	0.24%	2.54%	13.95%
115% to 120%	0.00%	15.42%	0.02%	0.03%	86.82%	0.13%	0.14%	84.56%
120% to 125%	0.00%	10.84%	0.00%	0.02%	0.01%	0.05%	0.02%	0.72%
125% to 200%	0.00%	56.35%	0.00%	0.10%	0.00%	0.08%	0.00%	0.38%
200% to 400%	0.00%	0.03%	0.00%	0.00%	0.00%	0.00%	0.00%	0.01%
400% to 600%	0.00%	0.00%	0.00%	0.00%	0.00%	0.01%	0.00%	0.00%
Total Above	0.58%	99.77%	99.83%	20.63%	99.89%	32.58%	99.47%	99.81%


SREL C4 Edwards Break- Water Jet Specific Energy (all columns)
Water jet specific energy (MJ/m)
250
225
200
175
150
Column ID
WH-A1
WH-A2
WH-A3
WH-A4
WH-TC1
WH-TC2
WH-A5
WH-A6
WH-TC3
WH-TC4
WH-A7
WH-A8
WH-A9
WH-A10
WH-A11
WH-A12
WH-A13
WH-A14
WH-A15
WH-A16
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WH-A32
WH-A33
WH-A34
WH-A35
WH-A36
WH-A37
WH-TC5
WH-TC6
WH-A38
WH-A39
WH-A40
WH-A41
WH-A42
WH-A43
WH-A44
WH-A45
WH-A46
Target(8')
90% Target
Max(8')
μ + 2σ(8')
μ + σ(8')
Mean(8')
μ - σ(8')
μ - 2σ(8')
Min(8')
2 per. Mov. Avg. (Mean(8'))

Closing Summary

CLOSING SUMMARY

- **DMM Challenges and Solutions**
  - ❖ Waiver of the bench scale test section and increased coring and in-situ corehole testing saved schedule.
  - ❖ Difficulty in coring after covering the top of wall with fill.
  - ❖ Cavities in the top of wall that required cleanout and backfill.
- **Jet Grouting Challenges and Solution**
  - ❖ Buried 24” high-pressure natural gas line required modification of approach.
  - ❖ Extensive sinkhole development and associated risk to rig safety, personnel safety, and adjacent structure integrity addressed through shallow, support jet grout columns and additional monitoring.
  - ❖ Difficult site conditions including breach fill debris, boulder, cobble, dense gravel, buried trees/wood. → High drilling deviation, corehole collapse, fluid loss, and schedule delay.

A HUGE THANK YOU TO OUR SPONSORS AND TEAM MEMBERS

- USACE, Sacramento District
- Stakeholders
  - ❖ Sacramento Area Flood Control Agency (SAFCA)
  - ❖ City of Sacramento Department of Utilities
  - ❖ State of California Department of Water Resources
  - ❖ Central Valley Flood Protection Board
- Consultants
  - ❖ Kleinfelder, Stantec
- Contractors
  - ❖ Maloney Odin, JV, Raito, Treviicos

QUESTIONS

U.S. Army Corps of Engineers
Sacramento District

Kleinfelder - Stantec
USACE | A JOINT VENTURE



PARALLEL SESSION B3
Golden Designs

Evaluation of Piles' Ultimate Capacity Utilizing Direct CPT Methods Considering Consolidation and Aging Setup

Murad Abu-Farsakh, Ph.D., P.E., F-ASCE
Isam Khasib, Ph.D. Student

Louisiana Transportation Research Center
Louisiana State University

Disclaimer: The opinions expressed in this presentation and on the following slides are solely those of the presenter(s) and not necessarily those of DFI. DFI does not guarantee the accuracy or reliability of the information nor endorse the recommendations offered by the presenter(s)

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LSU

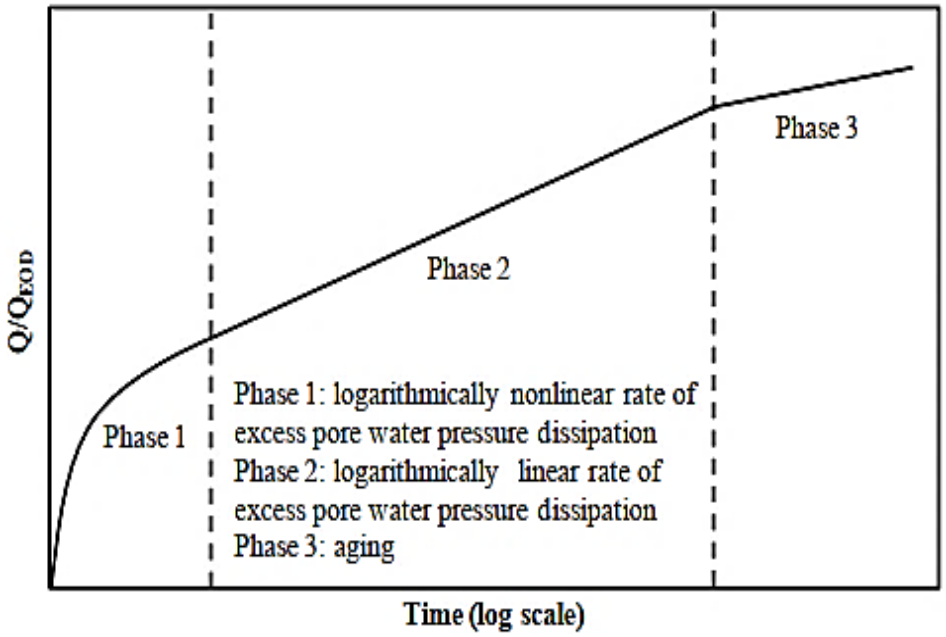
Objective of Study

- Evaluate the axial capacity of in-place piles for in-service bridge structures subjected to scour using available methods based on CPT/PCPT data.
- This imply evaluating the long-term increase in axial pile capacity due to consolidation setup and aging setup.

Brief Introduction

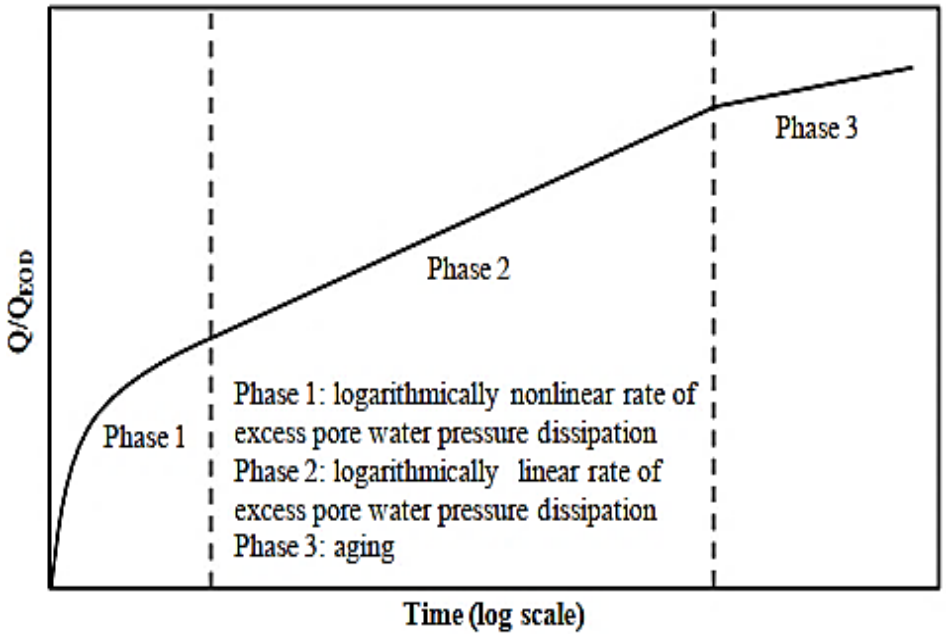
- Piles driven into granular or fine-grained soils usually exhibit an increase in capacity over time.
- This increase in pile resistance is known as pile "setup" or "freeze".
- Pile setup mechanism can be divided into three phases:

1. Logarithmically nonlinear part,
2. Logarithmically linear part,
3. Aging.



Brief Introduction

- **Phase 1**: due to the high disturbance and remolding of surrounding soil during pile driving. The rate of excess pore pressure (PWP) dissipation is nonlinear with respect to log time.
- **In Phase 2**: the rate of excess PWP dissipation becomes linear with respect to log time (consolidation setup).
- **Phase 3 (Aging)**: can be related to thixotropy, secondary compression, particle interference, and clay dispersion. During aging, the shear strength and stiffness of the surrounding soil increase, and the pile-soil interface angle increases.



Brief Introduction

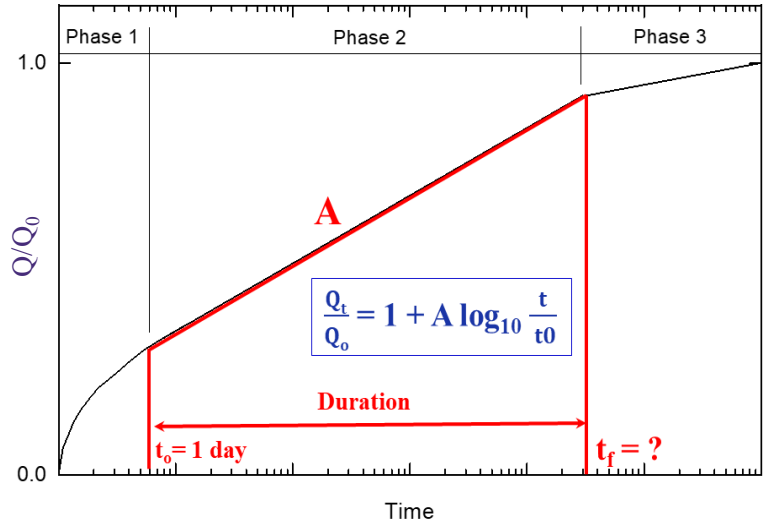
- The current practice in design of pile foundations in Louisiana (and other states) is based on conducting pile load tests at 14 days after pile driving, ignoring any pile setup after that, leading to a conservative pile design.
- For reuse of existing pile foundations (or to calculate capacity of in-place piles subjected to scour), it is important to know how to estimate the increase in pile resistance that could have been mobilized after decades of piles in place due to consolidation and aging setup.

Brief Introduction

- Most of previous studies on pile setup proposed the following correlation between pile setup and the elapsed time, t, for the linear logarithmic phase 2 (due to excess pore dissipation):

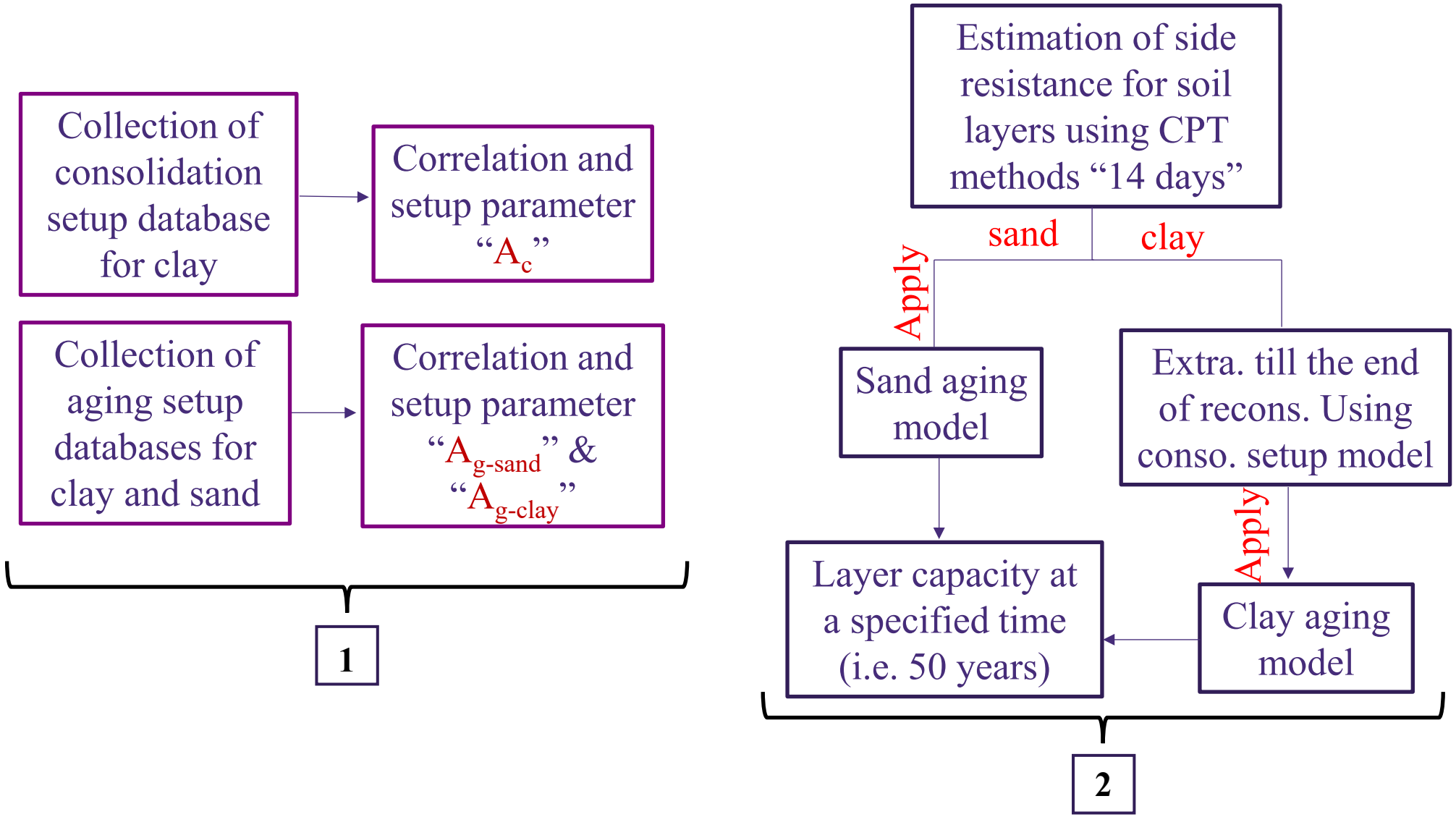
$$\frac{Q}{Q_0} = A \times \log\left(\frac{t}{t_0}\right) + 1$$

Where
Q/Q_0 = ratio of pile resistance at time t to pile resistance at the initial refer
t/t_0 = ratio of elapsed time to the initial reference time from the end of the
A = setup parameter.



- Different researchers proposed different setup A factors:
  - Skov and Denver (1988) suggested using A = 0.2 for sand and A = 0.6 for clay, with t_0 = 0.5 day for sand and t_0 = 1 day for clay.
  - Abu-Farsakh et al. (2016) suggested using A = 0.15 for sand and A = 0.31 for clay, with t_0 = 1 day

Methodology



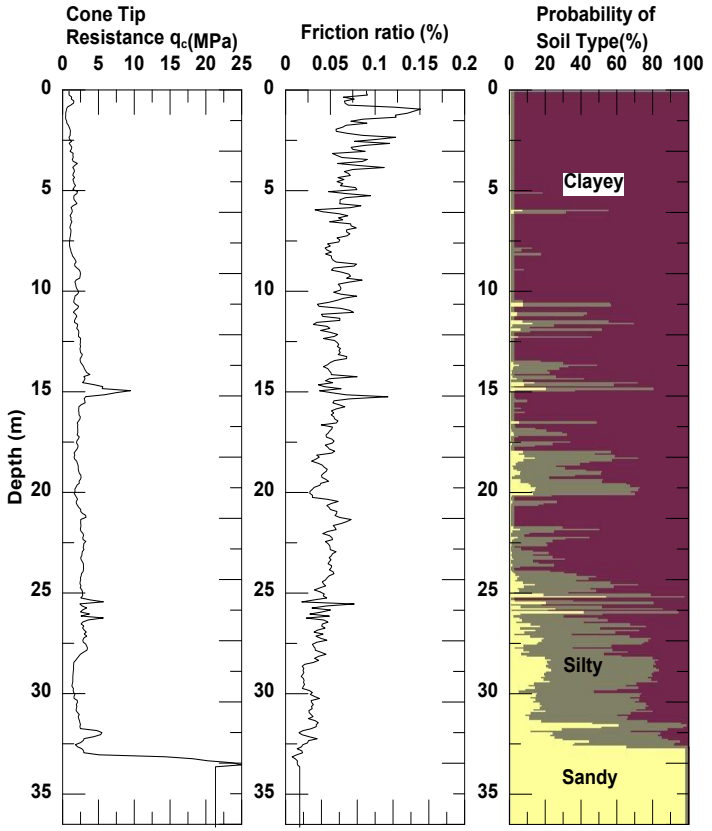
Database of Consolidation Setup

- An extensive study was conducted by Abu-Farsakh et al. (2016) using 10 instrumented test piles to estimate the magnitude of setup and the setup parameter, A, based on the increase of side resistance for individual soil layers.

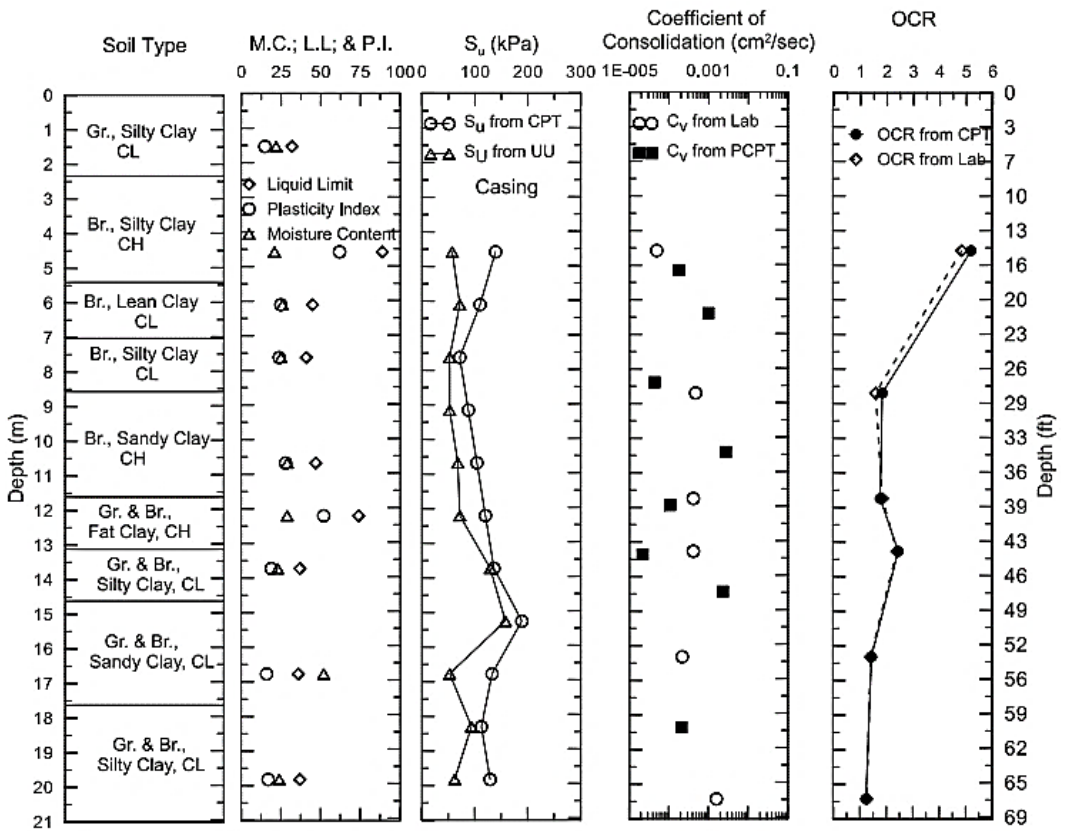
Project Name	Pile ID	Width, in (mm)	Embedded length, ft. (m)	Scour elevation, ft. (m)
Bayou Lacassine	BL-1	30 (762)	67 (20.4)	21 (6.4)
	BL-3	30 (762)	67 (20.4)	21 (6.4)
Bayou Zourie	BZ	24 (610)	51 (15.5)	20 (6.1)
Bayou Boeuf	BB3	30 (762)	130 (39.6)	6 (1.8)
LA-1	LA1-2	16 (406)	120 (36.6)	-
	LA1-3	30 (762)	180 (54.9)	70 (21.3)
	LA1-4a	24 (610)	150 (45.7)	-
	LA1-4b	24 (610)	201 (61.3)	-
	LA1-5a	24 (610)	139 (42.4)	-
	LA1-5b	24 (610)	160 (49.7)	-

Database of Consolidation Setup

- The subsurface soil conditions and soil properties for the four tested sites in Louisiana were evaluated using extensive laboratory and in-situ testing programs. The subsurface soil conditions are predominated by clay layers.



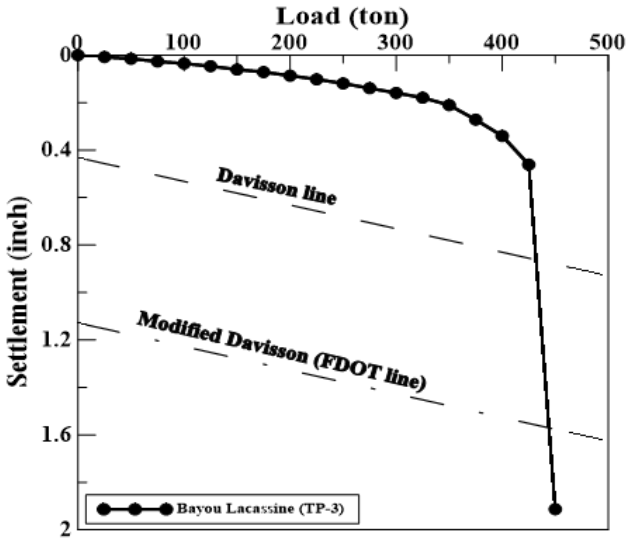
CPT data and CPT soil classification at Bayou Lacassine site - test pile BL3:



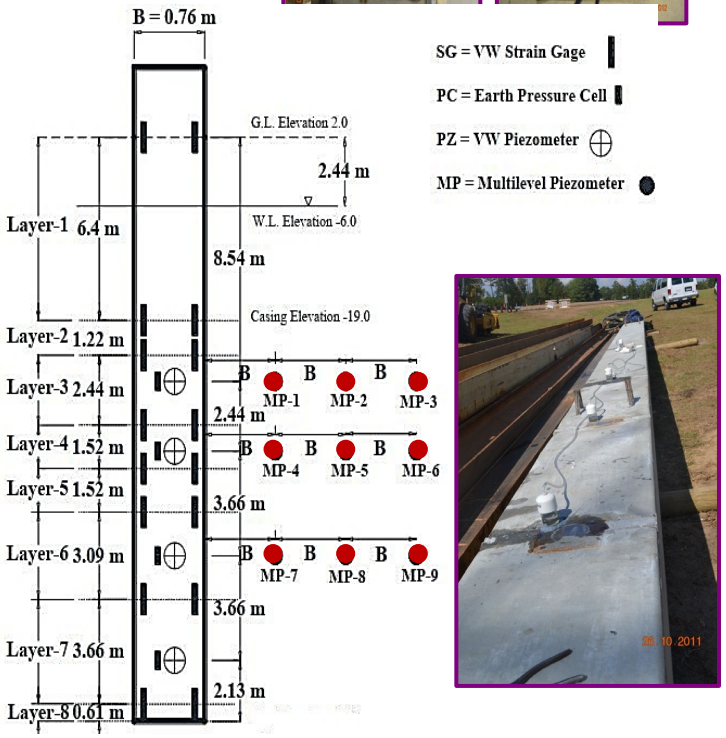
Soil profile and properties for the Bayou Lacassine site

Database of Consolidation Setup

- The 10 test piles were instrumented with strain gauges in pairs, earth pressure cells and piezometers on the pile face, and multi-level piezometers on the surrounding soil.
- The piezometers installed on the piles' faces and surrounding soils were used to measure the excess pore pressure with time.
- Davisson and modified Davisson (pile width > 24") interpolation criterion were used to estimate the ultimate resistance of instrumented piles from load-settlement curves.

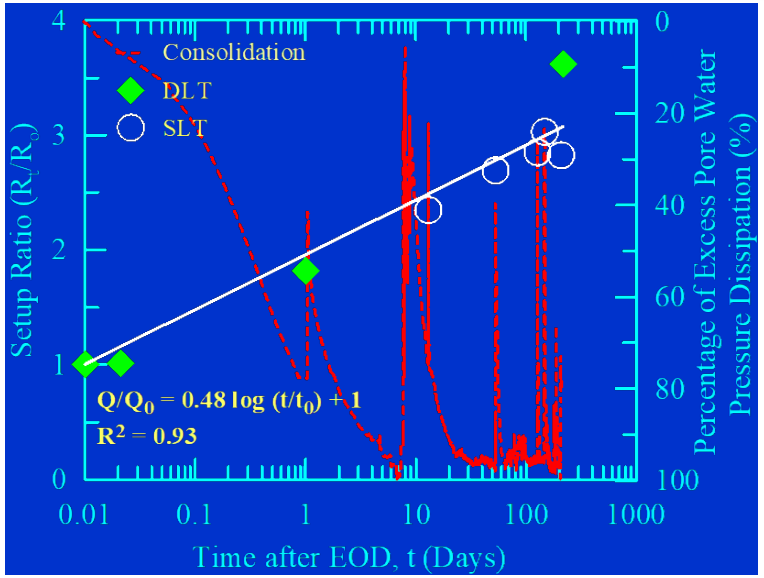


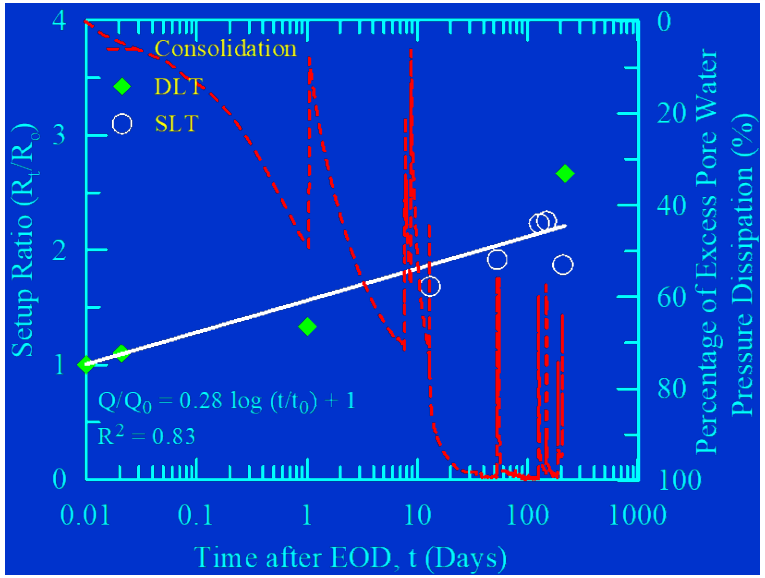
Davisson and Modified Davisson failure criteria for estimating ultimate pile resistance



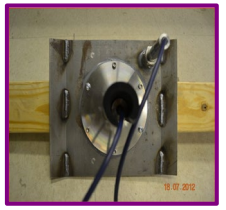
Instrumentation plan for test pile BL3 at Bayou Lacassine (Abu-Farsakh et al. 2016)

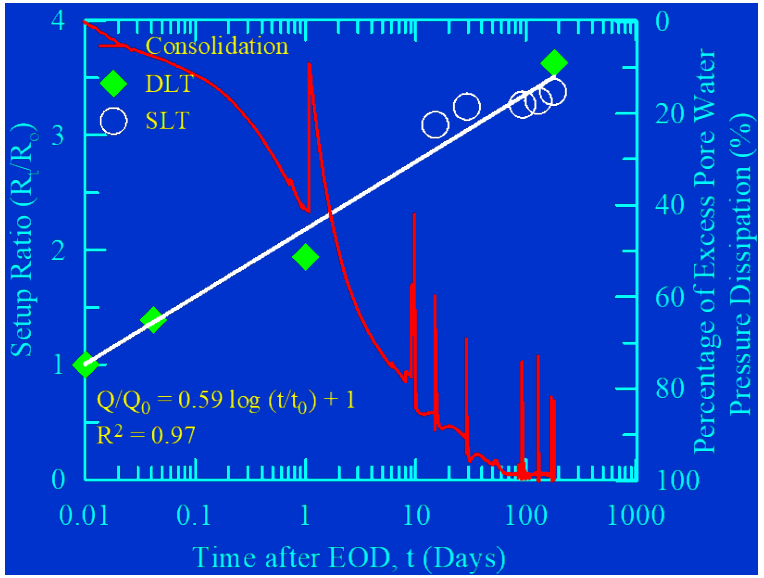
Database of Consolidation Setup

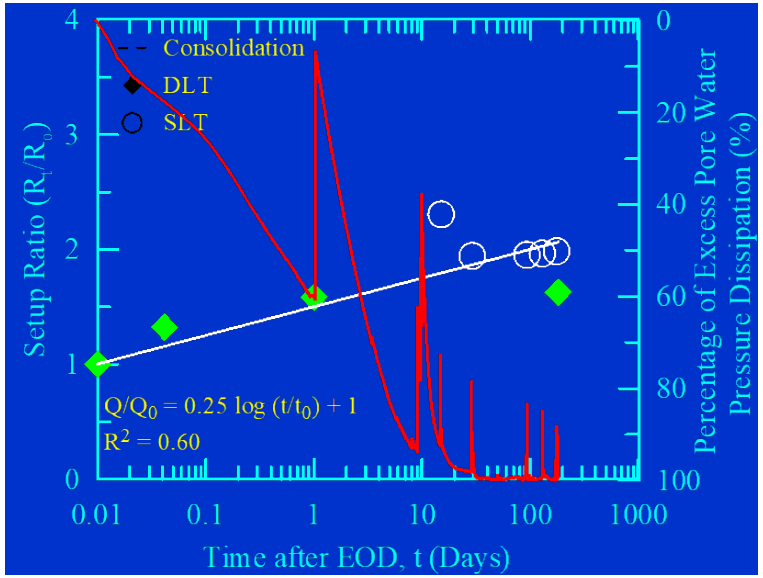








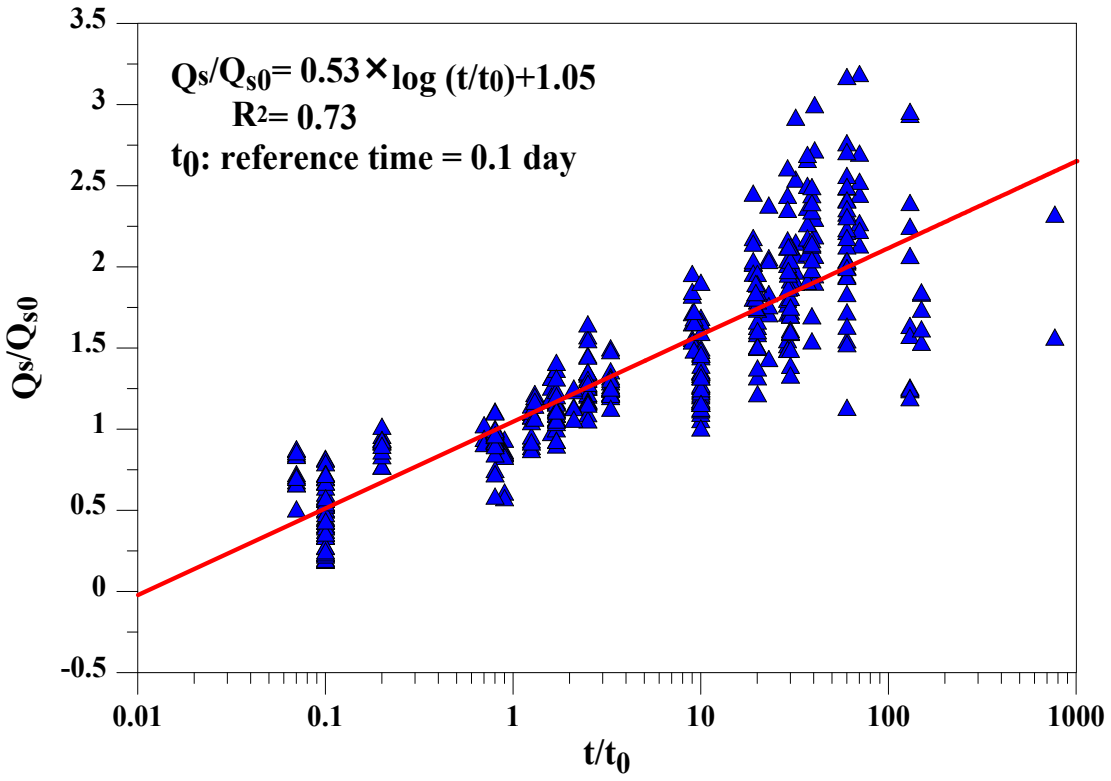






Database of Consolidation Setup

- The pile resistances were plotted versus log time, to evaluate the consolidation setup factor (A_c) for clay soil layers.
- The side resistances of individual soil layers (Q_s) at different times were normalized with respect to a reference resistance (Q_{s0}), which is the resistance measured at $t_0 = 0.1$ days.
- The average value of A_c is 0.53, which is within the range of 0.2 - 0.8 as reported by other researchers in literature.



Database of Aging Setup

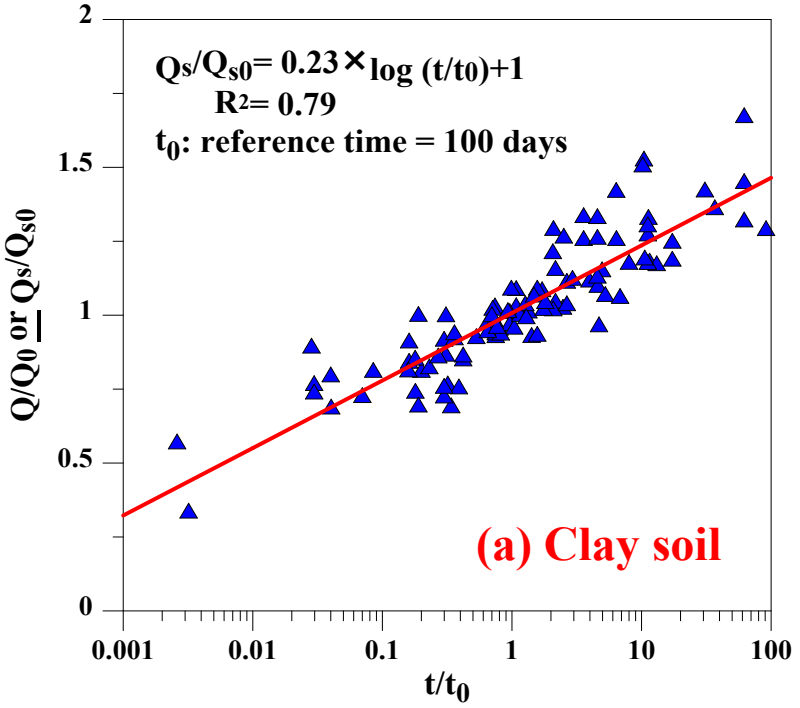
- A total of 17 case studies were collected (in Table) from literature in which pile aging was tested in sand and clay soils.
- Pile types: precast concrete, steel, or wood.
- Pile diameters: ranged from 4 in. to 30 in.
- Pile lengths: ranged between 16 ft. to 230 ft.

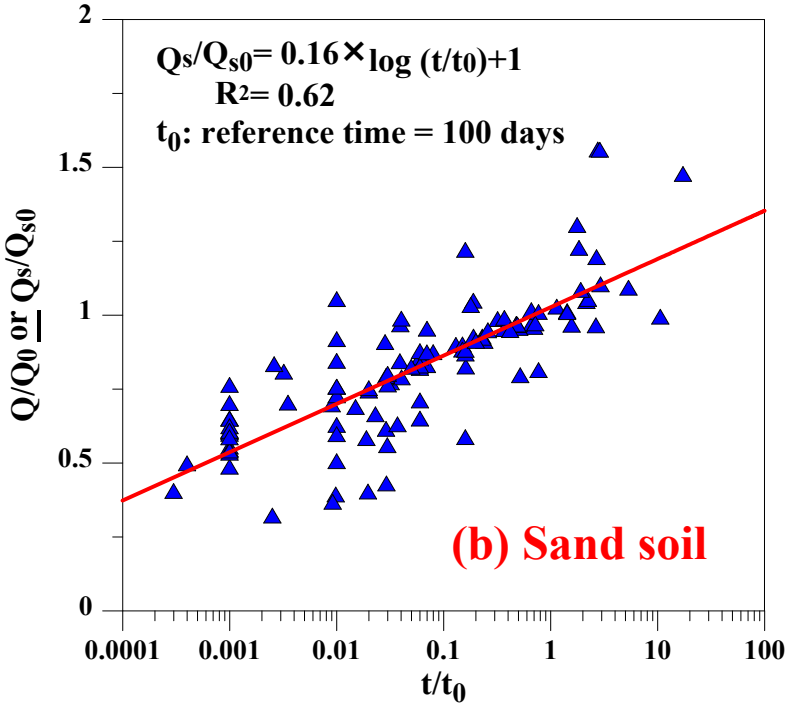
Site	Soil type	Pile material	Pile diameter/ width (inch)	# of pile load tests	Reference
Cowden	Clay	Steel/open ended	18	4	(43,23)
Canons park	Clay	Steel/closed ended	7	15	
Houston	Clay	Steel/closed ended	11	3	(44,45)
Nitsund	Clay	Timber	7	10	(46)
Drammen	Clay	Timber	10	3	(47)
Ska-Edeby	Clay	Timber	4	17	(48)
Belfast	Clay	PPC	10	7	(3)
Haga	Clay	Steel/closed ended	6	6	(49)
Bothkennar	Clay	PPC	4	2	(50)
Buckman Bridge	Sand	PPC	18	4	(31,17)*
Aucilla River	Sand & clay	PPC	18	10 clay; 6 sand	
Vilano Bridge East	Sand & clay	PPC	18	3 clay; 4 sand	
Vilano Bridge West	Sand & clay	PPC	18	6 clay; 4 sand	
Seabreeze Bridge	Sand & clay	PPC	18	5 clay; 6 sand	
West Delta	Clay	Steel/open ended	30	2	(51)
Bayou Zourie	Sand	PPC	24	2	(4)*
Bayou Lacassine	Clay	PPC	30	9	
Bayou Teche	Sand	PPC	24	3	
LA-1	Sand	PPC		25	
Alberta	Sand	H-Steel	HP (12.20× 3.1)	2	(52)
Hamburg	Sand	PPC	14	4	(1)
Alborg	Sand	PPC	10	3	
Stockholm	Sand	PPC	10	13	(11)
New York	Glacial Sand	Steel/monotube	14	30	(53)
Victoria	Sand	PPC	18	2	(54)
Dunkirk	Sand	Steel/open ended	12	4	(19)

*Instrumented piles were used with strain gauges. The sum of the resistance from clay (or sand) layers at different times was used

Database of Aging Setup

- The normalized aging database (after excess PWP dissipation) (Q_s/Q_{s0} or Q/Q_0) were plotted against the log t/t_0.
- The reference time used for both sand and clay soils is t_0 = 100 days. The value of A_g(clay) = 0.23, and A_g(sand) = 0.16.





Direct Pile-CPT Methods

- The 8 top-performed Pile-CPT methods evaluated in a previous study (Amirmojahedi & Abu-Farsakh, 2019) were first used to calculate the pile resistances.
- The methods were evaluated using a database of 80 pile load tests conducted 14 days after pile driving.
- The side resistance of each soil layer was extrapolated with time using the consolidation setup factor (A_c) until the completion of excess pore pressure dissipation.

Pile-CPT method	Calculations for determining q_b (unit end bearing) and f (unit shaft resistance)
Schmertmann	$q_b = (q_{c1} + q_{c2})/2 \leq 15\ MPa$, <u>In clay</u>: $f = k_c\ f_s \leq 120\ kPa$, <u>In sand:</u> $Q_S = K \sum_{d=0}^{8b} y\,(8D) f_S A_S + \sum_{D=8B}^{L} f_S A_S$ q_{c1}: minimum of the averages of qc values of zones ranging from 0.7D to 4D below tip; D:pile diameter; q_{c2}: average of minimum q_c values 8D above the cone tip; k_c= 0.2 – 1.25 for clay; K value depends on d/D ratio.
De Ruiter and Beringen	<u>In clay</u>: $q_b = N_c S_u \leq 15\ MPa$, <u>In sand</u>: q_b similar to Schmertmann *(66)*, <u>In clay</u>: $f = \alpha S_u \leq 120\ kPa$, <u>In sand</u>: $f = \min(f_s, q_{cd}/300, 120kPa)$ $S_u = q_{cd}/N_k$, N_c=9, N_k= 15 to 20; α=1 for NC clay and 0.5 for OC clay.
LCPC	$q_b = k_{b1} q_{eq}(tip)$, $f = q_{eq}(side)$ $k_{SI} \leq f_{max}$ k_{b1} = 0.15-0.60 depending on soil type and installation procedure; q_{eq}: equivalent average of qc values of zone ranging from 1.5D below pile tip to 1.5D above pile tip; k_{S1} = 30-150depends on soil type, pile type and installation procedure
UF	$q_b = k_b q_{ca}$ (tip), $f = \alpha\ q_{cs}/F_s \leq 120\ kPa$ k_b : depends on soil type = 0.4 for sand, 0.45 for silt and 1 for clay; as d 1.25 for PPC driven piles; FS = 50–300 depending on soil type.
Probabilistic	$q_b = k_b q_{eq} \leq 15$ MPa , $f = k_s\ f_s$ $k_b = 0.3$ Pr(sand)+ 0.4 Pr(silt)+ 0.5 Pr(clay); $k_s = k_{s(silt\text{-}clay)}$ Pr(silt-clay) + $k_{s\ (silt\text{-}clay)}$ $k_{s(sand)} = 1/(0.61+0.7fs)$
Philipponnat	$q_b = k_b q_{eq}$ (tip), $f = \alpha\ q_{c\ (side)}/F_s$ k_b : depends on soil type = 0.4 for sand, 0.45 for silt and 0.5 for clay; α_3 = 1.25 for PPCdriven piles; F_S = 50-200 depending on soil type.
Eurocode 7	$q_b = \alpha_p\ \beta\ s\ q_{eq} \leq 15$ MPa α_p, β and s values are 1.0 for precast concrete piles. $f = \alpha_s\ q_c$; α_s depends on soil type $0.02 \leq \alpha_s \leq 0.03$ for cohesive soil; $0.005 \leq \alpha_s \leq 0.01$ for non-cohesive soil depend on pile type
ERTC3	q_b is calculated exactly similar to Eurocode 7 method $f = \alpha_s\ q_c$; α_s depends on soil type $0.003 \leq \alpha_s \leq 0.01$ for Non-cohesive soil; $0.015 \leq \alpha_s \leq 0.055$ for cohesive soil

Direct Pile-CPT Methods

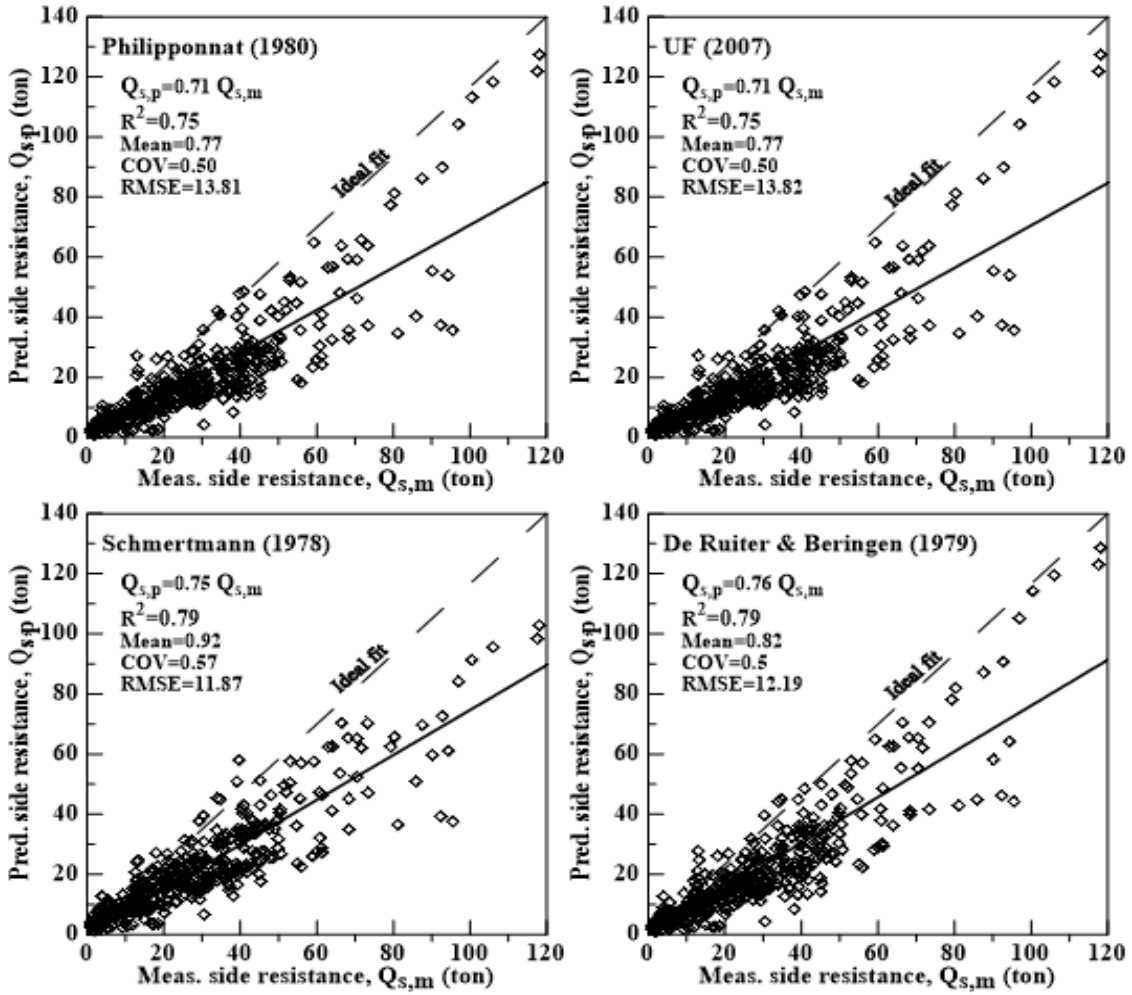
- Statistical analyses of predicted/measured side capacities ($Q_{s,p}/Q_{s,m}$) (μ, COV) were first calculated for the 8 Pile-CPT methods for the clay soil layers at 14 days.

Pile-CPT Method	$Q_{s,p}/Q_{s,m}$ For 10 instrumented piles	
	μ	COV
Philipponnat	0.74	0.48
De Ruiter	0.78	0.47
Schmertmann	0.85	0.44
LCPC	0.74	0.49
UF	0.73	0.48
Probabilistic	0.78	0.46
Eurocode7	0.71	0.46
ERTC3	0.73	0.64

Pile-CPT Method	Q_p/Q_m For 80 pile load tests	
	μ	COV
Philipponnat	1.02	0.37
De Ruiter	0.95	0.36
Schmertmann	1.21	0.35
LCPC	1.07	0.39
UF	1.04	0.35
Probabilistic	1.03	0.33
Eurocode7	1.17	0.48
ERTC3	1.08	0.35

Estimating the Resistance of Clay Layers within Consolidation Setup using Pile-CPT Methods

- The estimated side resistances of clay layers using the 8 Pile-CPT methods (14 days) were extrapolated using the A_c factor until the completion of excess pore pressure dissipation,
- The results were compared with the measured resistances from pile load tests at end of consolidation setup.
- The comparison shows that the combination of the direct Pile-CPT methods and consolidation setup model have fair estimations of the measured side resistance with time.



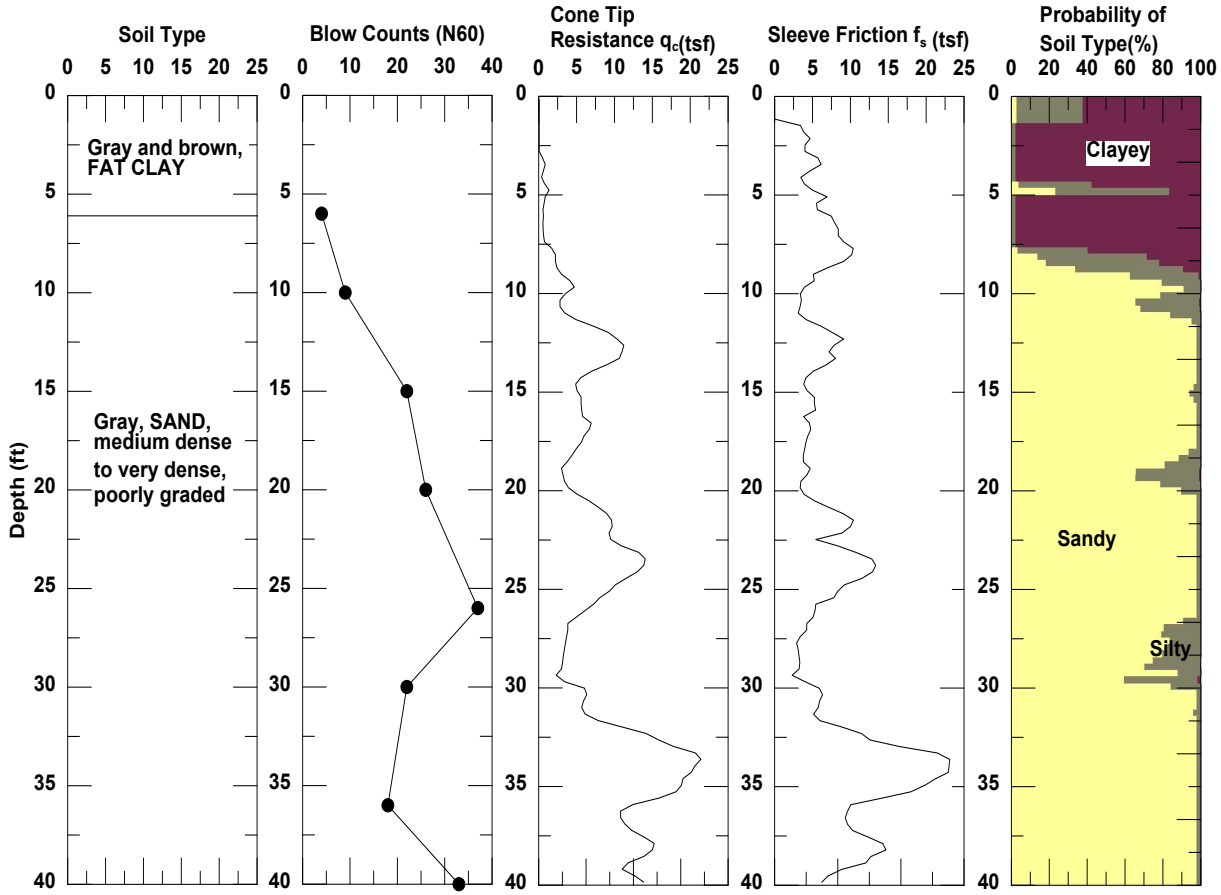
Estimation of Side Resistance Considering Aging Setup

- A study conducted by Missouri Department of Transportation (MDOT) was used to evaluate the 8 Pile-CPT methods with the aging model.
- The MDOT evaluated the axial resistance of in-place piles of a bridge built in the 1960s and was replaced in 2017 (the UU bridge). The UU Bridge was supported by octagonal precast concrete piles.
- Five piles were tested on the UU bridge (4 vertical and 1 battered).

Pile #	Embedded depth (ft.)	Pile inclination	Type of load testing	Side resistance (ton) / end-bearing resistance (ton), static test result (ton)
1	21.3	Vertical	Dynamic, Static	93.5 / 30, (**>134**)
2	30.1	Vertical	Dynamic	132.5 / 35
3	30.2	Vertical	Dynamic	118 / 32.5
4	23.5	Battered	Dynamic	90.5 / 32.5
5	25.1	Vertical	Dynamic	72 /31

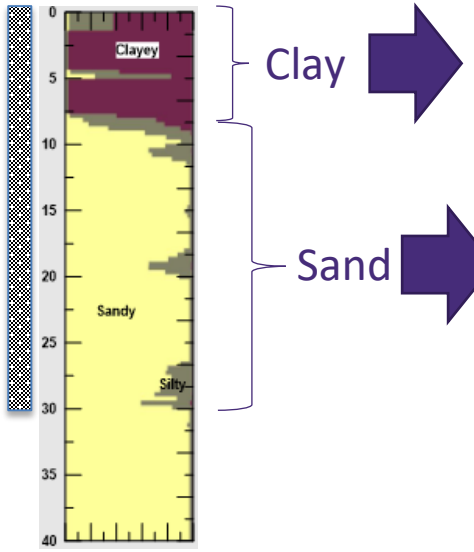
Estimation of Side Resistance Considering Aging Setup

- The subsurface soil conditions at the UU Bridge consist of high plasticity soft clays on the top 5 to 8 ft. underlain by medium dense to very dense poorly graded sand to a depth of 66 ft.
- The soil boring was conducted at one embankment side while the CPT test was performed at the other side.



Estimation of Side Resistance Considering Aging Setup

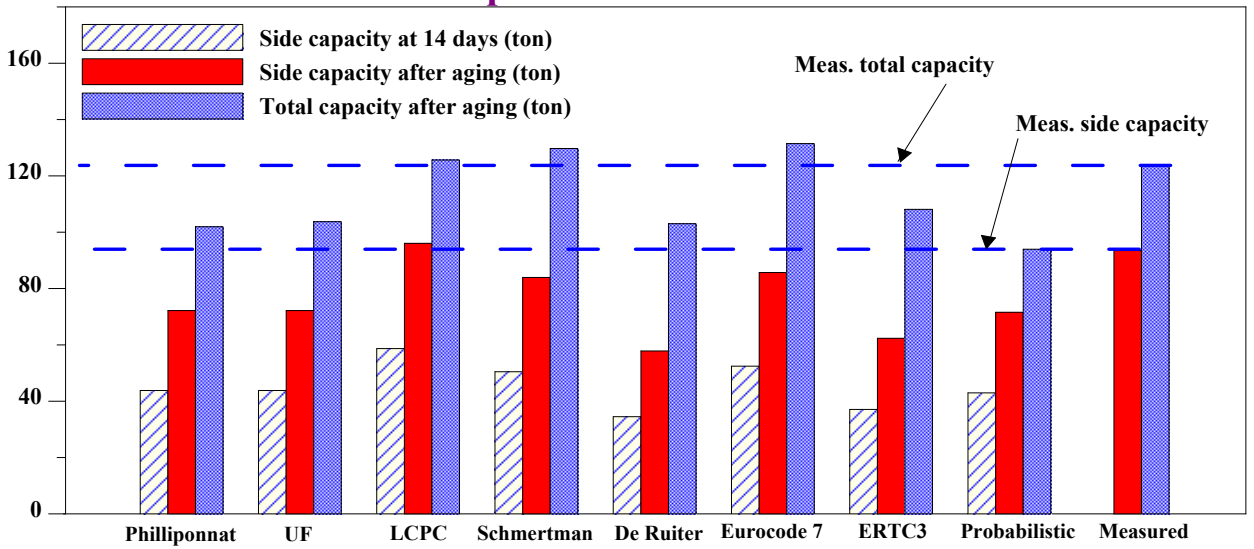
- The 8 top-performed direct Pile-CPT methods were used to estimate the axial resistance of these piles.
- The Pile-CPT methods were first applied to estimate pile capacities at 14 days and then the aging factor (A_g) was applied to extrapolate the pile capacities to 50 years.



Clay → **Resis. at 14 days + Clay aging model**

Sand → **Resis. at 14 days + Sand aging model**

Predicted resistance of pile #1 based on the 8 direct-CPT methods



Statistical analyses of predicted versus measured capacities

CPT method+ Aging model	$Q_{s,p}/Q_{s,m}$			Q_p/Q_m		
	μ	COV	RMSE	μ	COV	RMSE
Philipponnat	1.01	0.24	17.72	0.97	0.13	15.09
UF	1.01	0.24	17.61	0.98	0.12	13.59
LCPC	1.29	0.28	31.87	1.20	0.21	30.29
Schmertman	1.12	0.26	19.71	1.12	0.15	19.44
De Ruiter	0.76	0.16	30.72	0.85	0.09	25.84
Eurocode 7	1.22	0.29	27.01	1.19	0.16	27.89
ERTC3	0.84	0.20	25.08	0.91	0.11	19.65
Probabilistic	0.95	0.21	17.66	0.89	0.13	21.76

Conclusions

- In clay, the consolidation setup parameter (A_c) was 0.53, and for aging, the setup parameter (A_g) was 0.23. In sand, A_g was 0.16.
- The resistances estimated before and after the completion of excess porewater pressure dissipation indicates that significant aging takes place after consolidation setup.
- The use of direct Pile-CPT methods in combination with consolidation setup tend to underestimate the side resistance of clay layers up to the end of consolidation setup. The mean, μ, of predicted/measured side resistance ($Q_{s,p}/Q_{s,m}$) ranges from 0.71 to 0.85.

Conclusions

- The statistical results of predicted/measured side resistance ($Q_{s,p}/Q_{s,m}$) using the combination of Pile-CPT methods and aging setup show that the mean, μ, of $Q_{s,p}/Q_{s,m}$ ranges from 0.76 (De Ruiter) to 1.29 (LCPC); while the mean, μ, of predicted/measured total resistance (Q_p/Q_m) ranges from 0.85 (De Ruiter) to 1.20 (LCPC).
- The Philipponnat and UF methods demonstrated the most accurate estimations of total resistance as compared to other Pile-CPT methods.
- Incorporating the aging models into the Pile-CPT methods significantly enhanced their accuracy.

THANK YOU



Questions ?



Probabilistic Analysis of Laterally Loaded Piles using Commercial Finite Difference Software

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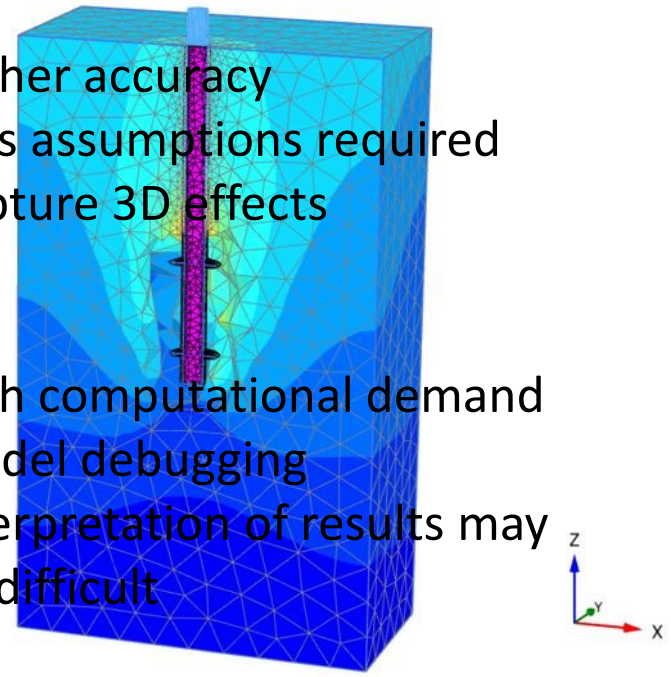
Outline

- Introduction
- Project Overview, Purpose, and Objectives
- Development of Computer Program (AllPile Randomizer)
- Parametric Study
- Practical Example – Pile Tip Deflection Calibration
- Conclusion

Introduction: Common Analysis Methods

Pros:
- Higher accuracy
- Less assumptions required
- Capture 3D effects

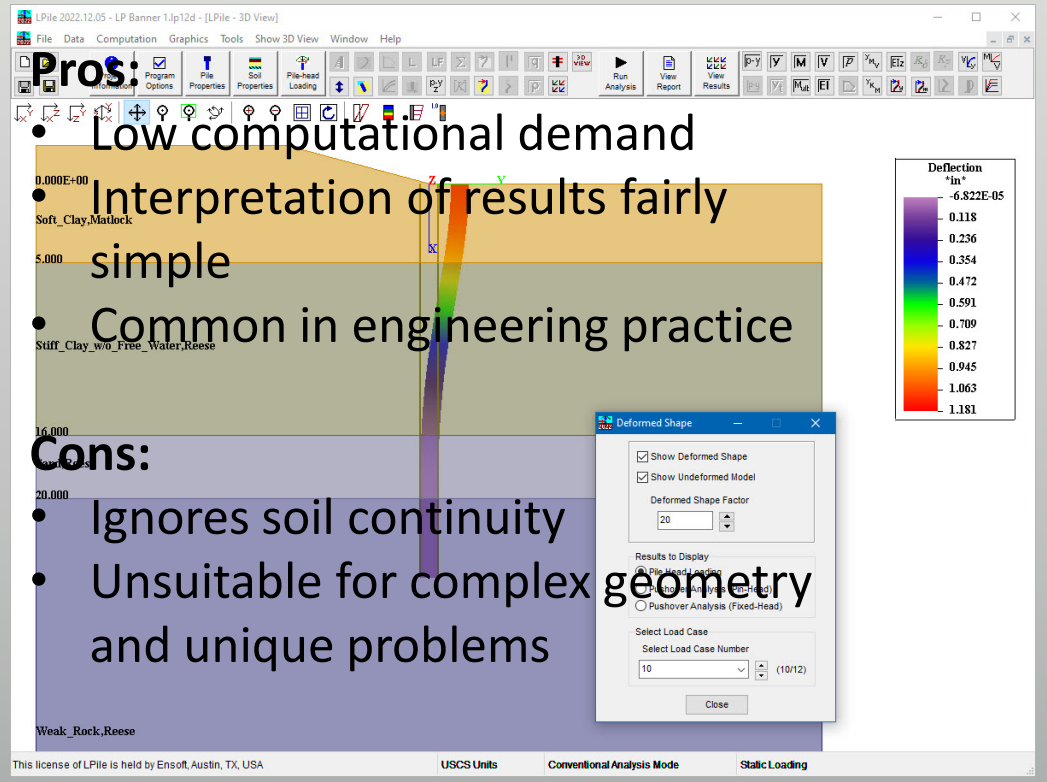
Cons:
- High computational demand
- Model debugging
- Interpretation of results may be difficult



1. Continuum (FE) Approach

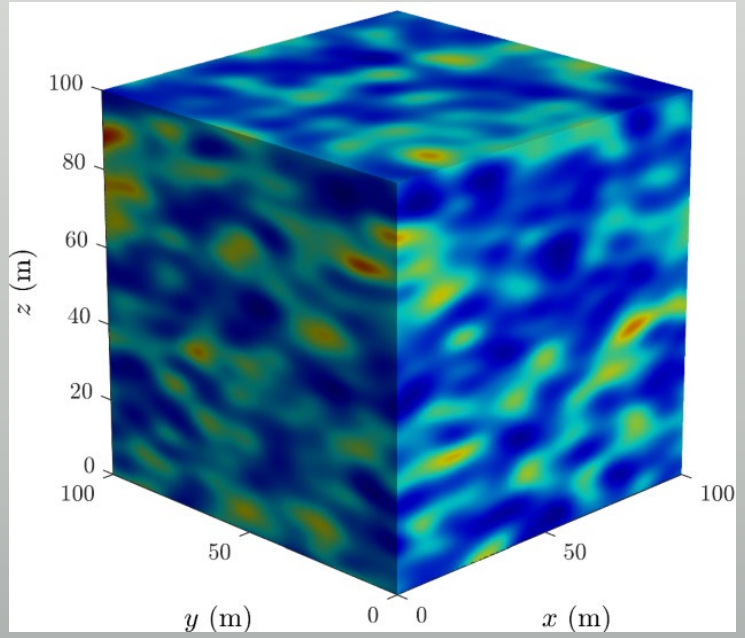
Pros:
- Low computational demand
- Interpretation of results fairly simple
- Common in engineering practice

Cons:
- Ignores soil continuity
- Unsuitable for complex geometry and unique problems

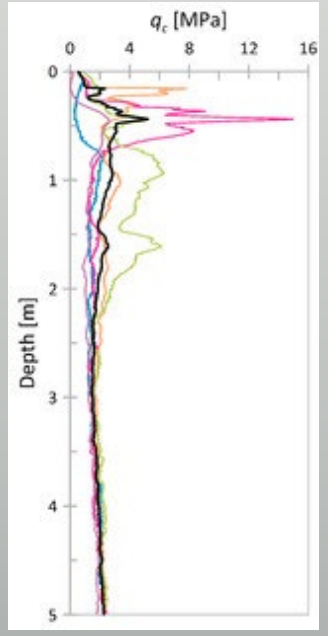


2. Finite Difference (p-y curves)

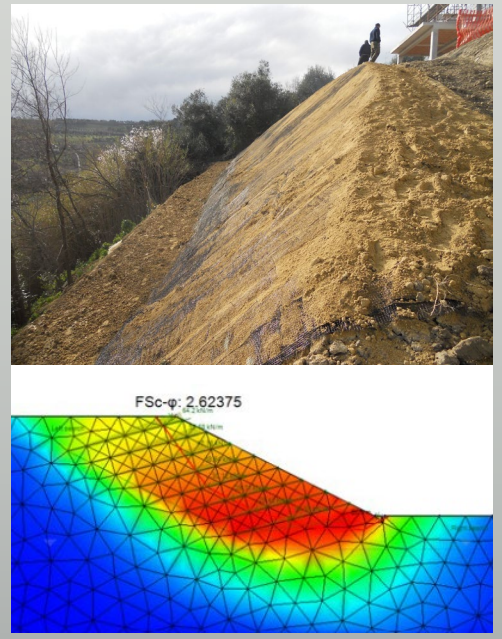
Introduction: Uncertainty of Soil



Spatial Variability



Measurement Uncertainty



Model Uncertainty

$$\phi' \approx 27.5 + 9.2 \times \log_{10}[(N_1)_{60}]$$
$$\phi' \approx \tan^{-1}[0.1 + 0.38 \times \log_{10}(q_t/\sigma'_v)]$$

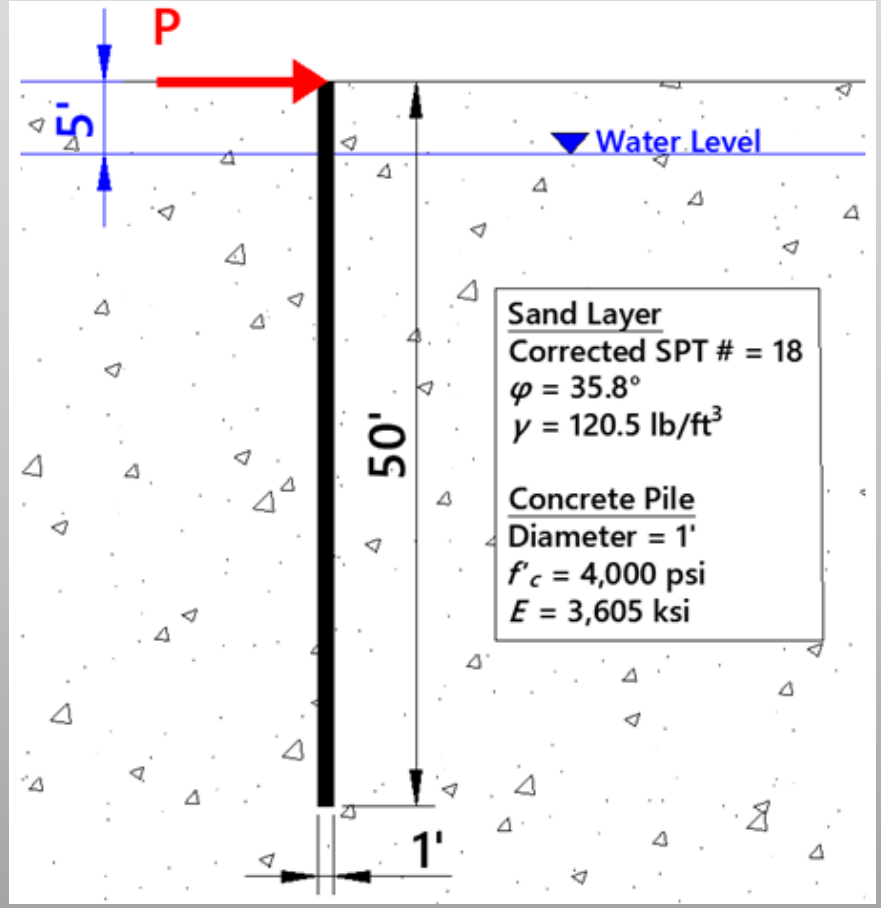
$$\phi' \approx 17.6 + 11 \times \log_{10}(q_{t1})$$

$$\phi' \approx \phi_{cv} + 3 \times (D_r[10 - \ln(p'_f)] - 1)$$
$$\phi' \approx \phi_{cv} + 3 \times (D_r[8.3 - \ln(p'_f)] - 0.69)$$
$$\phi' \approx \sqrt{15.4 \cdot (N_1)_{60}} + 20$$

Transformation Uncertainty

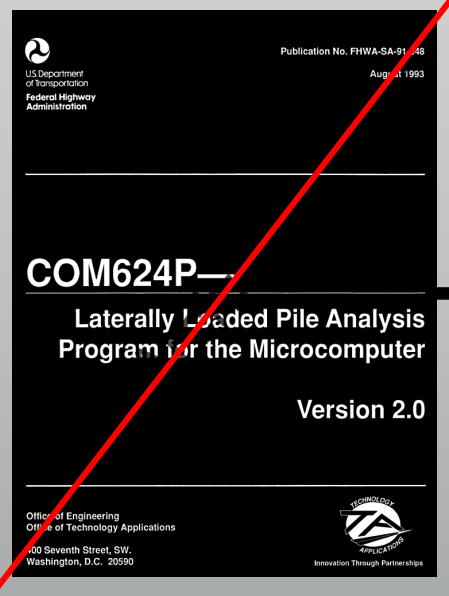
Project Overview, Purpose, and Objectives

- Conduct probabilistic analysis (Monte Carlo simulation) of laterally loaded piles using commercial finite difference software.
- Study the effect of varying water table and soil properties on the safety of a 50’ concrete pile embedded in sand.
- Develop a user-friendly tool that practitioners and researchers can use to conduct probabilistic analysis of laterally loaded piles.



Development of Allpile Randomizer
Background

No longer freely available



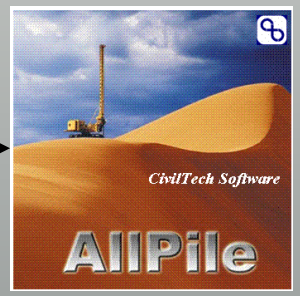
Initial plan

Assign random soil properties to COM624 input files

Commercial Software

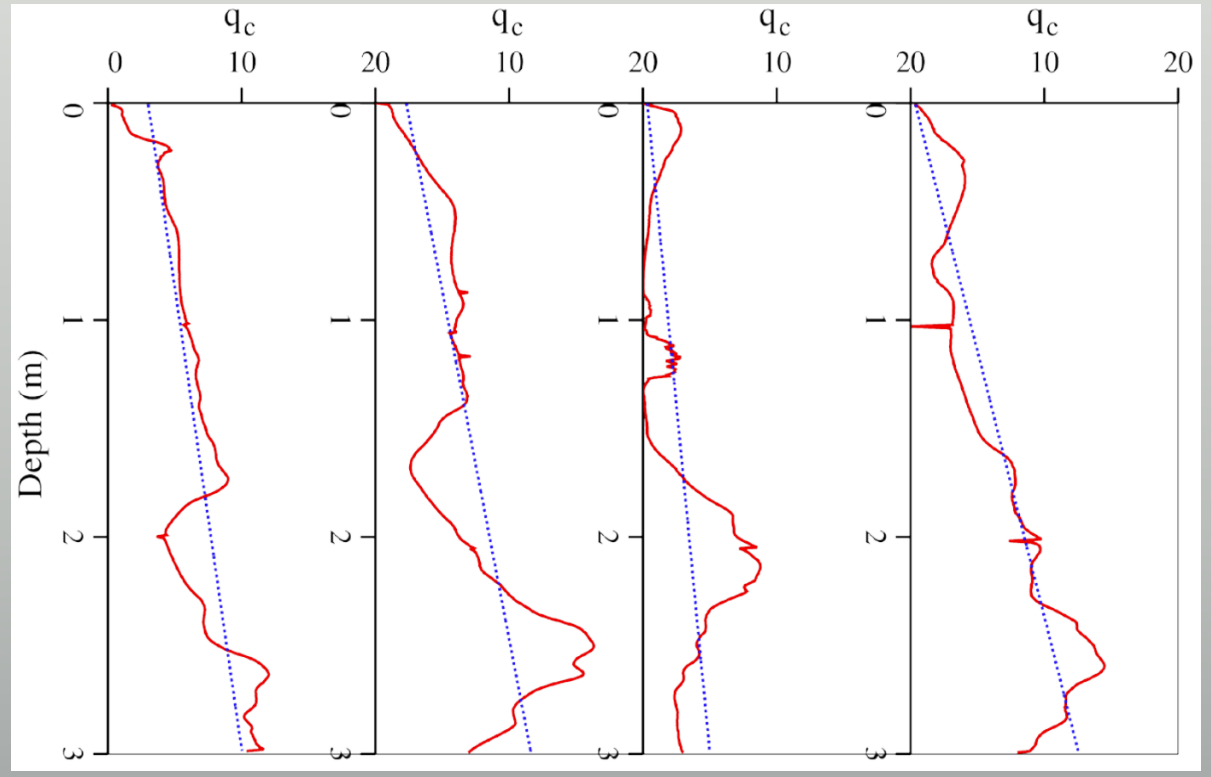


- No method to run LPILE from command line



- Main engine consists of a solver called all6.exe
- Solver does not require license
- COM624 input file must be called 1six24l.txt

Development of Allpile Randomizer
Background cont.



Initial plan

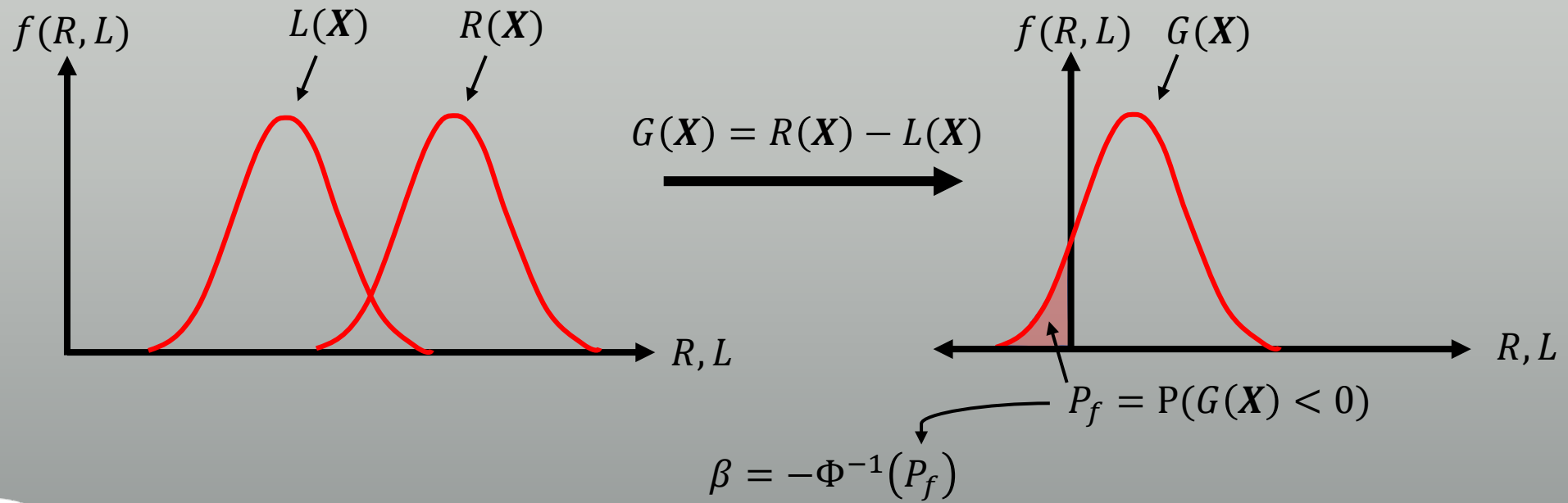
Assign spatially varying material properties through random fields

- Each random field realization requires soil layer definition
- High variability = High number of soil layers
- AllPile only allows a max of 10 soil layers which is insufficient to represent a random field with a high degree of fluctuation.

Random variable approach with no spatial distribution

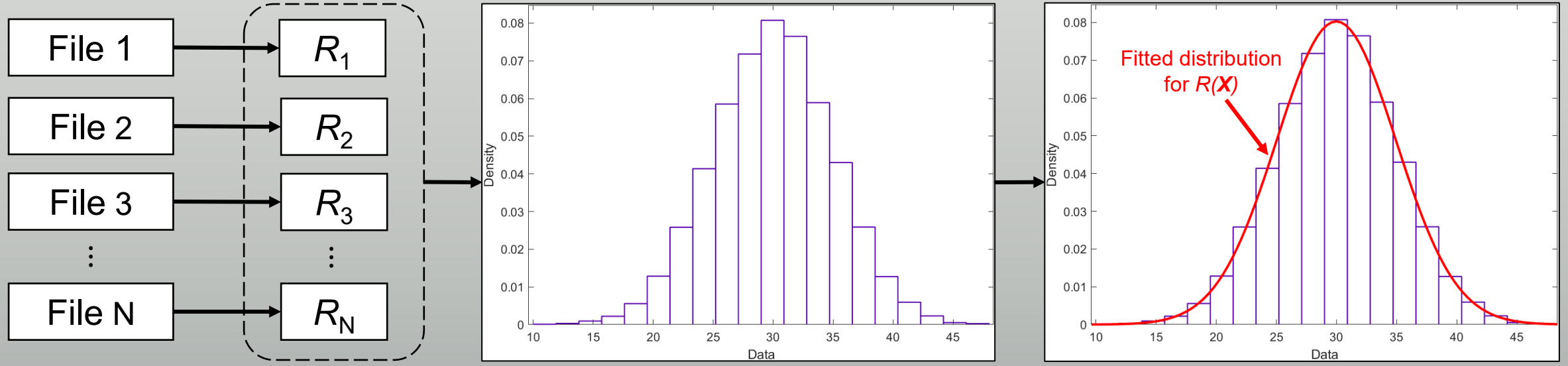
Development of Allpile Randomizer
Reliability analysis using Monte Carlo simulation

- **Reliability index, β** – measure of safety directly related to probability of failure, P_f.
  - Probability of failure can also be defined as probability of unsatisfactory performance
- **Limit state or performance function**, $G(\boldsymbol{X})$ – mathematical representation of limit state, typically using a resistance model, $R(\boldsymbol{X})$, and load model, $L(\boldsymbol{X})$.



where Φ^{-1} = inverse of normal cumulative distribution function

Development of Allpile Randomizer
Reliability analysis using Monte Carlo simulation cont.

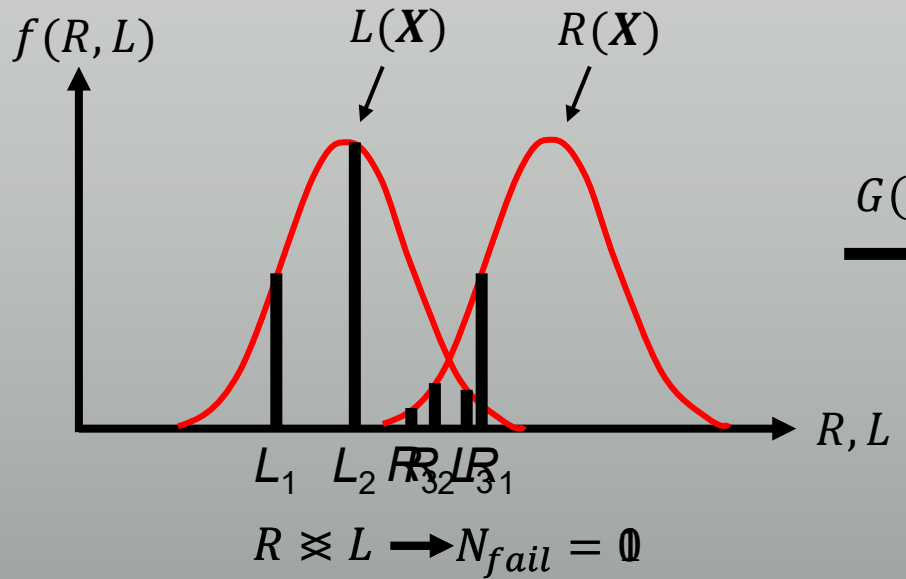


1. Generate files **2. Run and extract loads at failure** **3. Plot histogram** **4. Fit distribution**

*L(**X**)* can be established based on known statistical distribution and parameters or based on loads that correspond to a specific response (pile displacement, rotation, etc.) before failure.

Development of Allpile Randomizer
Reliability analysis using Monte Carlo simulation cont.

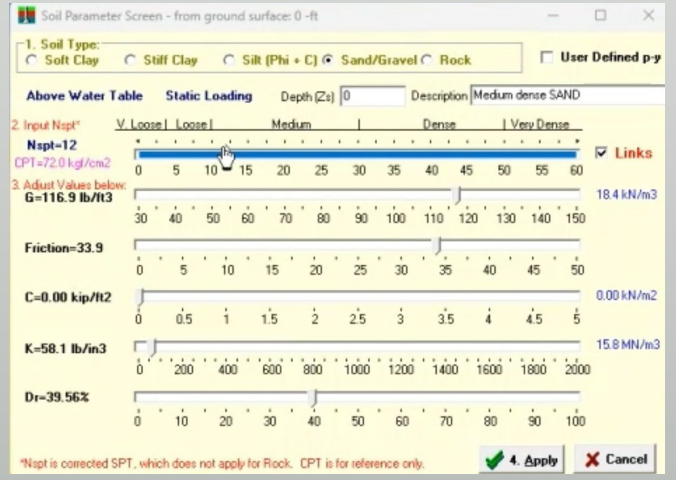
Randomly generate samples using established distributions, $L(\boldsymbol{X})$ and $R(\boldsymbol{X})$, and evaluate performance function, $G(\boldsymbol{X})$.

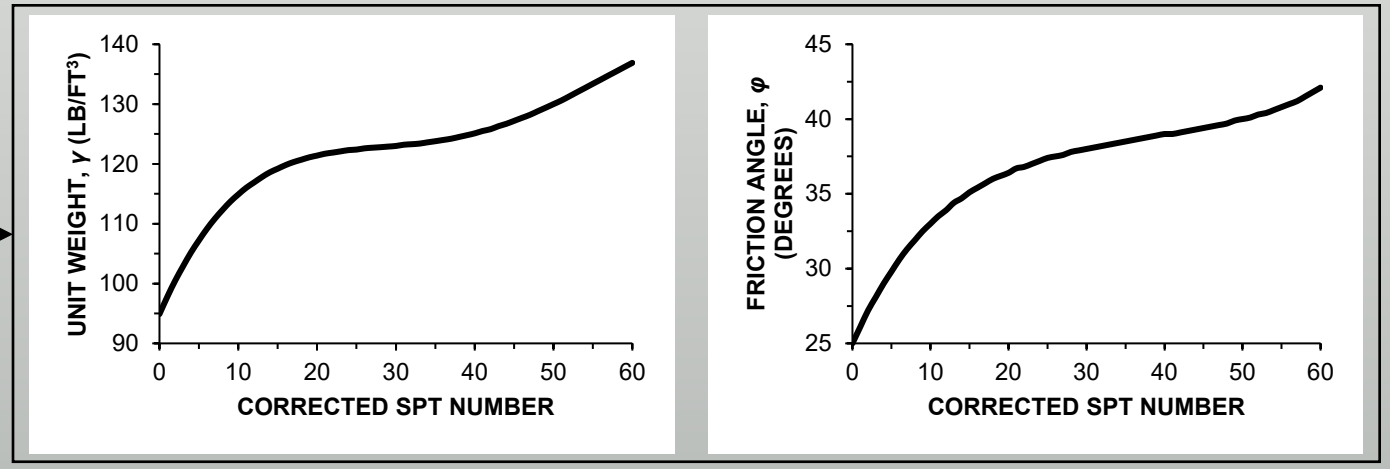


$$G(\boldsymbol{X}) = R(\boldsymbol{X}) - L(\boldsymbol{X}) \rightarrow P_f = \mathrm{P}(G(\boldsymbol{X}) < 0) = \frac{N_{fail}}{N_{trials}} \rightarrow \beta = -\Phi^{-1}(P_f)$$

Typically, in the order of **hundreds of thousands** to **millions** of trials. Number dependent on expected probability of failure.

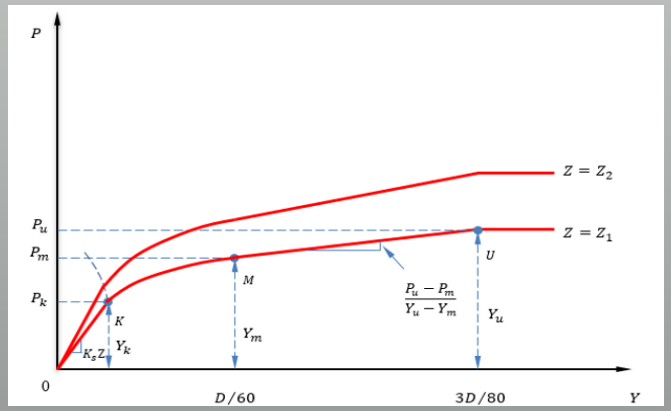
Development of Allpile Randomizer
Mapping SPT for Sand/Gravel





Nspt or $(N_1)_{60}$
Corrected for effect of overburden stress →

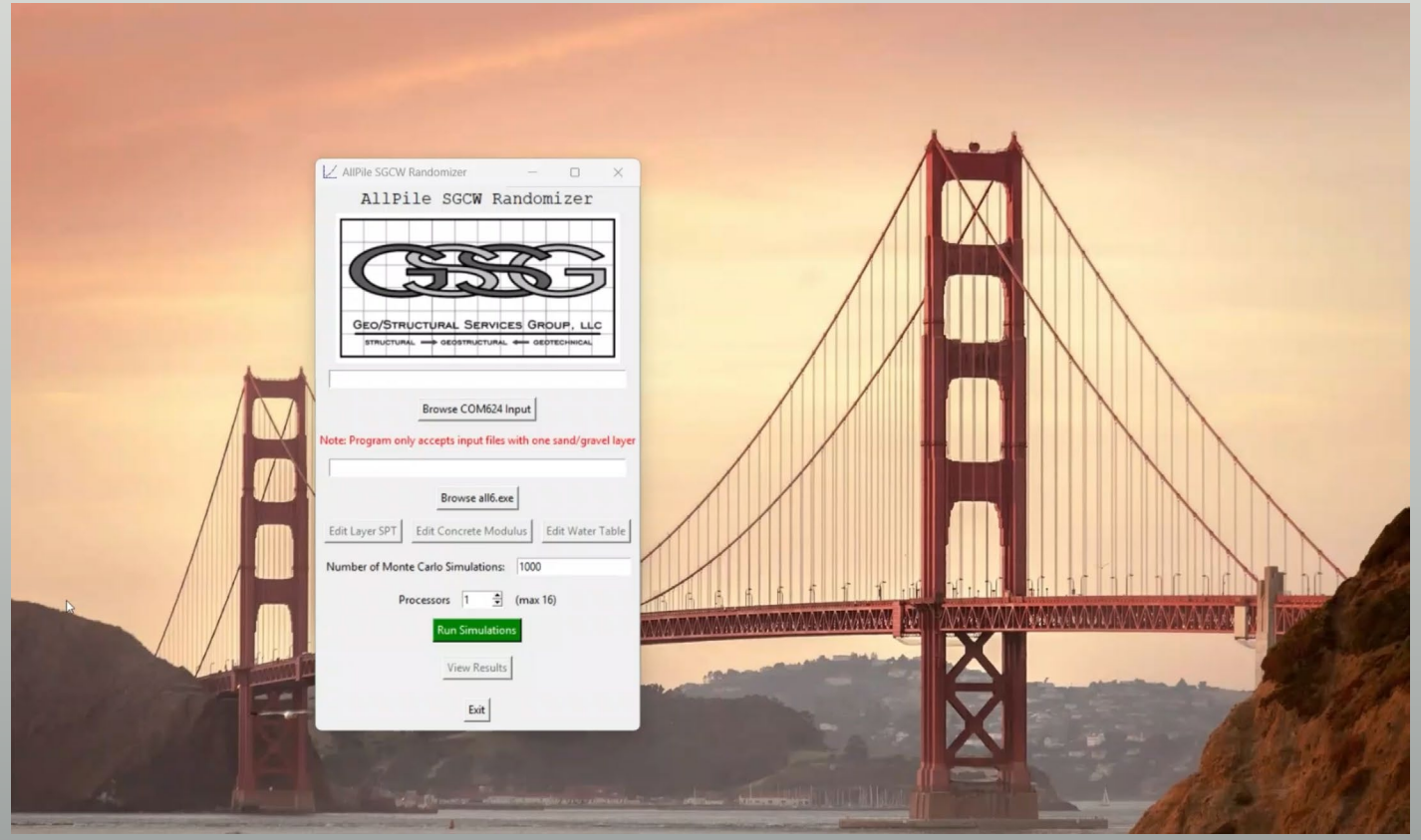
1. Unit weight
2. Friction angle
3. Subgrade modulus ← Low influence on variation
4. Relative density ← Does not affect p-y curve



Reese et al. (1974)

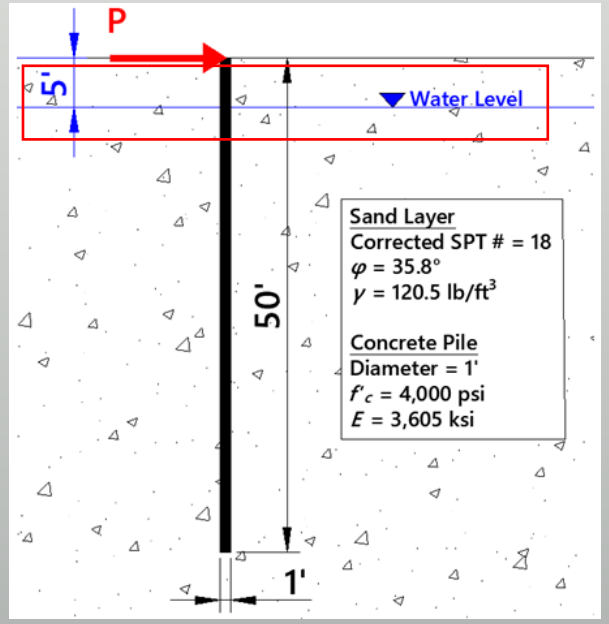
Development of Allpile Randomizer

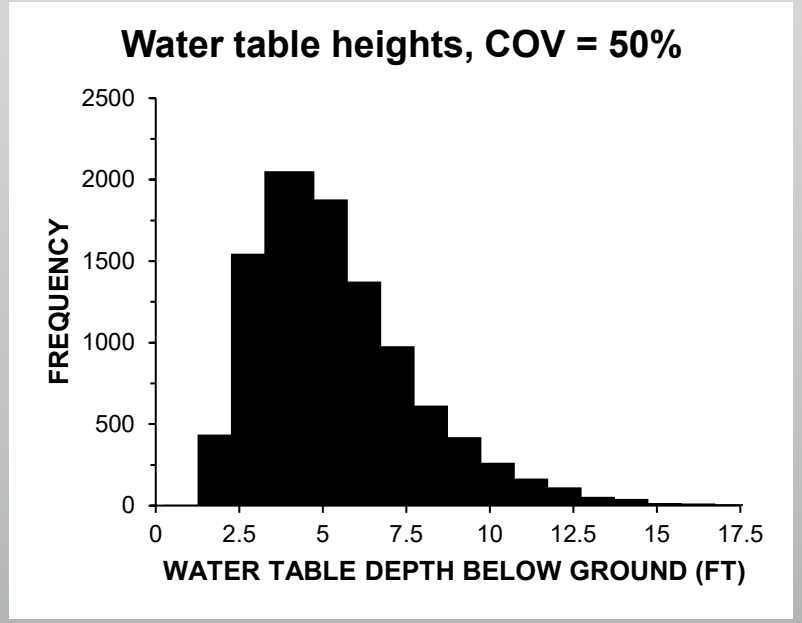
Overview

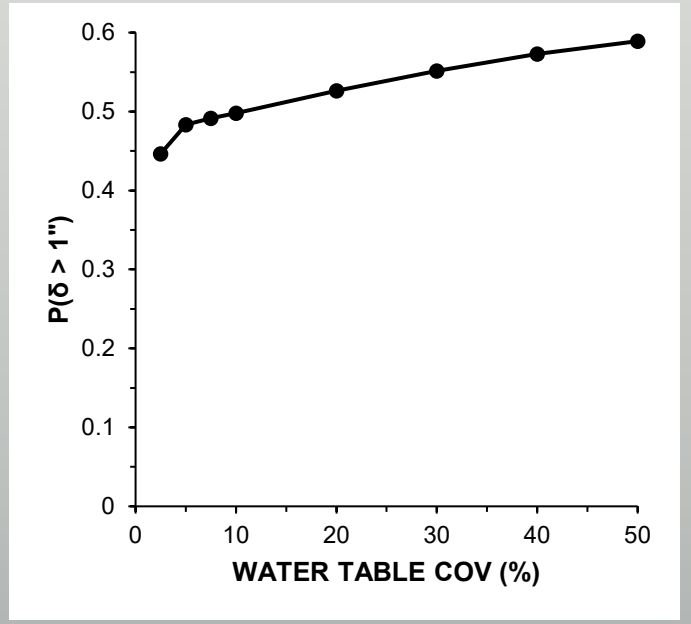


1. Define COM624 input file path
2. Define all6.exe path
3. Define random or deterministic properties for SPT number, pile elastic modulus, and water table
4. Define number of Monte Carlo simulations/trials
5. Define number of processors
6. Run simulations
7. View results (optional)

Parametric Study
Influence of varying water table

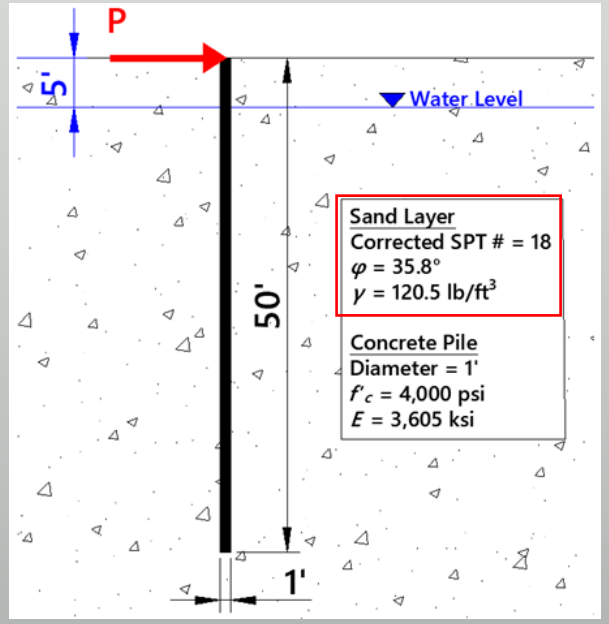


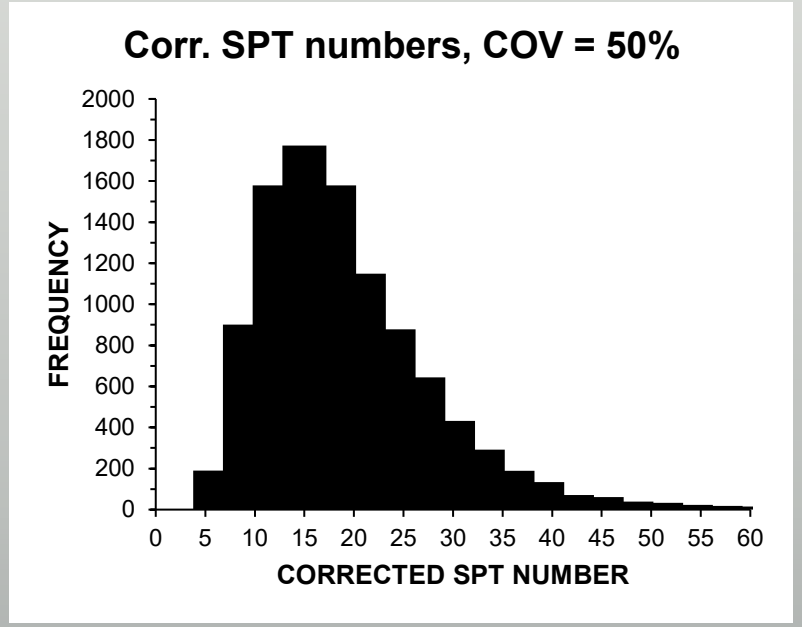


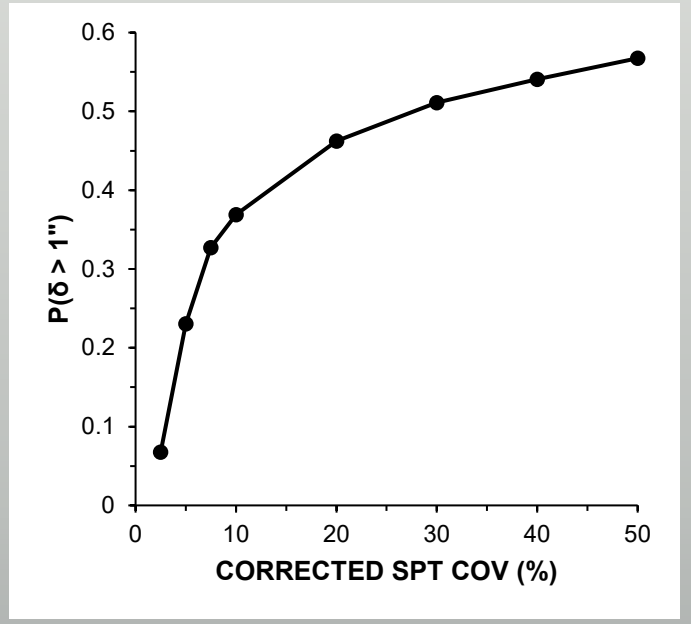


- Lognormally distributed water table with mean of 5 ft and COV values ranging from 2.5% to 50%.
- All other properties kept deterministic (load, soil properties, concrete properties).
- Varying water table did not have a strong influence on the probability of failure.

Parametric Study
Influence of varying soil properties







- Lognormally distributed corrected SPT with mean of 18 and COV ranging from 2.5% to 50%.
- All other properties kept deterministic (load, water table, concrete properties).
- Varying has a strong influence on the probability of failure, especially for COV values below 20%.

Practical Example – Pile Tip Deflection Calibration

Corrected SPT (Lognormally Distributed)

Mean = 18

COV* = 40%

Water Table (Lognormally Distributed)

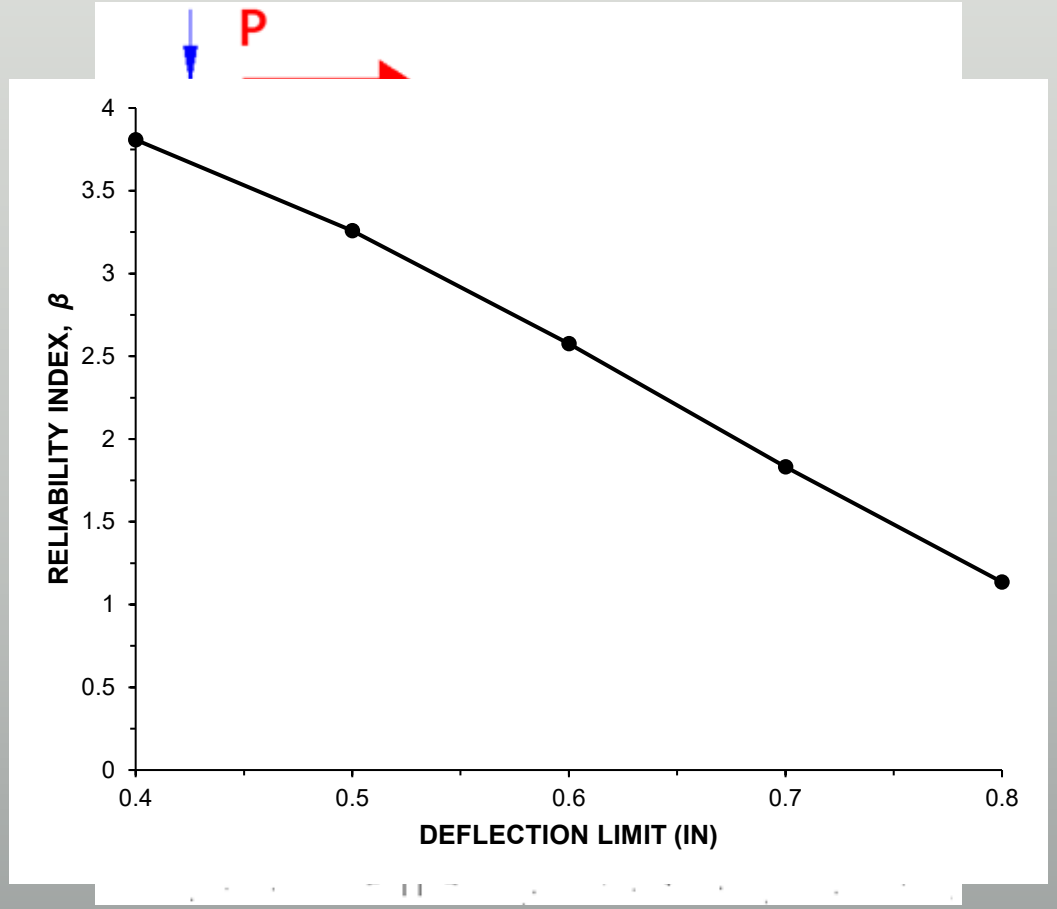
Mean = 5 ft

COV* = 30%

*Values selected where probability of failure generally plateaued.

Load (Deterministic)

Deflection (in)	Applied load at pile head, P (kip)
0.4	10.6
0.5	12.3
0.6	13.9
0.7	15.4
0.8	16.8



Conclusion

- **AllPile Randomizer** introduces probabilistic analysis into commercial finite difference software using Monte Carlo simulation.
  - Currently limited to only one sand/gravel layer. Will be further developed.
- Uncertainty in corrected SPT numbers had a **stronger** effect on pile performance than water table fluctuations.
- Reliability-based calibration of pile deflection limits enables **safer and more optimized** designs.
  - Project-specific response limits can be calibrated through risk-based approach.
- Commercial software should allow **more soil layers** to better capture spatial variability and should implement **probabilistic analysis methods**.
  - RS2 (2D FE) allows for random variable analysis.
  - Slide2 (LEM) allows for random field analysis.

GitHub Download Link:

https://github.com/Geo-Structural-Services-Group-LLC/AllPile-Randomizer

Thank you

GEO/STRUCTURAL SERVICES GROUP, LLC

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www.geossg.com

Annual Conference on Deep Foundations
Nashville
October 20 - 23, 2025 | Nashville, Tennessee

Use of Nondestructive Testing To Establish Installation Disturbance Zone Criteria For Adjacent Continuous Flight Auger Pile Installation on Design Build Project

Presented By:
Saad Rehmatullah, EIT
Les Chernauskas, PE

Co-Authors:
Alfredas Daugiala, PE
Bill Oder, PE

GTR ENGINEERING SINCE 1995

Underpinning & Foundation

SKANSKA



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Agenda

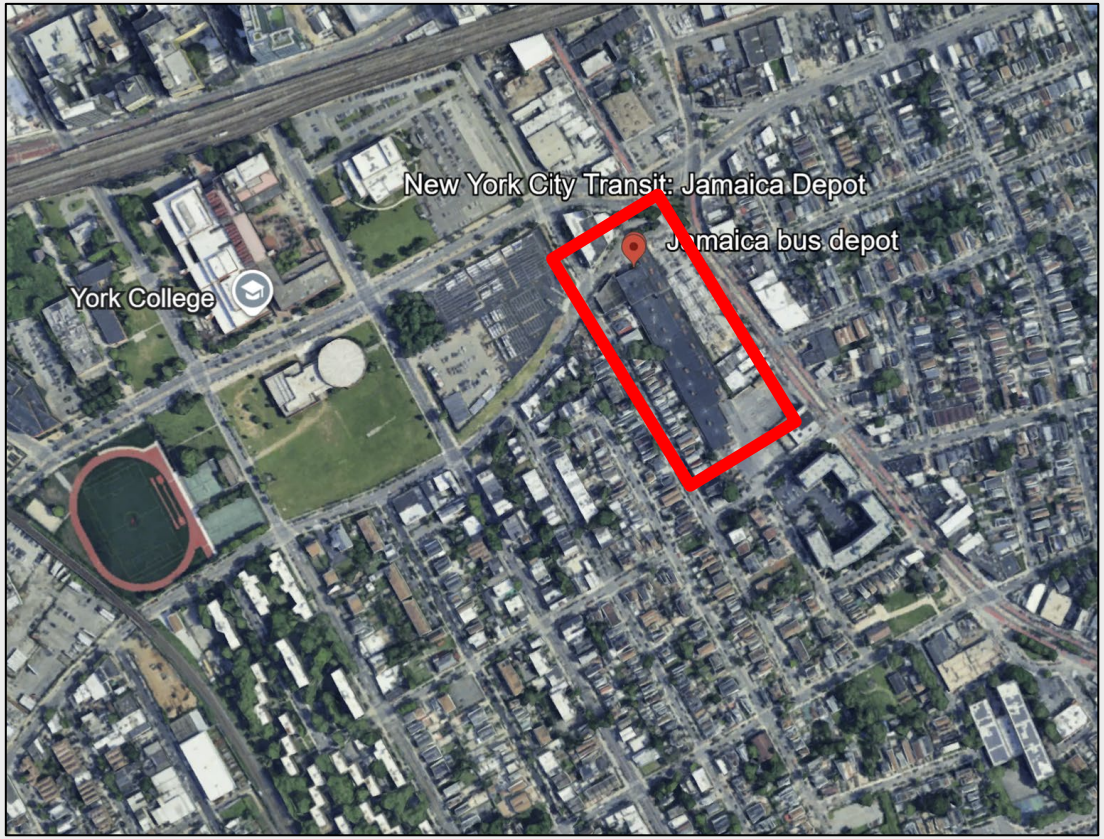
- Project introduction and geotechnical design
- Client requirements vs industry guidelines
- Proposed NDT during pile test program
- Analysis of NDT results
- Summary, conclusions, future work



$480M contract awarded to Design-Build team Skanska-STV

Rendering courtesy of STV, Inc.

Project Location - Jamaica, Queens, New York

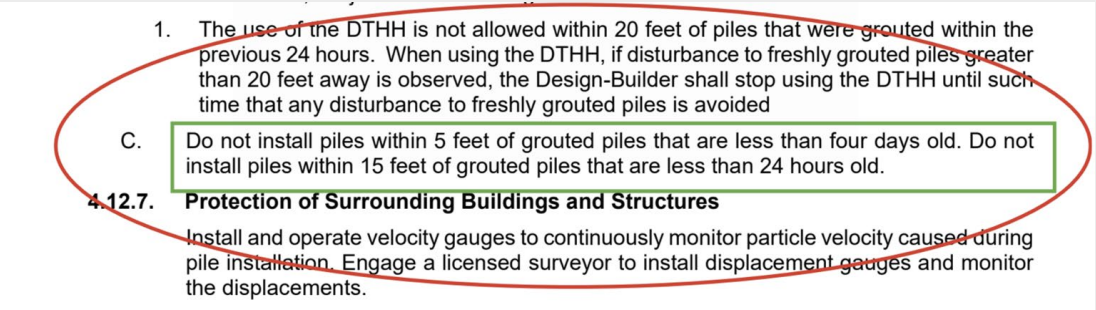


Via Google Earth accessed on 9/20/2025

Approximate project bounds

Project Design

- Approx. 1,000- 24in. Continuous Flight Auger (CFA) piles selected as foundation solution
  - Creates less vibrations and more suitable for stringent noise criteria
  - Reduced steel material and associated costs
  - Speed of installation (single pass)
- Client Project Requirements and Design Criteria (PRDC) limits pile installation within 15 feet of any grouted piles less than 24 hours old
- DB team proposed additional NDT during pile installation test program to justify a reduced drilling distance and installation time



1. The use of the DTHH is not allowed within 20 feet of piles that were grouted within the previous 24 hours. When using the DTHH, if disturbance to freshly grouted piles greater than 20 feet away is observed, the Design-Builder shall stop using the DTHH until such time that any disturbance to freshly grouted piles is avoided

C. Do not install piles within 5 feet of grouted piles that are less than four days old. Do not install piles within 15 feet of grouted piles that are less than 24 hours old.

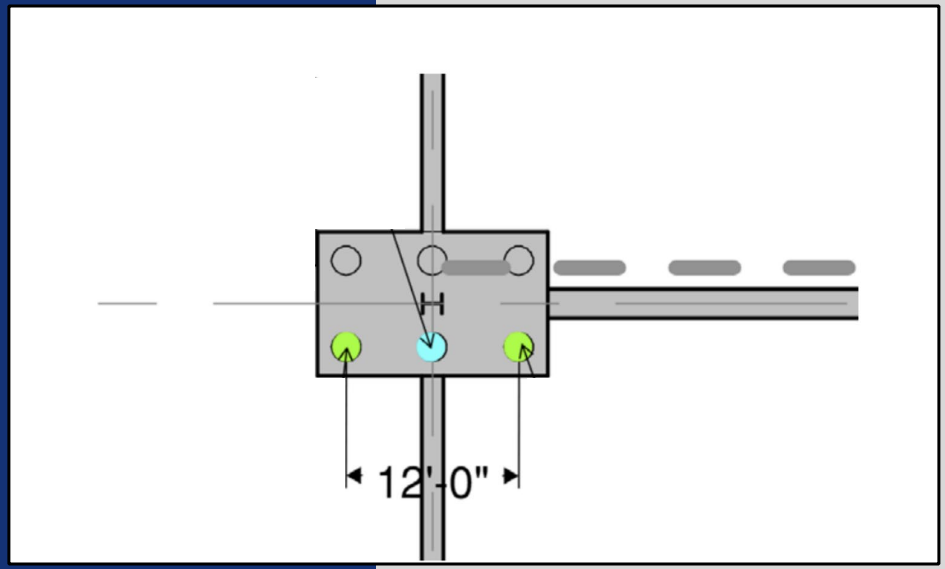
4.12.7. Protection of Surrounding Buildings and Structures

Install and operate velocity gauges to continuously monitor particle velocity caused during pile installation. Engage a licensed surveyor to install displacement gauges and monitor the displacements.

Typ. 6- pile group limited to 1 pile installation per 24 h, takes **6 days** to install

Proposed to complete in **4 days**

150% faster for approx. **1,000 CFA piles**



Generic Pile Group Layout

Industry Accepted Codes and Guidelines

Agency	Reference Document	Installation Distance of Adjacent Pile
Federal Highway Administration (FHWA)	FHWA-HIF-07-03 Geotechnical Engineering Circular No. 8	Adjacent piles within **six diameters**, center to center, of each other shall not be installed until it can be demonstrated by the CFA Pile Contractor that the grout or concrete in the first pile installed is fully set
Deep Foundations Institute (DFI)	Augered Cast-In-Place Piles Manual, 2nd Ed	Adjacent piles within **six (6) pile diameters**, center to center, shall not be installed within **12 hours** of each other
New York State Department of State	2020 Building Code of the State of New York, 1810.4.8	Augered cast-in-place piles shall not be installed within **six diameters** center to center of an element filled with concrete or grout less than **12 hours** old, unless approved by the code enforcement official
New York City Department of Buildings (NYC DOB)	2022 New York City Building Code, 1812.3.3	Augered cast-in-place piles shall not be installed within a distance equivalent to center-to-center spacing equivalent to **six times the diameter** of a drilled pile filled with concrete or grout less than **12 hours** old, unless approved by the engineer

Foundation industry guidelines: 6D spacing within 12 h → D(2ft)=12ft spacing

Client PRDC: 15 ft spacing within 24 h


Bauer BG 36 H Rotary Drill Rig
Grout pump and injection system
Spacer centralizers
Reinforcement cages outfitted with TIP wiring (yellow)

JBD C40355	**NONCONFORMANCE REPORT - *Construction***	Date 08/30/23

1. NCR Title	2. Responsible Party	3. NCR No.
CFA piles augured and cast too soon near adjacent piles	Skanska	SKA-003 Response Rev 1 - 12-18-23 T. Tyle

4. Description of Non Conformance (Attach applicable photographs)

It was observed by the Special Inspector that the Pile 2A was auger cast 12 feet away from Pile 6 which had been augered and cast shortly before. This does not comply with the limitation in the PDRC of not less than 15 feet within 24 hours. It does, however, comply with the limitation in Specification 31 63 16, Rev 1. (Special Inspector report from 08/25/23 attached).

5. Applicable Requirements and Reference Specification:

PDRC 04 Rev 3 Ad7 – Structural – 4.12.6 - Page 43 (attached)
Specification 31 63 16, Rev B, 08-18-23 (attached)

6. Prepared By:	7. Response Due Date :	8. Designer Concurrence Required?
T. Tyler 08/30/23	09-06-23	Yes

NDT During Pile Test Program

Low Strain Impact Integrity (Sonic Echo) Testing [ASTM D5882-16]

- Surface reflection technique which can detect change in pile impedance (e.g. necking)
- External integrity test
- Original NDT method required

Thermal Integrity Profile (TIP) Testing [ASTM D7949-14]

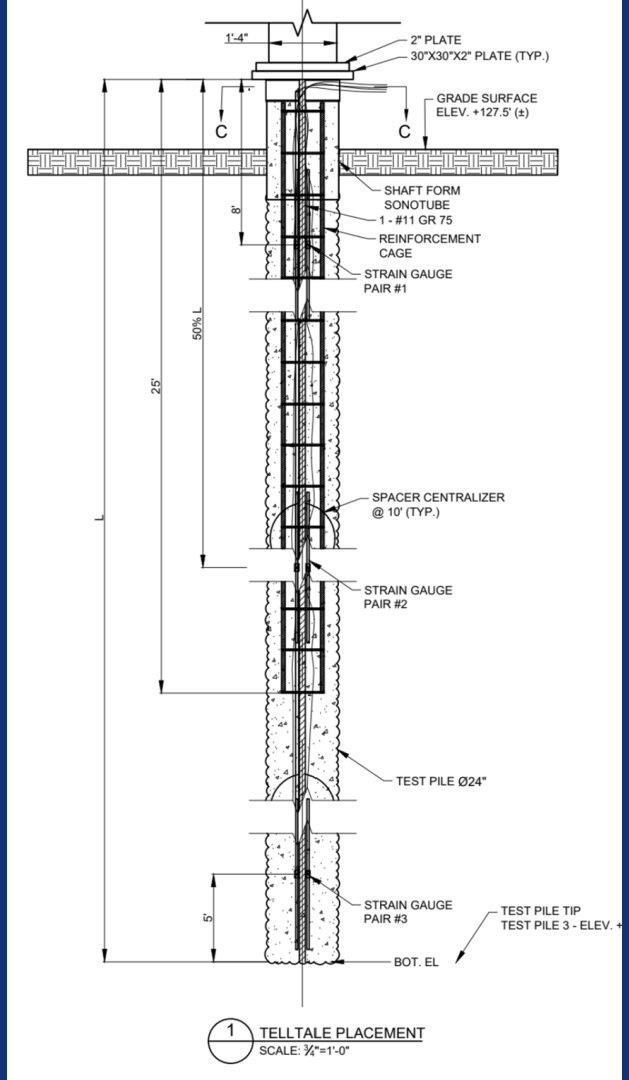
- Monitors temperature profile generated by exothermic heat of hydration reaction as concrete cures over time throughout the depth of shaft
- Internal integrity test
- Additional NDT method to investigate closer pile installation

Reinforcement Details

Rebar cage (27.75ft):
8 - #8 bar

Center reinforcement (48ft): #18 bar

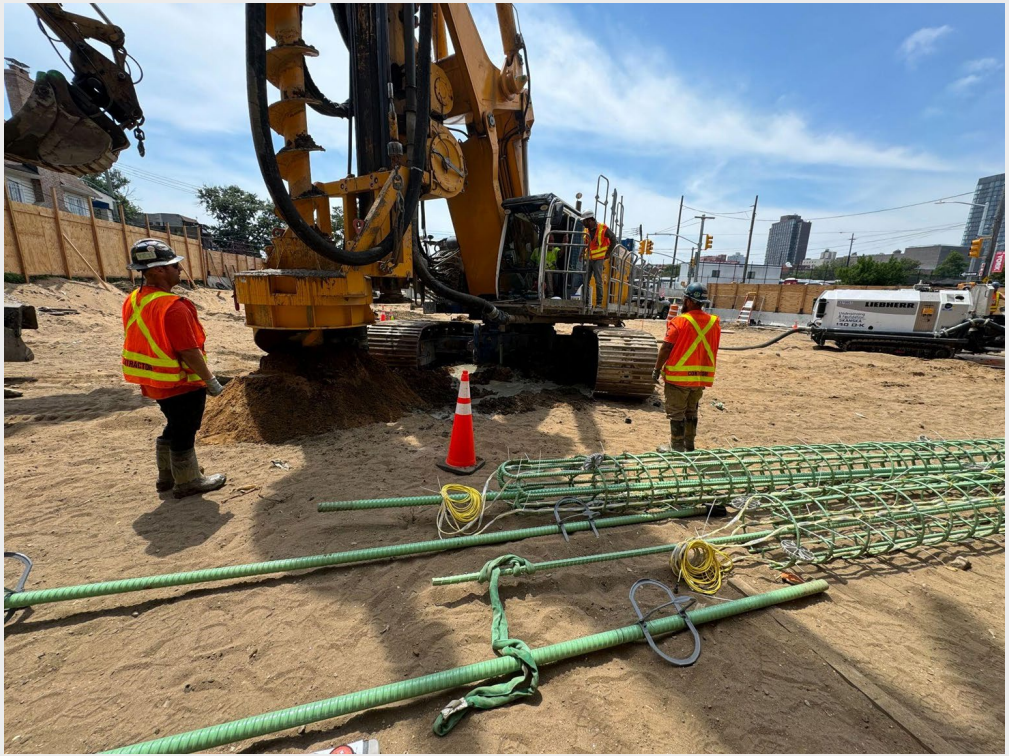
Centralizers at 10 ft O.C.





TIP Testing Field Instrumentation





TIP Testing Field Instrumentation

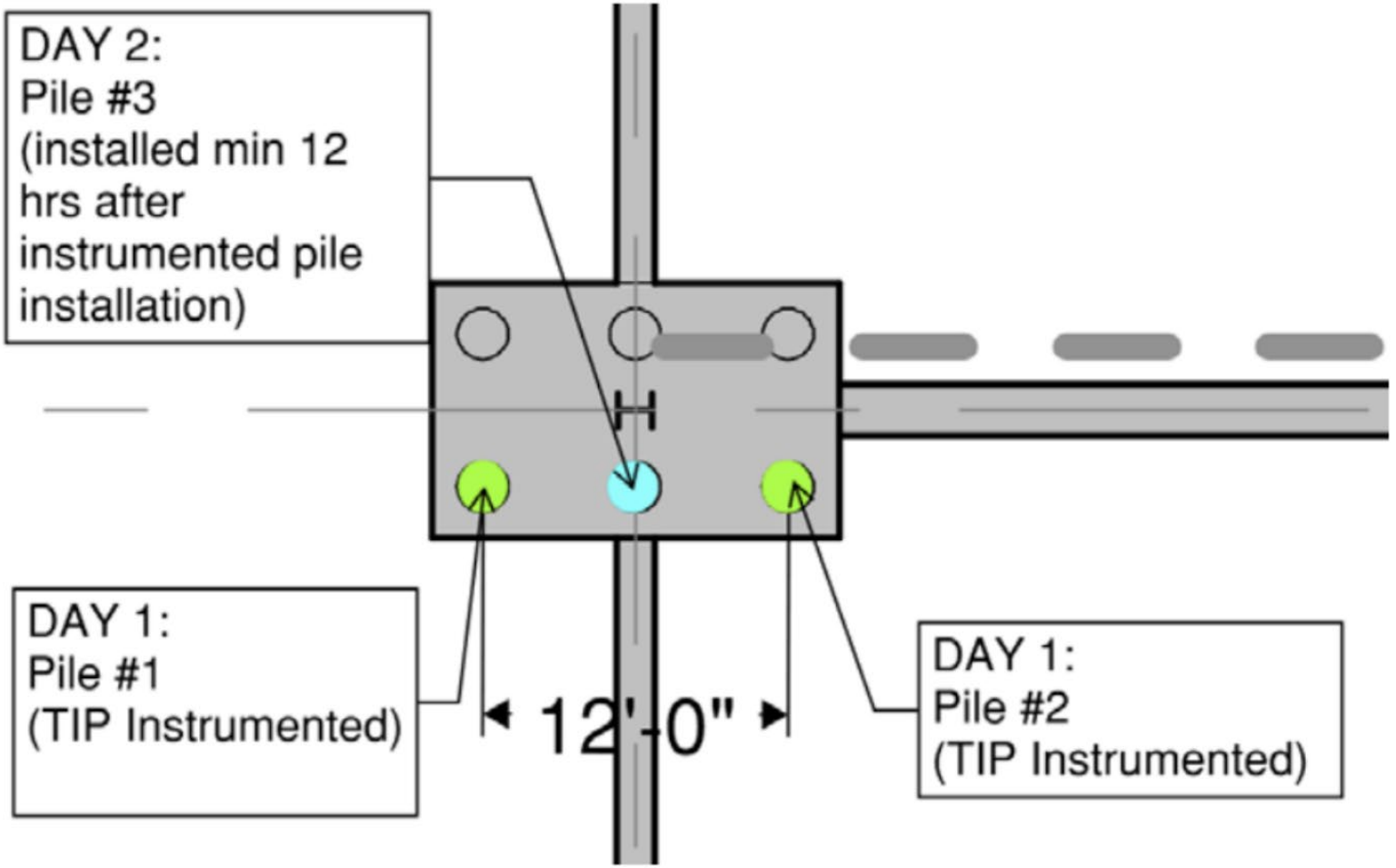




Grout Mix



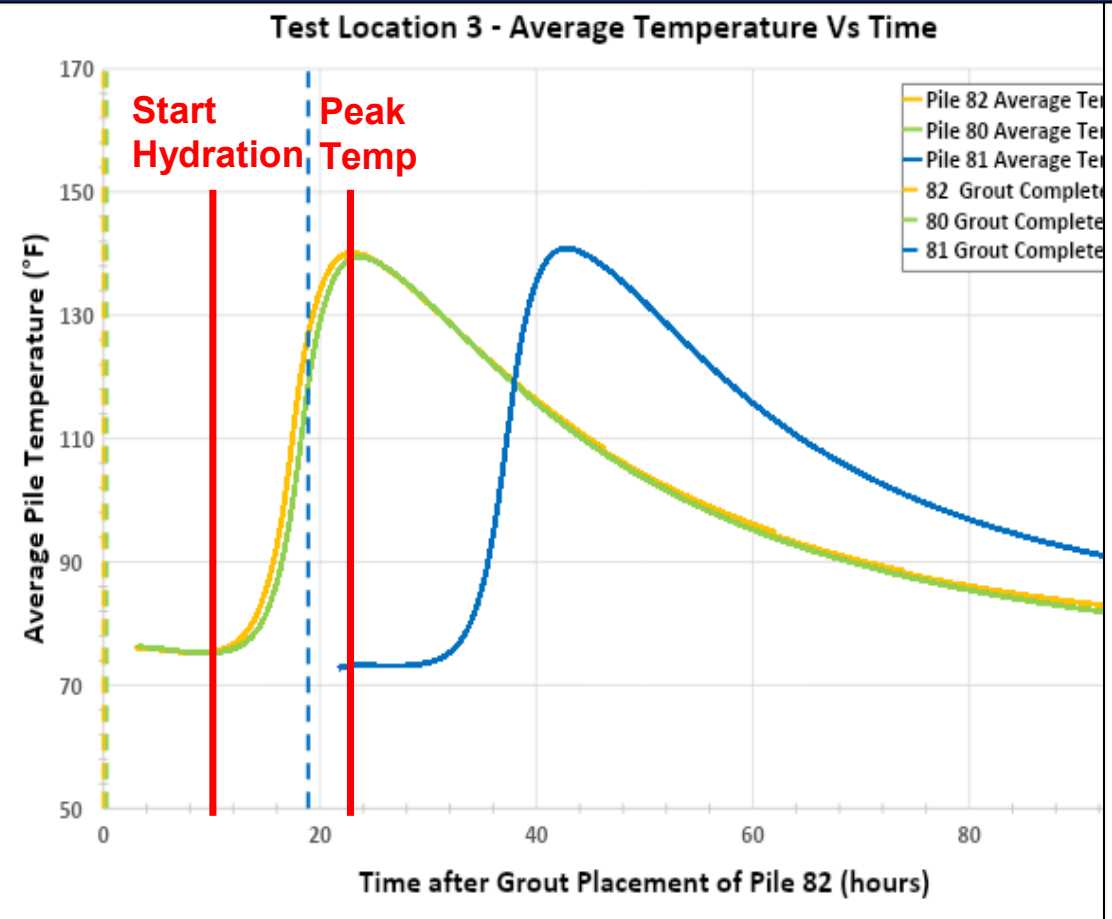


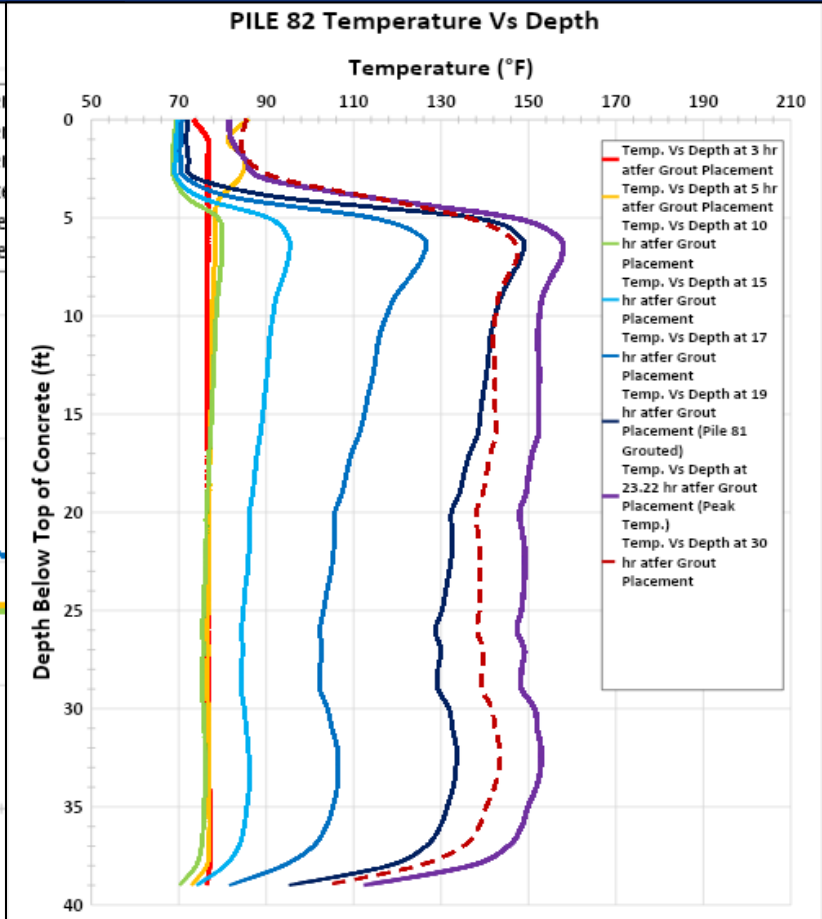

DAY 2:
Pile #3
(installed min 12 hrs after instrumented pile installation)
H
DAY 1:
Pile #1
(TIP Instrumented)
12'-0"
DAY 1:
Pile #2
(TIP Instrumented)

Results of NDT Test Program

- Two (2) anomalies identified out of 9 tested piles:
- Pile 497 in TL1 (3rd pile installed)
- Pile S-0 in TL2 (1st pile installed)
- The anomalies not a result of the installation of adjacent piles, but rather occurred during construction of the pile itself
- Anomaly is data that deviates from expectations and needs to be further evaluated (for TIP most commonly is a significant temperature reduction)

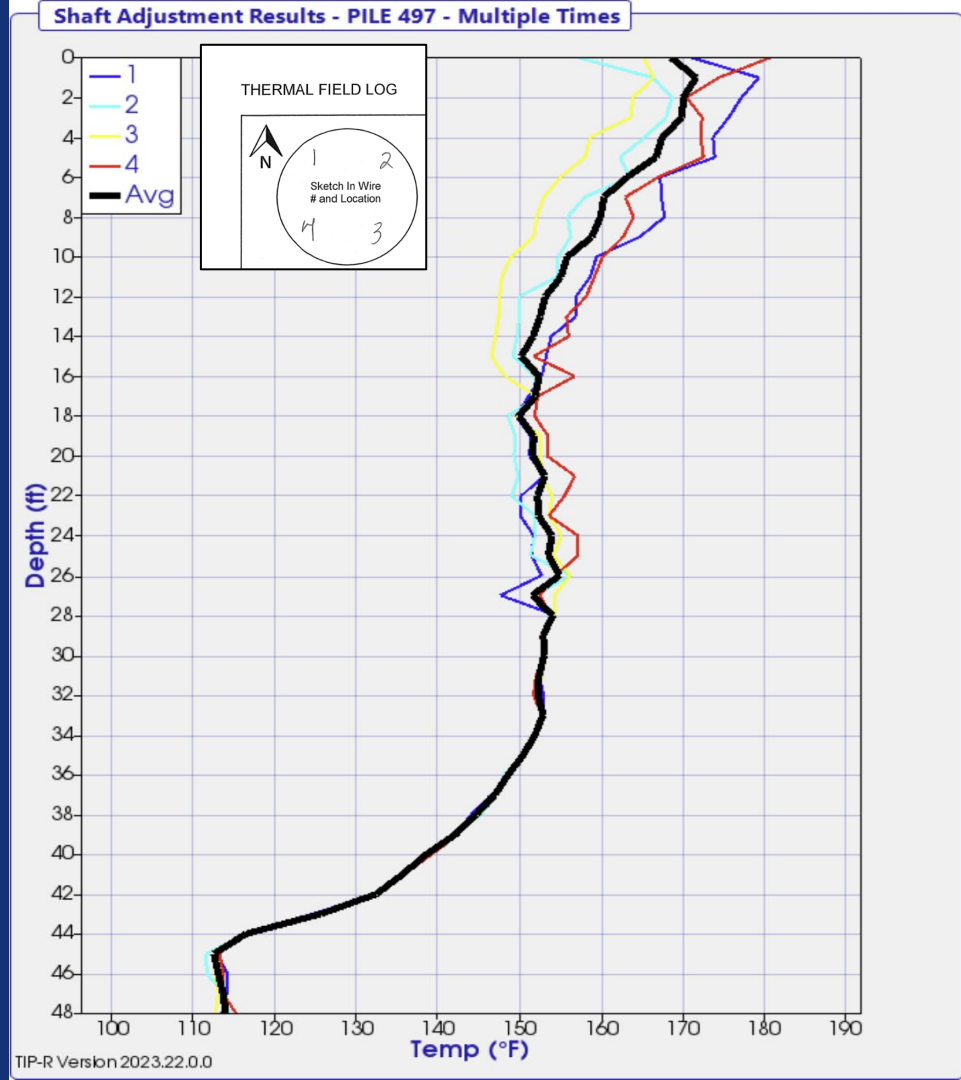
TIP Testing Example (Test Location 3)



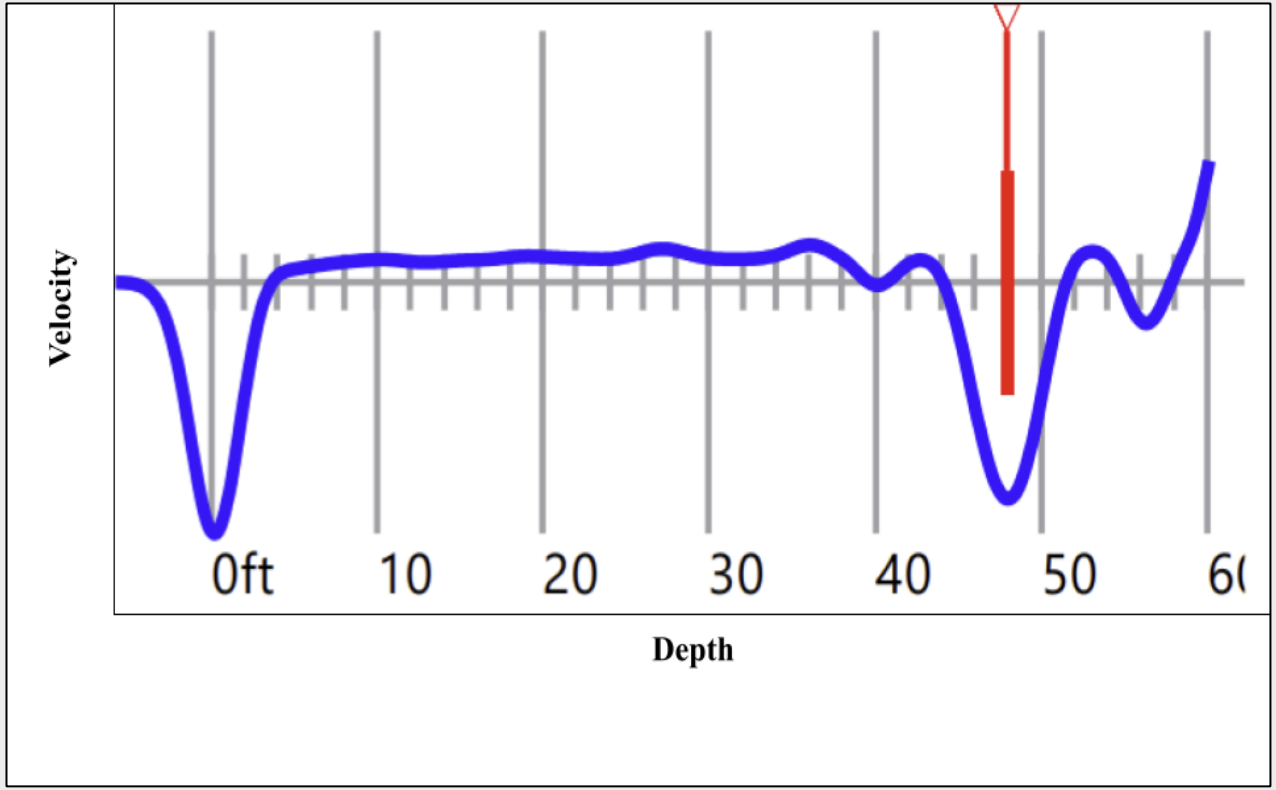


Pile 497:
Temp Vs Depth

- Individual wire temperatures with depth (converges at end of rebar cage at 28ft)
- Temperature difference between wire 1 (blue) and wire 3 (yellow) upper 15 ft
- Temperatures converge at 16 ft depth, then wire 1 exhibits lower temperatures as cage is closer to the soil in this area
- Gradual reduction in temperature from 30 ft to 45 ft
- Most likely the reinforcement bar is not plumb, resulting in decreased grout cover on TIP wires
- Possible neck at very bottom of pile (45ft to 48ft)



Pile 497: SE Test Results



Pile 497

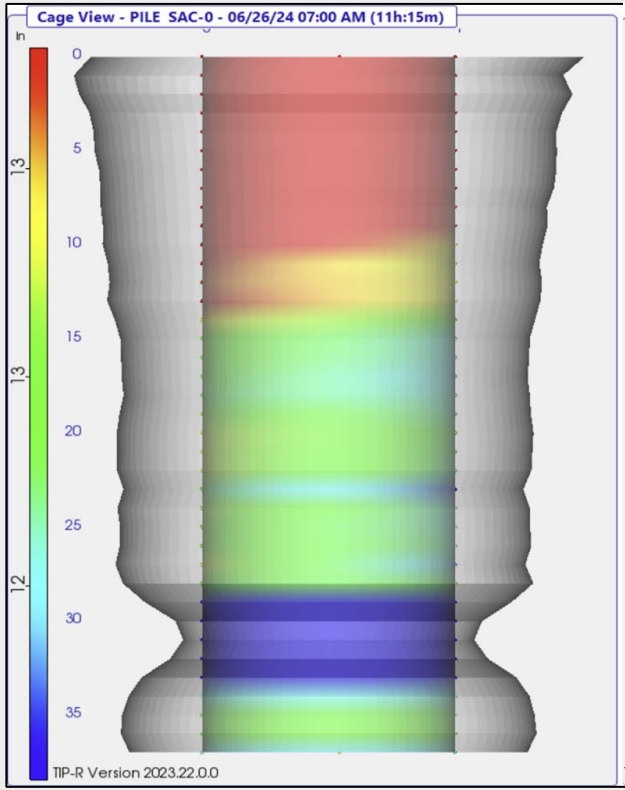




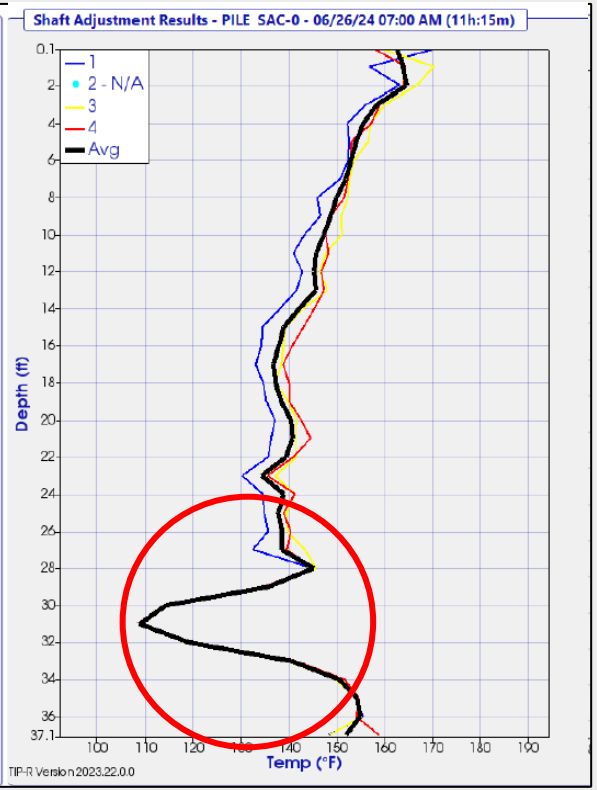
Photos show UniSpacer™ Single Bar Centralizer and ShaftSpacer® Rebar Wheel

- Installed on day 2 after the other two piles, therefore there is no adjacent pile installation disturbance
- Anomaly attributed to cage misalignment and bar angle
- Also anomaly indicative of possible necking near bottom
- Underwent successful load test
- Increased focus and attention to cage and rebar placement and the use of the centralizing wheels
- Grout mix design refined with increased retardant (4h to 10h)

Pile S-0 TIP Testing Results

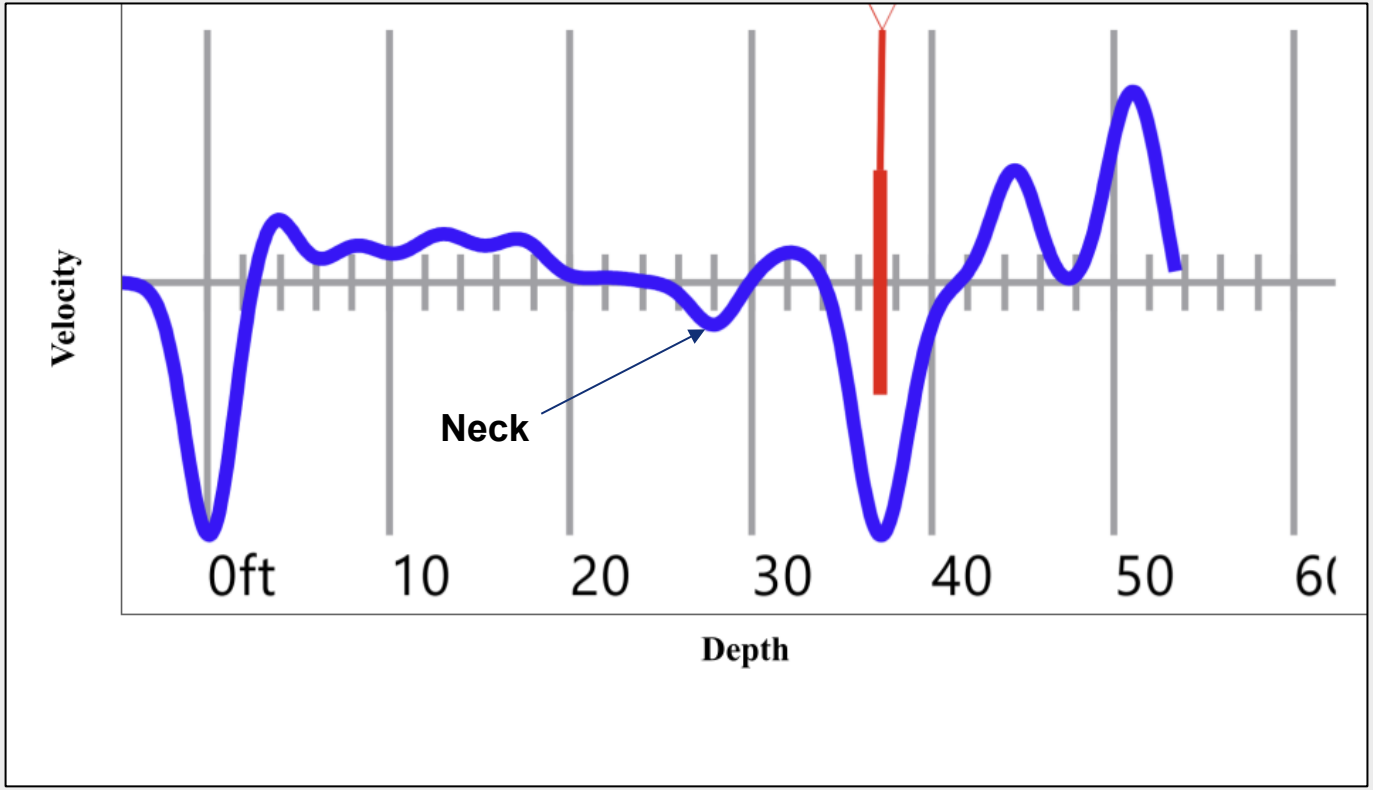


TIP Pile Shape



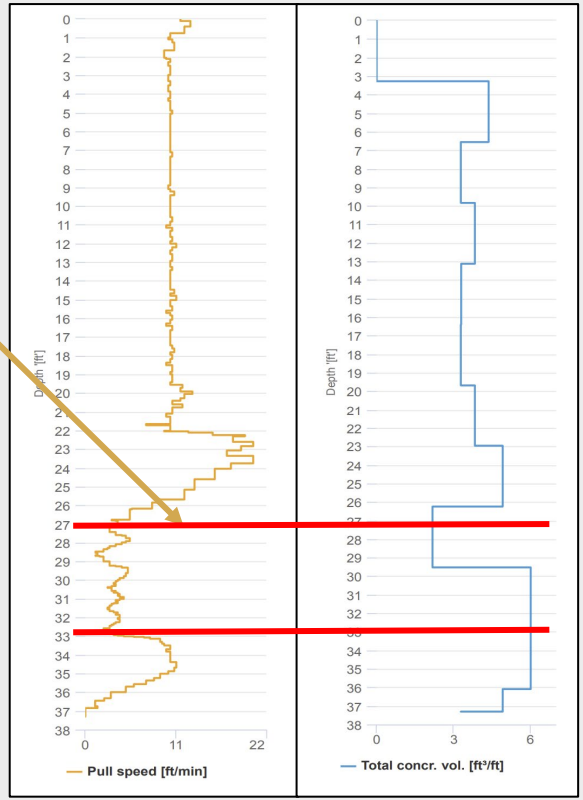
Temperature Vs Depth

Pile S-0: SE Test Results

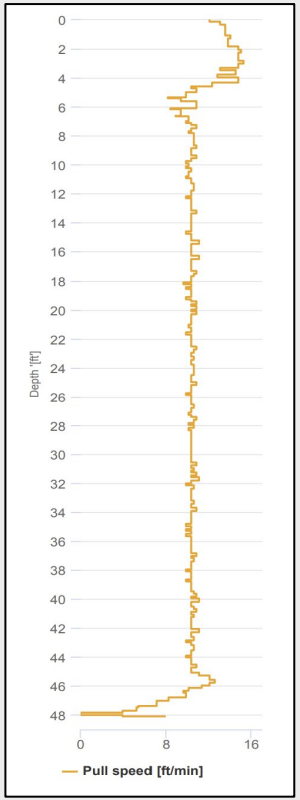


Pile S-0 Installation Logs

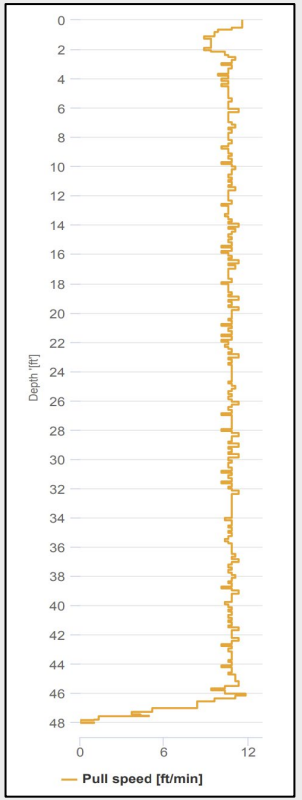
- Variable pull speeds (4 to 20 ft/min) in zone of anomaly (28ft - 34ft)
- Reduction of grout volume
- Pile installation logs for other 2 piles in TL have uniform pull speeds and no anomalies



S-0



681



679

Pile S-0

- Anomaly/defect not a result of installation sequencing from nearby piles, but instead occurred during construction
- Prompted more focus and better procedure for auger withdrawal and grout placement
- Sacrificial pile used as reaction pile for successful load test

Summary and Conclusion

- Results from TIP and SE testing prove acceptable and more efficient pile installation sequence
- Two anomalies identified were not a result of installation sequencing from nearby piles, but instead occurred during construction
    - More attention placed on fabrication/placement of cage, rebar and centralizers – added retarder
    - Refined auger withdrawal rate and grout placement procedure
- Project continued with 5% TIP testing requirement on production piles and no anomalies were detected
- Results confirm drilling 6D from freshly grouted piles and 3D from piles which have cured for about 12 hours does not impact integrity of pile
- Further evaluation using TIP to investigate sequence of pile cure time and pile spacing can shed more light on disturbance zones around piles

Acknowledgement



Underpinning and Foundation Skanska:

Alfredas Daugiala
Saad Rehmatullah
Andrew Burns
Bobby Sacaris (IUOE Local 15)
Maurice Diong
Syed Asghar
Scheedie Russell
Lisa Papandrea
Alfranio Prado
Miguel Echeverry Perez
Casey Marraccino
Matthew Lozito



Geosciences Testing and Research (GTR):

Les Chernauskas
William Oder
Alex Maurice





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THANK YOU!
Questions?

Green Infrastructure Partners

Engineering Heritage: Integrating North America's Largest Façade Retention System with Support of Excavation in the United Bldg. Project

Mark Reinders, P.Eng
Thomas Fiala, P.Eng
Lauren Speakman



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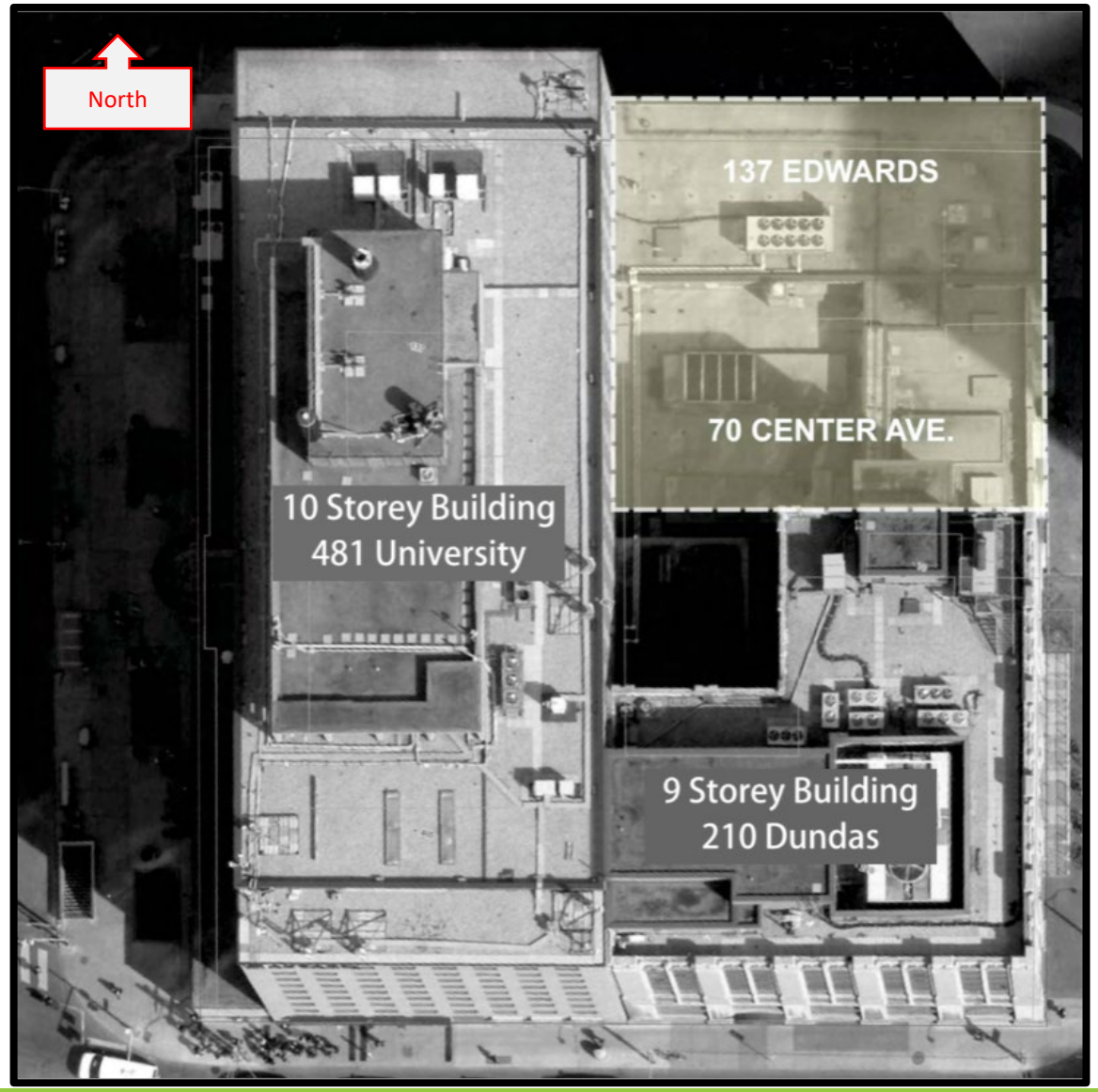
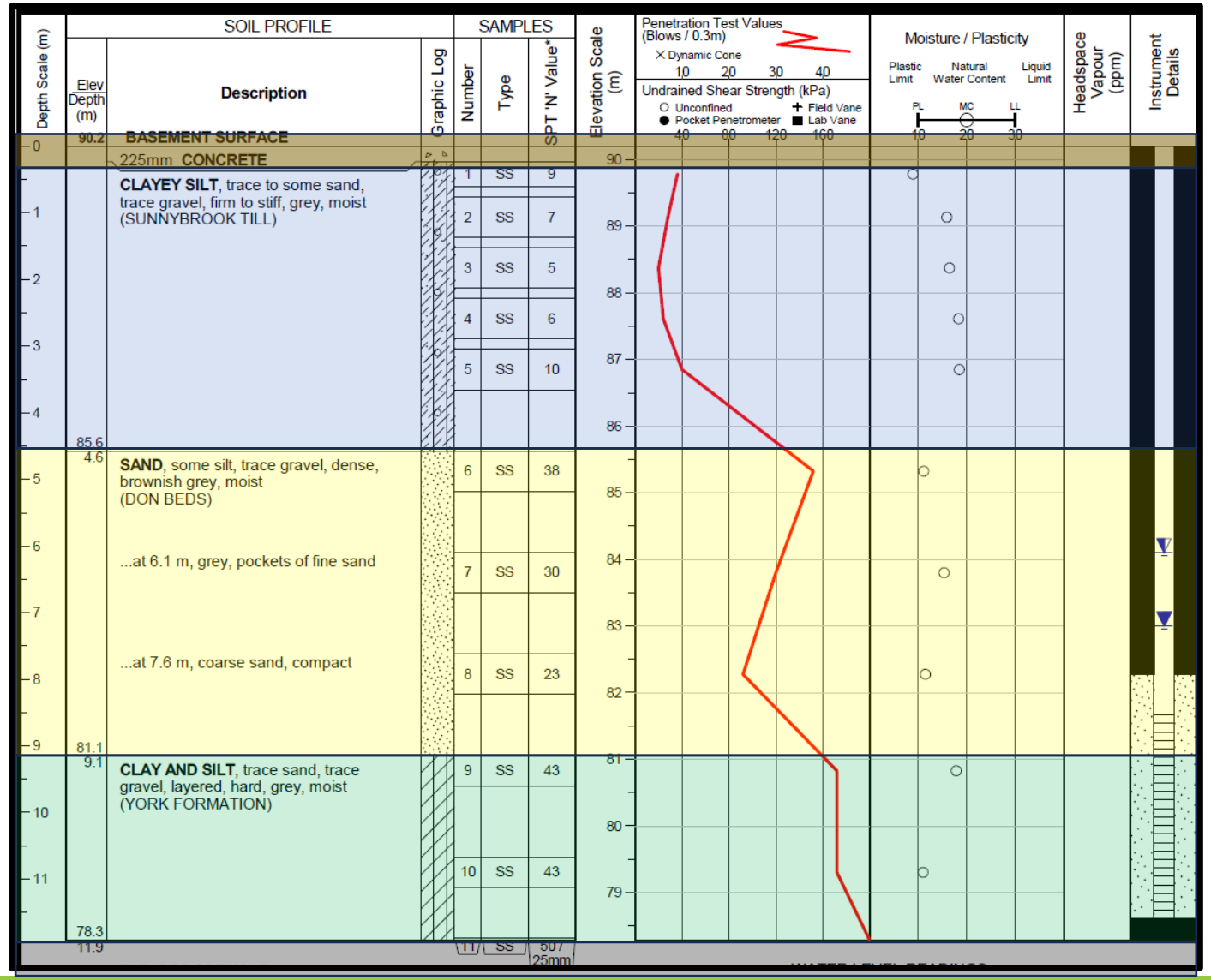
Location – 481 University Ave., Toronto, CAN



Location - Geotechnical Conditions

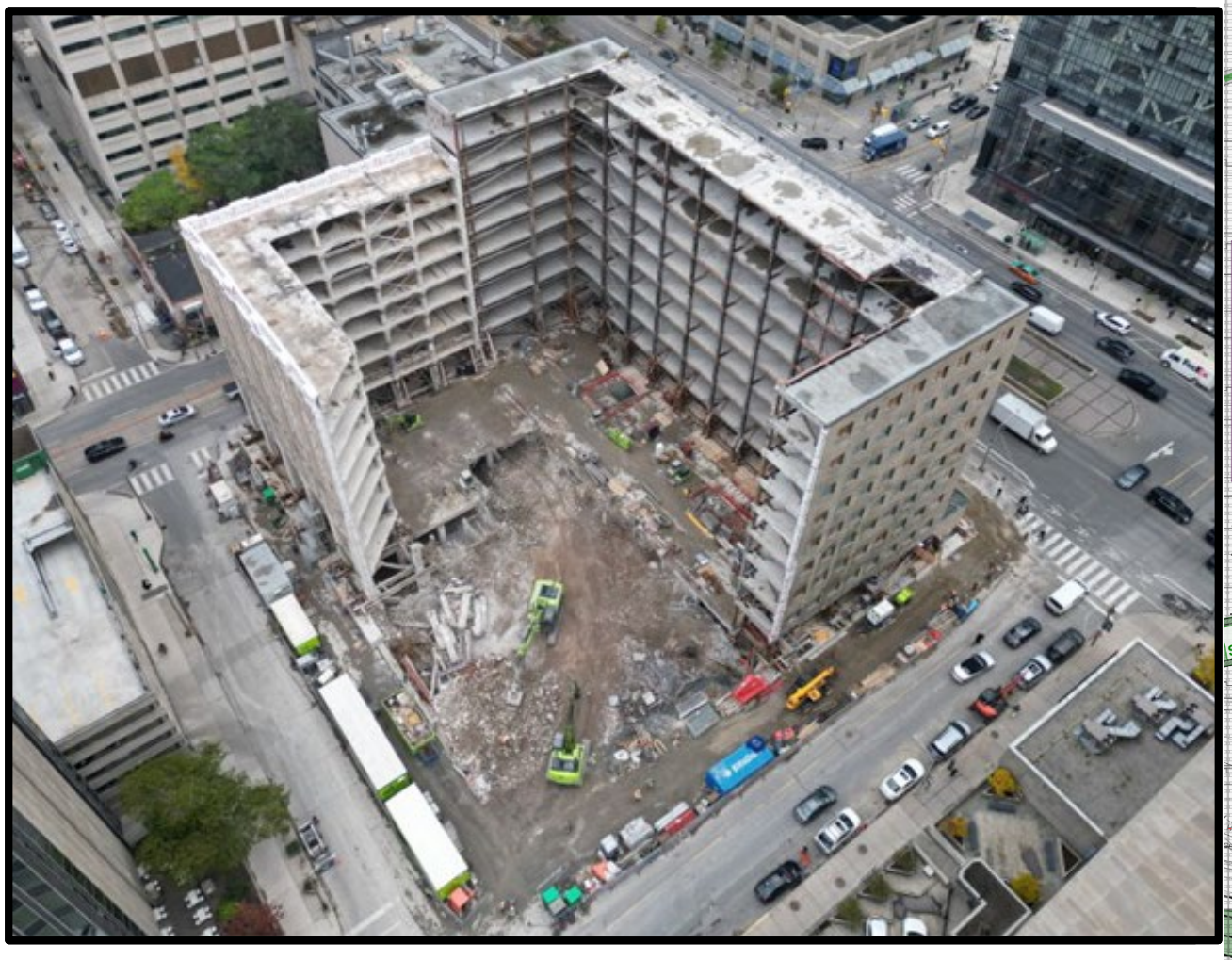
- Typical for Downtown Toronto
- All Boreholes within Basement of Structures

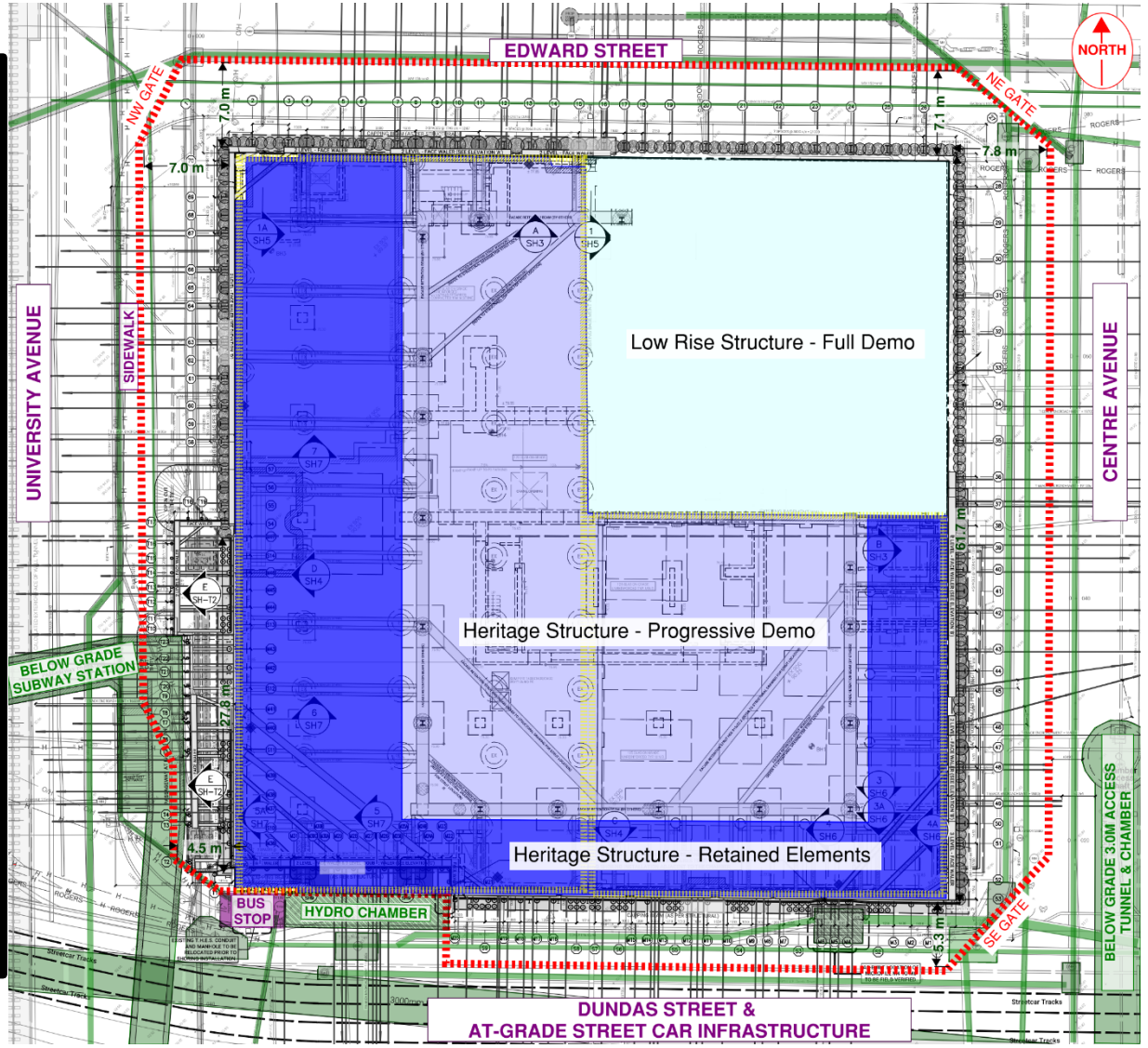


Elev in meters above seal level (MASL)

- Fill (94.3 – 89.0)
- Clayey Silt (89.0 – 85.0)
- Silty Sand (85.0 – 81.7)
- Clay & Silt (81.7 – 78.7)
- Shale Bedrock (78.7 onward)

Demolition

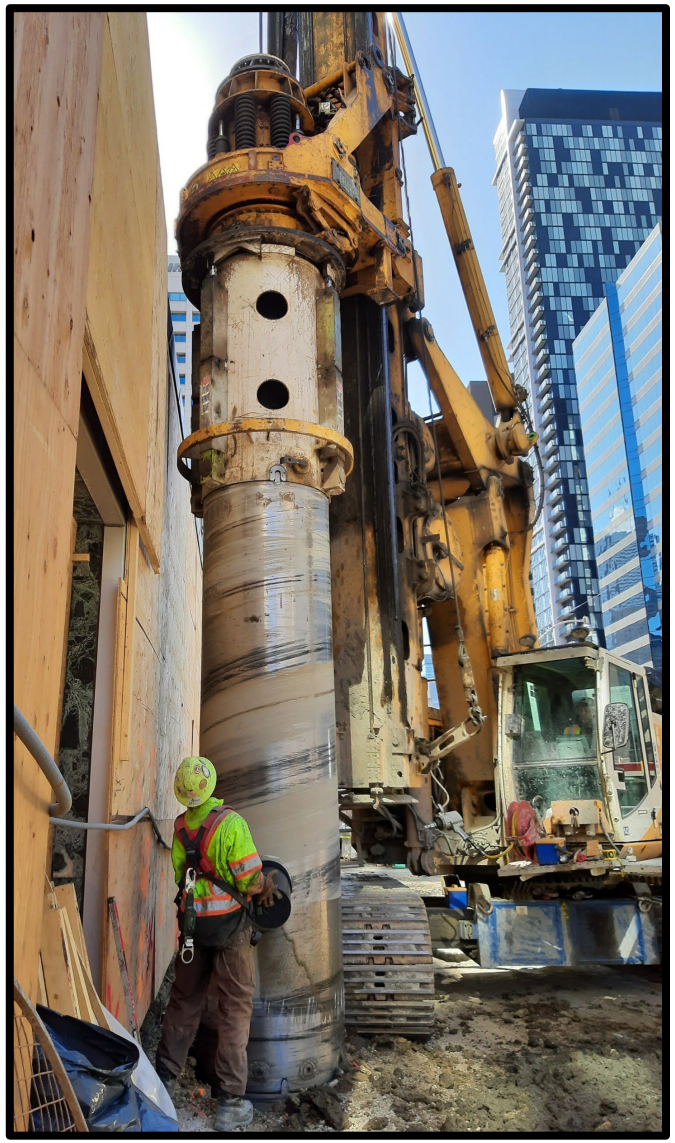


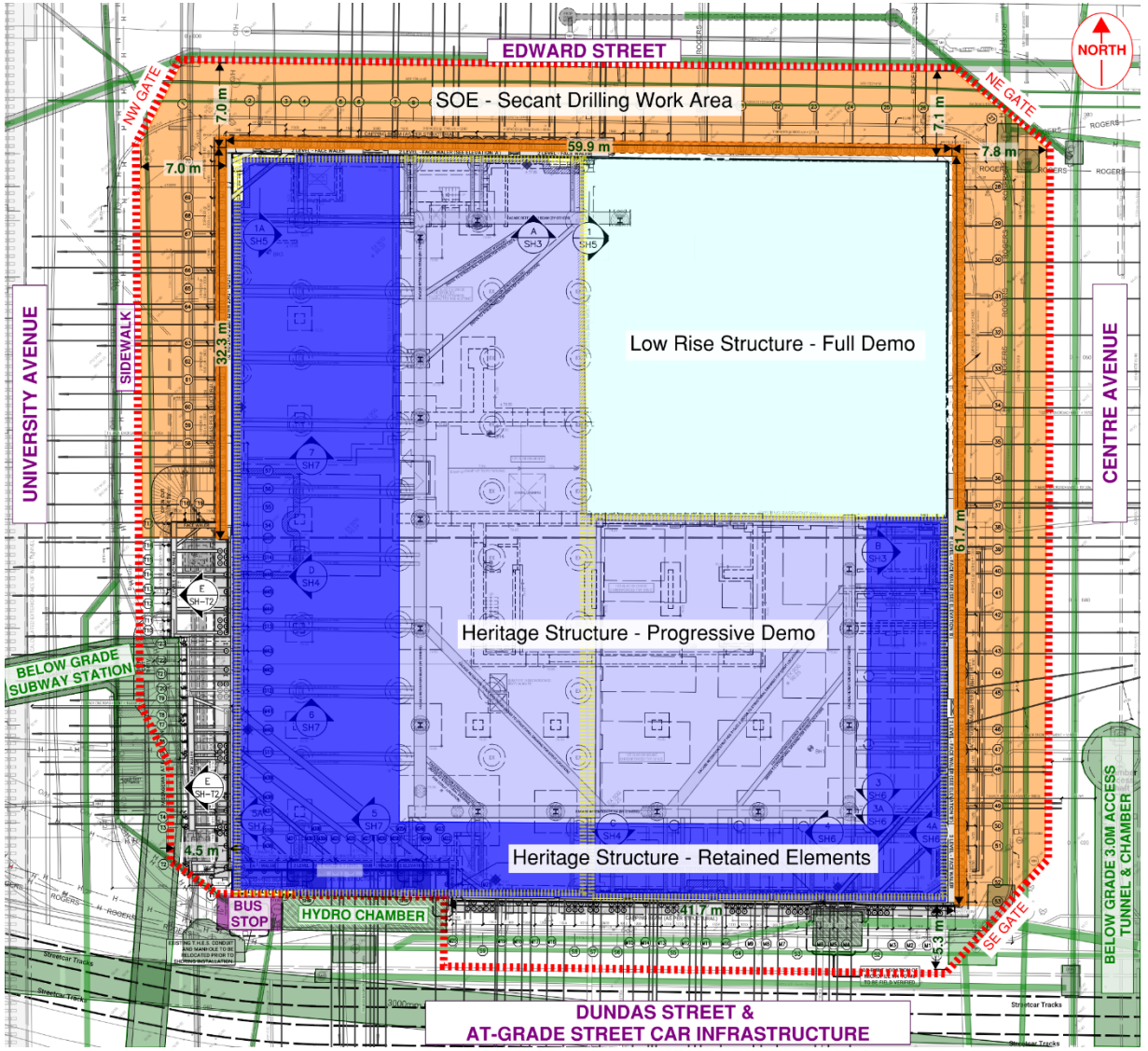

EDWARD STREET
NORTH
NW GATE
NE GATE
SE GATE
7.0 m
7.0 m
7.1 m
7.8 m
UNIVERSITY AVENUE
SIDEWALK
CENTRE AVENUE
Low Rise Structure - Full Demo
Heritage Structure - Progressive Demo
Heritage Structure - Retained Elements
BELOW GRADE SUBWAY STATION
27.8 m
4.5 m
BUS STOP
HYDRO CHAMBER
5.3 m
BELOW GRADE 3.0M ACCESS TUNNEL & CHAMBER
DUNDAS STREET &
AT-GRADE STREET CAR INFRASTRUCTURE

Exterior Support - SOE Secant Wall

- Bauer BG36
  - 23' Work Area
  - Casing 1075mm off façade
  - Engineered Work Pad
- SOE Piles:
  - ~1040mm Dia
  - ~69.35m Depth
  - W610x174 to W610x217
  - 70 MPa Concrete
- SOE/FRS Piles:
  - ~1300mm Dia
  - ~69.55m Depth
  - W690x244 to W760x357
  - High Axial Load Piles:
    - Nelson Studs on Web
    - 35 MPa Concrete

(Imperial Units)

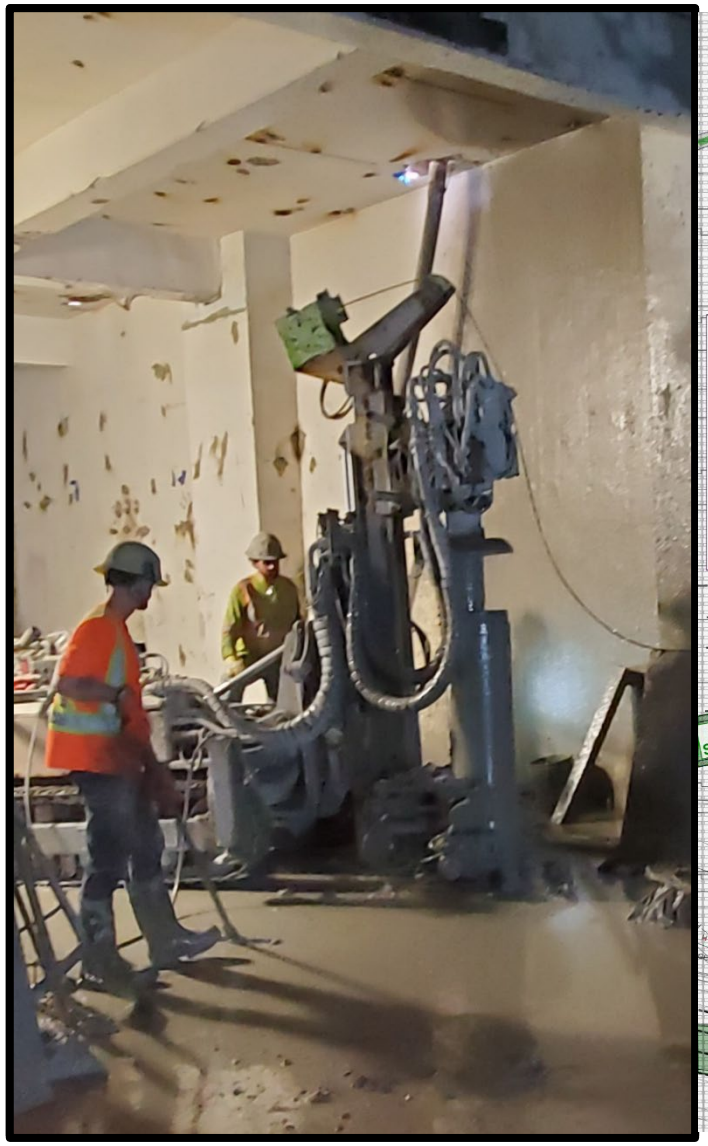


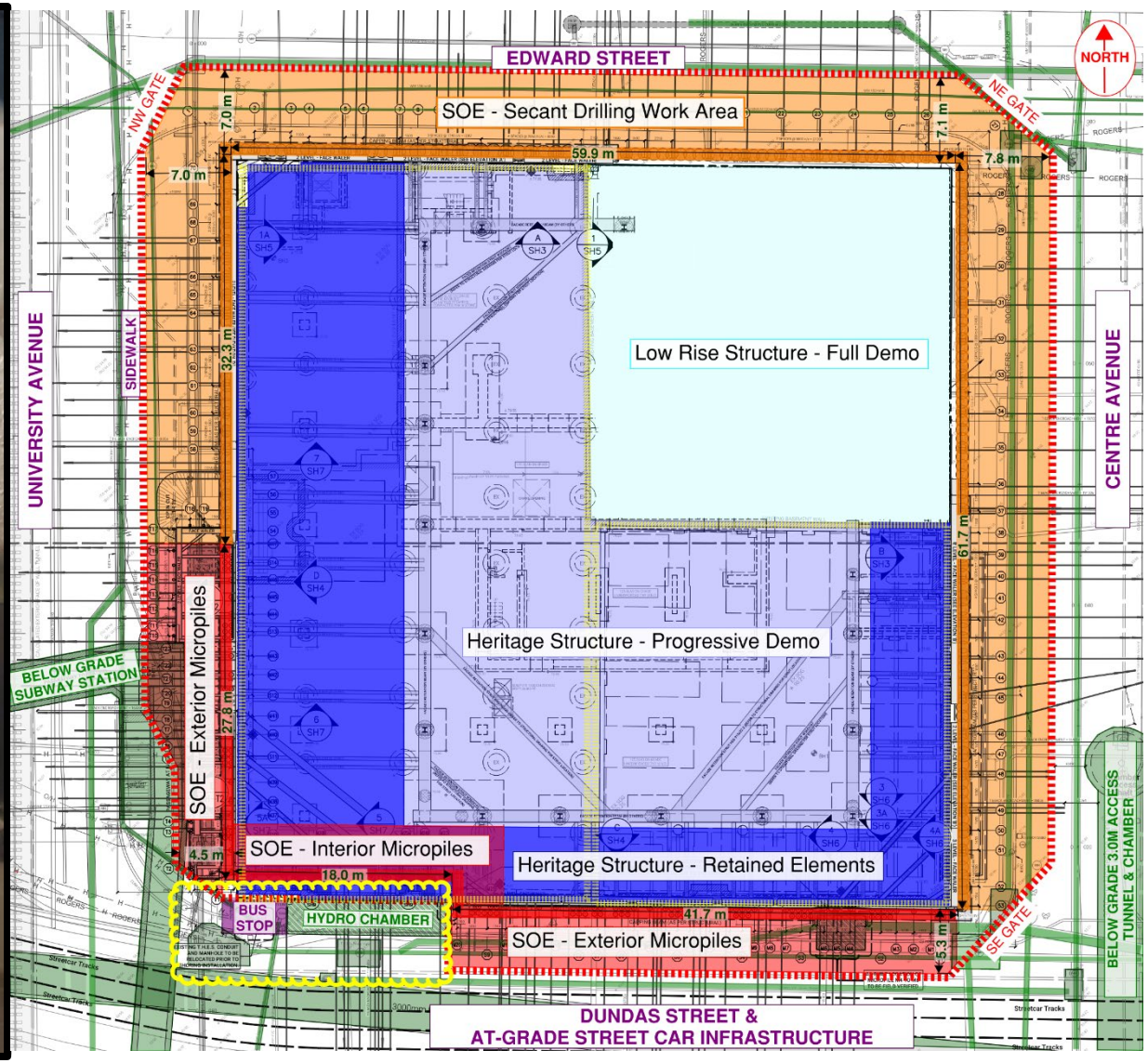


Exterior Support - SOE Micropiles

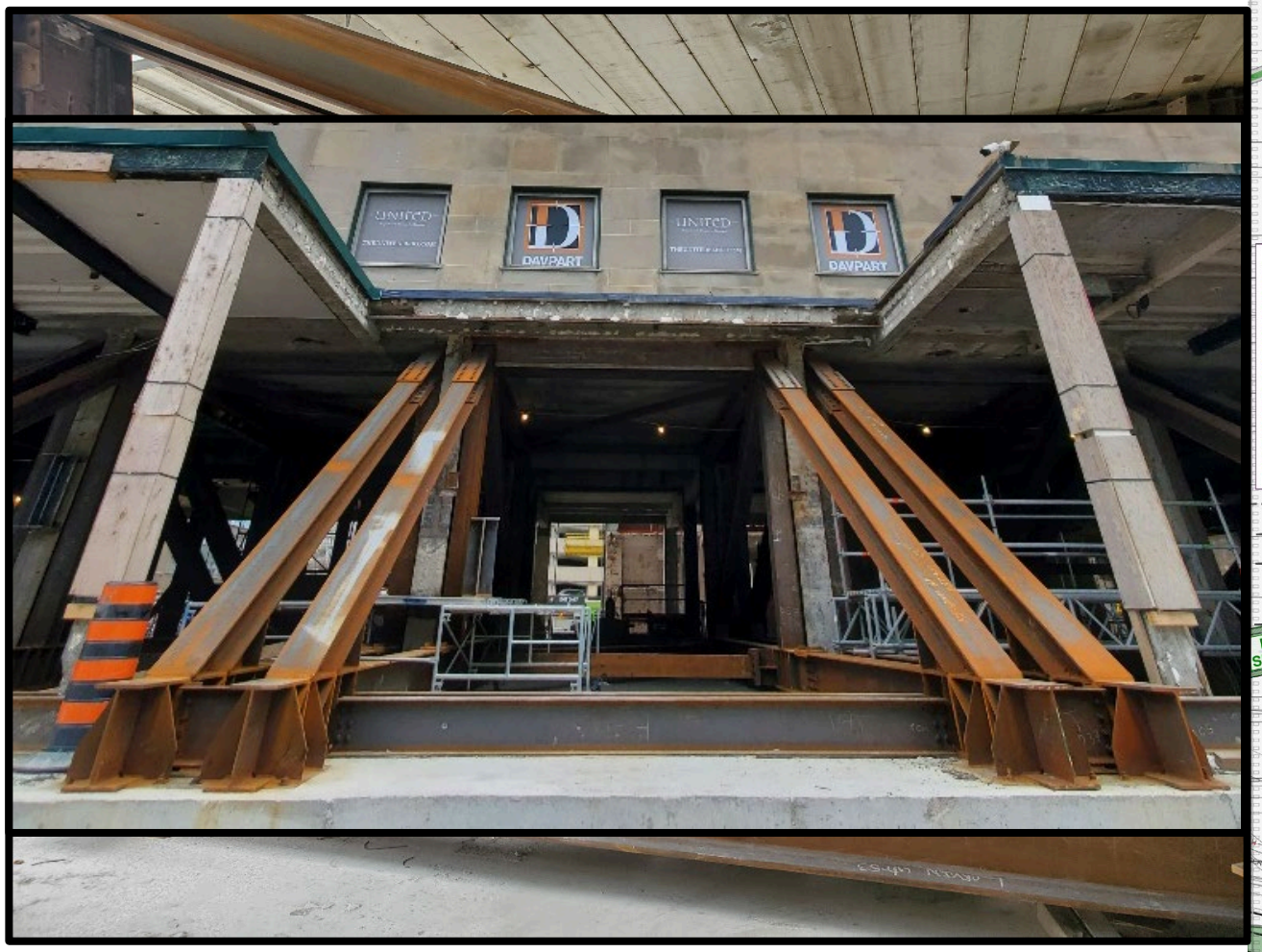
- Street Level:
  - Klemm 806-5G
  - Manipulate to work in 16.5’ area with O/H Wire 2.5’ off façade
  - Drill 20” min. off façade
  - HSS 406x15.5” @ 30’ c/c with passive central threadbar
  - 30 MPa Grout
- Basement:
  - Klemm 702
  - 15’ headroom for 3’ casing sections
  - HSS 356x12.5” @ 30’ c/c with passive central threadbar
  - 30 MPa Grout

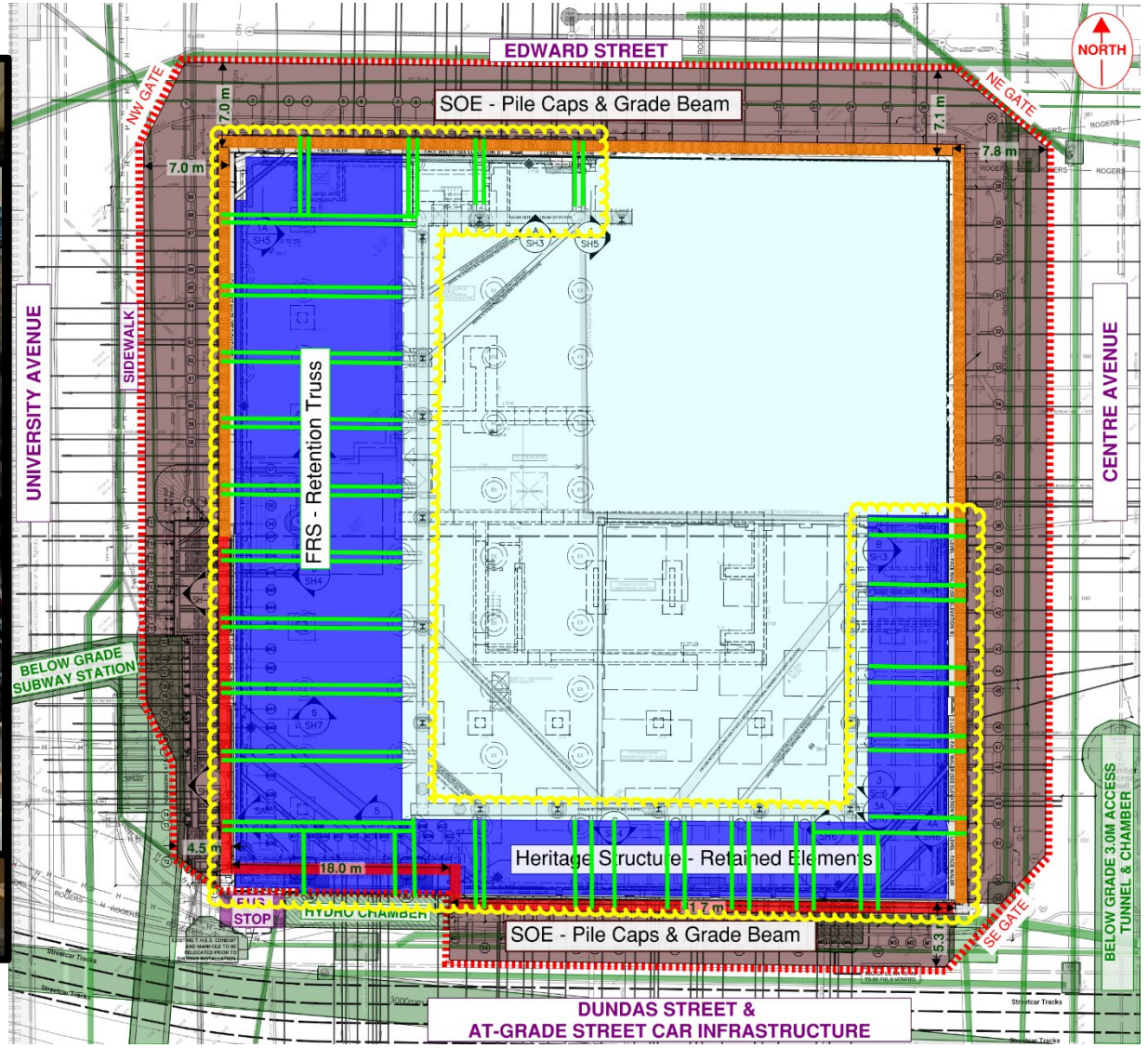
(Imperial Units)





Structural Steel – FRS Retention Truss



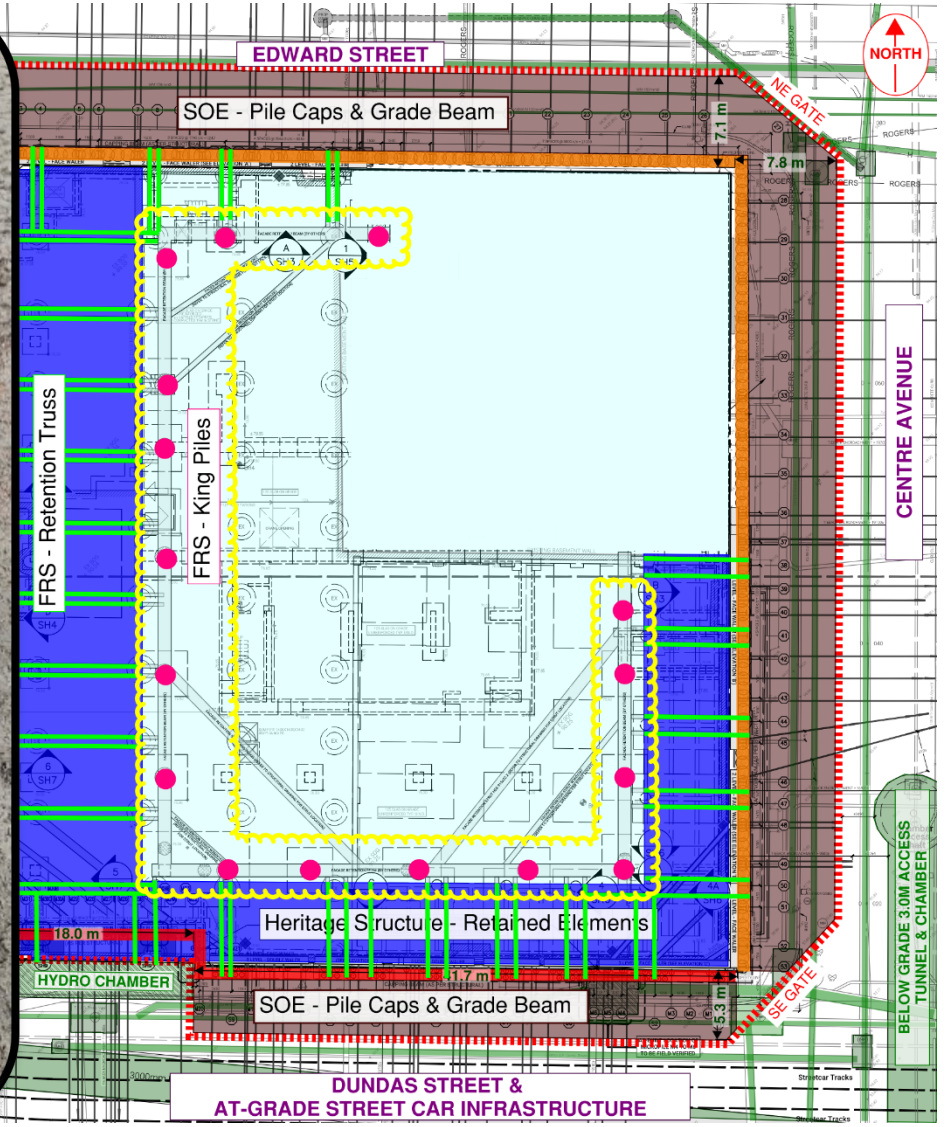


Interior Support - FRS King Piles

- Bauer BG28
  - Engineered Work Pad at Basement Elevation
  - Casing ~6'-0" off face of façade to support beams
- FRS King Piles:
  - 43'-0" Drilled Rock Socket
  - ~57'-5" Depth with a 2'-6" toe below BOE
  - W360x634 and W360x509 (~ 15.5 Tonnes)
  - 108 Studs on Pile Toe
  - Toe with 80 MPa Self-Consolidating Concrete. U-fill above.

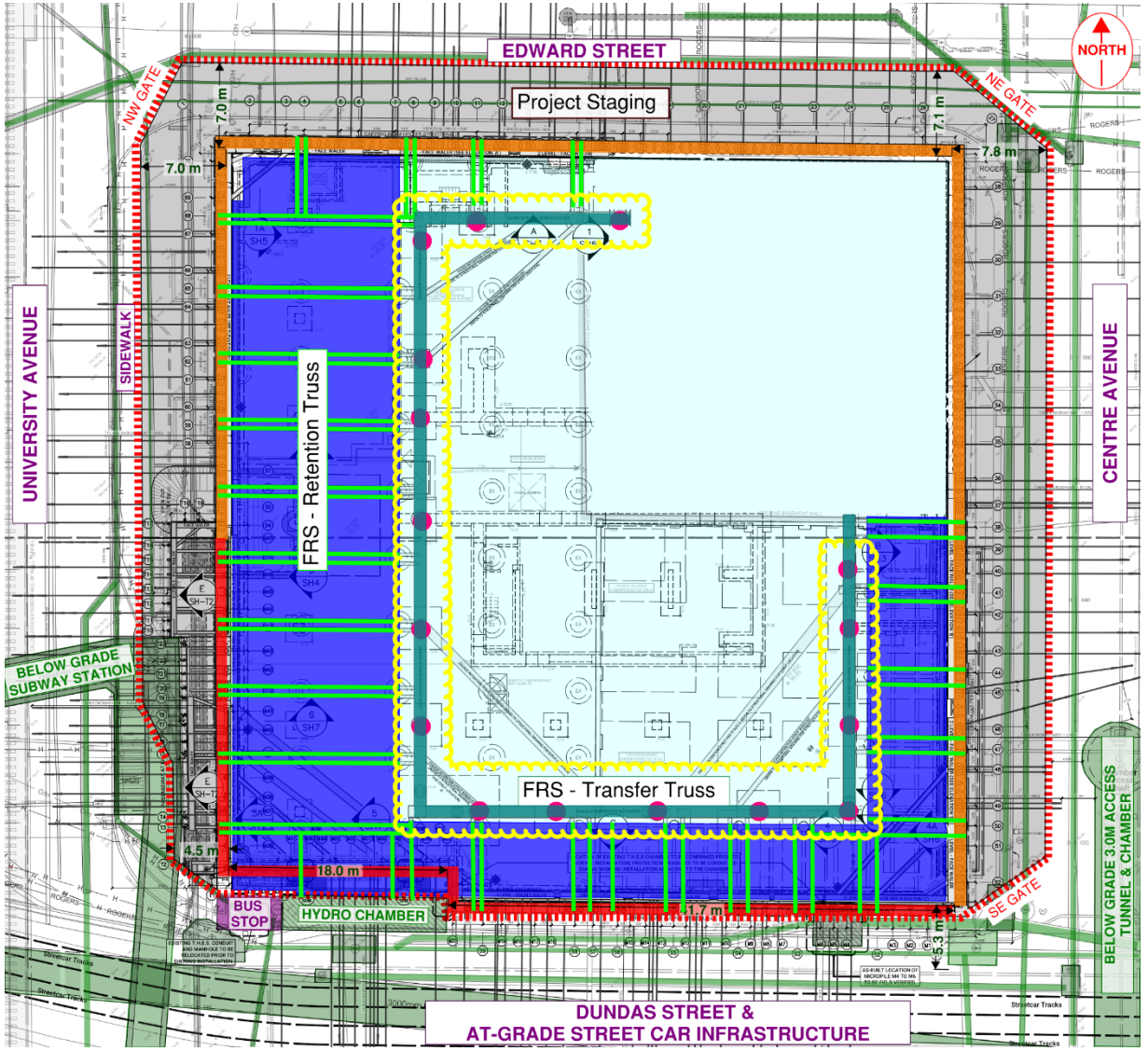
(Imperial Units)





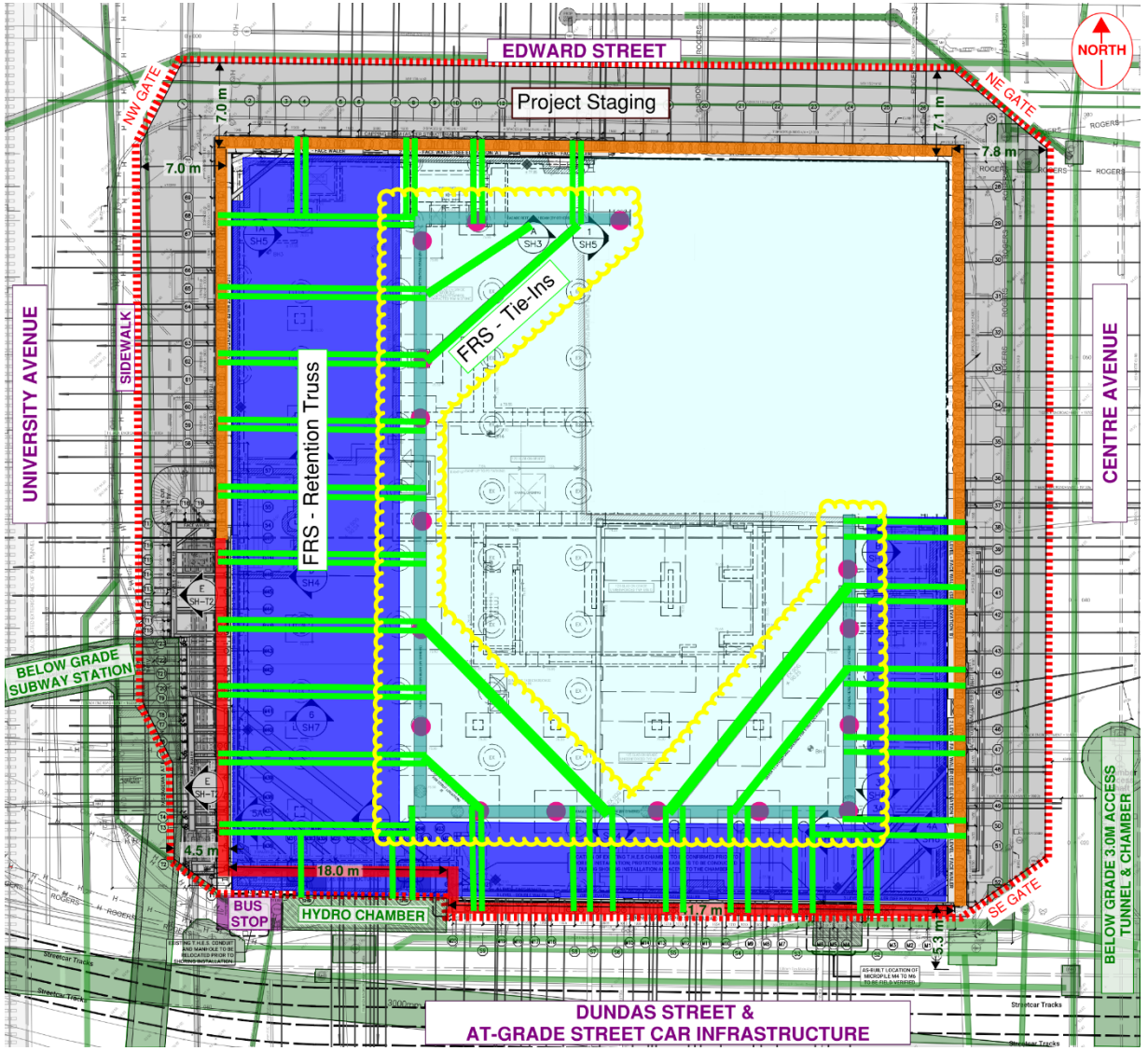
Interior Support - FRS Transfer Truss





Structural Steel – FRS Tie-Ins

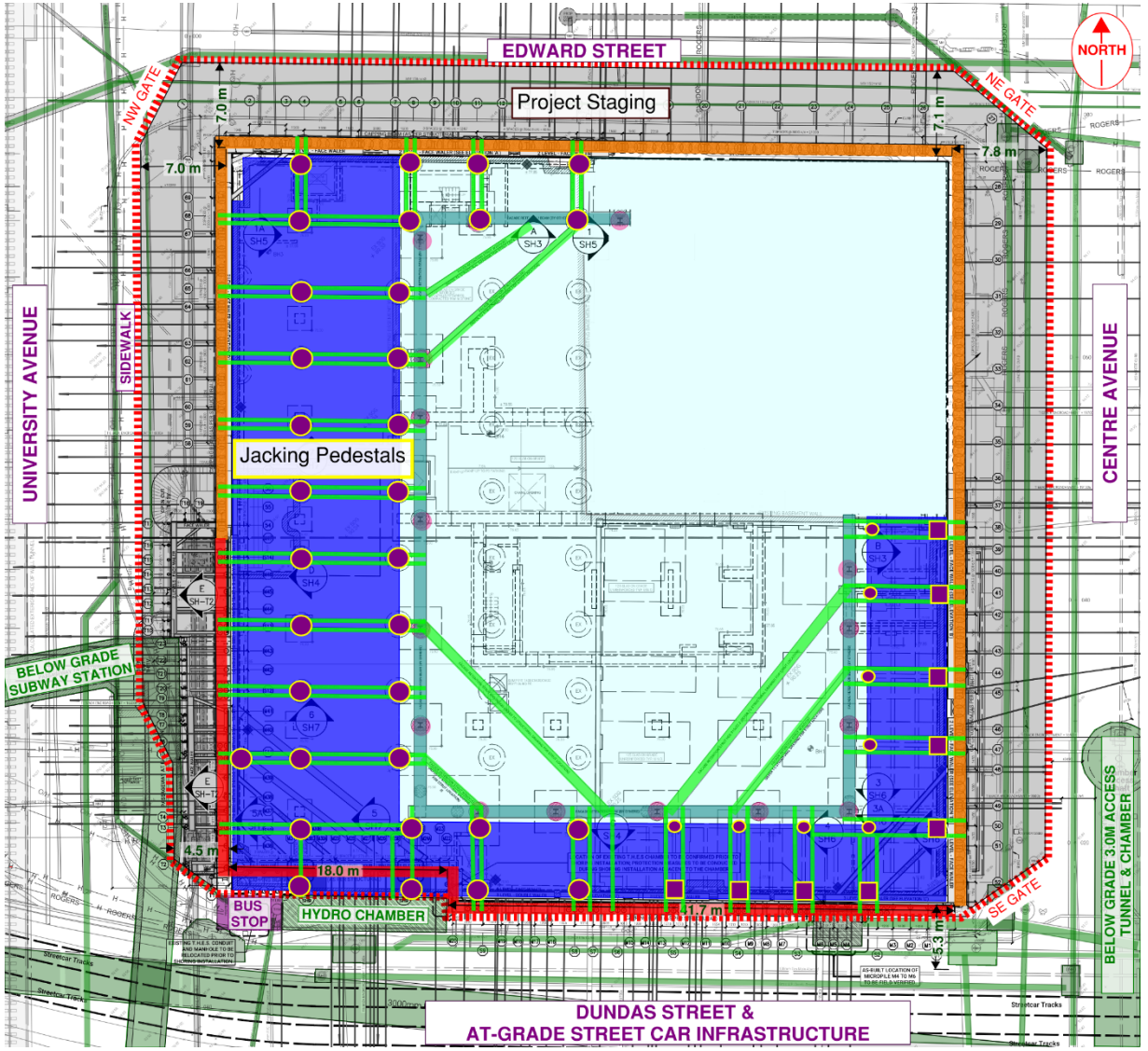




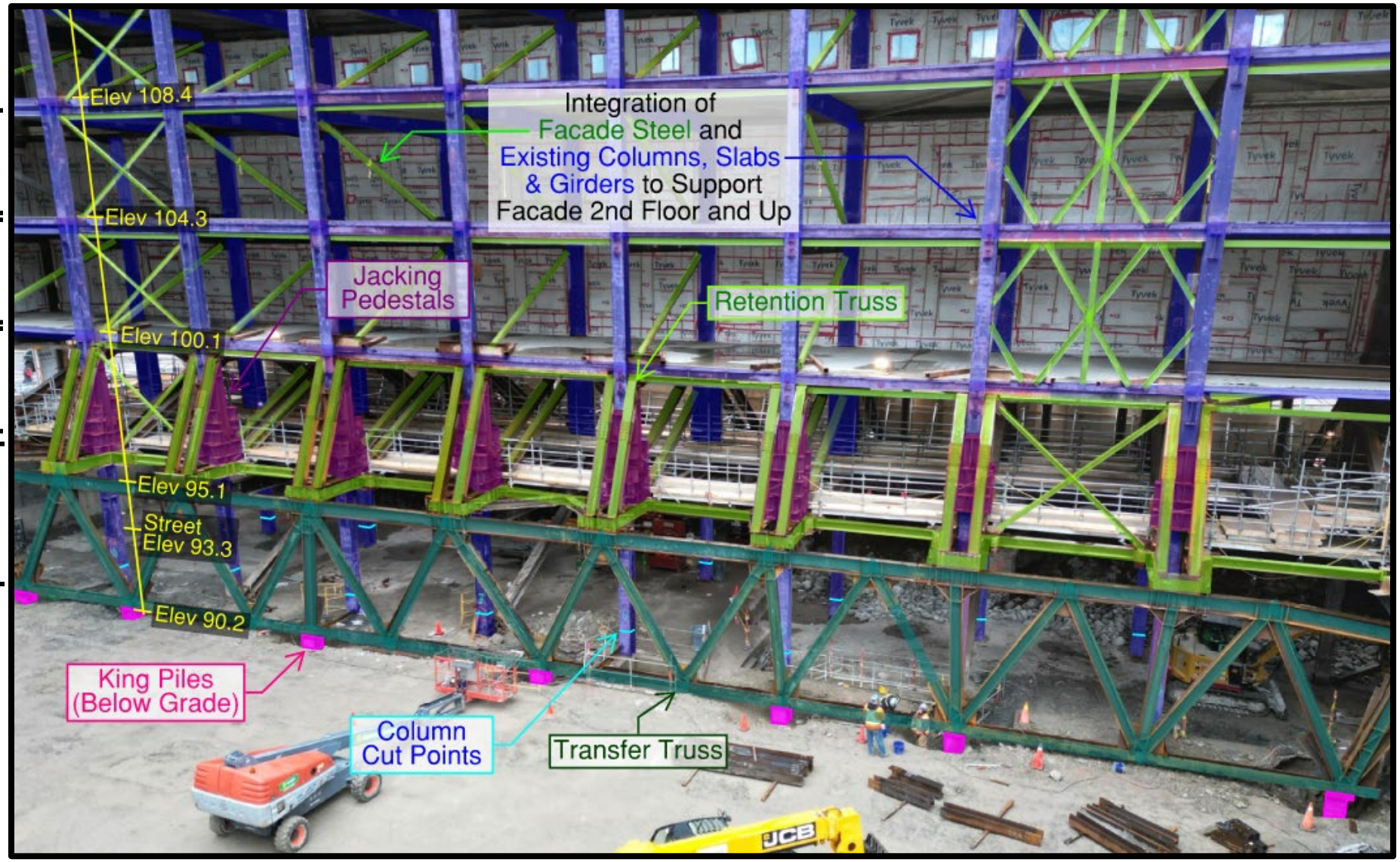
Structural Steel – FRS Jacking Pedestals







FRS / SOE – Integration

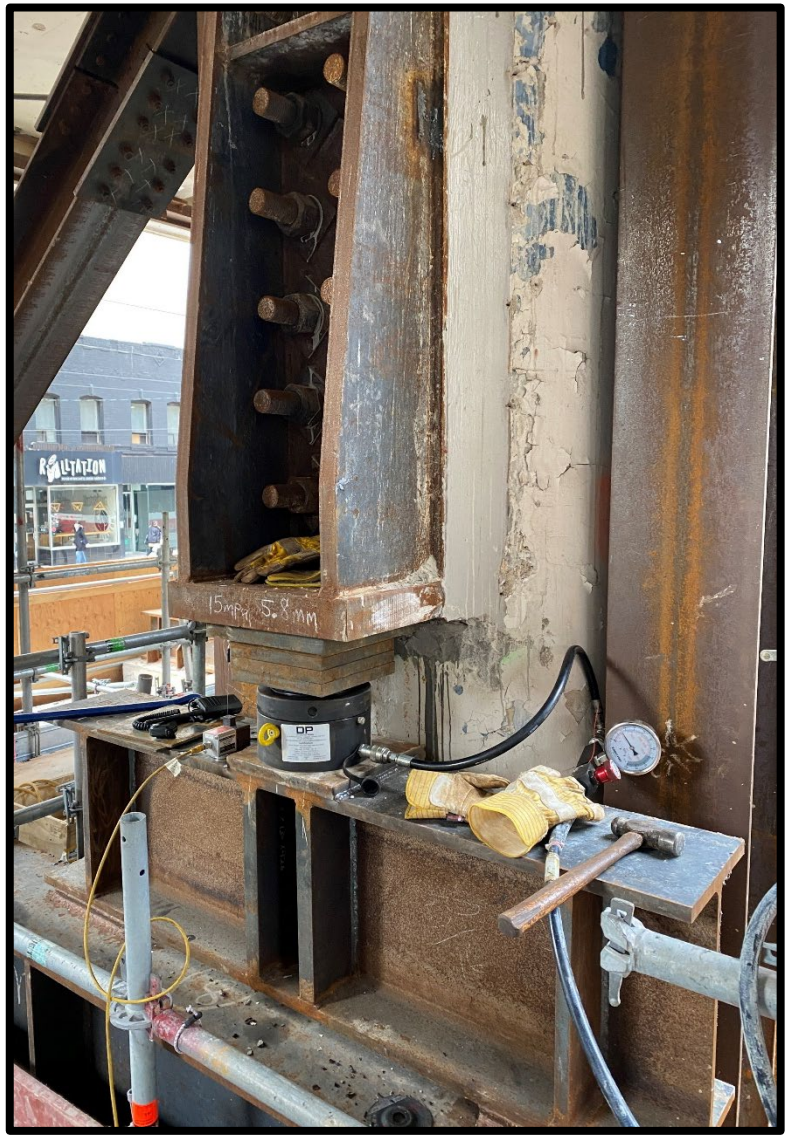


FRS / SOE – Load Transfer

- Estimated Load: **20,000 metric tons (22,000 tons) +/- 3,000** (15%)
  - Each of the 50 remaining columns could have a load between 340 and 460 MT
  - Under or over estimation could lead to excess movement at top of façade during load transfer process – potential for damage or system failure.
- Multi-step jacking process to transfer load incrementally in segments.
  - Three or Four 150 MT hydraulic jacks for each column; some sacrificial.
  - 5 segments for 481 University & 3 segments for 210 Dundas St.
- Monitoring!
  - Façade heavily monitored for vertical & lateral displacement.
  - Strain gauges & displacement gauges located below jacks near cut point.
- Jacking Sequence divided into 5 Steps.
  - 5 steps x 8 segments = 40 steps through entire load transfer.

FRS / SOE – Jacking Sequence – Step 1 & 2

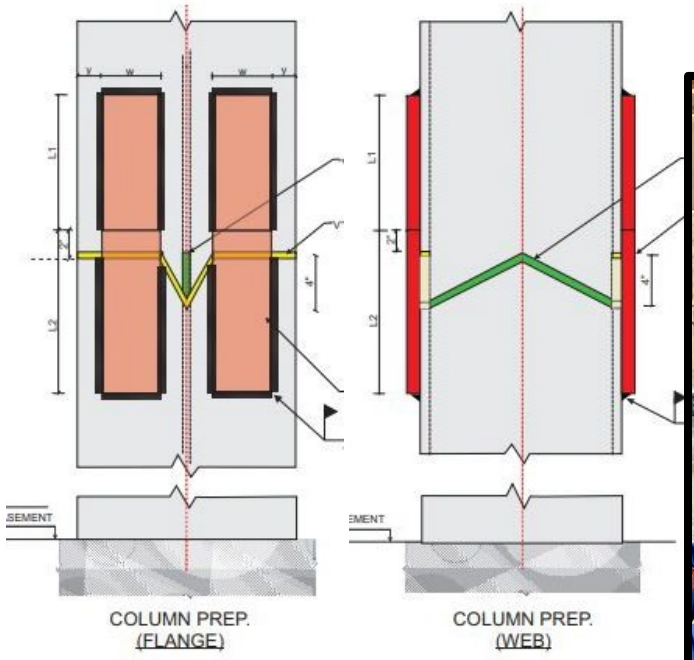
- Step 1 – Load to 50%
  - Jacks loaded in parallel to ~200 MT.
  - Check relative strain in gauge @ 50%.
  - Secure with temporary shims.
  - Remove jacks.
  - Check strain gauge again.
- Step 2 – Load to 75%
  - Jacks loaded in parallel to ~300 MT.
  - Check relative strain in gauge @ 75%.
  - Secure with temporary shims.
  - Remove jacks.
  - Check strain gauge again.

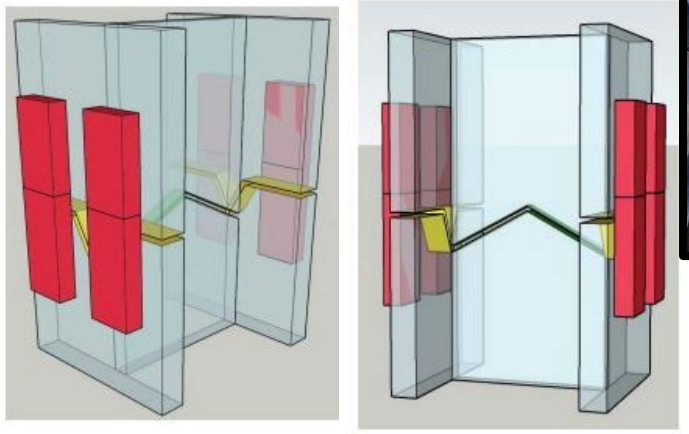




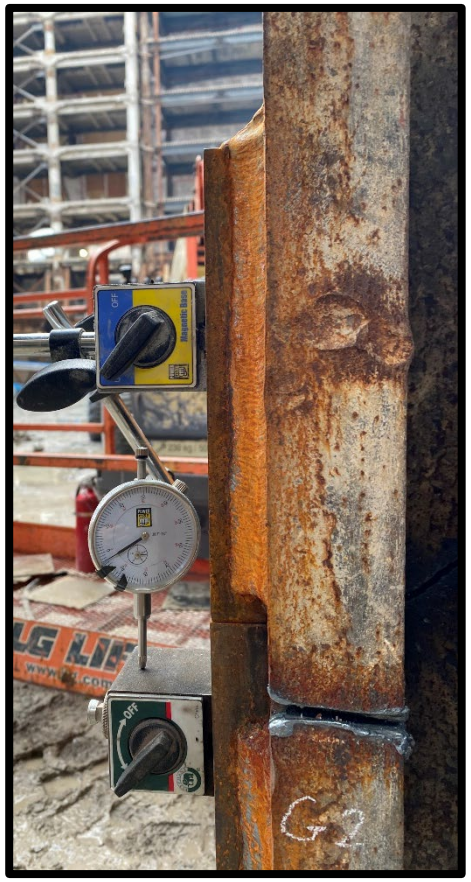
FRS / SOE – Jacking Sequence – Step 3a

- Step 3a – Separate **STEEL** Columns
  - Torch cut one flange, weld in upper & lower compression plate across cut joint. Repeat for all four flanges
  - Torch cut web.
  - Place dial gauge across joint



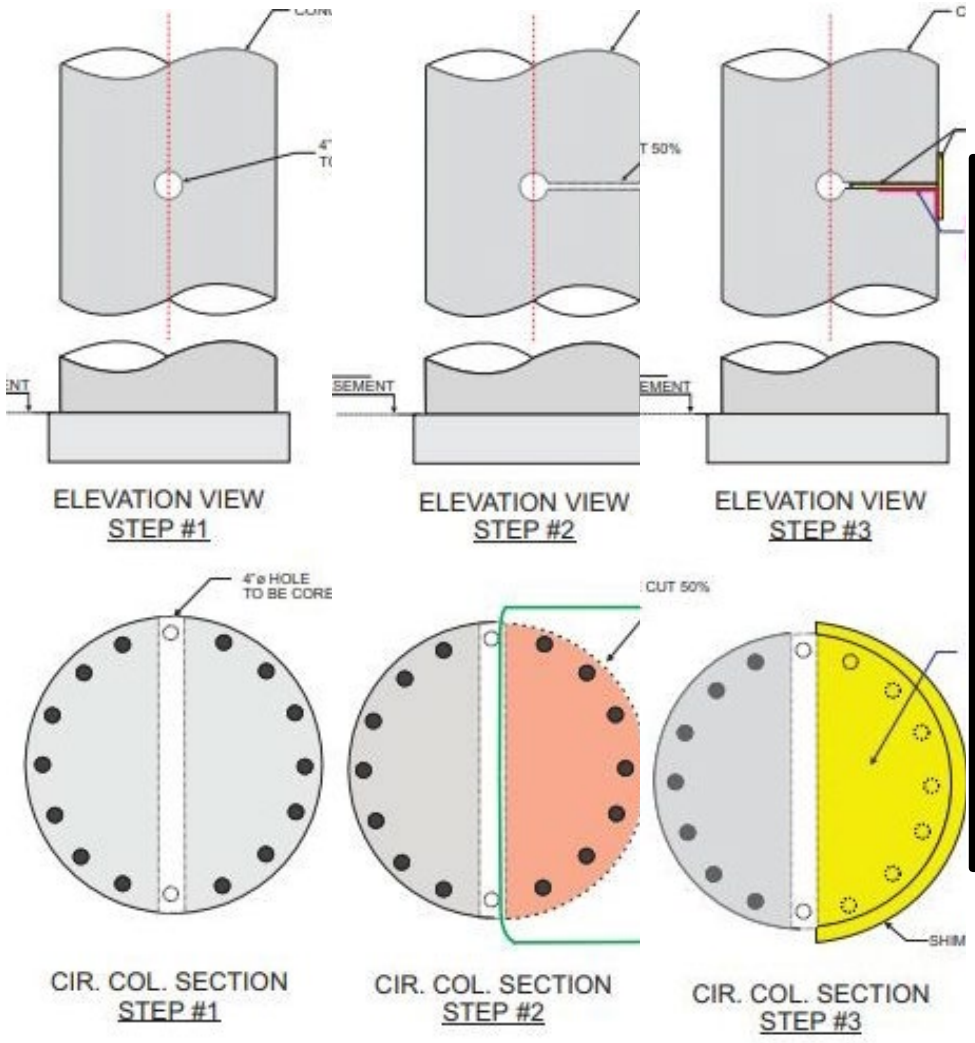






FRS / SOE – Jacking Sequence – Step 3b

- Step 3b – Separate **CONCRETE** Columns
  - Core through center of column.
  - Wire 25mm (1”) slot from side to core.
  - Place bond-breaker on bottom of cut, then grout on top.
  - Repeat for 2nd half.
  - Insert Steel bar through cored hole as a shear key.
  - Place dial gauge across joint.
  - Check strain gauge.







FRS / SOE – Jacking Sequence – Step 4 & 5

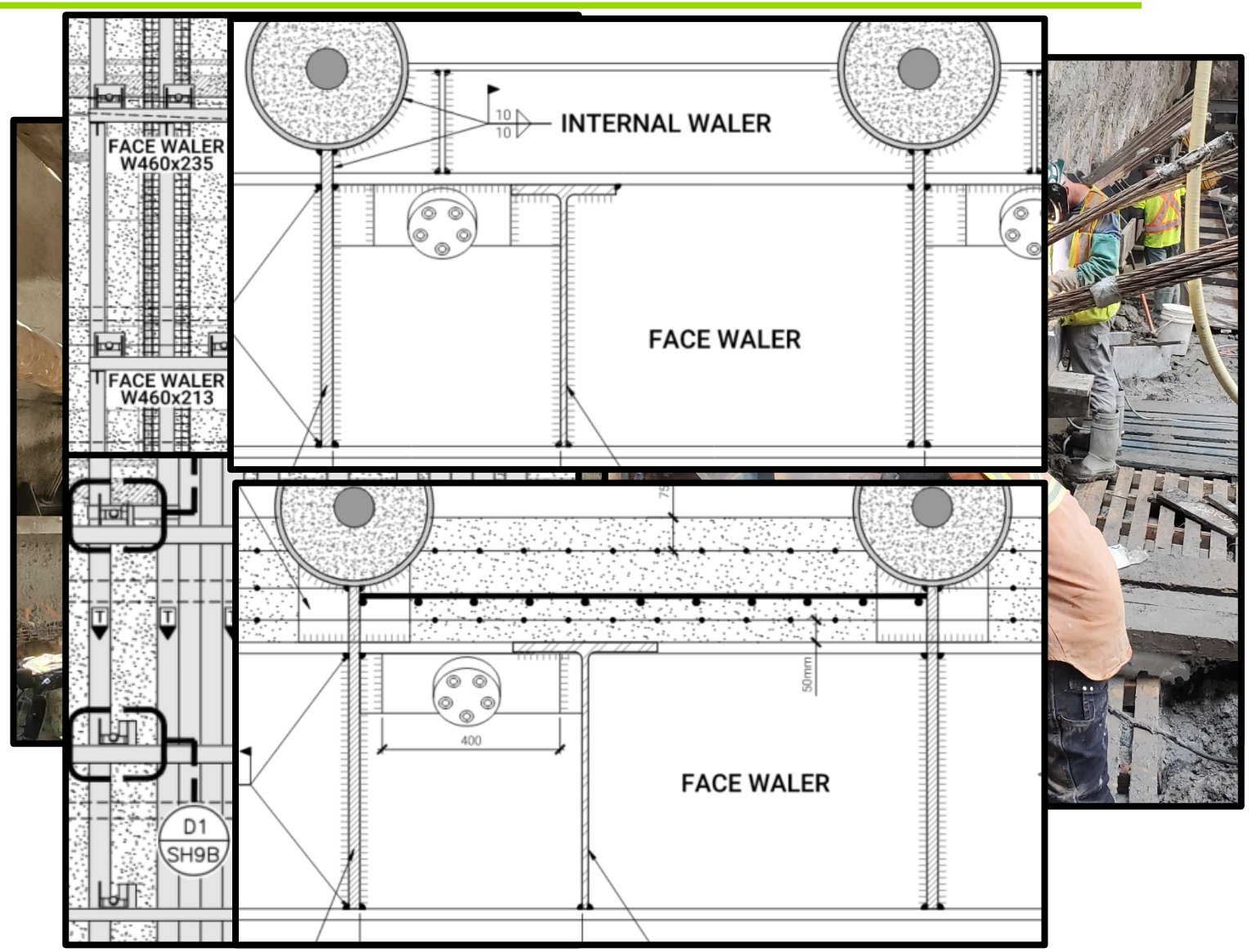
- Step 4 – Load until…
  - Jacks in parallel achieve 100% (400 MT), *or*
  - Strain gauge matches 100% of expected strain, *or*
  - Dial gauge across joint identifies LIFT-OFF.
  - Investigate anomalies.
  - Secure with temporary shims.
- Step 5 – Secure It
  - Stop jacking & hold load.
  - Insert load transfer plates, or adjustable shims.
  - Check gauges.
  - Remove jacks.





SOE – Excavation

- Tiebacks:
  - Maximized for efficiency.
  - Use ROW were possible – adjust skew & position.
- Walers:
  - Discrete to strap FRS secant.
  - Continuous + Internal on micropiles.
- Shotcrete:
  - 150-200 (6-8”) facing with double mesh layer.
  - High early strength allowed replacing Internal Waler with 250mm shotcrete/rebar panel.

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MSSP Software & Testing

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HELICAL PILE SIZING SOFTWARE: A COMPARATIVE ANALYSIS

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MAGNUM GEOENGINEERING designing solutions

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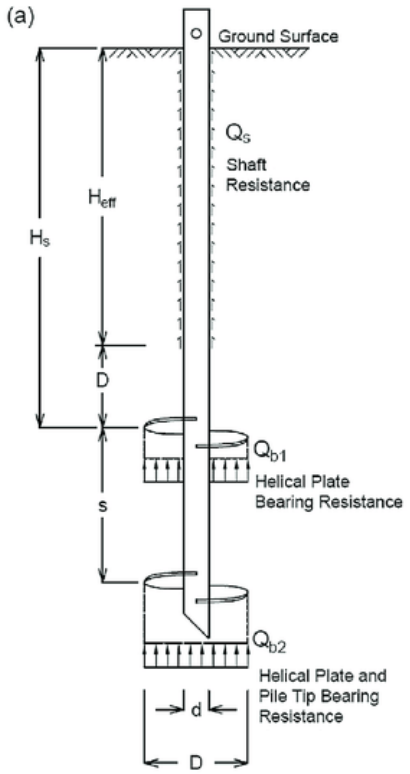
Introduction/Background

- Methods used to size helical piles
- Analysis of different helical pile sizing software
  - Magnum® Geo-App®
  - HeliCAP®
  - HelixPile by Deep Excavation
- Summary of Analysis

Theory Behind Sizing Helical Piles

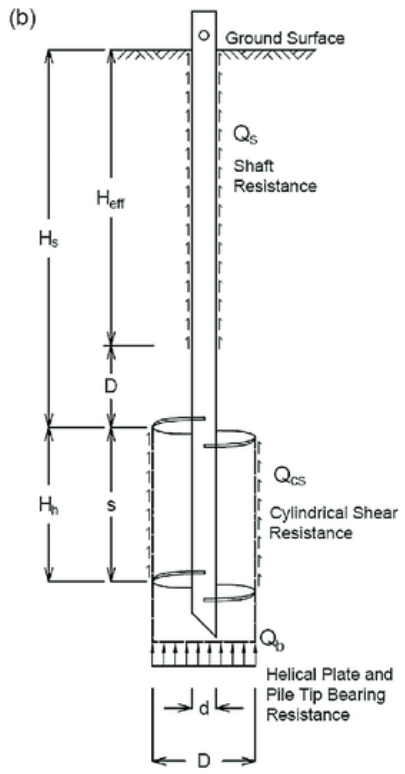
Individual Bearing

$$P_u = \sum_n q_{ult} A_n + \alpha z(\pi d)$$



Cylindrical Shear

$$P_u = q_{ult} A_1 + (n-1)s\pi D_{AVG} + \alpha H(\pi d)$$



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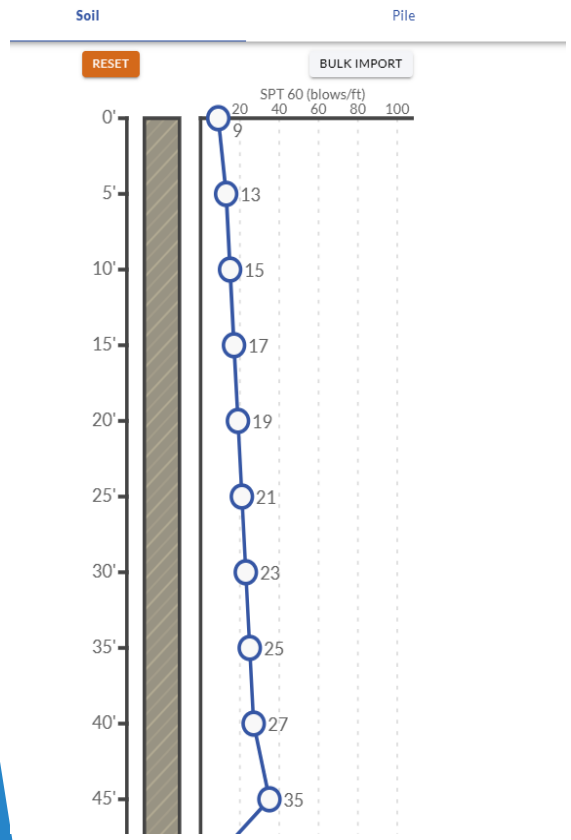

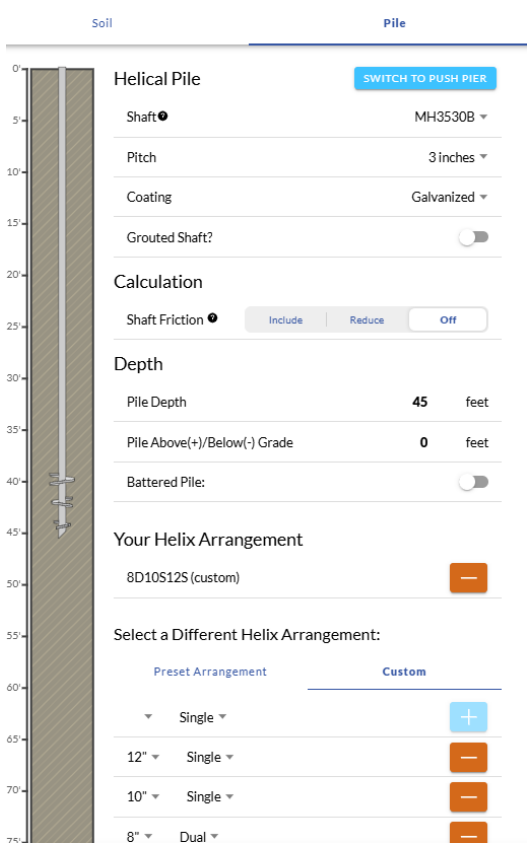

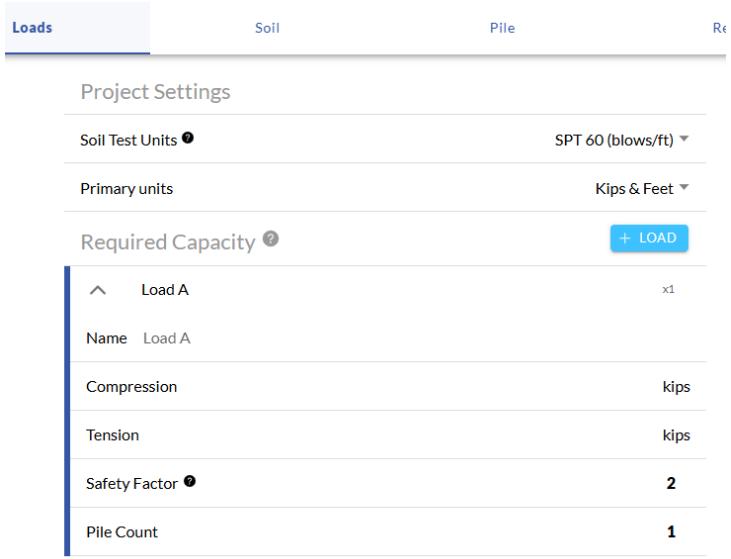

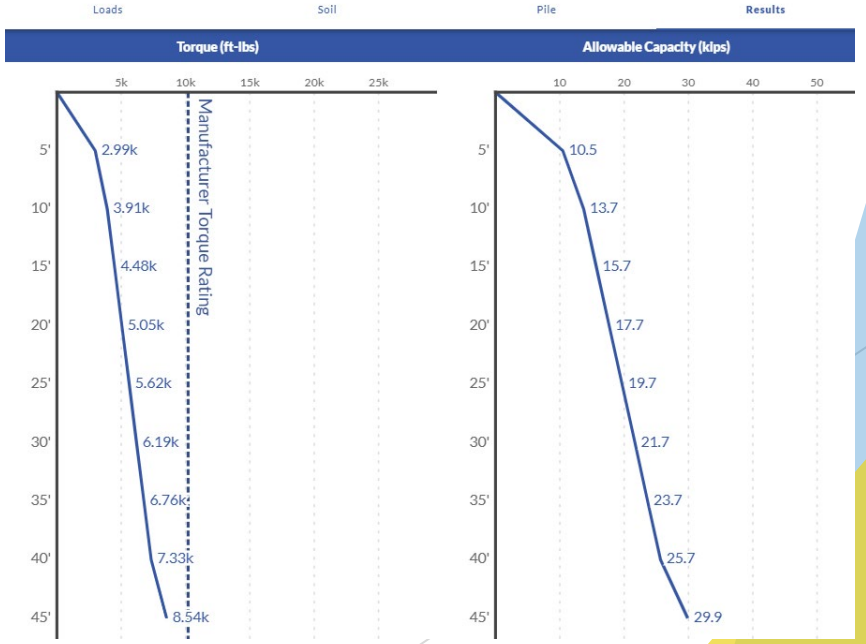

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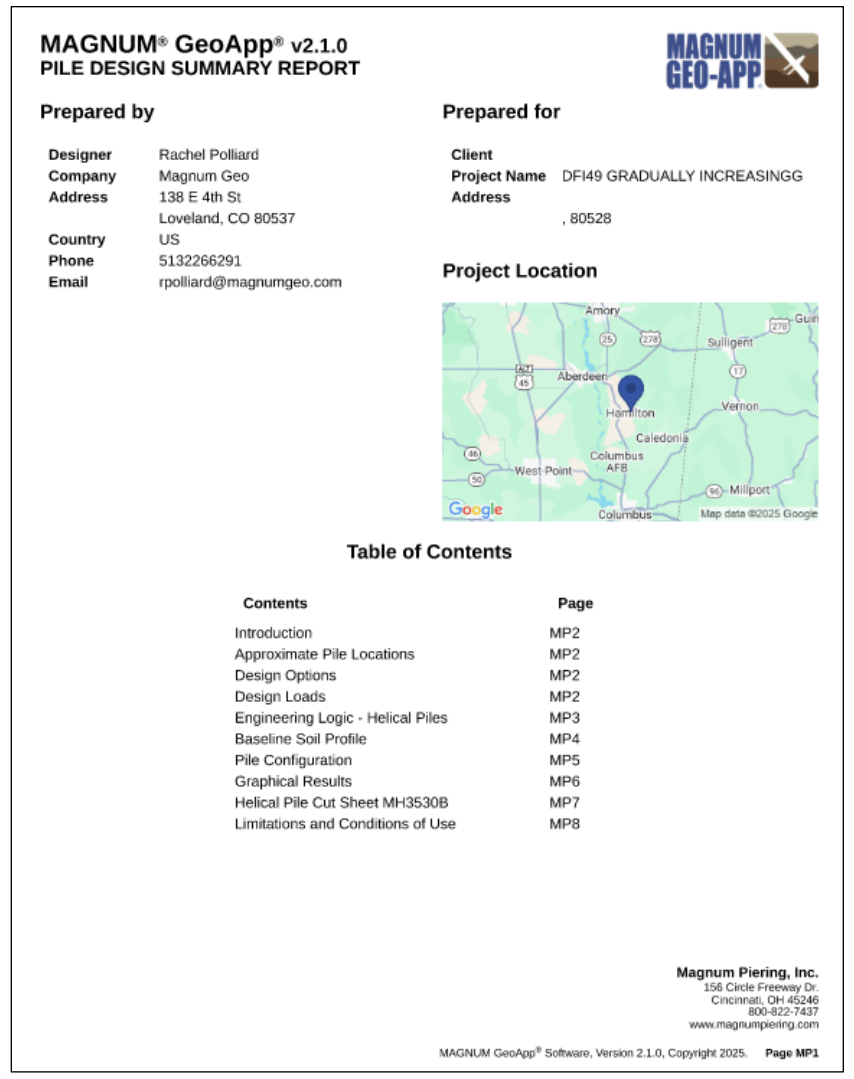

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PILE DESIGN SUMMARY REPORT

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Project Location

Table of Contents

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Pile Configuration	MP5
Graphical Results	MP6
Helical Pile Cut Sheet MH3530B	MP7
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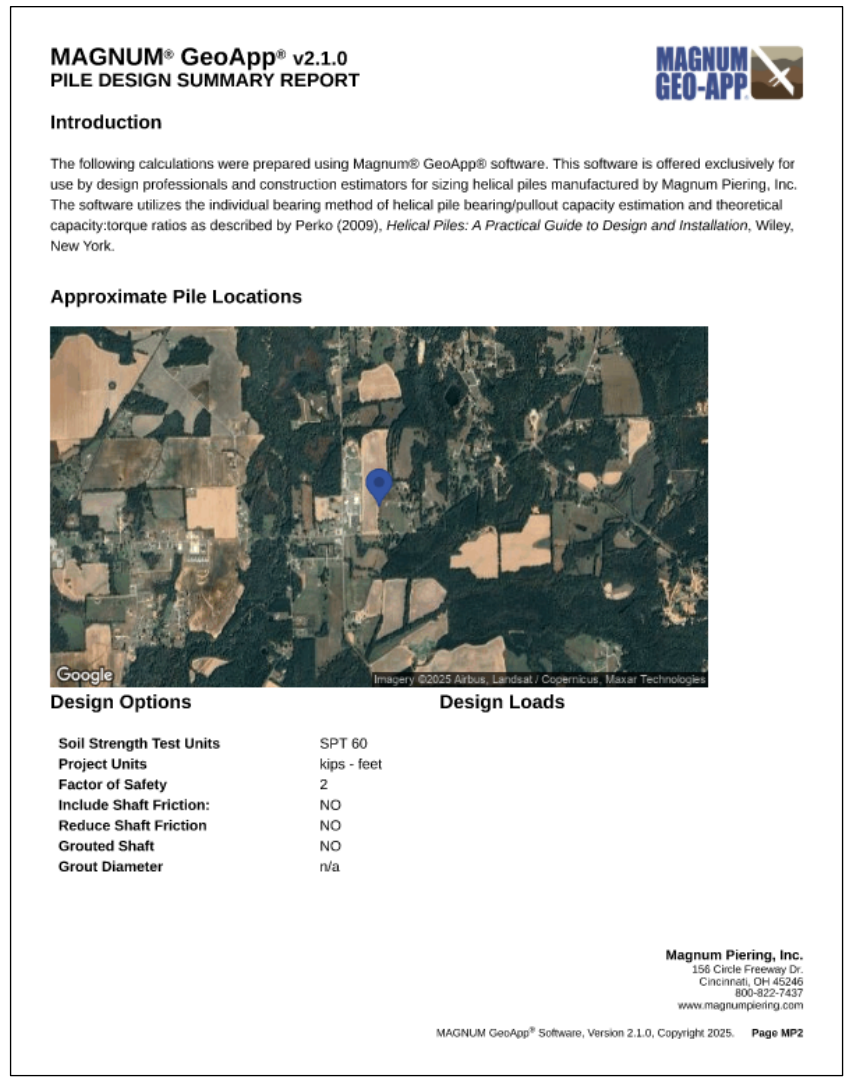
156 Circle Freeway Dr.
Cincinnati, OH 45246
800-822-7437
www.magnumpiering.com



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PILE DESIGN SUMMARY REPORT

Introduction

The following calculations were prepared using Magnum® GeoApp® software. This software is offered exclusively for use by design professionals and construction estimators for sizing helical piles manufactured by Magnum Piering, Inc. The software utilizes the individual bearing method of helical pile bearing/pullout capacity estimation and theoretical capacity:torque ratios as described by Perko (2009), *Helical Piles: A Practical Guide to Design and Installation*, Wiley, New York.

Approximate Pile Locations

Design Options

Soil Strength Test Units	SPT 60
Project Units	kips - feet
Factor of Safety	2
Include Shaft Friction:	NO
Reduce Shaft Friction	NO
Grouted Shaft	NO
Grout Diameter	n/a

Design Loads

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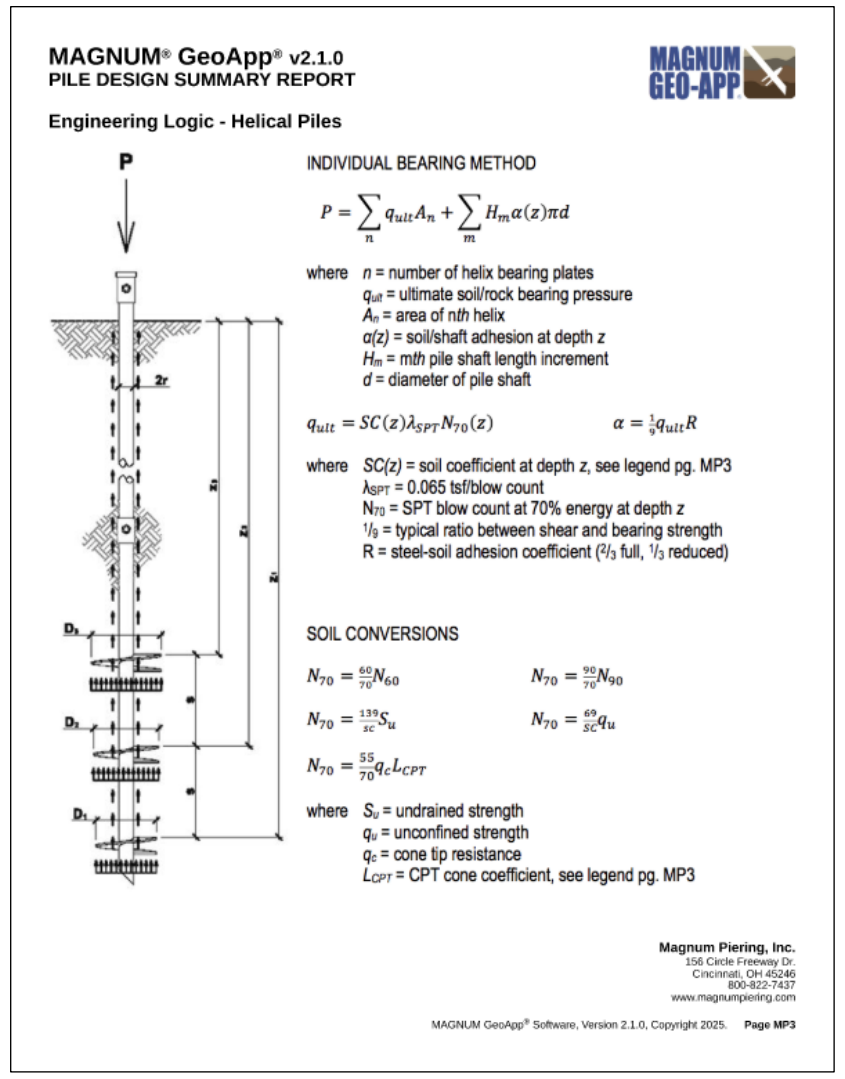

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PILE DESIGN SUMMARY REPORT

Engineering Logic - Helical Piles

INDIVIDUAL BEARING METHOD

$$P = \sum_n q_{ult} A_n + \sum_m H_m \alpha(z) \pi d$$

where n = number of helix bearing plates
q_{ult} = ultimate soil/rock bearing pressure
A_n = area of nth helix
$\alpha(z)$ = soil/shaft adhesion at depth z
H_m = mth pile shaft length increment
d = diameter of pile shaft

$$q_{ult} = SC(z)\lambda_{SPT} N_{70}(z) \qquad \alpha = \tfrac{1}{9} q_{ult} R$$

where $SC(z)$ = soil coefficient at depth z, see legend pg. MP3
λ_{SPT} = 0.065 tsf/blow count
N_{70} = SPT blow count at 70% energy at depth z
$1/9$ = typical ratio between shear and bearing strength
R = steel-soil adhesion coefficient ($2/3$ full, $1/3$ reduced)

SOIL CONVERSIONS

$$N_{70} = \tfrac{60}{70} N_{60} \qquad N_{70} = \tfrac{90}{70} N_{90}$$

$$N_{70} = \tfrac{139}{sc} S_u \qquad N_{70} = \tfrac{69}{SC} q_u$$

$$N_{70} = \tfrac{55}{70} q_c L_{CPT}$$

where S_u = undrained strength
q_u = unconfined strength
q_c = cone tip resistance
L_{CPT} = CPT cone coefficient, see legend pg. MP3

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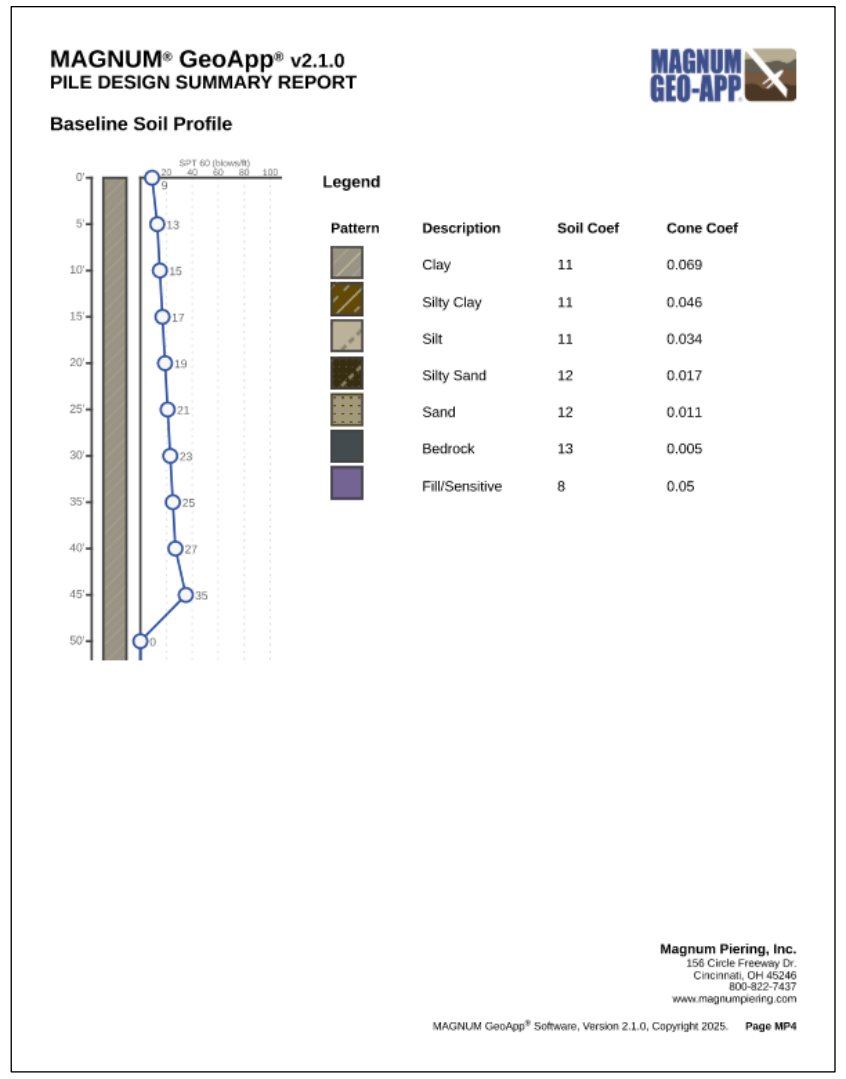

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PILE DESIGN SUMMARY REPORT

Baseline Soil Profile

Legend

Pattern	Description	Soil Coef	Cone Coef
	Clay	11	0.069
	Silty Clay	11	0.046
	Silt	11	0.034
	Silty Sand	12	0.017
	Sand	12	0.011
	Bedrock	13	0.005
	Fill/Sensitive	8	0.05

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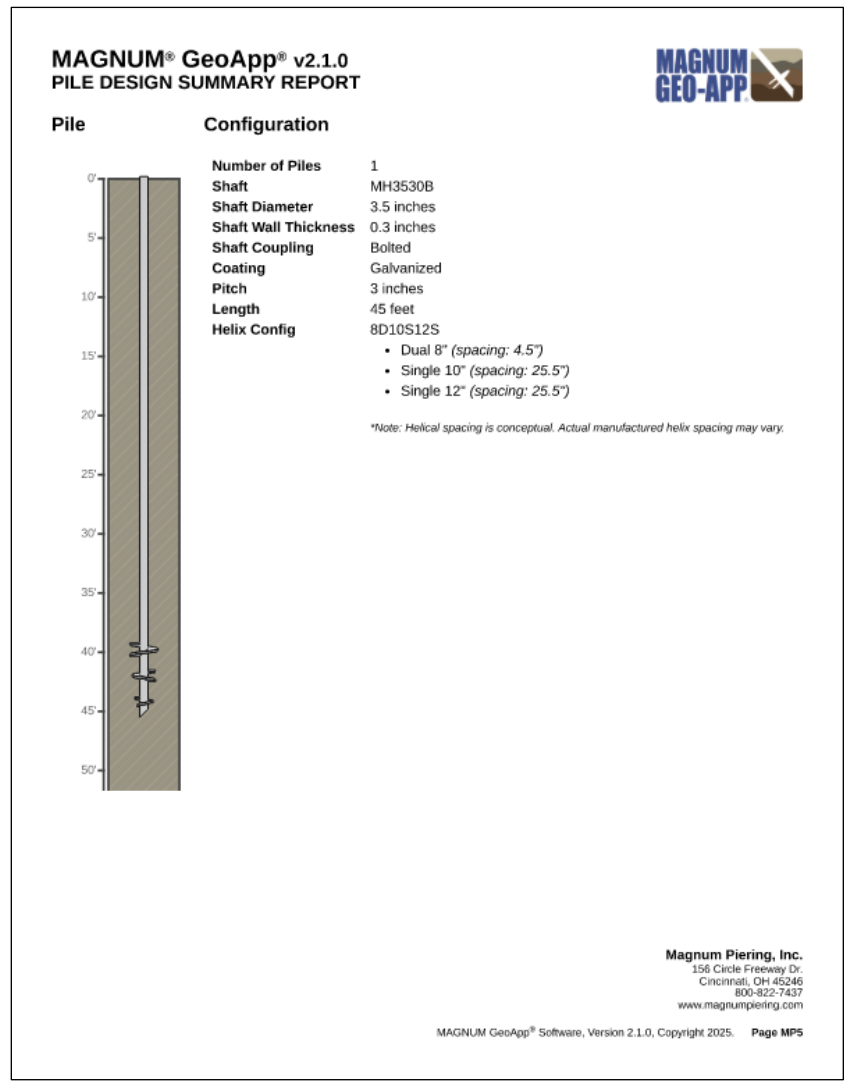

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PILE DESIGN SUMMARY REPORT

Pile

Configuration

Number of Piles	1
Shaft	MH3530B
Shaft Diameter	3.5 inches
Shaft Wall Thickness	0.3 inches
Shaft Coupling	Bolted
Coating	Galvanized
Pitch	3 inches
Length	45 feet
Helix Config	8D10S12S

- Dual 8" *(spacing: 4.5")*
- Single 10" *(spacing: 25.5")*
- Single 12" *(spacing: 25.5")*

**Note: Helical spacing is conceptual. Actual manufactured helix spacing may vary.*

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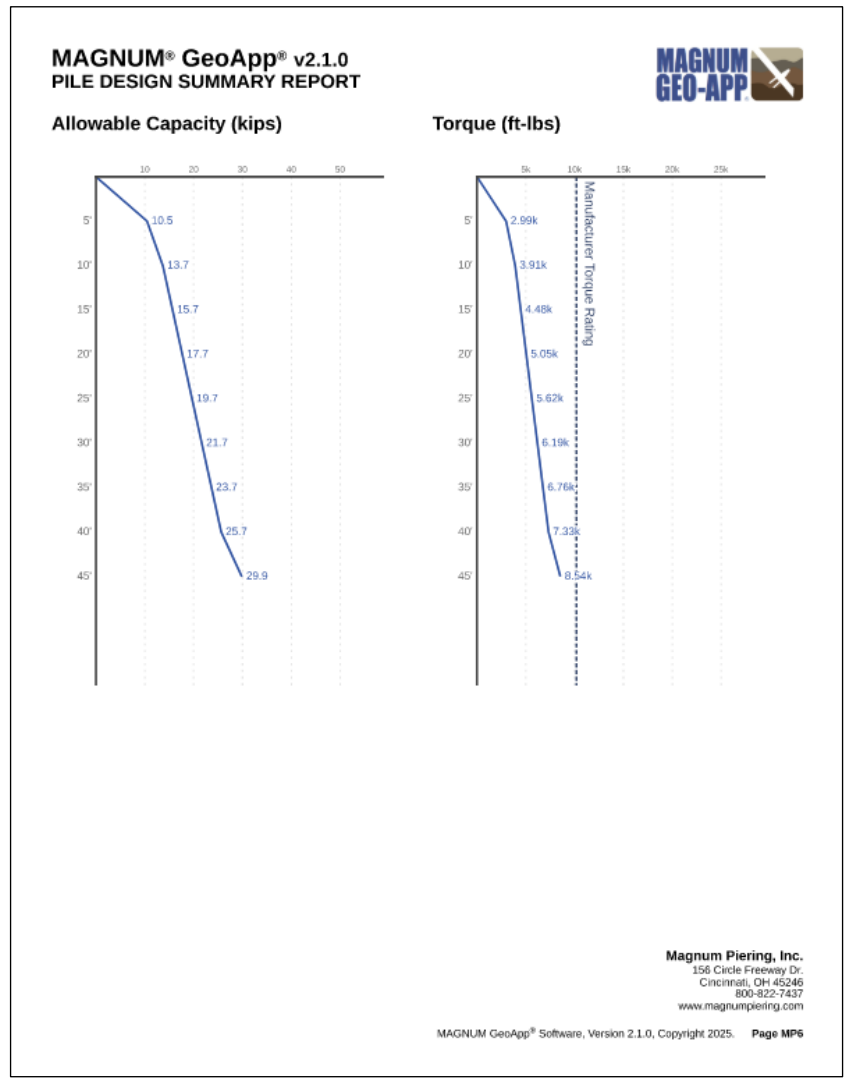

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Allowable Capacity (kips)

Torque (ft-lbs)

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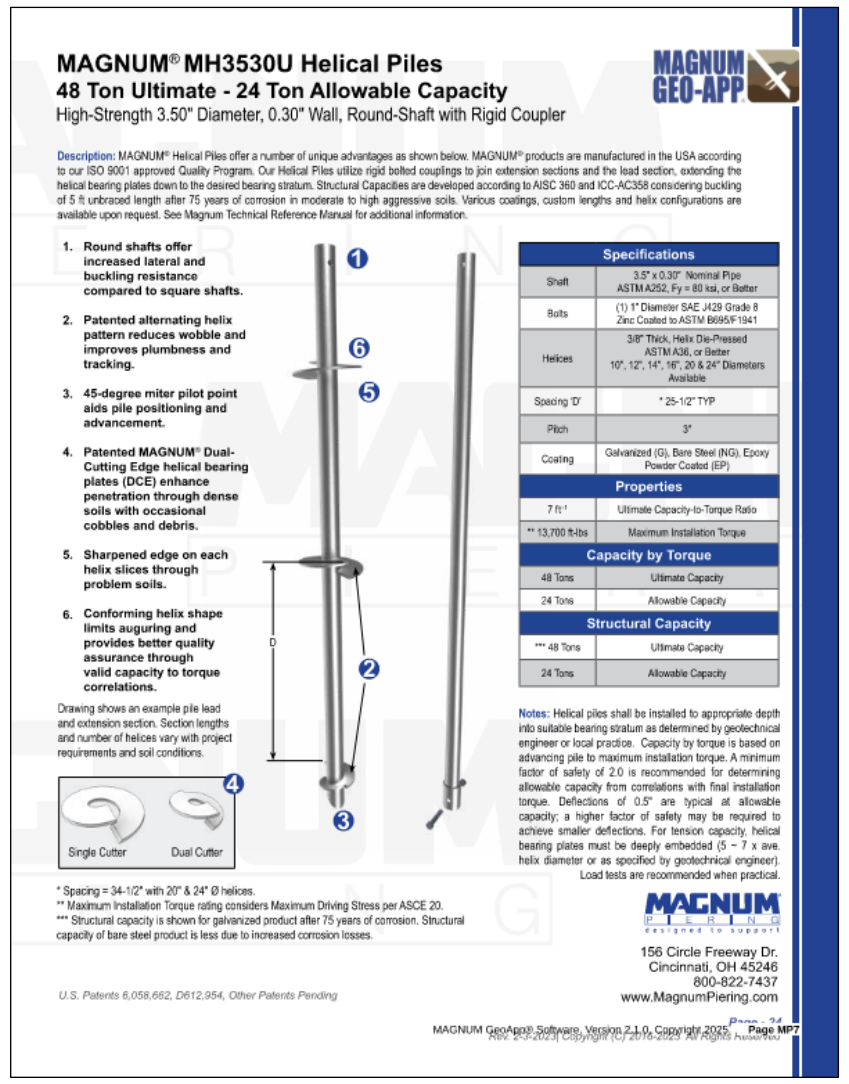

MAGNUM® MH3530U Helical Piles
48 Ton Ultimate - 24 Ton Allowable Capacity
High-Strength 3.50" Diameter, 0.30" Wall, Round-Shaft with Rigid Coupler

Description: MAGNUM® Helical Piles offer a number of unique advantages as shown below. MAGNUM® products are manufactured in the USA according to our ISO 9001 approved Quality Program. Our Helical Piles utilize rigid bolted couplings to join extension sections and the lead section, extending the helical bearing plates down to the desired bearing stratum. Structural Capacities are developed according to AISC 360 and ICC-AC358 considering buckling of 5 ft unbraced length after 75 years of corrosion in moderate to high aggressive soils. Various coatings, custom lengths and helix configurations are available upon request. See Magnum Technical Reference Manual for additional information.

1. **Round shafts offer increased lateral and buckling resistance compared to square shafts.**
2. **Patented alternating helix pattern reduces wobble and improves plumbness and tracking.**
3. **45-degree miter pilot point aids pile positioning and advancement.**
4. **Patented MAGNUM® Dual-Cutting Edge helical bearing plates (DCE) enhance penetration through dense soils with occasional cobbles and debris.**
5. **Sharpened edge on each helix slices through problem soils.**
6. **Conforming helix shape limits auguring and provides better quality assurance through valid capacity to torque correlations.**

Drawing shows an example pile lead and extension section. Section lengths and number of helices vary with project requirements and soil conditions.

Specifications	
Shaft	3.5" x 0.30" Nominal Pipe ASTM A252, Fy = 80 ksi, or Better
Bolts	(1) 1" Diameter SAE J429 Grade 8 Zinc Coated to ASTM B695/F1941
Helices	3/8" Thick, Helix Die-Pressed ASTM A36, or Better 10", 12", 14", 16", 20 & 24" Diameters Available
Spacing 'D'	* 25-1/2" TYP
Pitch	3"
Coating	Galvanized (G), Bare Steel (NG), Epoxy Powder Coated (EP)
Properties	
7 ft⁻¹	Ultimate Capacity-to-Torque Ratio
** 13,700 ft-lbs	Maximum Installation Torque
Capacity by Torque	
48 Tons	Ultimate Capacity
24 Tons	Allowable Capacity
Structural Capacity	
*** 48 Tons	Ultimate Capacity
24 Tons	Allowable Capacity

Notes: Helical piles shall be installed to appropriate depth into suitable bearing stratum as determined by geotechnical engineer or local practice. Capacity by torque is based on advancing pile to maximum installation torque. A minimum factor of safety of 2.0 is recommended for determining allowable capacity from correlations with final installation torque. Deflections of 0.5" are typical at allowable capacity; a higher factor of safety may be required to achieve smaller deflections. For tension capacity, helical bearing plates must be deeply embedded (5 ~ 7 x ave. helix diameter or as specified by geotechnical engineer). Load tests are recommended when practical.

* Spacing = 34-1/2" with 20" & 24" Ø helices.
** Maximum Installation Torque rating considers Maximum Driving Stress per ASCE 20.
*** Structural capacity is shown for galvanized product after 75 years of corrosion. Structural capacity of bare steel product is less due to increased corrosion losses.

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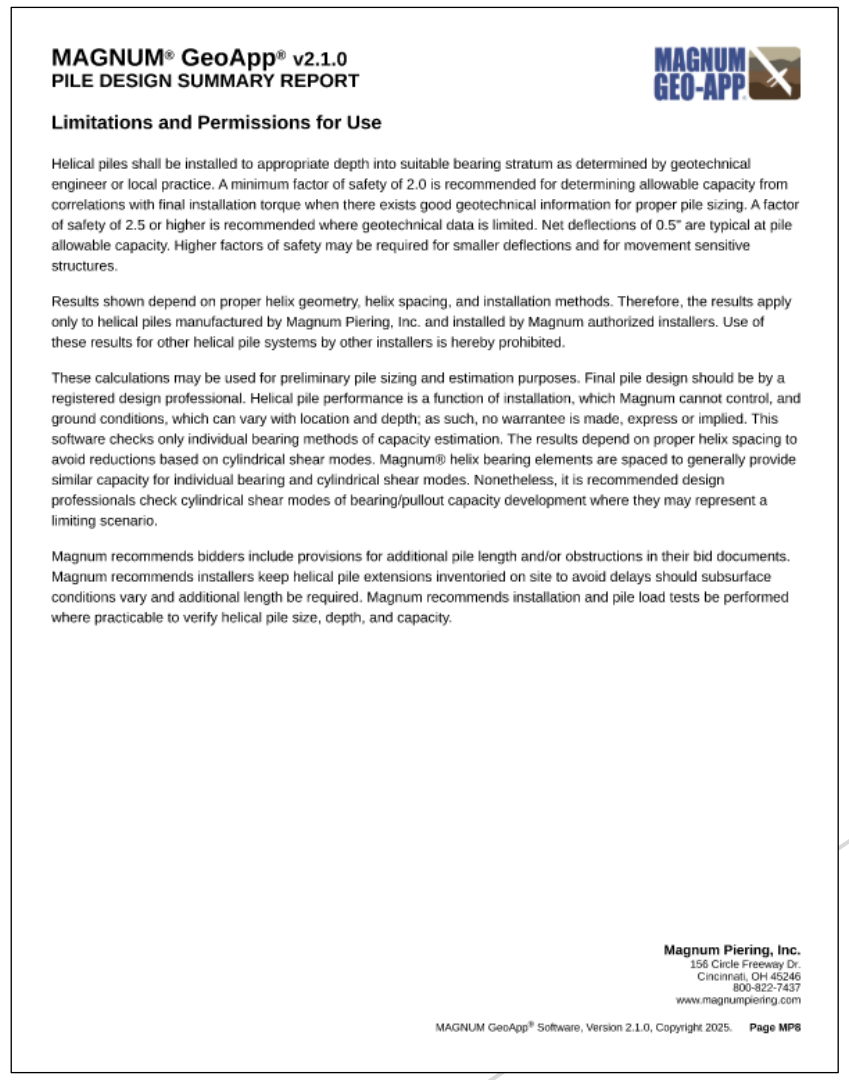

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PILE DESIGN SUMMARY REPORT

Limitations and Permissions for Use

Helical piles shall be installed to appropriate depth into suitable bearing stratum as determined by geotechnical engineer or local practice. A minimum factor of safety of 2.0 is recommended for determining allowable capacity from correlations with final installation torque when there exists good geotechnical information for proper pile sizing. A factor of safety of 2.5 or higher is recommended where geotechnical data is limited. Net deflections of 0.5" are typical at pile allowable capacity. Higher factors of safety may be required for smaller deflections and for movement sensitive structures.

Results shown depend on proper helix geometry, helix spacing, and installation methods. Therefore, the results apply only to helical piles manufactured by Magnum Piering, Inc. and installed by Magnum authorized installers. Use of these results for other helical pile systems by other installers is hereby prohibited.

These calculations may be used for preliminary pile sizing and estimation purposes. Final pile design should be by a registered design professional. Helical pile performance is a function of installation, which Magnum cannot control, and ground conditions, which can vary with location and depth; as such, no warrantee is made, express or implied. This software checks only individual bearing methods of capacity estimation. The results depend on proper helix spacing to avoid reductions based on cylindrical shear modes. Magnum® helix bearing elements are spaced to generally provide similar capacity for individual bearing and cylindrical shear modes. Nonetheless, it is recommended design professionals check cylindrical shear modes of bearing/pullout capacity development where they may represent a limiting scenario.

Magnum recommends bidders include provisions for additional pile length and/or obstructions in their bid documents. Magnum recommends installers keep helical pile extensions inventoried on site to avoid delays should subsurface conditions vary and additional length be required. Magnum recommends installation and pile load tests be performed where practicable to verify helical pile size, depth, and capacity.

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HELICAP

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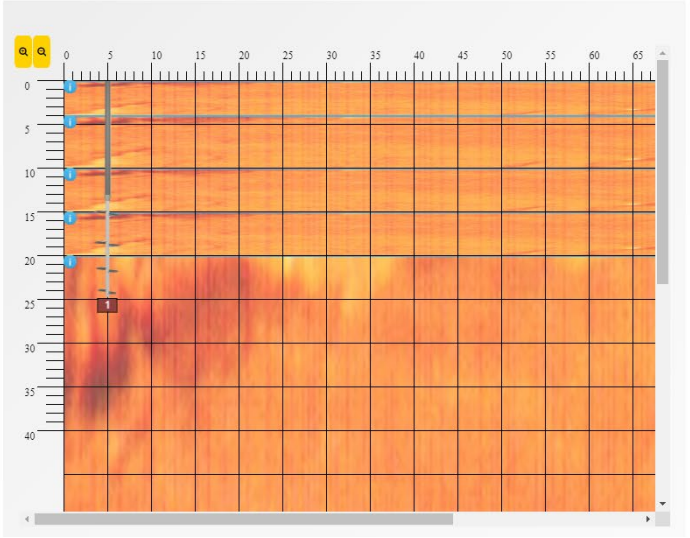
B-3

Application Mode: Compression | Display Depth: 40 | Grid Lines: ON | SAVE



Water Table Depth (ft): None | Hammer Efficiency: Auto Hammer (80%) | Critical Depth (ft): None

SOIL PROFILE

	Depth (ft)	Soil Type	N	N_{60}	Cohesion (psf)	Angle of Internal Friction (degrees)	In Situ Unit Weight (pcf)	Clay Bearing Capacity Factor (N_c)	Sand Bearing Capacity Factor (N_q)	Bond Value (psf)
1	0	Clay	7	9	1125	0	98	9	0	
2	4	Clay	9	12	1500	0	104	9	0	
3	10	Clay	7	9	1125	0	98	9	0	
4	15	Clay	8	11	1375	0	102	9	0	
5	20	Clay	19	25	3125	0	120	9	0	
6										

SAVE | CALCULATE | PRINT

HELICAL PILE: 1

Pile Length (ft): 20 | Installation Angle (degrees): 90 | Datum Depth (ft): 0

Pile Type: Square Shaft (SS) | Product Series: SS125 | Calculate: Bearing Only

User Defined K_t: 10 | Number of Helices: 2 | Configuration: 10,12

Items in red are custom user inputs.

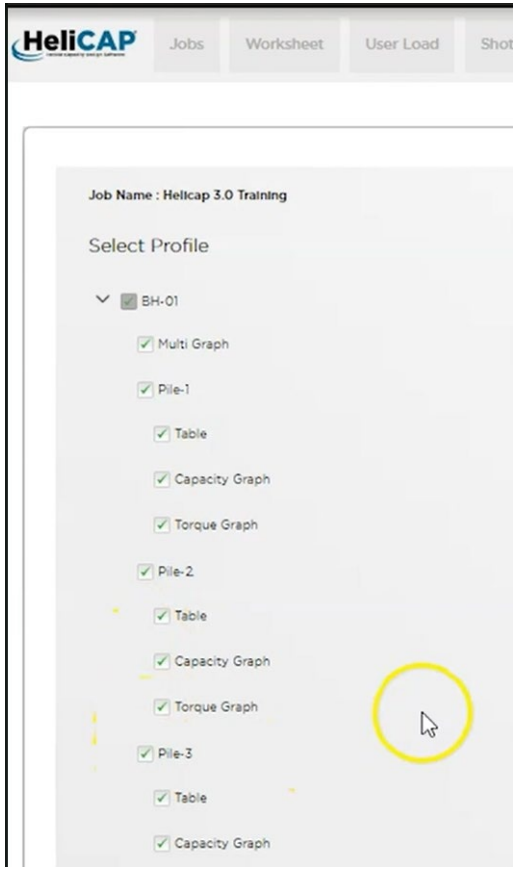
Profile | Series | Table | Capacity Graph | Torque Graph

Helical Pile/Anchor: 1 — INSERT PILE | UPDATE | DELETE

Highlight row below to select your anchor/pile.

	Length	Install Angle	Datum Depth	Series	# of Helices
1	20	90	25	SS125	2
2	40	90	0	SS5 / RS2875.203BC	3

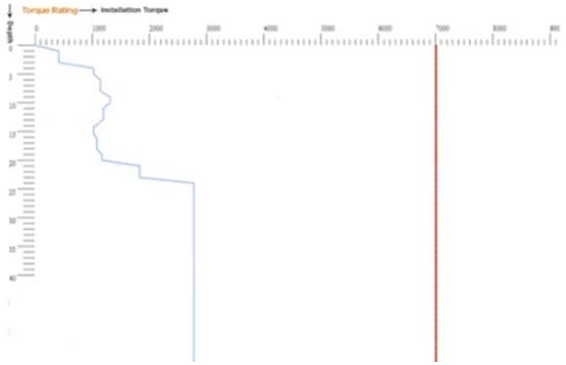
HELICAP REPORT

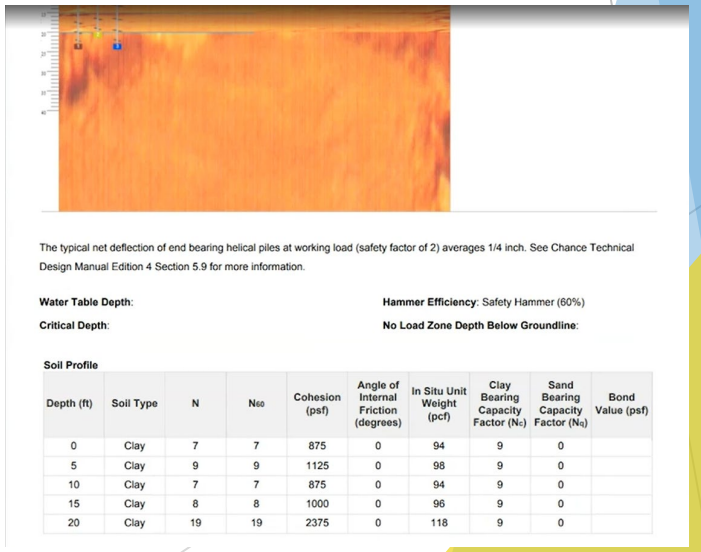


HeliCAP
Helical Capacity Design Software

Summary Report

Helical Pile Number: 2	Product: SS150		Installation Torque: 1170 ft·lb	Effective Torque: 1141 ft·lb
Length: 20.0 ft	Angle: 90.0 degree	Datum Depth: 0.0 ft		
Helix Diameter (in)	**Helix Depth (ft)**	**Ultimate Helix Geotechnical Capacity (q_G) (kip)**	**Nominal Helix Strength (kip)**	**Ultimate Helix Recommended Capacity (q_R) (kip)**
12	17.0	6.6t 6.9c	49.7	6.6t 6.9c
10	19.5	4.8t 9.1c	61.9	4.8t 9.1c
Total Ultimate Helix Geotechnical Capacity (Q_G) (kip)		11.4t 16.1c		
Total Ultimate Helix Recommended Capacity (Q_R) (kip)				11.4t 16.1c





The typical net deflection of end bearing helical piles at working load (safety factor of 2) averages 1/4 inch. See Chance Technical Design Manual Edition 4 Section 5.9 for more information.

Water Table Depth:

Hammer Efficiency: Safety Hammer (60%)

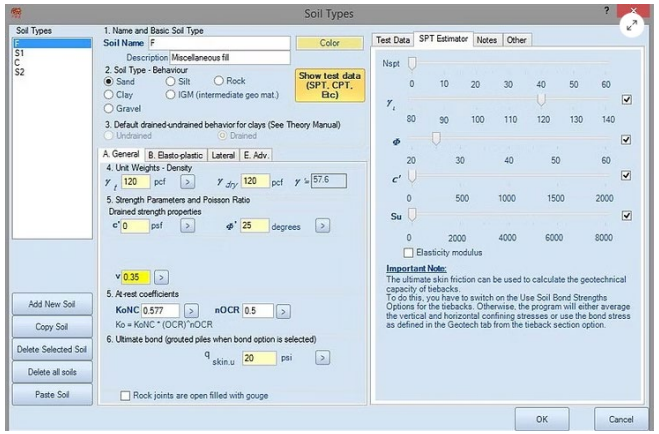
Critical Depth:

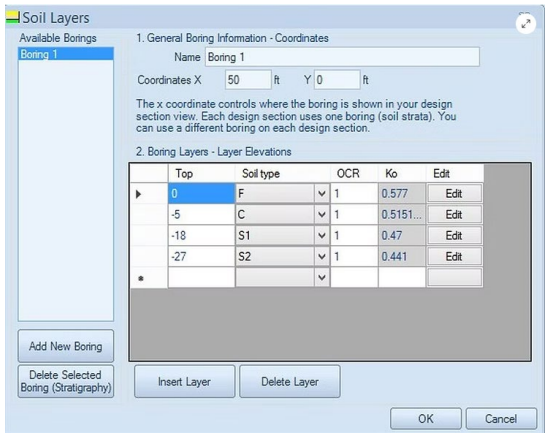
No Load Zone Depth Below Groundline:

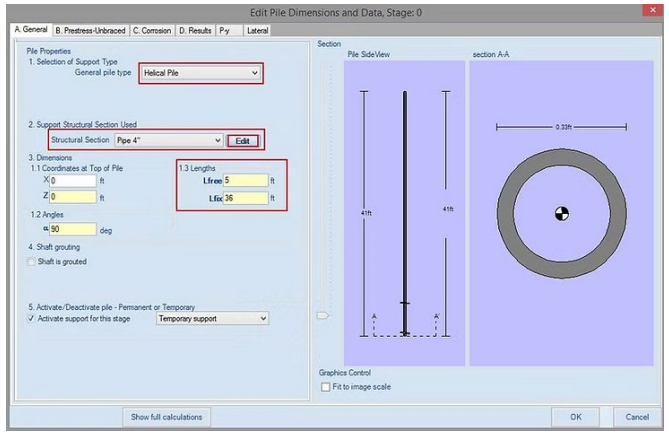
Soil Profile

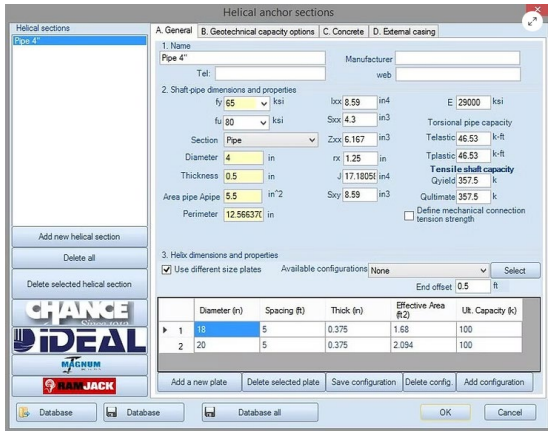
Depth (ft)	Soil Type	N	N_{60}	Cohesion (psf)	Angle of Internal Friction (degrees)	In Situ Unit Weight (pcf)	Clay Bearing Capacity Factor (N_c)	Sand Bearing Capacity Factor (N_q)	Bond Value (psf)
0	Clay	7	7	875	0	94	9	0	
5	Clay	9	9	1125	0	98	9	0	
10	Clay	7	7	875	0	94	9	0	
15	Clay	8	8	1000	0	96	9	0	
20	Clay	19	19	2375	0	118	9	0	

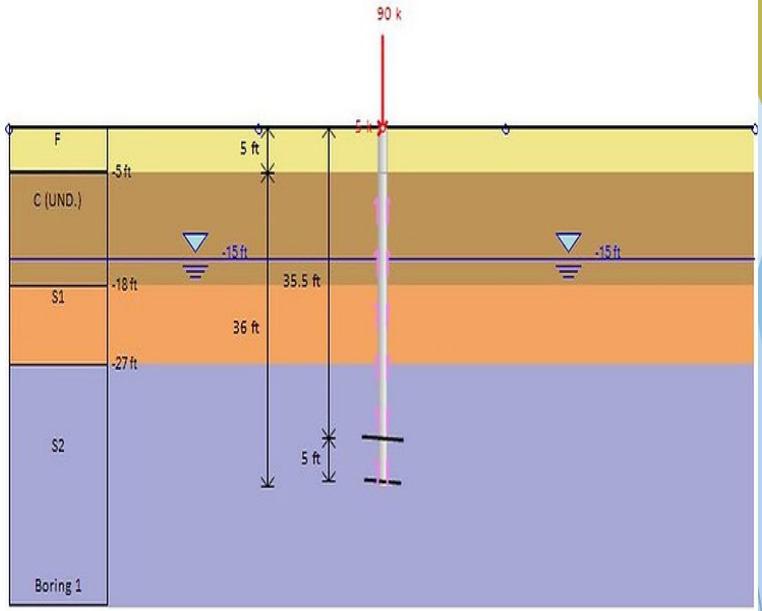
HELIXPILE


Soil Types
Soil Types
1. Name and Basic Soil Type
Soil Name F
Description Miscellaneous fill
Color
2. Soil Type - Behaviour
Sand
Silt
Rock
Clay
IGM (intermediate geo mat.)
Gravel
Show test data (SPT, CPT, Etc)
3. Default drained-undrained behavior for clays (See Theory Manual)
Undrained
Drained
A. General
B. Blasto-plastic
Lateral
E. Adv.
4. Unit Weights - Density
5. Strength Parameters and Poisson Ratio
Drained strength properties
degrees
6. Ultimate bond (grouted piles when bond option is selected)
Test Data
SPT Estimator
Notes
Other
Nspt
Elasticity modulus
Important Note:
The ultimate skin friction can be used to calculate the geotechnical capacity of tiebacks.
To do this, you have to switch on the Use Soil Bond Strengths Options for the tiebacks. Otherwise, the program will either average the vertical and horizontal confining stresses or use the bond stress as defined in the Geotech tab from the tieback section option.
Add New Soil
Copy Soil
Delete Selected Soil
Delete all soils
Paste Soil
5. At-rest coefficients
KoNC 0.577
nOCR 0.5
Ko = KoNC * (OCR)^nOCR
Rock joints are open filled with gouge
OK
Cancel

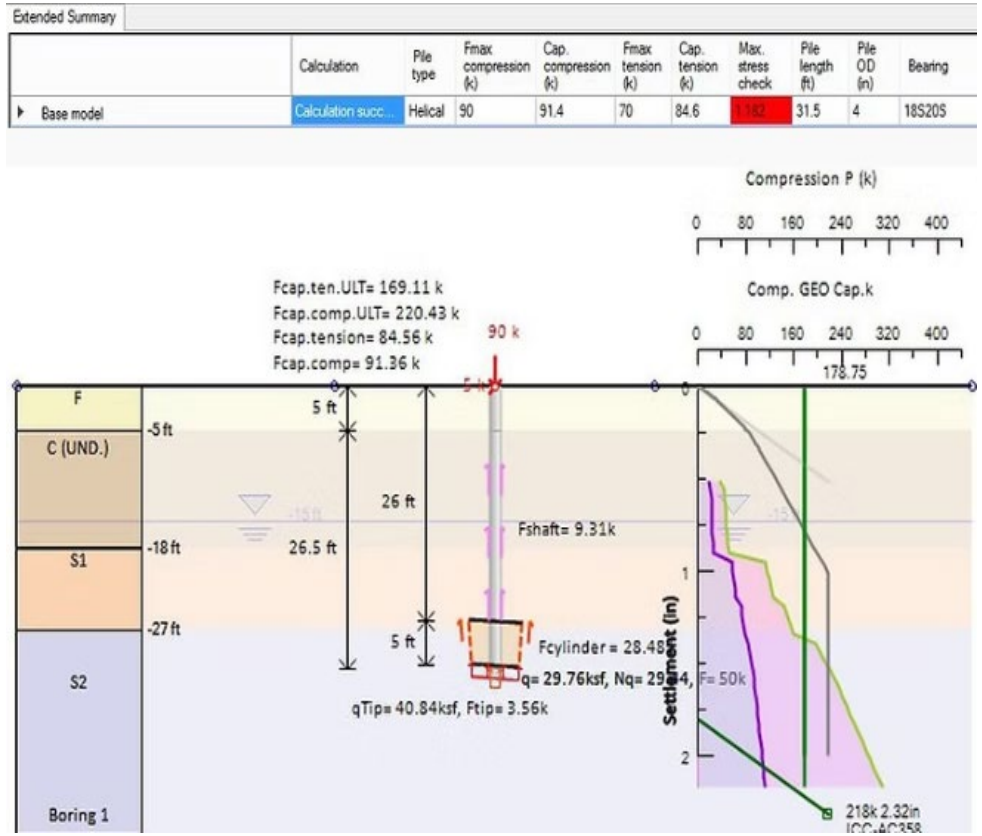

Soil Layers
Available Borings
Boring 1
1. General Boring Information - Coordinates
Name Boring 1
Coordinates X 50 ft Y 0 ft
The x coordinate controls where the boring is shown in your design section view. Each design section uses one boring (soil strata). You can use a different boring on each design section.
2. Boring Layers - Layer Elevations
Top
Soil type
OCR
Ko
Edit
0 F 1 0.577 Edit
-5 C 1 0.5151... Edit
-18 S1 1 0.47 Edit
-27 S2 1 0.441 Edit
Add New Boring
Delete Selected Boring (Stratigraphy)
Insert Layer
Delete Layer
OK
Cancel


Edit Pile Dimensions and Data, Stage: 0
A. General
B. Prestress-Unbraced
C. Corrosion
D. Results
P-y
Lateral
Pile Properties
1. Selection of Support Type
General pile type
Helical Pile
2. Support Structural Section Used
Structural Section
Pipe 4"
Edit
3. Dimensions
1.1 Coordinates at Top of Pile
1.3 Lengths
1.2 Angles
4. Shaft grouting
Shaft is grouted
5. Activate/Deactivate pile - Permanent or Temporary
Activate support for this stage
Temporary support
Section
Pile SideView
section A-A
Graphics Control
Fit to image scale
Show full calculations
OK
Cancel


Helical anchor sections
Helical sections
Pipe 4"
A. General
B. Geotechnical capacity options
C. Concrete
D. External casing
1. Name
Pipe 4"
Manufacturer
2. Shaft-pipe dimensions and properties
Section Pipe
Diameter 4 in
Thickness 0.5 in
Area pipe Apipe 5.5 in^2
Perimeter 12.56637(in
Torsional pipe capacity
Tensile shaft capacity
Define mechanical connection tension strength
3. Helix dimensions and properties
Use different size plates
Available configurations None
Select
End offset 0.5 ft
Diameter (in)
Spacing (ft)
Thick (in)
Effective Area (ft2)
Ult. Capacity (k)
1 18 5 0.375 1.68 100
2 20 5 0.375 2.094 100
Add new helical section
Delete all
Delete selected helical section
Add a new plate
Delete selected plate
Save configuration
Delete config.
Add configuration
Database
Database all
OK
Cancel


90 k
5 k
F
5 ft
-5 ft
C (UND.)
-15 ft
-15 ft
-18 ft
S1
35.5 ft
36 ft
-27 ft
S2
5 ft
Boring 1

HELIXPILE REPORT



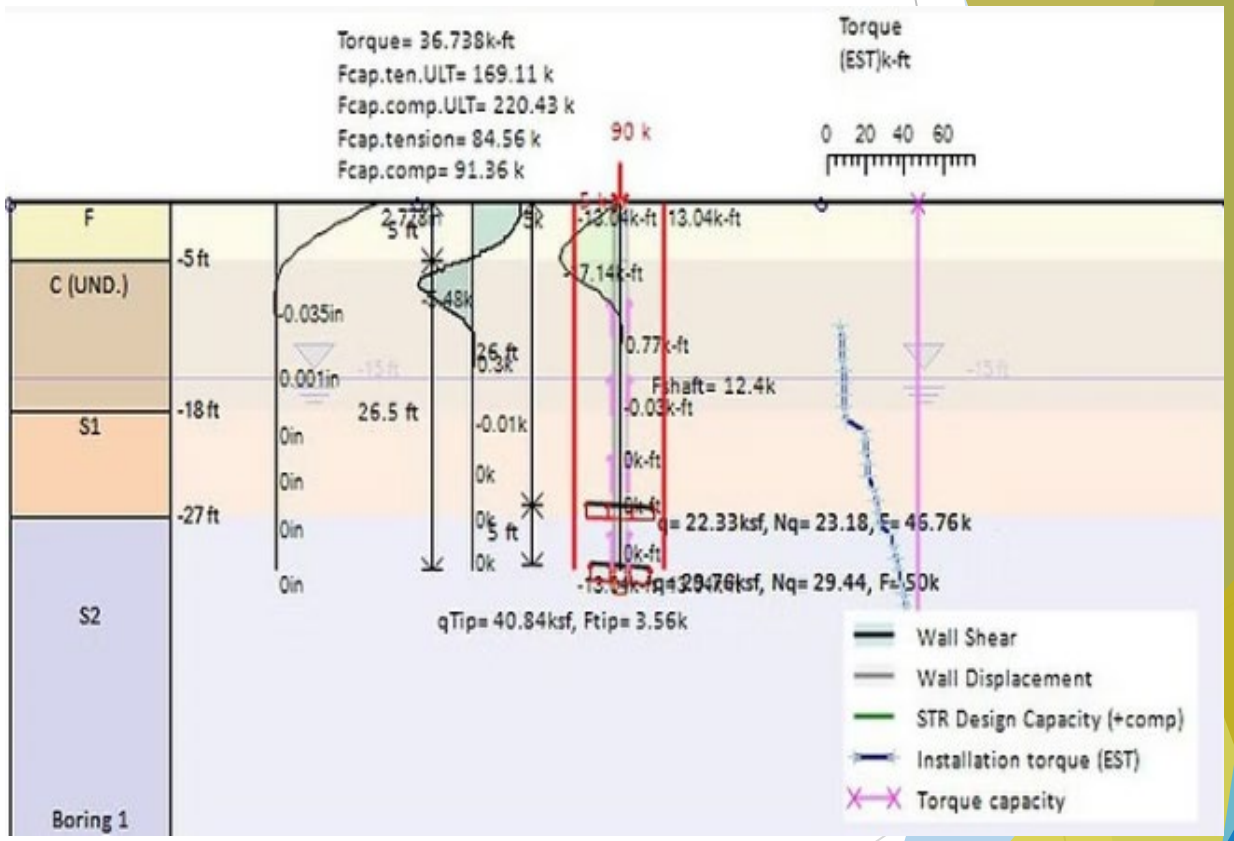


Table 1. Stiff Clay, Soft Clay, Medium Sand Input Parameters

Soil Classification	SPT N_{60} (bpf)	Shaft Diameter (in)	Wall Thickness (in)	Helix Configuration	Pile Length (ft)	F.S.
Soft Clay	3					
Stiff Clay	12	3.50	0.30	8-10-12	15	2
Medium Sand	15					

Table 2. Software Outputs for Soft Clay, Stiff Clay, and Medium Sand

Soil Classification	Magnum® Geo-App®		HeliCAP®		HelixPile	
	Torque (ft-lbs)	Capacity (kips)	Torque (ft-lbs)	Capacity (kips)	Torque (ft-lbs)	Capacity (kips)
Soft Clay	574	4	791	5.5	660	4.6
Stiff Clay	3,450	24	3,163	22	2,200	15.6
Medium Sand	4,700	32	4,662	35.4	10,460	36.6

Table 3. Gradually Increasing Clay Soil Inputs

Depth	Soil Classification	N_{60} (bpf)
0	Clay	9
5	Clay	13
10	Clay	15
15	Clay	17
20	Clay	19
25	Clay	21
30	Clay	23
35	Clay	25
40	Clay	27
45	Clay	35

Table 4. Layered Soil Profile Inputs

Depth	Soil Classification	N_{60} (bpf)
0	Clay	12
5	Clay	30
10	Clay	20
15	Sand	12
20	Sand	8
25	Sand	4
30	Sand	12
35	Clay	15
40	Clay	35
45	Clay	35

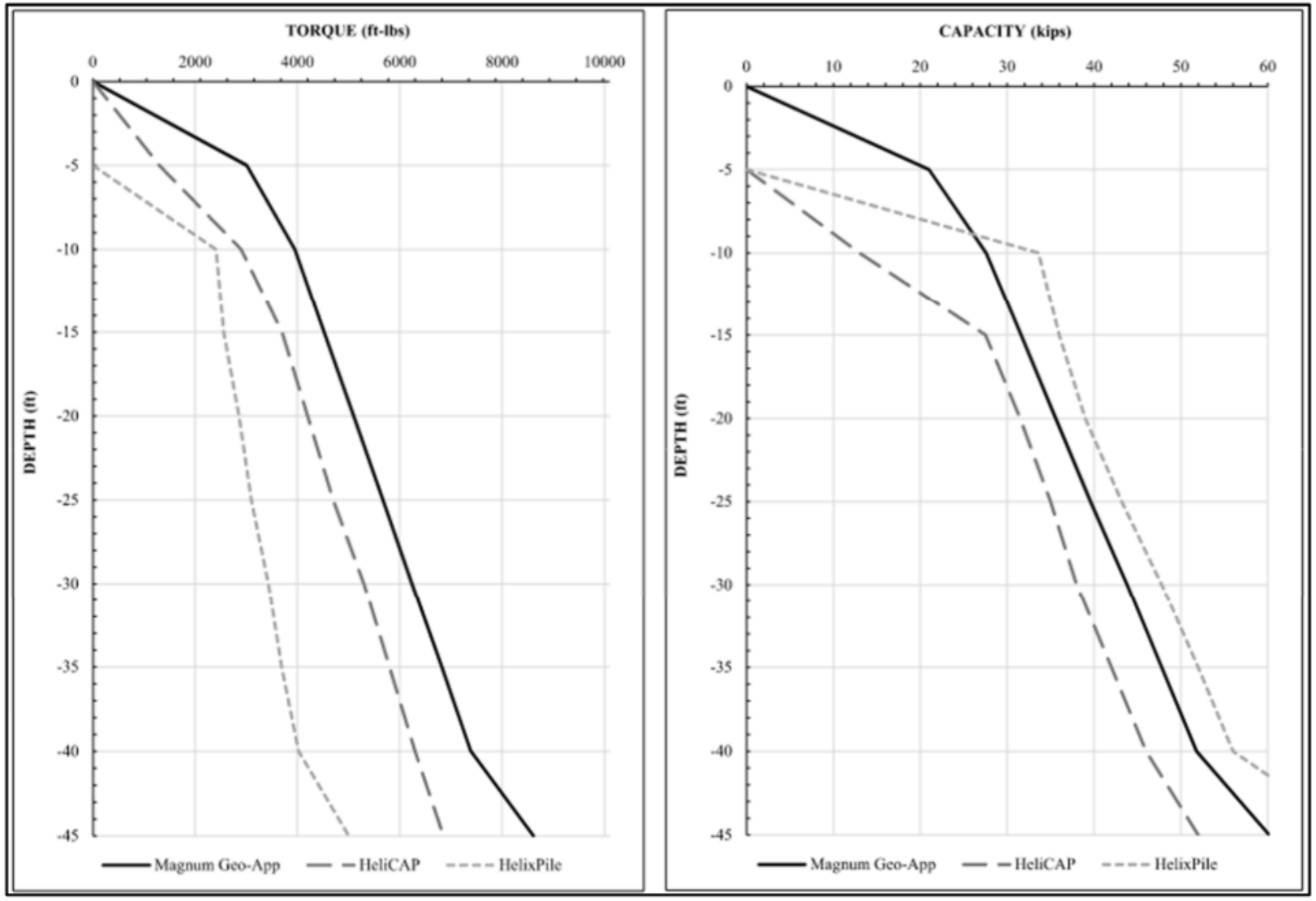


Fig. 1. Torque and capacity readings for a soil profile with gradually increasing stiffness.

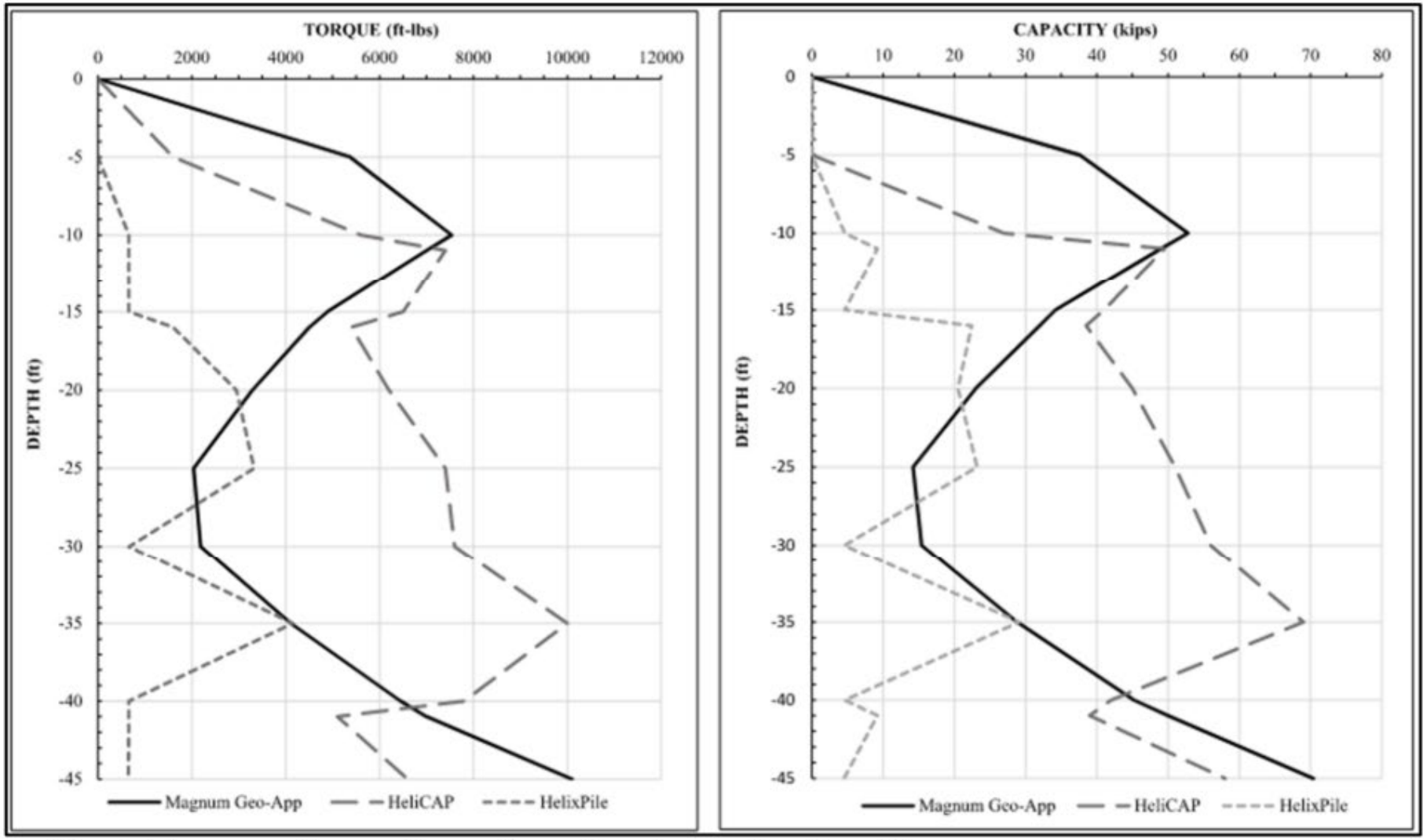
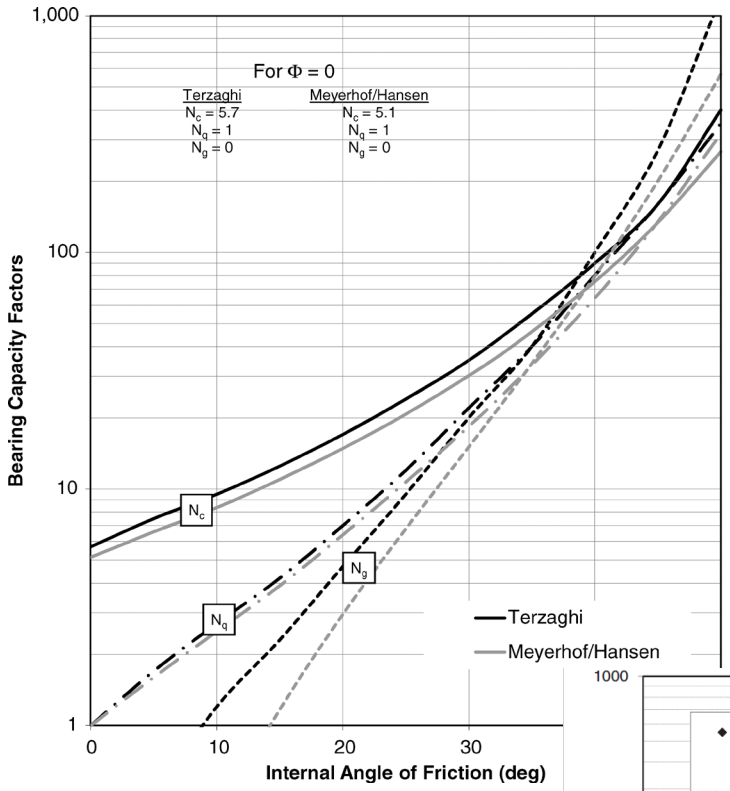
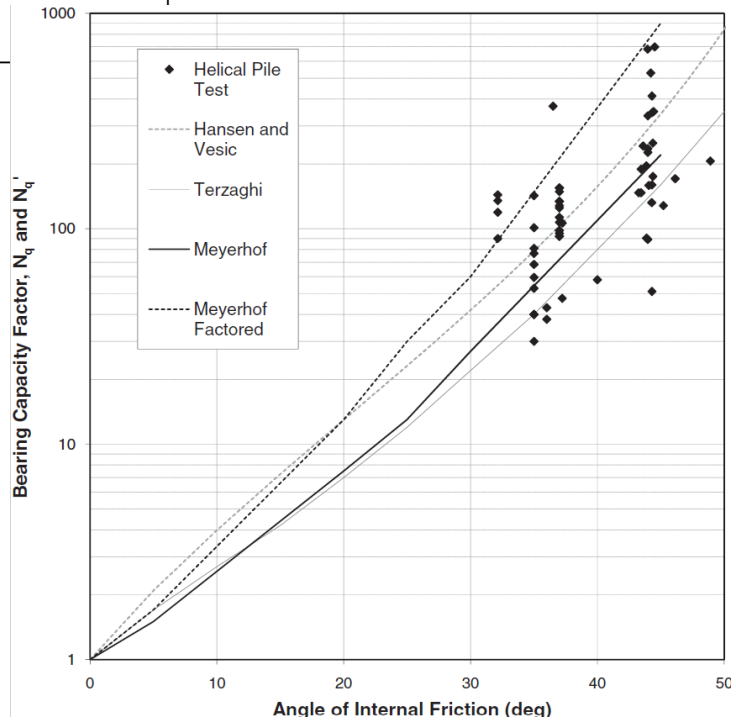


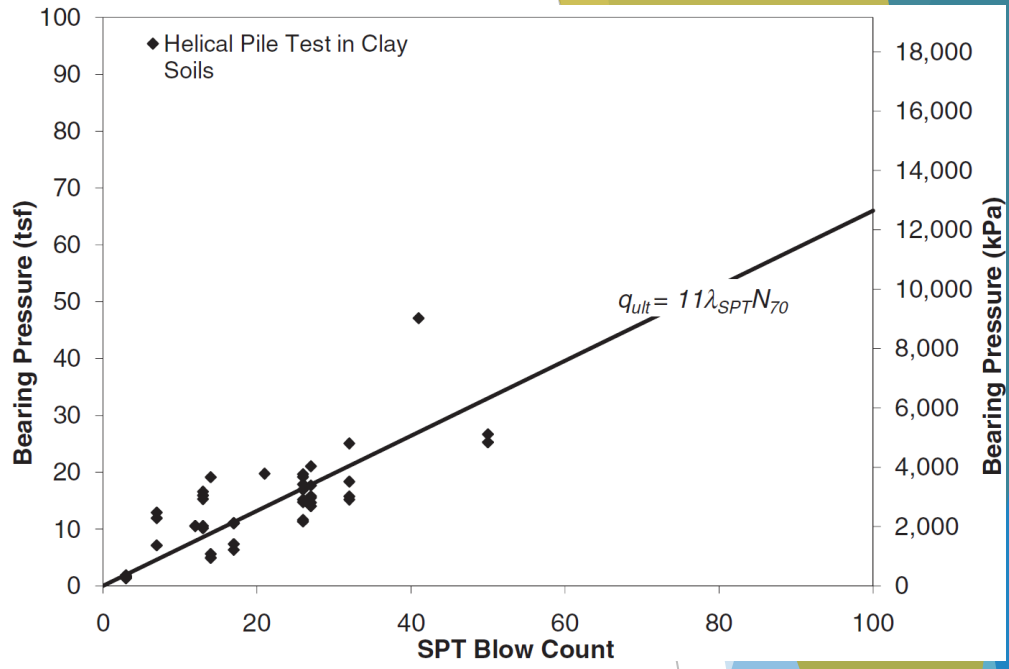
Fig. 2. Torque and capacity readings for a layered soil profile.

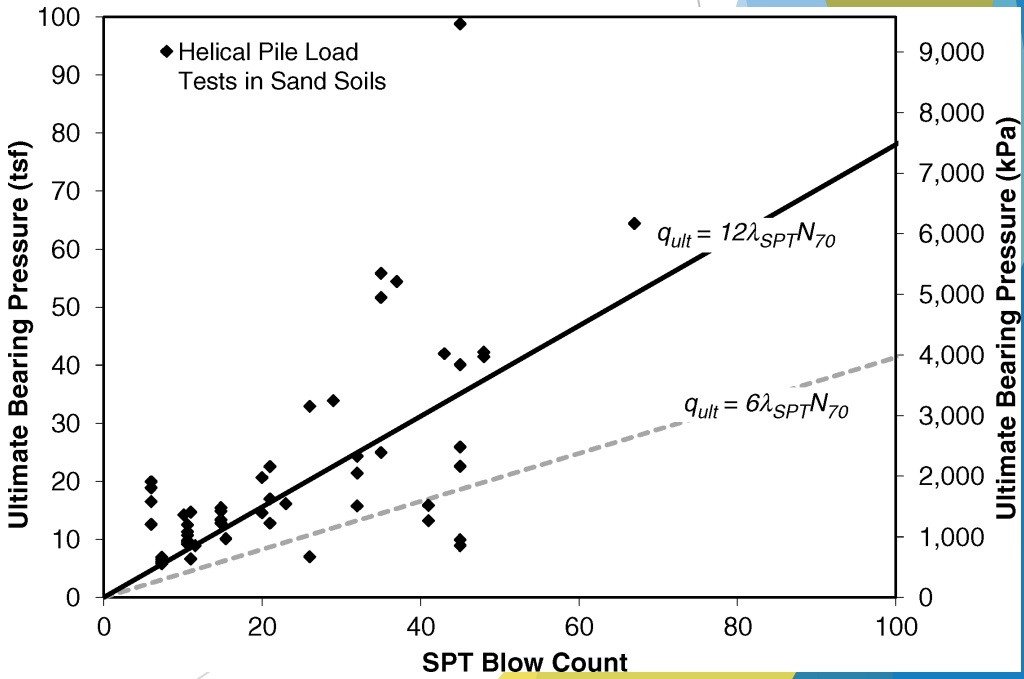
What Causes Different Capacity Results?

- Variations in Subsurface Conditions
- Limits in Geotechnical Sampling Data
- Errors in SPT and Other Tests
- Correlations between Field Test and Shear Strength
- Human Error
- Differences in Sizing Theory
- Differences in Software Assumptions
- Groundwater Table Fluctuations
- Changes in Moisture Content
- Changes in Time


For Φ = 0
Terzaghi
N_c = 5.7
N_q = 1
N_g = 0
Meyerhof/Hansen
N_c = 5.1
N_q = 1
N_g = 0
Bearing Capacity Factors
Internal Angle of Friction (deg)
N_c
N_g
N_q
Terzaghi
Meyerhof/Hansen


Helical Pile Test
Hansen and Vesic
Terzaghi
Meyerhof
Meyerhof Factored
Bearing Capacity Factor, N_q and N_q'
Angle of Internal Friction (deg)


Helical Pile Test in Clay Soils
$q_{ult} = 11\lambda_{SPT}N_{70}$
Bearing Pressure (tsf)
Bearing Pressure (kPa)
SPT Blow Count


Helical Pile Load Tests in Sand Soils
$q_{ult} = 12\lambda_{SPT}N_{70}$
$q_{ult} = 6\lambda_{SPT}N_{70}$
Ultimate Bearing Pressure (tsf)
Ultimate Bearing Pressure (kPa)
SPT Blow Count

Monitoring sheet piling assets with the SmartSheetPile system

DFI50
Nashville, Tennessee

October 22, 2025

Pierre Werner
Business Development Manager North America

ArcelorMittal Sheet Piling – Luxembourg
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Based at JD Fields & Company, Houston TX



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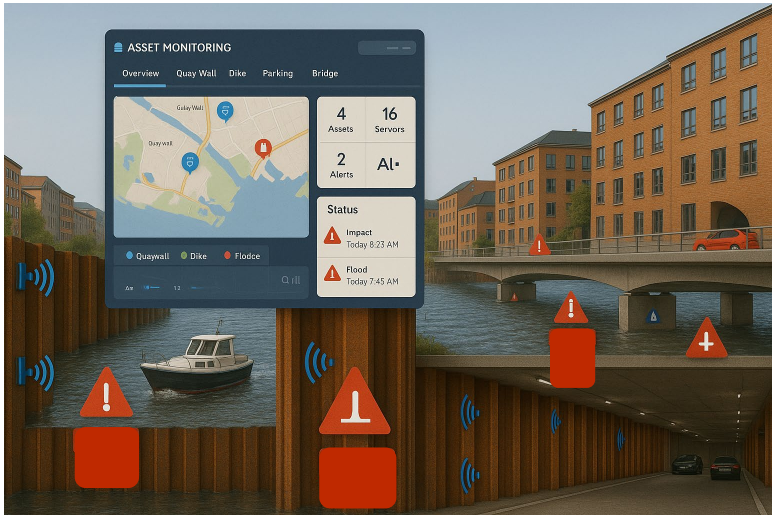
SmartSheetPile – Executive Summary

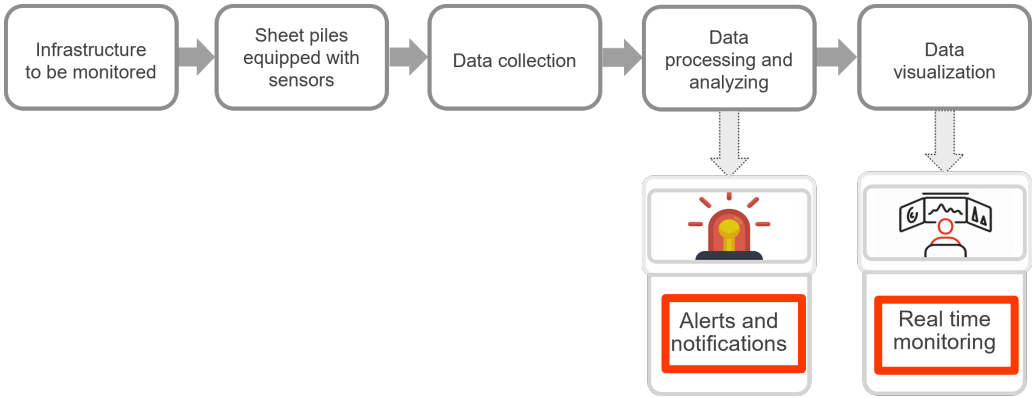
WHAT ?

- Structural Health Monitoring system : monitor, detect and forecast / predict / prevent damage based on current and future conditions.



WHY ?

- Effective asset management
- Uncover hidden quay wall capacities
- Monitor corrosion in real time
- Know when and where maintenance is needed
- Lifetime extension



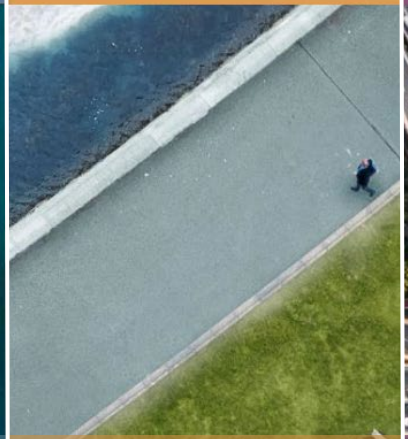
Applications for sheet piling

Water Transport Solutions



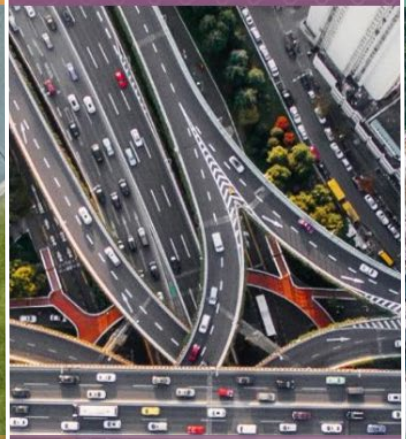
Breakwaters, quay walls, jetties, dolphins, locks, canals and land reclamation …

Hazard Protection Solutions



Dykes, dams, river embankments, flood protection barriers, tidal defences and reservoirs …

Mobility Infrastructure Solutions



Bridge abutments, tunnels, underpasses, noise barriers, foundations, basements and underground car parks …

Environmental Protection Solutions



Containment structures, water management, polluted soil remediation, landfill conversion and leachate cut-off …

Key benefits in the different applications



Water Transport

- Effective asset management
- Uncover hidden quay wall capacities
- Monitor corrosion in real time
- Know when and where maintenance is needed
- Lifetime extension



Hazard Protection

- Raise the alarm on increasing water levels
- Detect accidental and weather-induced damage
- Detect early signs of structural weaknesses
- Detect early signs of potential catastrophic collapse



Mobility infrastructure

- Monitor the structural health of bridge abutments
- Reduce traffic disruption and avoid unexpected closures with more effective preventive maintenance
- Save costs and limit inconvenience to users



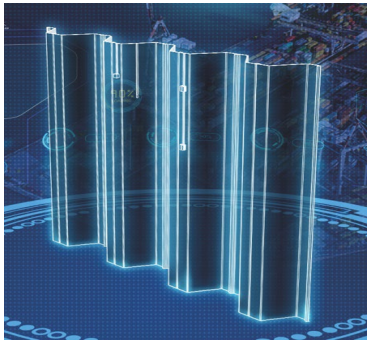
Environmental protection

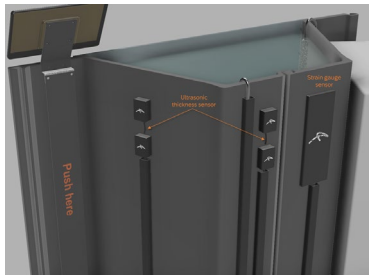
- Guarantee the integrity against declutching
- Ensure the imperviousness of the sheet pile wall
- Remove the need for costly inspections and unscheduled repair works

SmartSheetPile – System architecture

Hardware

Sheet pile with embedded / associated sensors

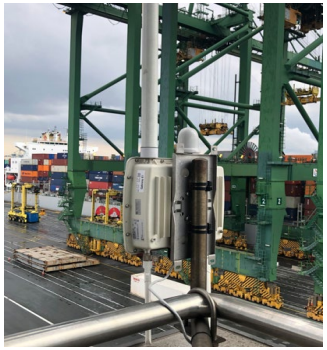




Wired

Acquisition unit





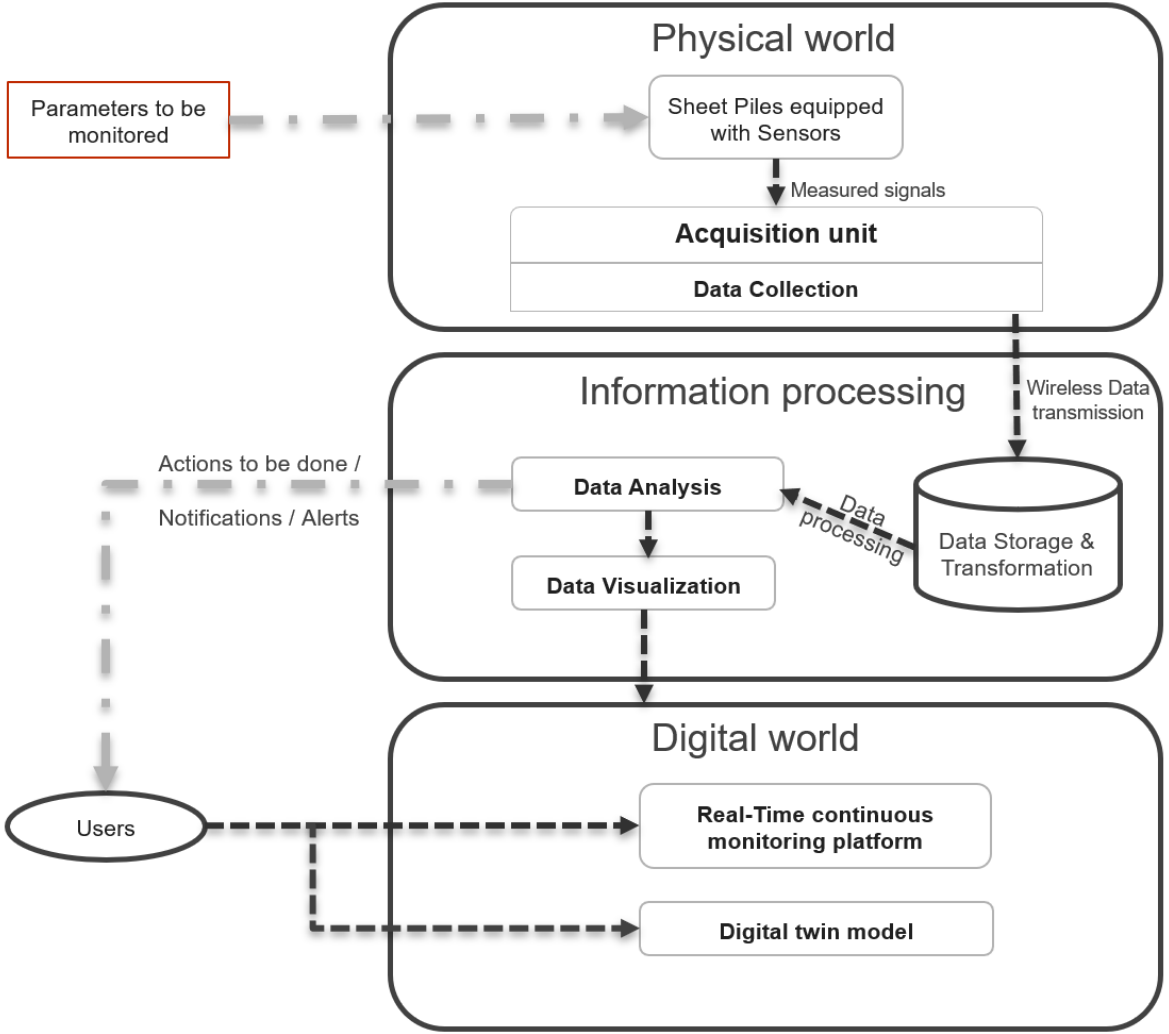
Wireless

Software

Cloud platform / Digital twin model / Monitoring system



Data flow


Physical world
Parameters to be monitored
Sheet Piles equipped with Sensors
Measured signals
Acquisition unit
Data Collection
Information processing
Wireless Data transmission
Actions to be done /
Notifications / Alerts
Data Analysis
Data processing
Data Storage & Transformation
Data Visualization
Digital world
Users
Real-Time continuous monitoring platform
Digital twin model

SmartSheetPile – the menu (non-exhaustive)

The SmartSheetPile is a modular product, so the end user can select from a menu list of parameters to track :

“Core” sheet pile behavior

- Corrosion / material loss
- Inclination / tilt
- Position / displacement
- Structural deformation (strain)
- Anchor tension
- Soil pressure
- Shock / vibrations / acceleration
- Force / load applied

Surrounding / environmental information:

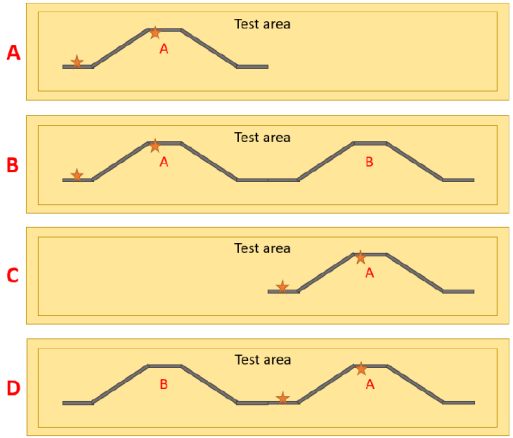
- Tidal levels
- Soil inclination
- Settlement
- Ground water levels
- River water level
- Temperature

Dashboard – Online monitoring (concept demonstrator)



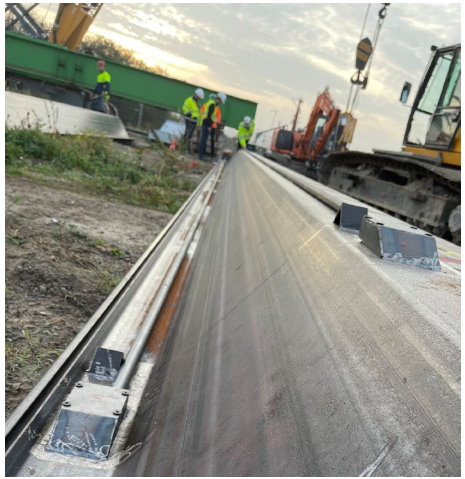
Driving tests

- Reliability of sensors/cables after sheet pile installation
- Simulate the most extreme conditions
  - Vibro hammer / Impact hammer



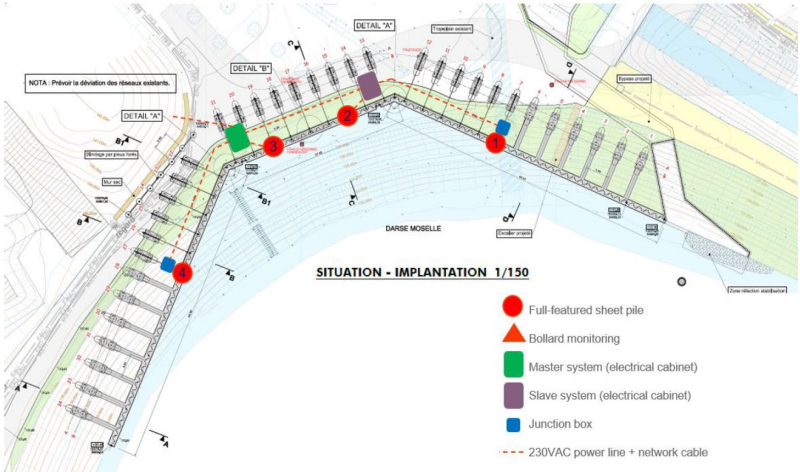


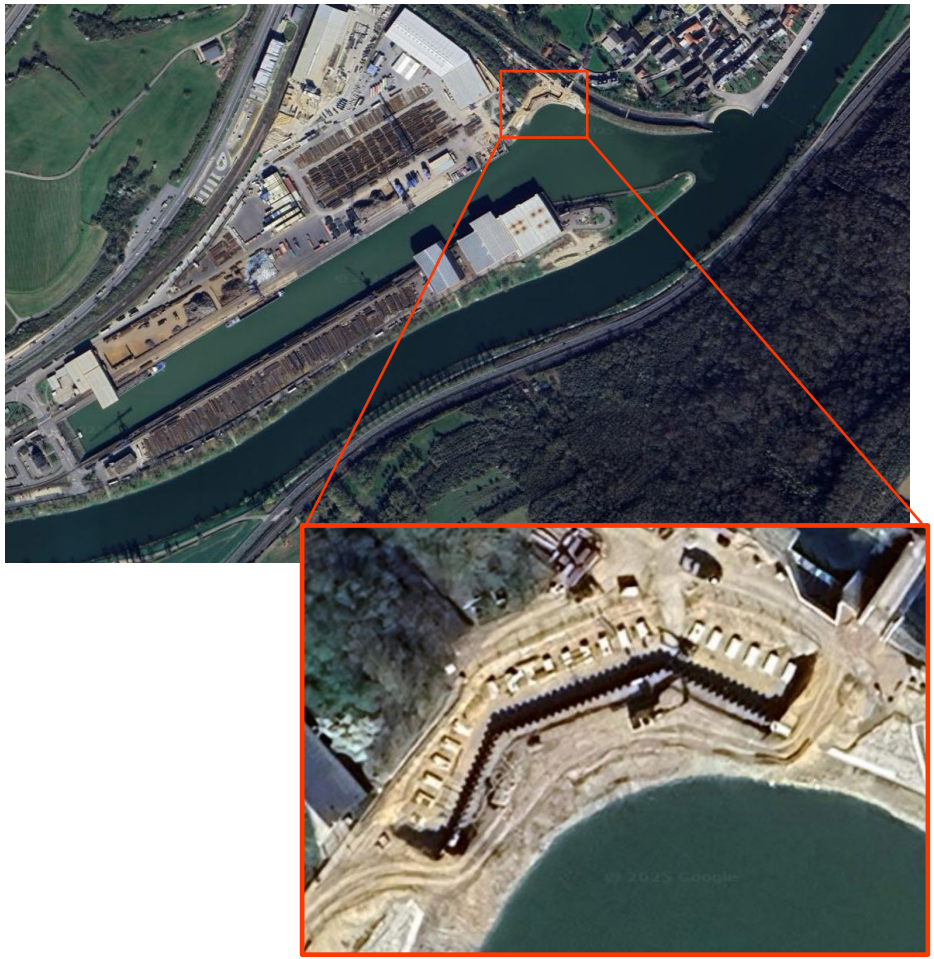




Pilot / breakthrough project : Port of Mertert – end user P&Ch (DOT)

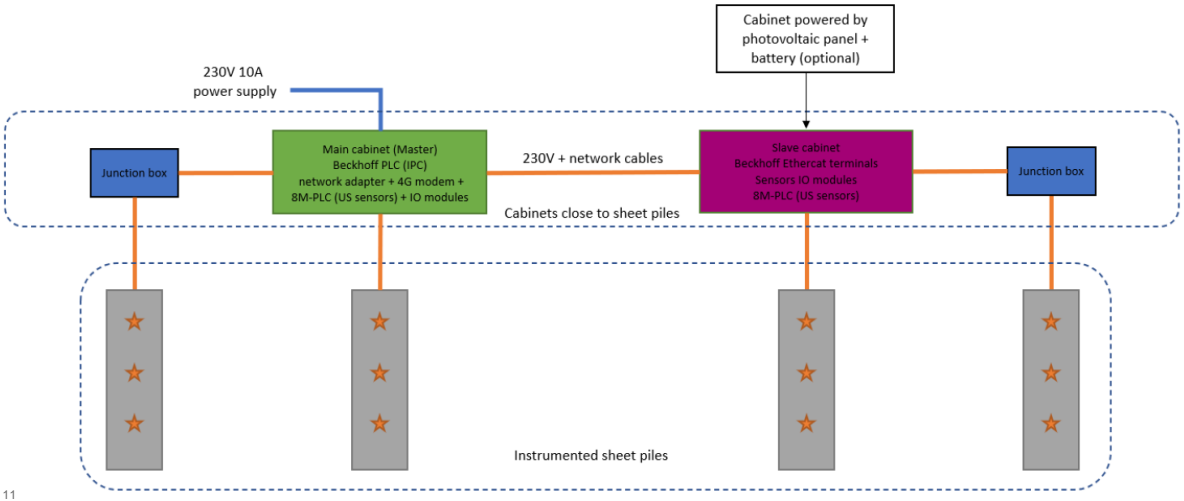
- River Port of Mertert (Luxembourg)
- New turning point for 135m / 450ft long vessels





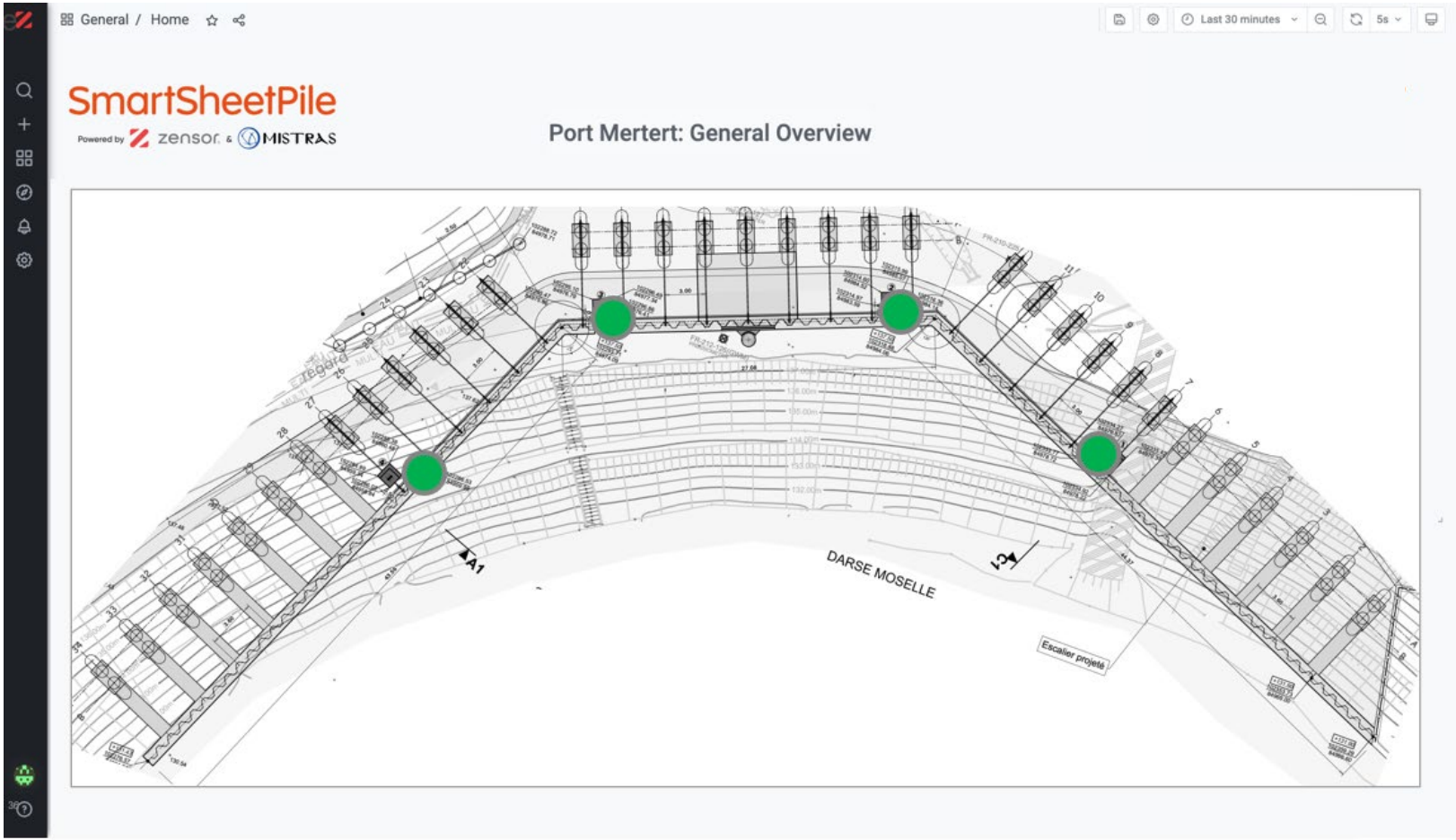
Pilot / breakthrough project:
Port of Mertert – end user P&Ch (DOT)

- Monitoring : corrosion, river water level, anchor tension, inclination, strain, bollard mounting, shock, temperature
- > 125 sensors covering some 10 different data types on 4 double piles
- 24/7 monitoring dashboard

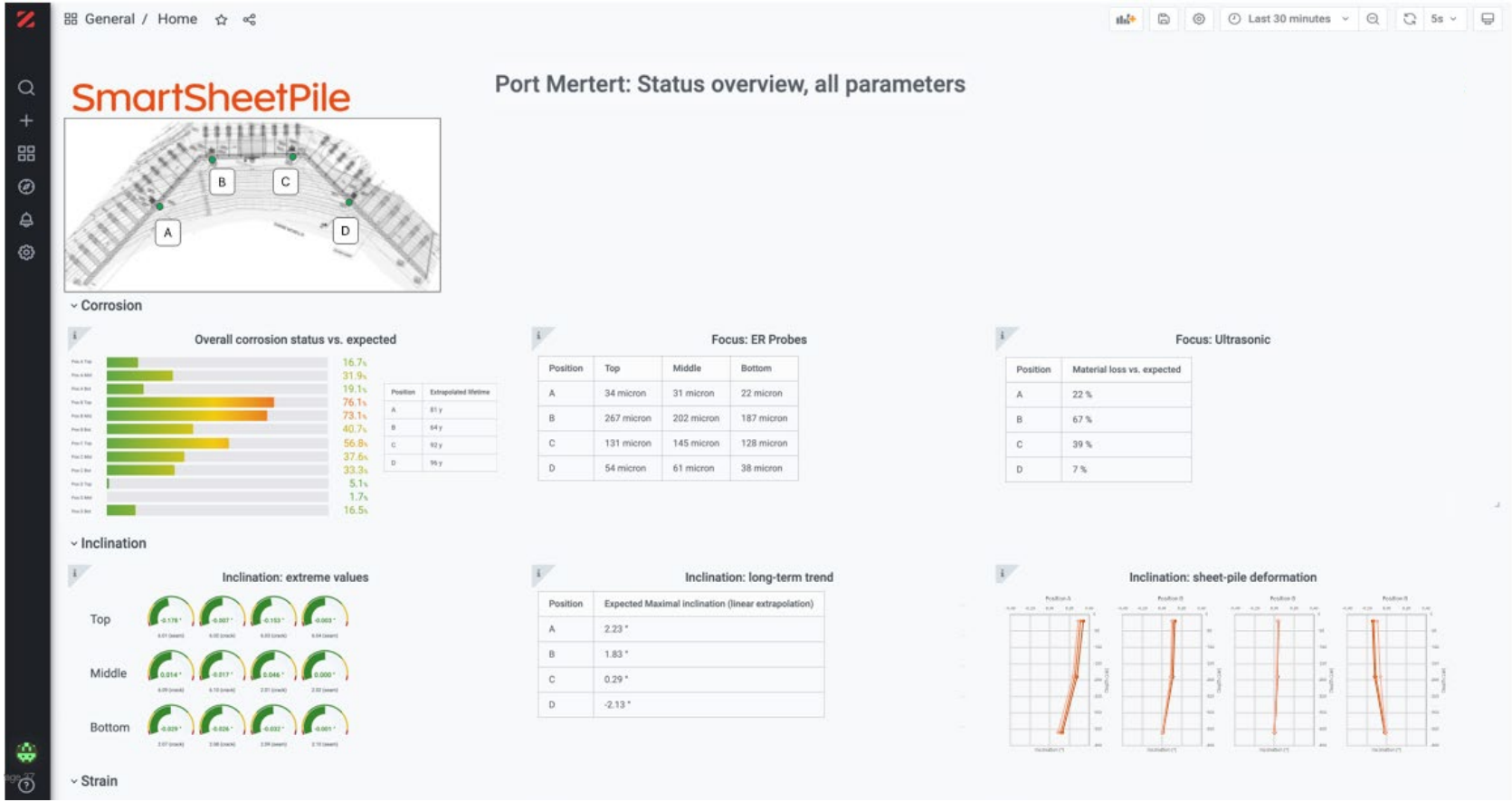




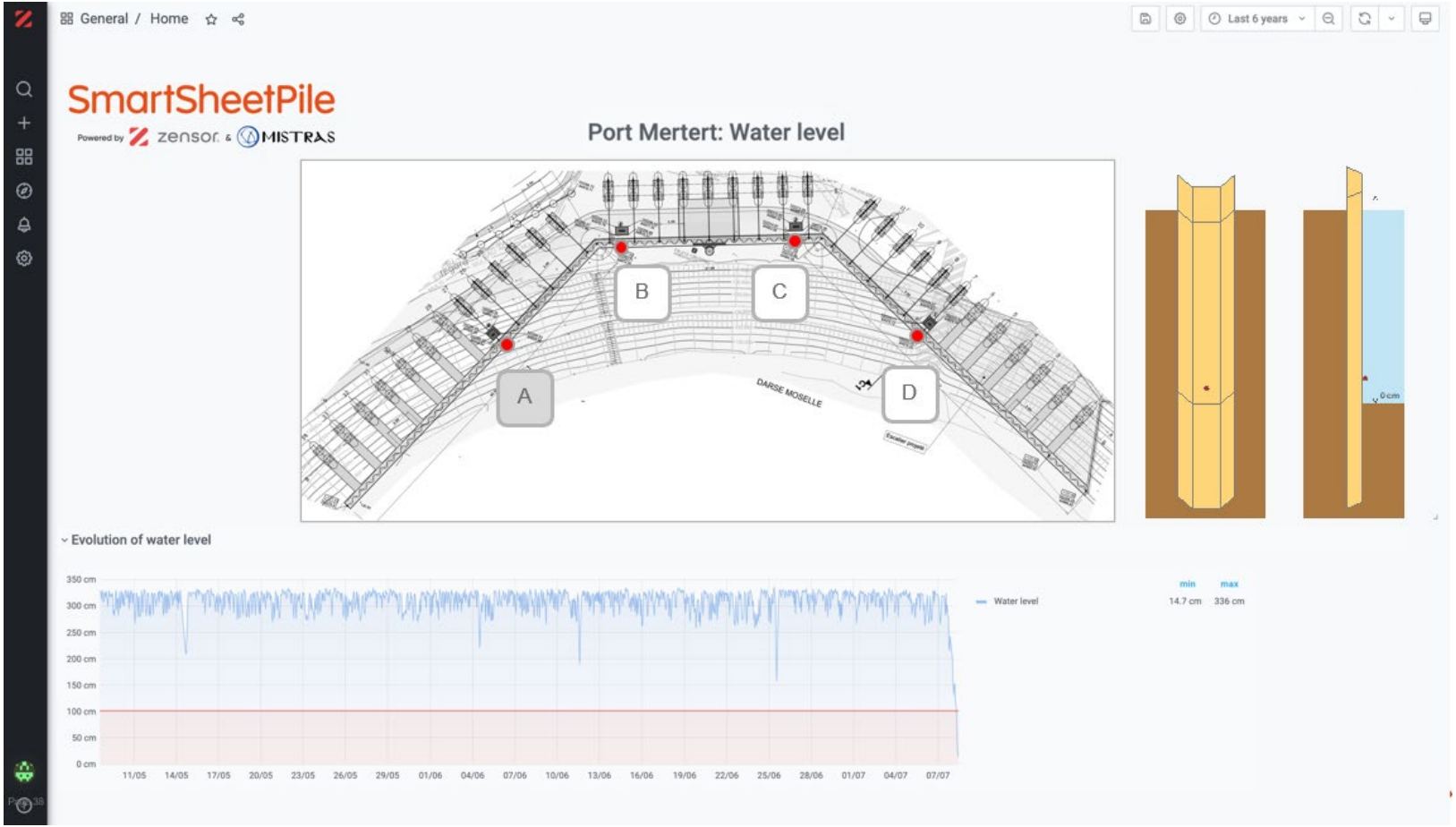
Dashboard examples



Dashboard examples



Dashboard examples



Thank you for your attention !



Practical Applications of the Finite Element Method to Deep Foundation Design

PRESENTATION TOPICS

- Overview and Separation of Design Responsibilities for Buildings
- Example: Pile Spacing for Lateral Loads
- Example: Pile Caps
- Example: Conflicting Code Provisions
- Example: Deep Foundation Systems
- Example: Rigid Inclusion Applications
- Example: Research and Development

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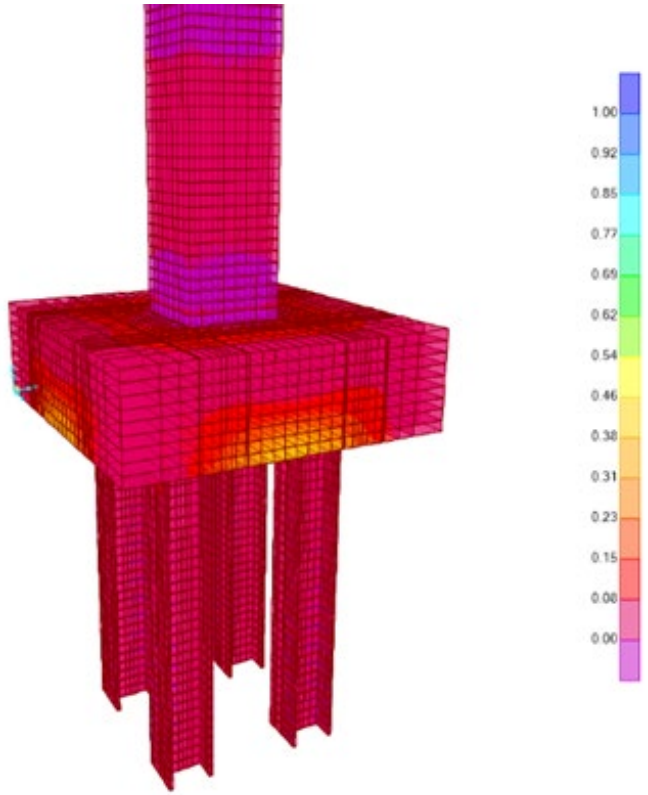
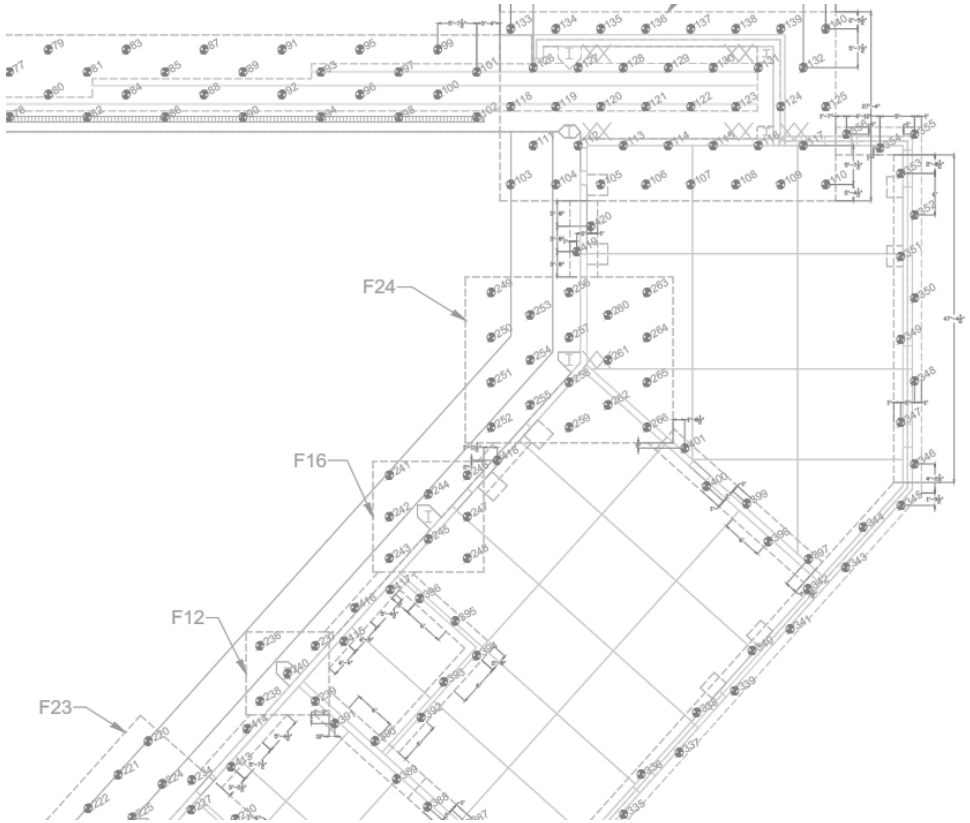


DFI 50th Annual Conference on Deep Foundations

Practical Applications of the Finite Element Method to Deep Foundation Design

Overview and Separation of Design Responsibilities for Buildings

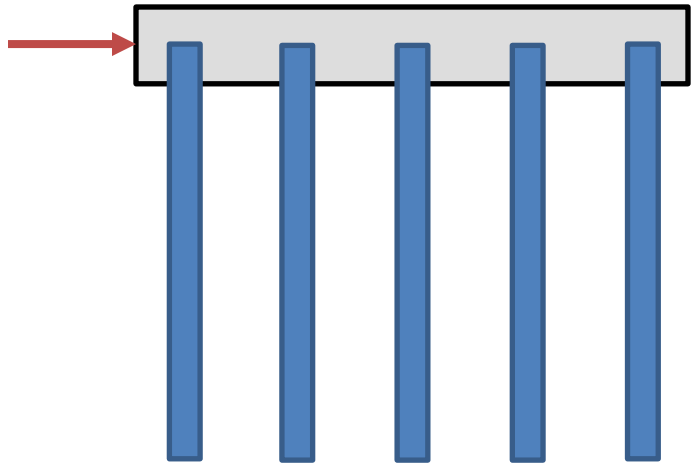
- ASCE 7 “fixed base” foundation allowance
- Delegated designer vs. EOR
- Elastic foundation response assumed even for seismic (R value via supported structure)
- Prescriptive code with conservative assumptions (head fixity, splice demands, lateral demands away from pile head)



Pile Spacing for Lateral Loads

How do we handle group effects for lateral loads?

- Required to be considered where center-to-center spacing of piles is less than 8 times the pile dimension (IBC 1810.2.5)
- Advanced analysis (group and FE modeling) with varying group factors recommended by the geotechnical engineer
- Average group factors for each group
- Conservative average group factor for the structure
- Something else? Generally accepted method required.

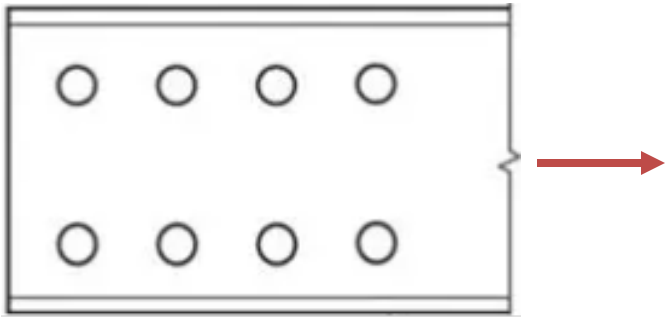


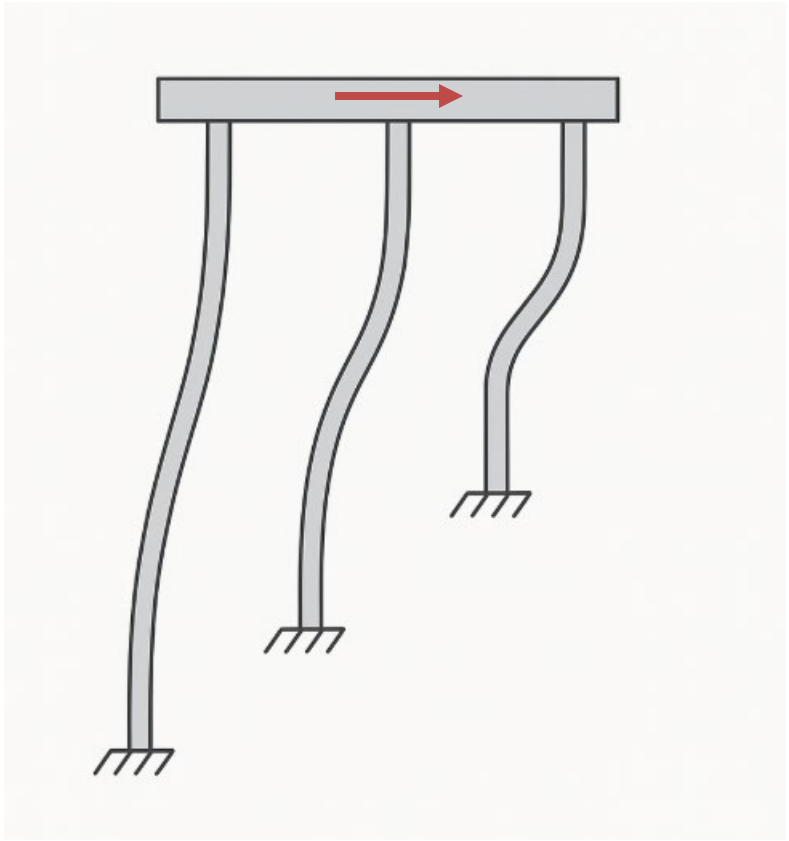


Pile Spacing for Lateral Loads (continued)

How does this model affect performance?

- Leading piles act as shorter piles
- Larger demands in leading piles
- Redistribution calculated using nonlinear springs
- Significant redistribution prior to failure





Conflicting Code Provisions

Critical Change: ACI 318-19 one-way shear strength equation for less than minimum stirrups and use of size effect factor

- Can require up to 2x thicker footings and 2.5 times or larger piles caps
- Size effect factor for pile caps per ACI 13.4.6.8 resolved (equal to 1.0) in ACI 318-25 but pile spacing must not exceed 4 pile dimensions for pile caps and 5 pile dimensions for mat foundations
- Resolution reducing pile cap thickness required by 50% effectively reduces lateral pile capacity by 50% in many cases

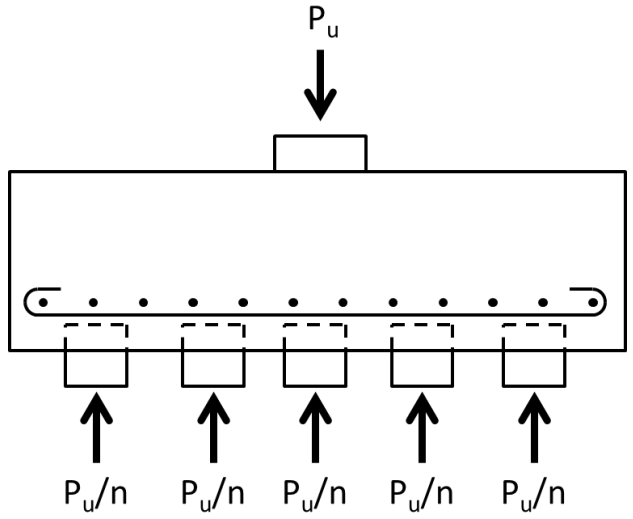
Practical Applications of the Finite Element Method to Deep Foundation Design

Pile Caps

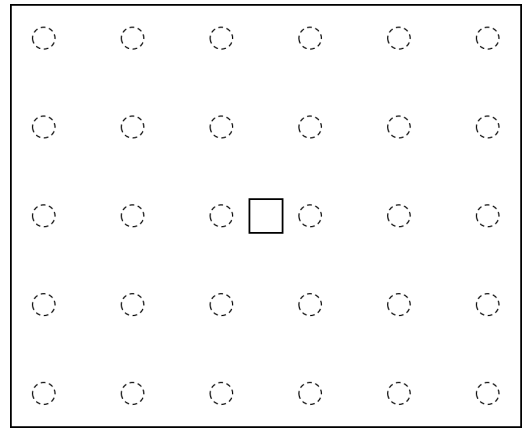
- Stiffness assumptions

Do all piles share compressive load equally?

- Piles have a stiffness that is related to (a) the soil t-z or vertical spring stiffness and (b) the axial stiffness of the pile as defined by the AE/L_{pile}, where A is the pile cross-sectional area, E is the pile modulus of elasticity, and L_{pile} is the overall pile length.



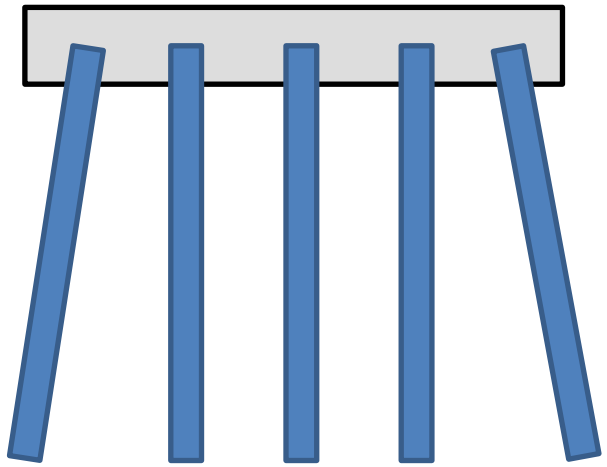
Vertical Pile Stiffness (k/in.)	P_{center} (2 piles)	P_{corner} (4 piles)	P_{other} (24 piles)
100	1/30	1/30	1/30
400	1/28	1/32	1/30.5 - 1/29.5
800	1/27	1/33	1/31 - 1/29
1,200	1/25	1/34	1/32 - 1/28
Rigid	1/7	1/82 (Tension)	1/80 - 1/10



Practical Applications of the Finite Element Method to Deep Foundation Design

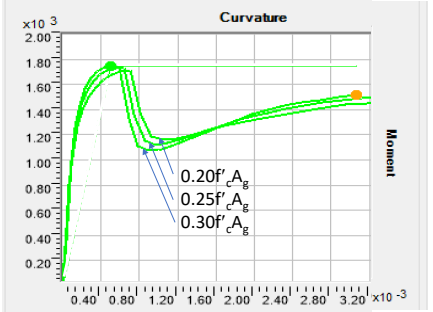
Deep Foundation Systems (continued)

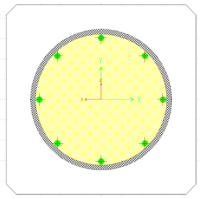
- Pile Models
- Batter Piles

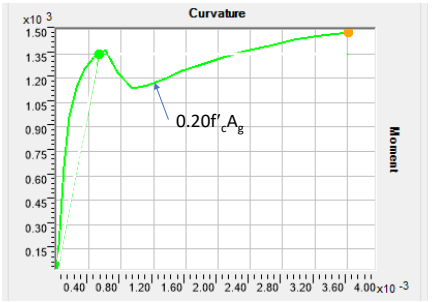


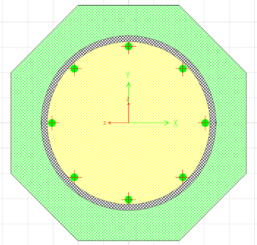
"Batter piles and their connection shall be designed to resist forces and moments that result from the application of seismic load effects including overstrength factor in accordance with Section 2.3.6 or 2.4.5 of ASCE 7"

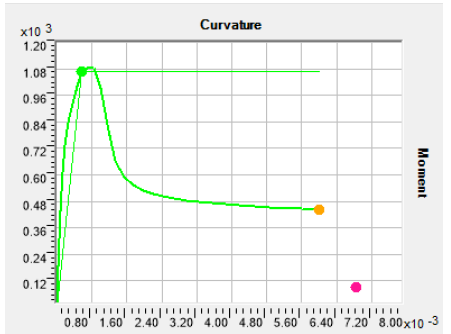
Note: Batter piles are **NOT** axial load only elements!

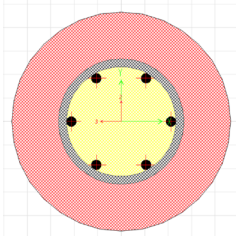












Rigid Inclusion Applications

- Engineers concerned about point loads on footings
- Structural/geotechnical engineering interaction (PLAXIS and SAP2000)
- Seismic design and detailing

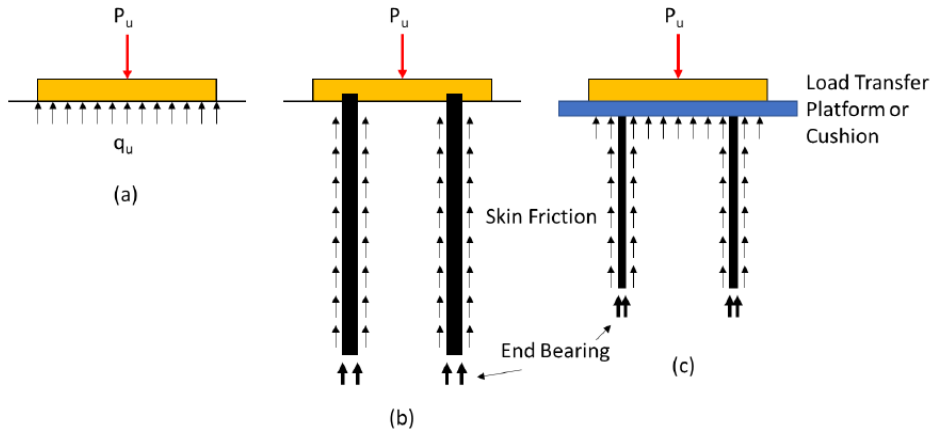


Figure 1. Typical foundation options: (a) shallow foundations (footings), (b) deep foundations (piles), (c) rigid inclusions.

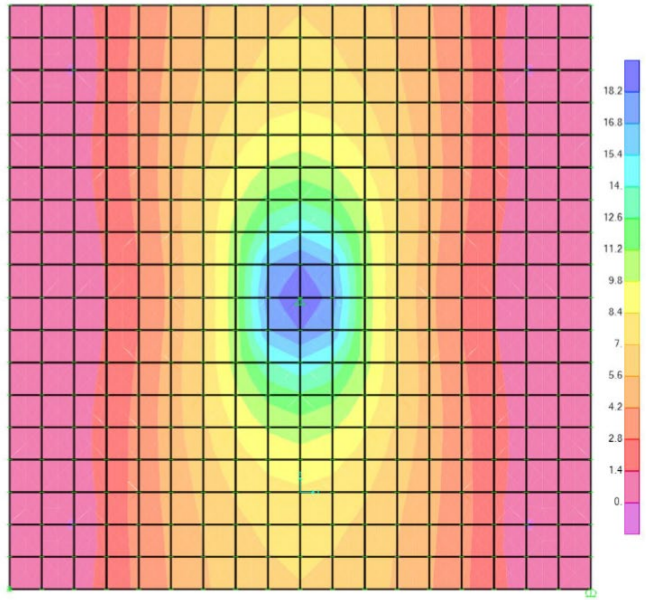
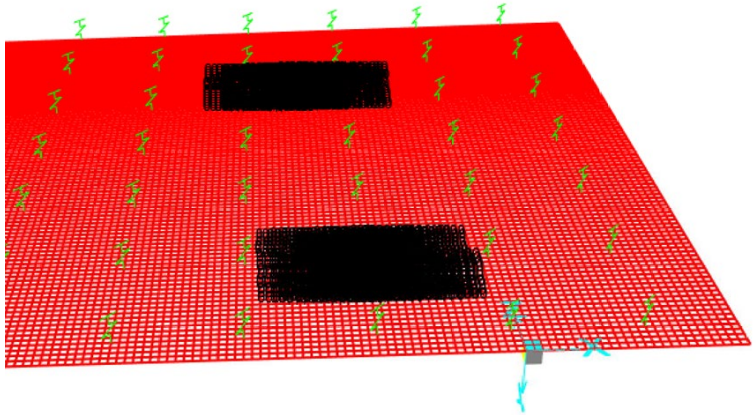
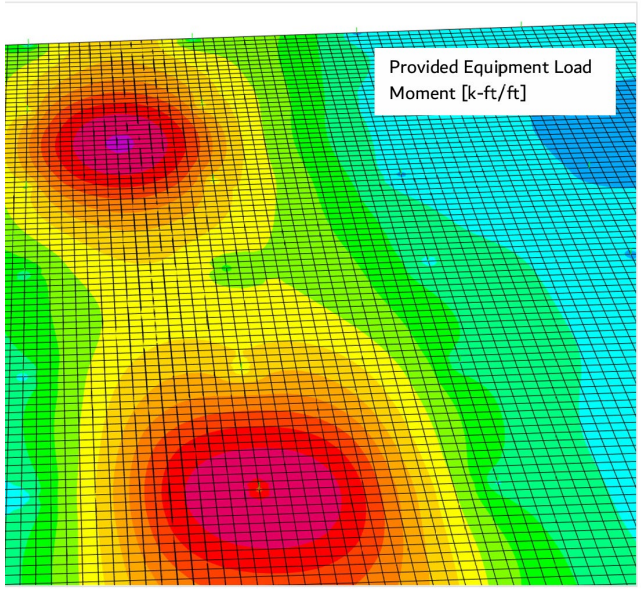


Figure 4. Finite element results for one directional bending moment (k-ft/ft) and rigid inclusions as shown in Figure 2.

Rigid Inclusion Applications (continued)

- Slab design for equipment loading and stringent serviceability criteria







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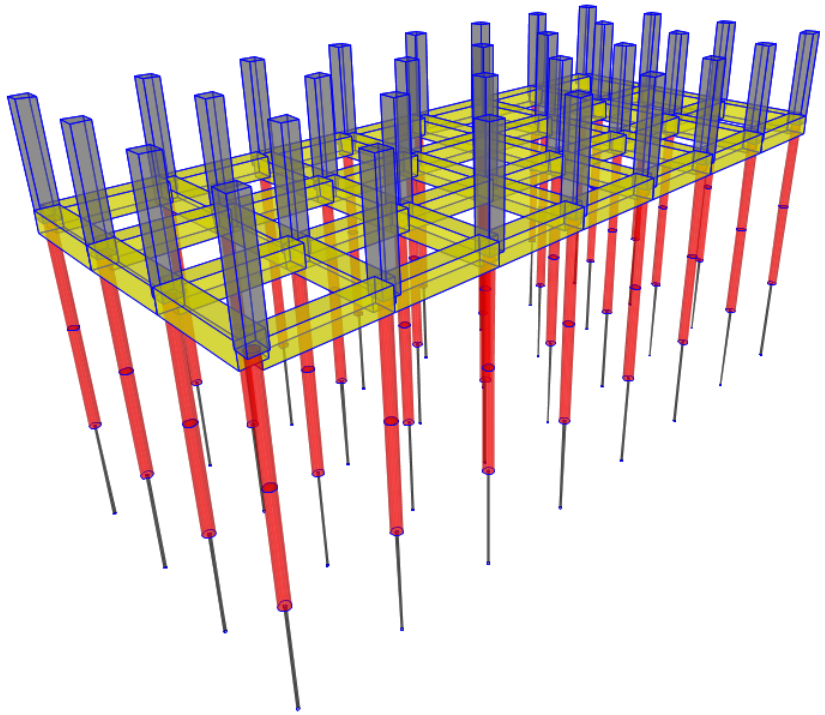
Practical Applications of the Finite Element Method to Deep Foundation Design

Research and Development

- Finite element analysis coupled with field testing (Citadel research study)
- Finite element studies to improve fabrication modifications





Speaker: Timothy W. Mays, Ph.D., P.E.

Evaluation of Post Proof Load Capacity of Piles in Coastal Plain Sands of Southern Nigeria

Adrian O Eberemu., John E. F. Usifo., John E. F. K. Usifo., and Ezekiel O. Akanbi

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Introduction

- Piled foundations are used in highway construction, buildings and other structures.
- Accurate and reliable determination of pile's capacity is an important design and construction consideration
- It is common practice at design stage to predict pile capacity by static analysis (formula) in advance of pile driving, based on the results of in-situ and laboratory tests
- On sufficiently large projects, loading tests may be performed at different stages during design or construction of the project

Introduction Contd'.

- Traditionally, the static pile load test is used to determine the load-carrying capacity of the pile-soil system, or the value of the service load to be supported by the pile
- Existing static loading test procedure could be classified into two groups.
- Procedures requiring a progressive loading without an intermediate unloading (Maintained load Test [MLT] and Constant Rate of Penetration, and the LPC [French Method]).
- Evaluation of pile bearing capacity is still a subject of many researchers.

Introduction Contd.

Several researchers Randolph et al. (1994), Awad (2002), Foray et al. (1998), Baca and Rybak (2021), Xie et al. (2024) mentioned that calculated pile bearing capacity using conventional methods often give poor agreement with load test results.

Leading to controversy about which value to adopt as a pile bearing capacity, especially from loading tests.

Background

Canadian Foundation Engineering Manual (1985) had divided the criteria for interpreting test loading results into two groups:

- (i). criteria governing the acceptance of tested piles
- (ii). Criteria defining the failure load of the tested pile.

Different failure criteria have been proposed in technical literature

- *Davisson offset Limit Load*:
- *De Beer Yield Load Approach*:
- *Brinch-Hansens's Failure Criterion*:
- *Chin-Kondner Failure Criterion*
- *Decourt* (1999) etc.

Background Contd.

- Majority of the pile load test carried out in Nigeria are to determine the load-settlement relationship, particularly in the region of the working load (W_L)
- Also to ensure that failure does not occur before a predetermined /selected load is reached.
- The selected load is a multiple of the working load ($1.5W_L$ or $2.0W_L$) and is referred to as Proof Load
- In accordance with BS 8004 (2015), which is in commonly used in Nigeria, the pile is regarded as capable if during the test process, it settles not more than 10% of its diameter.
- As piles are rarely taken to failure in Nigeria during testing.

Background Contd.

Balogun (1987) proposed the use of the equation

$$\delta = \frac{kP}{1 - \frac{P}{Pu}} \quad \text{..... (1)}$$



(Where ∂=settlement of pile head, K= a constant depending on pile- soil properties and reactions, P= load on pile, and P_U=ultimate pile load) for estimating the ultimate load-carrying capacity of pile.



He obtained higher values of factor of safety that ranged from 2.3 - 4.2 on some projects in Lagos based on estimated ultimate pile capacity values from load test using the equation

Background Contd.

The obvious disadvantage of the proof load test (i.e testing a pile to $1.5W_L$ or $2.0W_L$) is that the pile is never designed to its full working capacity.

Previous attempts to determine the ultimate load-carrying capacity of piles involve complex graphical procedure, subjective assumption or mathematical relations that contain constants, which cannot be readily determined

Aim and Objective

The aim of this work is therefore to propose simple straightforward graphical method of evaluating ultimate pile load of piles, from conventional pile load test results, without loading to failure.

Case study was considered from 20 different sites in this paper to show the practical results obtained.

Geologic Setting

- The sites on which the tests were carried out are located in the southern part of Nigeria.
- They include Kirikiri area of Lagos state, Rivers state (Bonny Island, Project sites within Port Harcourt metropolitan area), Bayelsa state (Kolo 2, Otuegila-Ogbia, Odi, Amasoma) and Delta State (Kiagbodo).
- They are located in the Benin Formation; a geological formation locally referred to as Coastal Plain Sands.
- The Benin formation, which is tertiary, is the terminal strategic unit of the three formations that make up the Niger Delta.
- Its type of section is made up of poorly cemented sand and gravel, with clay and sand intercalation

Methodology

- The pile load tests reported in this work were carried out between June 2001 and July 2013.
- The piles are precast concrete, auger bored piles and steel-cased driven piles of different dimensions.
- The tests were carried out one to three months after they were installed.
- The Maintained Load Test method, was used for the test and the proof load was taken to twice the working load, as recommended by the relevant code of practice BS 8004 (2015), ASTM D1143
- The details, including the designed working load are presented in Table 1.

Table 1:Details of test piles.

Test Pile	Site	Type	Size	Length (m)	Working Load (tons)
1	A	Precast concrete	300 x 300mm	16.0	25
2	A	Precast concrete	300 x 300mm	17.5	25
3	B	Precast concrete	450 x 450mm	15.0	40
4	B	Precast concrete	450 x 450mm	16.2	58
5	B	Precast concrete	450 x 450mm	15.5	40
6	C	Precast concrete	300 x 300mm	14.8	25
7	C	Precast concrete	300 x 300mm	14.75	25
8	D	Steel cased	406mm dia.	14.9	50
9	E	Bored	600mm dia.	25.0	166
10	F	Steel cased	508mm dia.	30.0	40
11	G	Bored	900mm dia.	21.0	203
12	H	Steel cased	406mm dia.	35.0	67
13	H	Steel cased	406mm dia.	35.0	67
14	I	Steel cased	350mm dia.	20.0	86
15	J	Steel cased	600mm dia.	12.0	28
16	K	Precast concrete	360 x 360mm	15.0	167
17	L	Precast concrete	350 x 350mm	9.0	65
18	M	Steel cased	334mm dia.	24.0	30
19	N	Steel cased	450mm dia.	25.6	65
20	N	Steel cased	450mm dia.	25.0	65

Methodology Contd.

- The following widely recognized criteria (BS 8004 2015; ASTM D1143; Tomlinson 1995) were used to interpret the load-settlement curve and to determine whether the piles failed or were failing:
  - ✓the load at which settlement continues to increase without any further increase in load.
  - ✓the load having a gross settlement of 10% of least pile width
  - ✓the load beyond which there is an increase in gross settlement, disproportionate to the increase in load.
  - ✓the load beyond which there is an increase in net settlement, disproportionate to the increase in load
  - ✓the load at which the slope of net settlement is equal to 0.25mm per 10kN (0.01inch per ton) of the load.

Methodology Contd.

- The platform was fabricated with Universal Beams and steel casings.
- The kentledge, which consists of reinforced concrete piles of cuts and 300 x 600mm x 12m long concrete sheet piles and/or 1.0m x 1.0m concrete cubes, was installed on the platform with a 25 - 50 ton P&H mobile crane .
- The test pile was loaded by jacking against a fixed platform, with a 230 – 600-ton hydraulic jack depending on the test load.
- The settlement-measuring devices, which consist of two dial gauges, were set up on opposite sides of the test pile.

Methodology Contd.

- Each of the dial gauges was placed at about 500mm (not less than the pile's width) from the test pile to ensure minimum disturbances and allow for convenient reading.
- The load was applied to the pile in a series of increment equal to 25% of the designated working load of the pile
- Each load increment was maintained until the rate of settlement was not greater than 0.25mm/hr or 2 hours, whichever occurs first.
- The proof load was held for a period of 24hr, before the commencement of unloading.

Results and Discussion

- The settlement values of test piles 1 to 20 under working load and proof load, including their residual settlements after all the loads have been removed are shown in Table 2.
- The limiting settlements (10% of pile's diameter), which are supposed to be the settlements at failure are also indicated in the Table for comparison.
- Typical load-settlement curves of Piles 1 and 6 are displayed in Figures 1 and 2 respectively.
- The load-settlement curves of the remaining piles are like those illustrated in figures 1 and 2.

TABLE 2– Maximum and residual settlements of test piles.

Test Pile	Settlement at Working Load (mm)	Settlement at Proof Load (mm)	Residual Settlement (mm)	Limiting Settlement (0.1d) (mm)
1	0.170	1.580	0.72	30
2	1.06	1.365	1.305	30
3	0.015	0.23	0.10	45
4	0.05	0.44	0.30	45
5	0.048	0.475	0.415	45
6	0.943	4.365	2.66	30
7	0.335	3.943	0.195	30
8	0.110	0.56	0.12	40.6
9	0.38	1.15	0.691	60
10	0.05	0.17	0.145	50.8
11	2.47	33.247	25.6	90
12	0.82	2.25	2.2	40.6
13	0.79	2.505	2.025	40.6
14	1.61	4.128	2.748	35
15	1.055	2.475	1.295	60
16	13.26	33.89	8.61	36
17	0.36	1.448	1.325	35
18	0.39	0.915	0.905	33.4
19	0.23	0.44	0.23	45
20	0.125	0.2325	0.1175	45

FIG. 1–Load versus settlement curve of test pile 1.

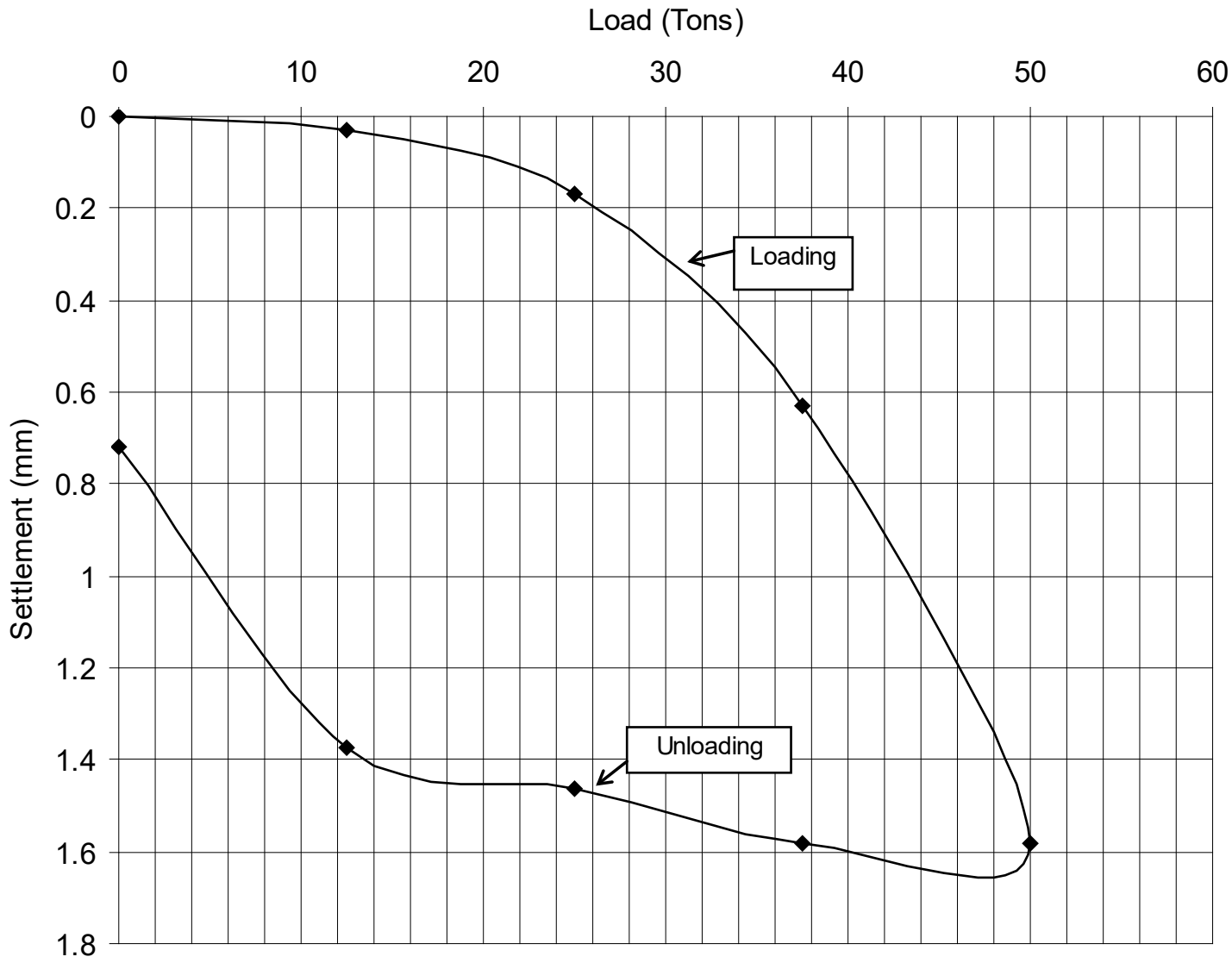
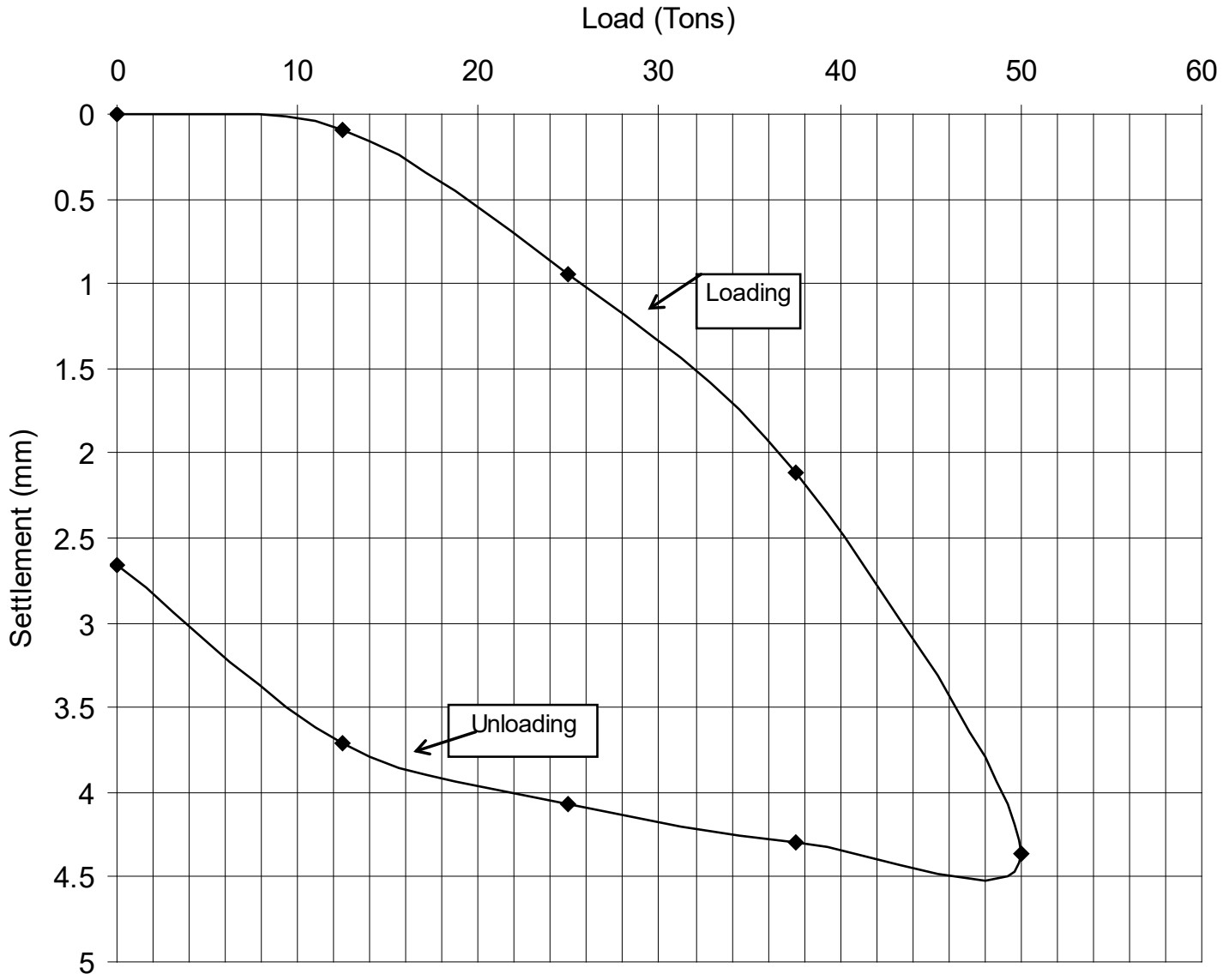


FIG. 2–Load versus settlement curve of test pile 6.



Results and Discussion Contd.

At the early stages of loading before the working load is attained, the settlement, in all cases, is very small.

The movement in this region is wholly elastic, and it is expected that if the pile load is removed the pile's head will rebound to original level (Tomlinson, 1995).

The figures show that as the load is increased the load-settlement curve stiffens.

The maximum settlements recorded for piles 1 to 20 are between 0.17mm and 33.89mm and all are less than the limiting value of 0.1d.

Results and Discussion Contd.

- The range of maximum settlement is obviously high due to the piles being of different types and sizes.
- And were installed into different sites, which are not expected to have the same subsurface conditions, including soil properties.
- The residual settlements obtained after 12 hours when all the loads were removed ranged from 0.1mm for Pile 3 (450 x 450 mm Precast concrete pile) to 25.6mm for Pile 11 (900 mm diameter bored pile).
- Tables 3, gives the reciprocal of the slopes of the loading part of the typical load-settlement curves at different working loads of the various test piles.

TABLE 3– Reciprocal (1/s) of load-settlement curves for test piles at different working load

Test Pile	At 50%W_L	At 75%W_L	At 100% W_L	At 150%W_L	At 200%W_L
1	140	110	42.6	17.5	5.7
2	21.0	14.3	13.5	12.0	3.8
3	∞	1204.8	1010.1	500	24
4	1040	729.9	429.2	165	159.2
5	933	490.2	420.2	179.9	145.8
6	23	14.5	12.5	7	1.3
7	77.6	70.6	59.9	40.0	9.6
8	444.4	330.8	229.9	187.6	170.1
9	1111	500	250	125	-
10	1111.1	833.3	625	416	261
11	166.7	50.8	41.7	7.24	4.60
12	84	84	71.4	55.6	33.4
13	10.42	8.4	6.2	4.8	3.34
14	666.7	52.6	51.3	27	16.7
15	28.6	18.2	17.5	8.8	-
16	41.7	10.4	8.3	5.12	-
17	400	166.7	166.7	142.9	58.8
18	142,9	111.1	37	23.3	-
19	833.3	555.6	200	83.3	-
20	1111.1	909.1	400	161.3	-

Results and Discussion Contd.

- Theoretically, at failure, the load- settlement curve would be parallel to the load axis, and at this stage the pile could even disappear into the ground.
- The slope (s) of the curve would therefore increase rapidly and approach infinity ($s \rightarrow \infty$).
- In other words, the reciprocal (1/s) of the slope of load-settlement curve will approach zero.
- The relationship between load (P) and reciprocal of the load-settlement curve (1/s) are illustrated in Figures 3 and 4 for Piles 1 and 6 respectively.

FIG. 3–Load versus reciprocal of settlement curve's slope for pile 1.

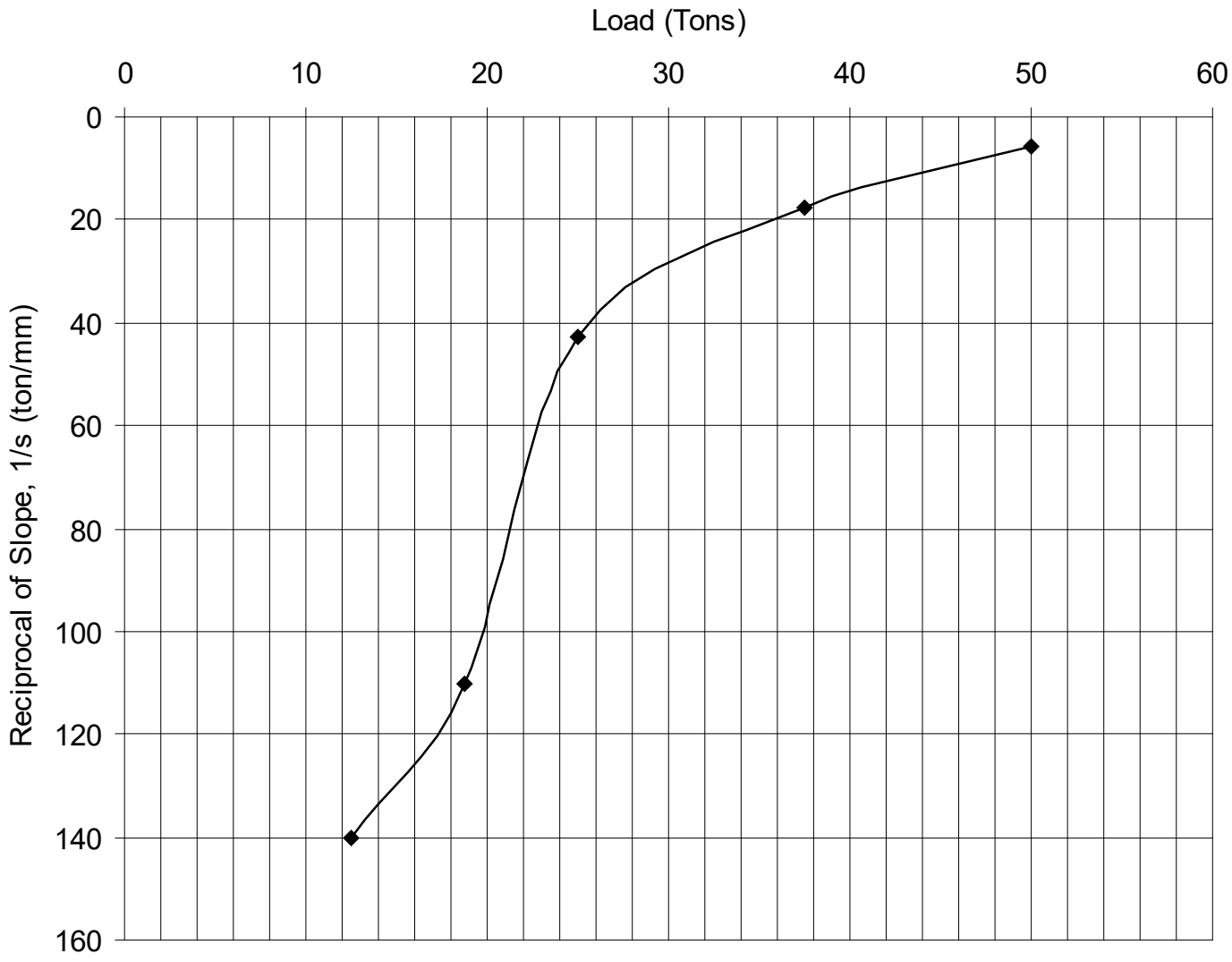
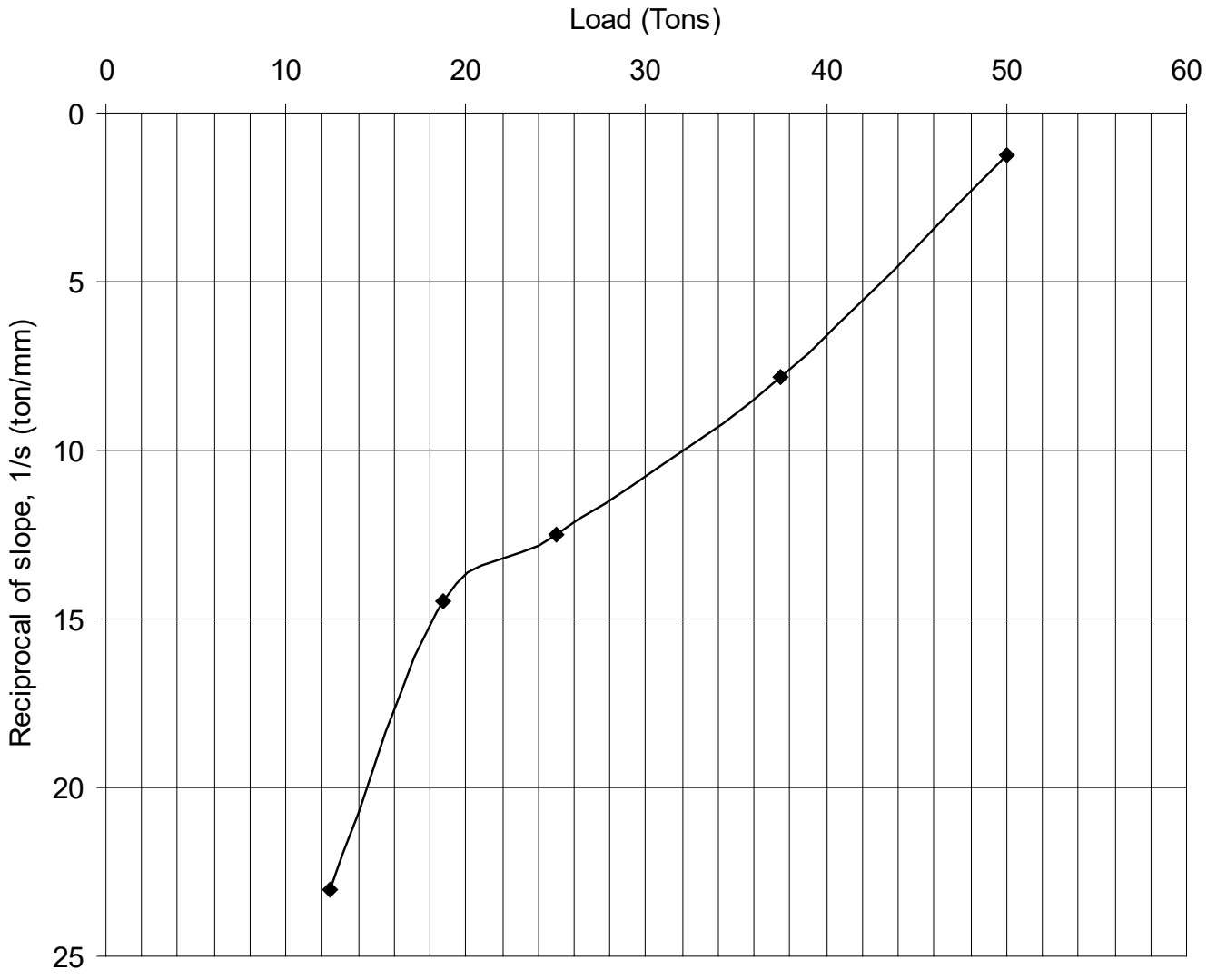


FIG. 4–Load versus reciprocal of settlement curve's slope for pile 6



Results and Discussion Contd.

- Extrapolating the load-reciprocal curve to cut the load axis (where the reciprocal 1/s is zero) gives the failure load of each pile.
- The failure loads are shown in Table 4 for the respective piles.
- These failure loads result in a factor of safety (F.O.S.) of between 2.0 and 3.64 for the piles 1 - 20.
- For all the piles, the F.O.S. obtained from the estimated Ultimate Failure load is greater than 2.0.
- Therefore, using proof load as limit of pile's capacity could result in gross underestimation.

TABLE 4–Estimated ultimate failure loads of test piles.

Test Pile	Working Load (tons)	Failure Load (tons)	Factor of Safety
1	25	60	2.40
2	25	65	2.60
3	40	82	2.05
4	58	130	2.24
5	40	120	3.00
6	25	53	2.12
7	25	56	2.24
8	50	182	3.64
9	165	340	2.06
10	40	116	2.9
11	203.2	720	3.54
12	67	185	2.76
13	67	190	2.84
14	86	250	2.9
15	28	58	2.07
16	167	380	2.27
17	65	154	2.4
18	30	95	3.2
19	65	130	2.0
20	65	130	2.0

Conclusion and Recommendation

- This work investigated the failure load of test piles using simple graphical method which was applied to 20 pile load test site.
- It assumed that at failure, settlement of the piles continues without increase in load and the load-settlement curve is parallel to the load axis.
- Hence, the reciprocal of the slope is zero at failure
- Extrapolating a curve of Load-Reciprocal of slope to cut the load axis (reciprocal is zero) gives an estimate of the failure load of the test pile.
- This was applied to twenty (20) different piles installed using different methods in different locations in southern Nigeria.
- The failure load ranged from 53 to 720 tons while the proof load ranged from 40 to 203.2 tons.

Conclusion and Recommendation Contd.

- Their F.O.S. therefore ranged from 2.0 to 3.64.
- The conventional method of merely using the static pile load test to ascertain that failure will not occur at proof load implies imposing a maximum F.O.S. of 2.0 on the pile.
- From the foregoing, this amounts to underestimation of the pile's capacity.
- Determining graphically the failure load could lead to exploiting the maximum potential of the pile.

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PARALLEL SESSION A4
Gold Innovations III



HERRENKNECHT AG

MECHANIZED SHAFT SINKING WITH VSM

DFI 2025_Nicola Stefanelli

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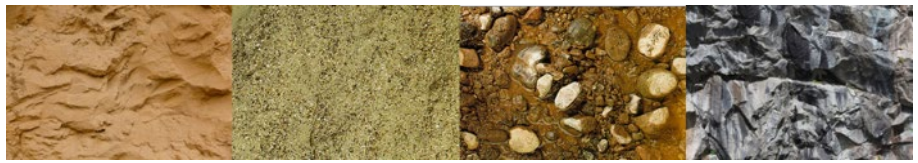
VSM APPLICATION RANGE
GROUND CONDITIONS & GROUNDWATER



- Submerge Excavation
- Application challenging or highly permeable soils
- Below groundwater,

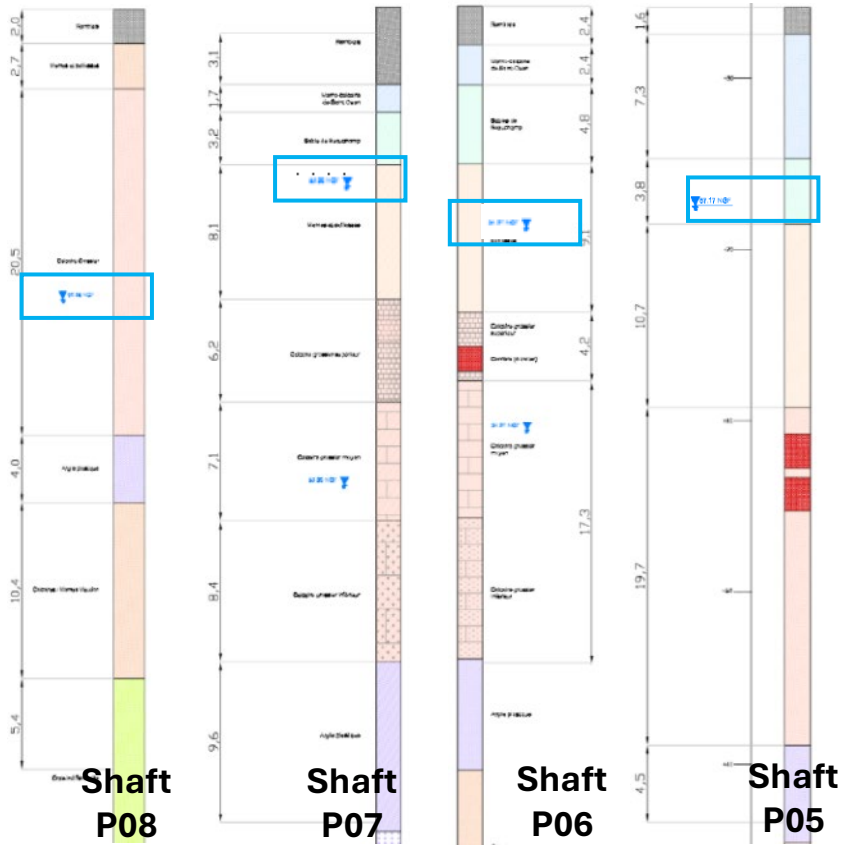
  max. 25 bar

- Applicable in almost all soils with strengths of up to

  max. 140 MPa

Setup example: GPE – Paris, France

Changing ground conditions, high permeability and groundwater level

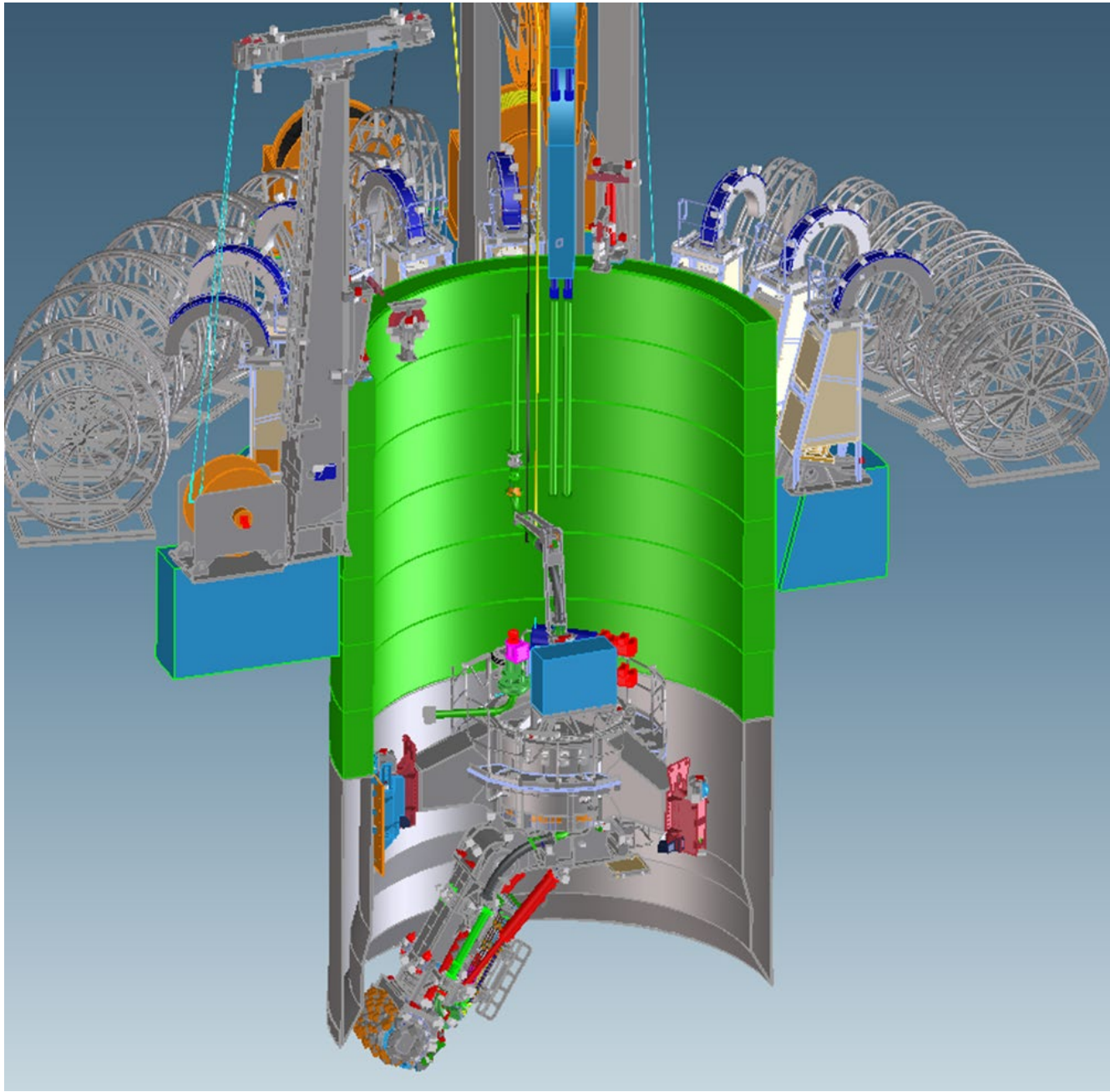


Ground conditions:

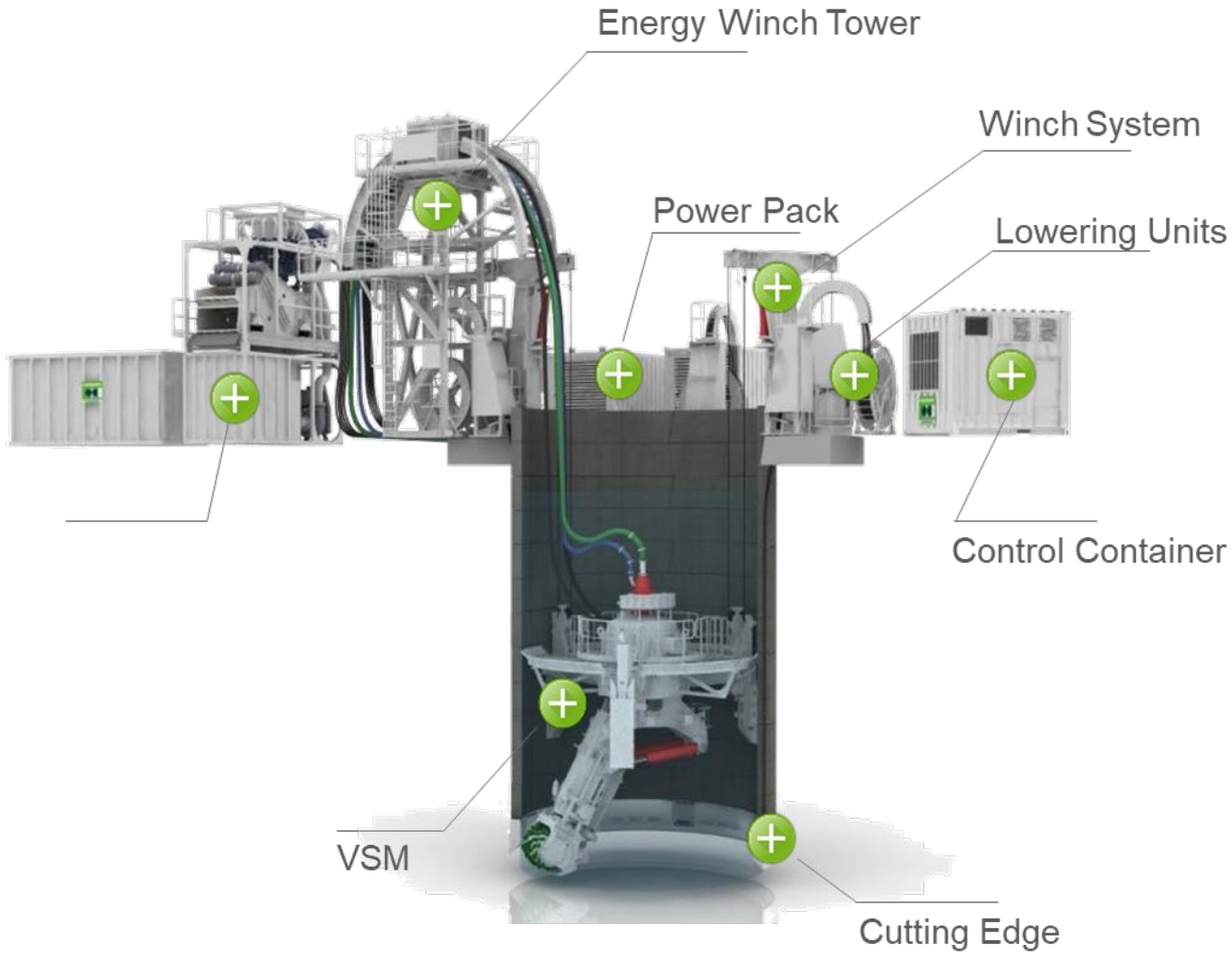
- Changing from loose sand to hard limestone with strengths of up to 100 MPa
- High permeability, fissured rock with high groundwater level



1 VSM MACHINE



VSM MACHINE AND EQUIPMENT OVERVIEW



VSM

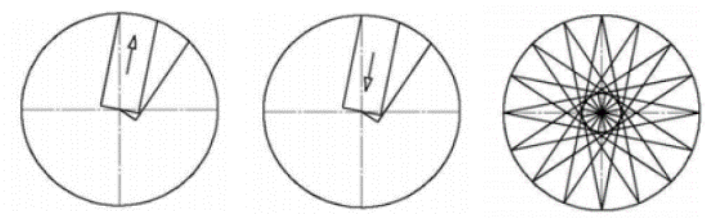


VSM working under groundwater

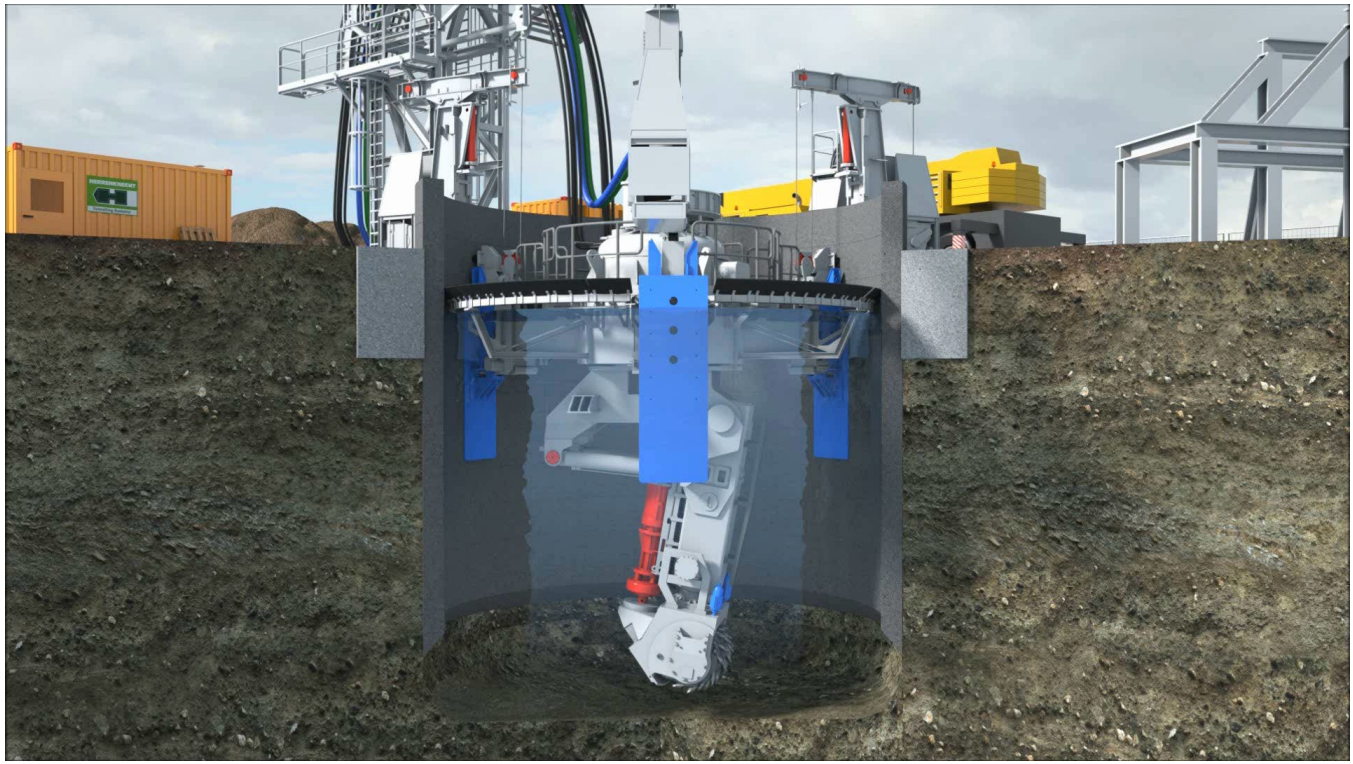
Mechanized Shaft Sinking

VSM FUNCTIONAL PRINCIPAL
EXCAVATION PROCESS

- Step-by-step excavation of complete cross section by flexible operation of the cutting boom



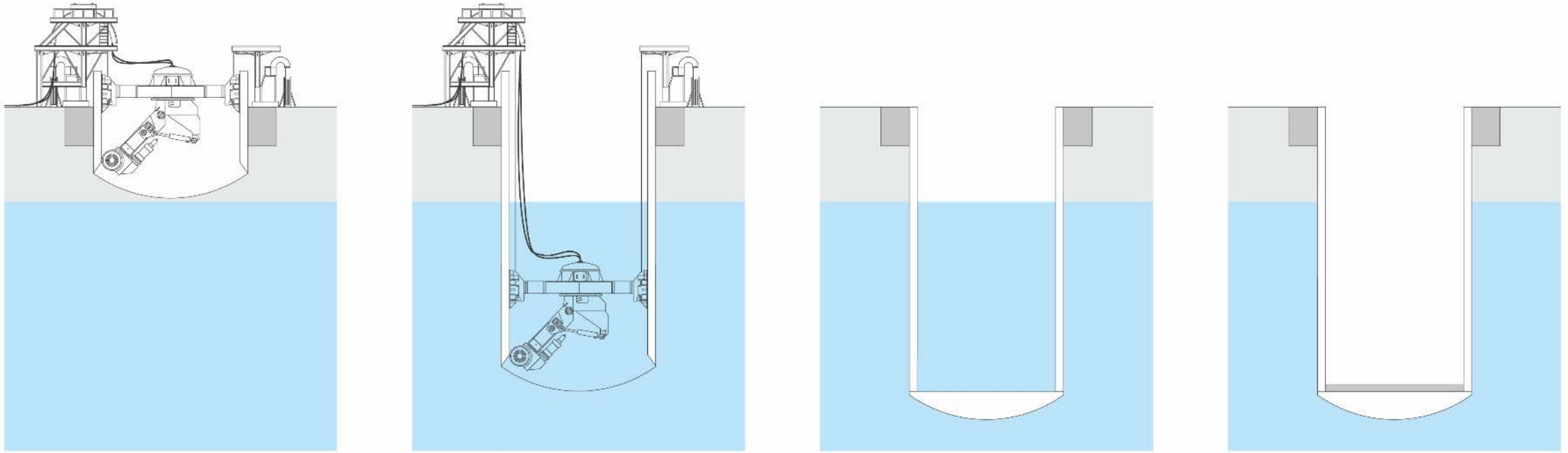
- Simultaneous excavation and ring building on the surface
- Slurry circuit transports excavated material to the separation plant on surface
- Full control of excavation and lowering of the shaft structure



See full video here

Mechanized Shaft Sinking

VSM SHAFT SINKING PROCESS UNDER GROUNDWATER



1. Jobsite setup;
   VSM installation in launch section.

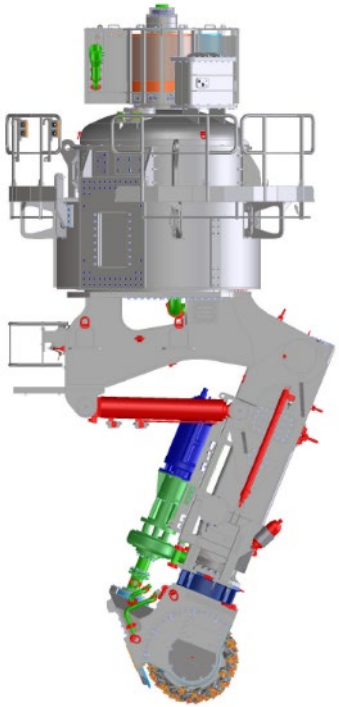
2. Excavation and shaft sinking
   in parallel;
   VSM is working under ground water
   level.

3. VSM recovery;
   installation of bottom plug;
   grouting of annulus.

4. Dewatering of shaft;
   construction of bottom plate.

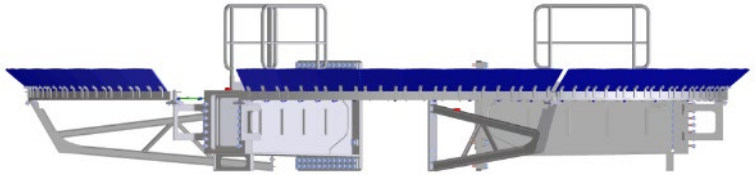
Mechanized Shaft Sinking

VSM MACHINE ADAPTATION TO DIFFERENT DIAMETERS

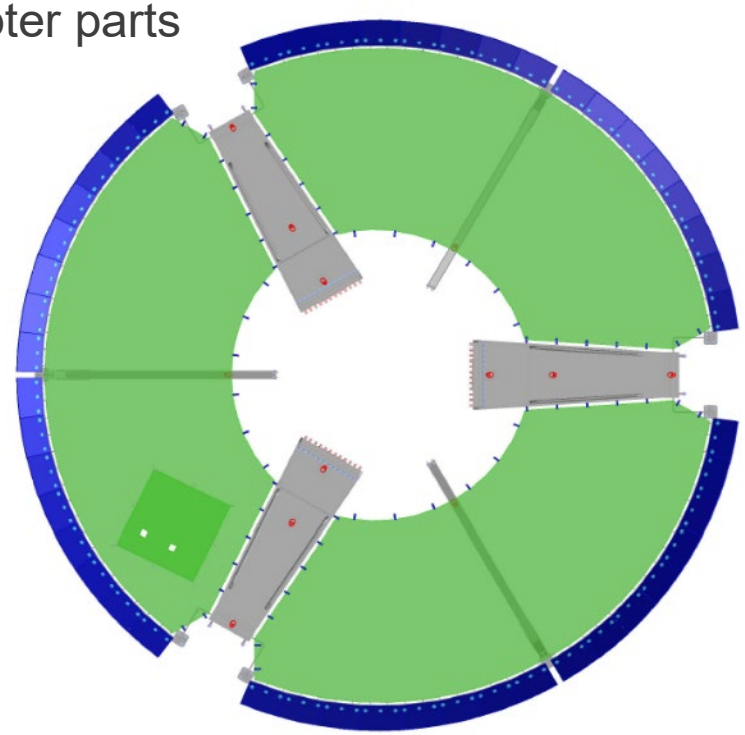


Core machine

+



Adapter parts



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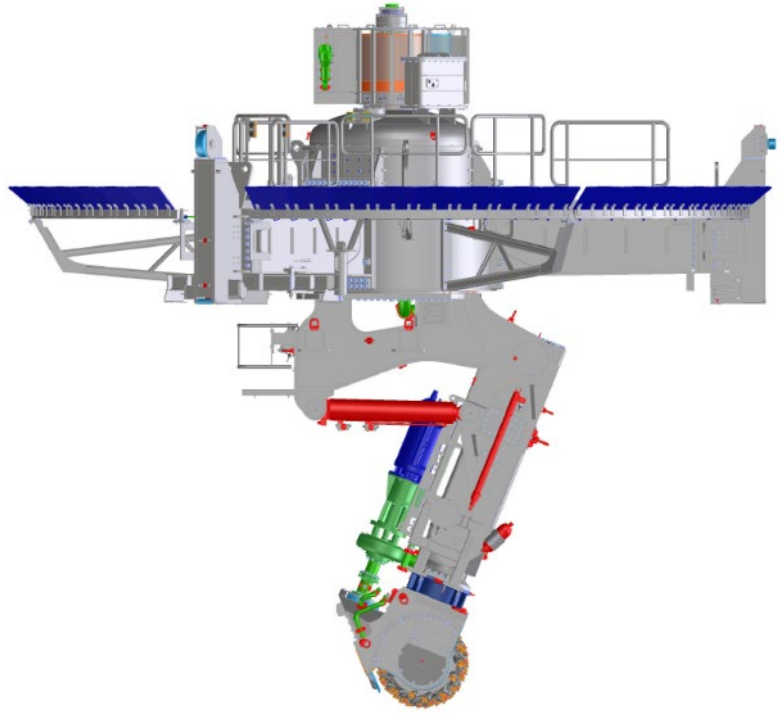
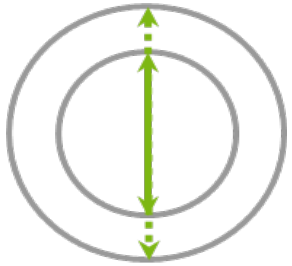
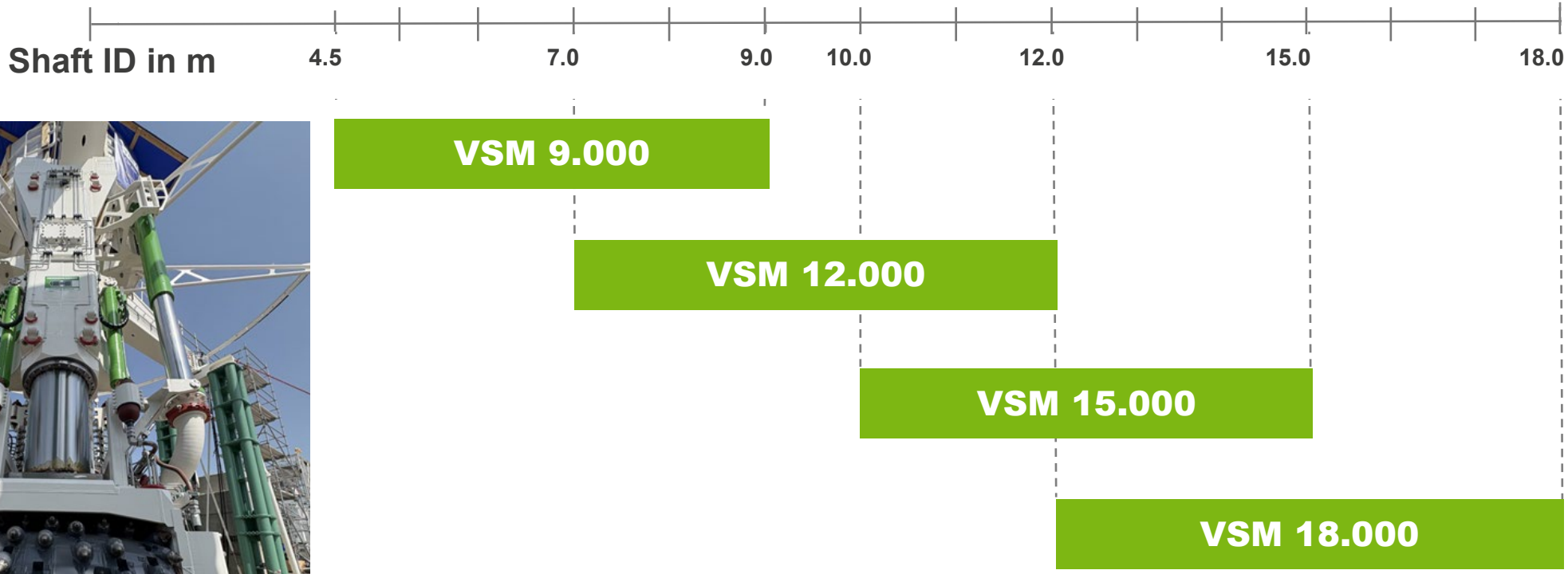


Final VSM configuration

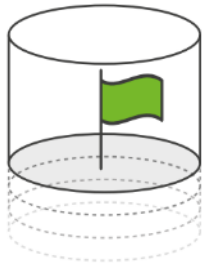
Mechanized Shaft Sinking

VSM MACHINE RANGE
WITH FLEXIBLE DIAMETERS









All machines can be designed for shaft depths of up **250 m**

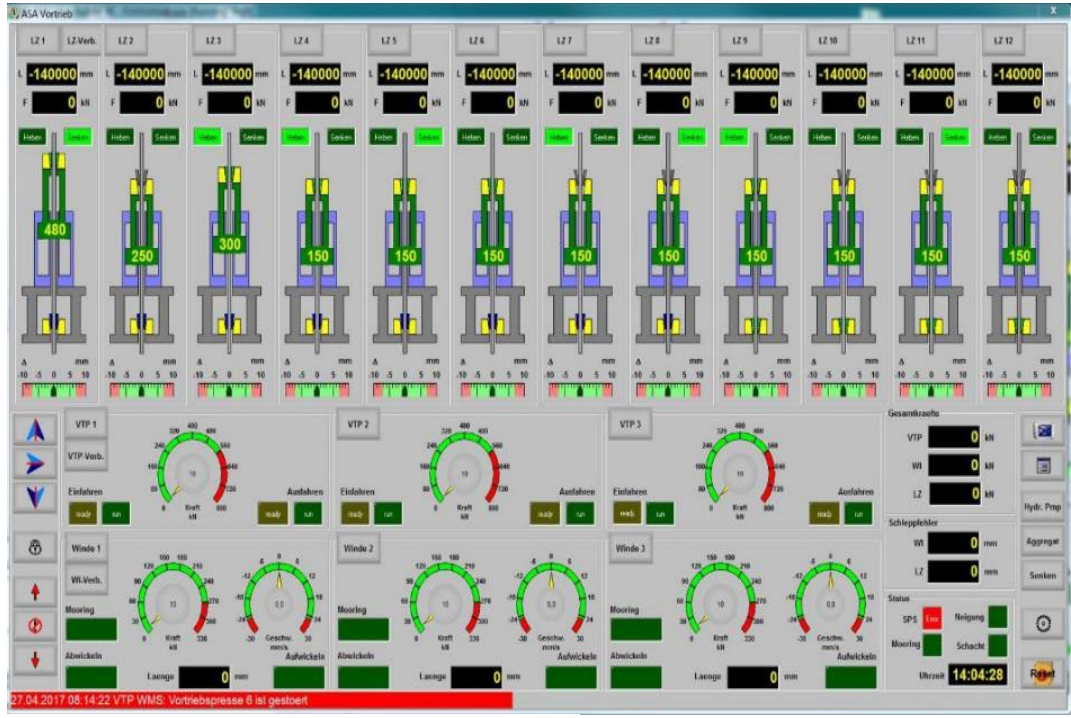
2

VSM EQUIPMENT OVERVIEW



VERTICAL SHAFT SINKING MACHINE
LOWERING UNIT SYSTEM

- Controlled sinking for high accuracy
- Complete control over all parameters during shaft installation
- Automated sinking process to increase operational safety
- Up to 12 lowering units can be operated simultaneously with one operating unit

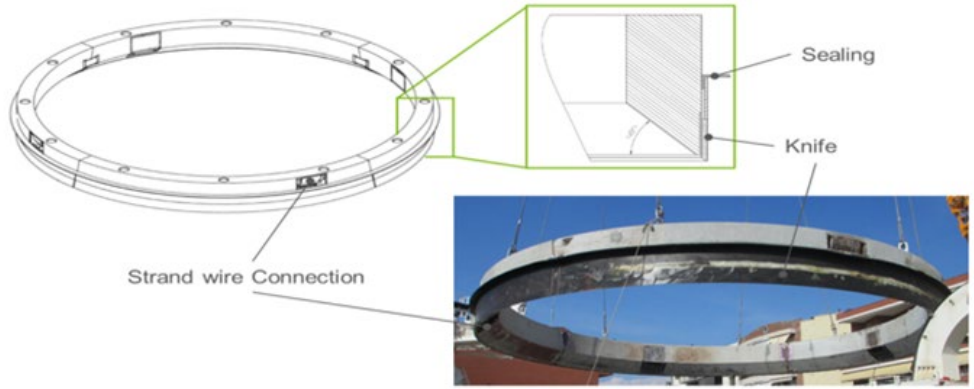






Shaft Lining – Cutting Edge.

- Cutting edge design includes a knife, a sealing and the strand wire connection



VSM Equipment Overview

VERTICAL SHAFT SINKING MACHINE
RECOVERY WINCH SYSTEM



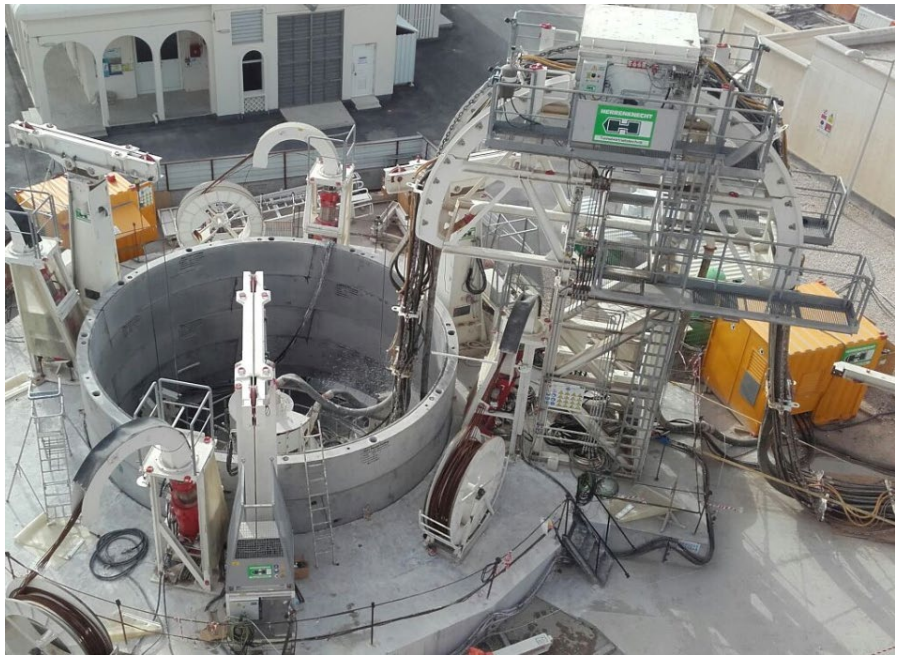


1. Integrated push system
2. Hook to machine
3. Hydraulic winch

VSM Equipment Overview

VERTICAL SHAFT SINKING MACHINE
ENERGY WINCH SYSTEM





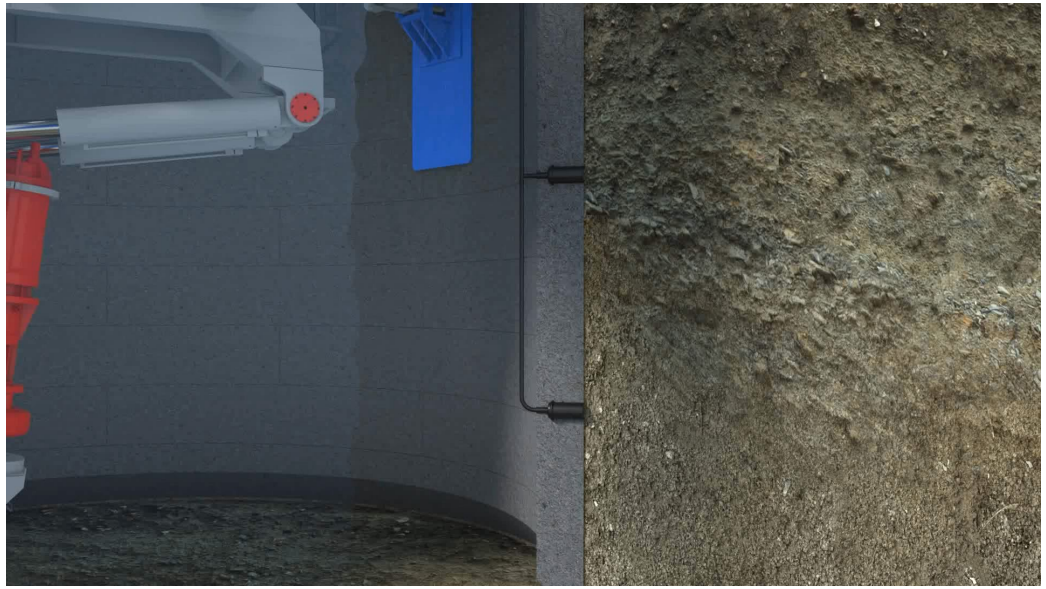
Energy winch = supply lines for:

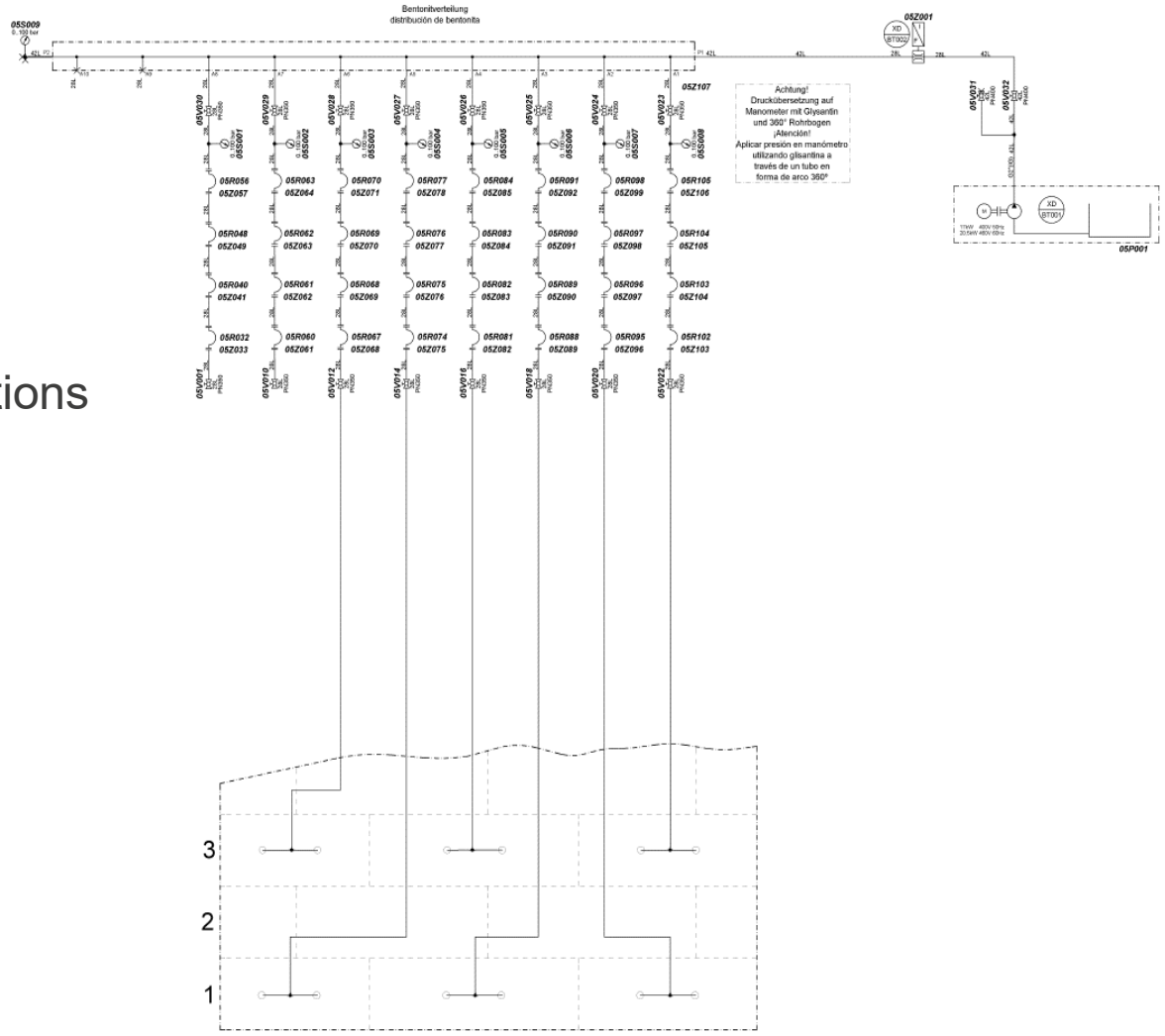
- slurry circuit
- bentonite lubrication
- Electrics & Hydraulics

VSM Equipment Overview

VERTICAL SHAFT SINKING MACHINE
BENTONITE LUBRICATION

- Bentonite injection volume: 0 – 8 m³/h
- Control of bentonite volume and pressure
- Compact bentonite **mixing and injection unit**
- **Injection ports** installed in the segments
  - Amount according to shaft depth and ground conditions



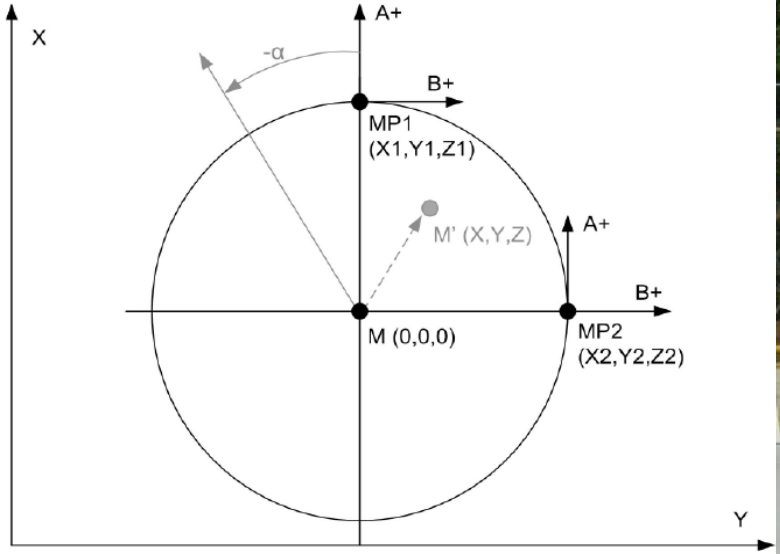


VSM Equipment Overview

VERTICAL SHAFT SINKING MACHINE NAVIGATION

- Surveying of shaft movement on two positions
- Monitoring of verticality, rotation and horizontal offset
- As build surveying of the entire shaft lining
- Surveying result displayed on operator screen

Reference values on exemplary projects				
City	Shaft ID [m]	Shaft Depth [m]	Deviation X	Deviation Y
St. Petersburg	7.8	78	26 mm	19 mm
Moscow	5.6	69	27 mm	30 mm
Dortmund	9.0	25	13 mm	14 mm
Naples	4.5	46	9 mm	11 mm
Barcelona	9.0	54	22 mm	25 mm
Hawaii	10.0	36	11 mm	6 mm



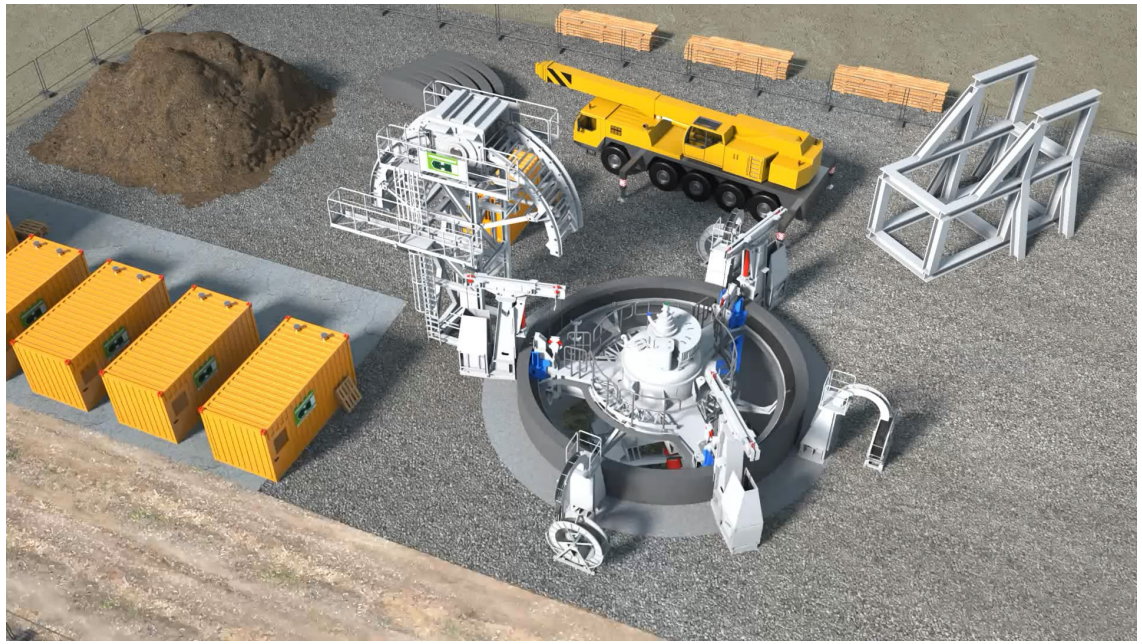


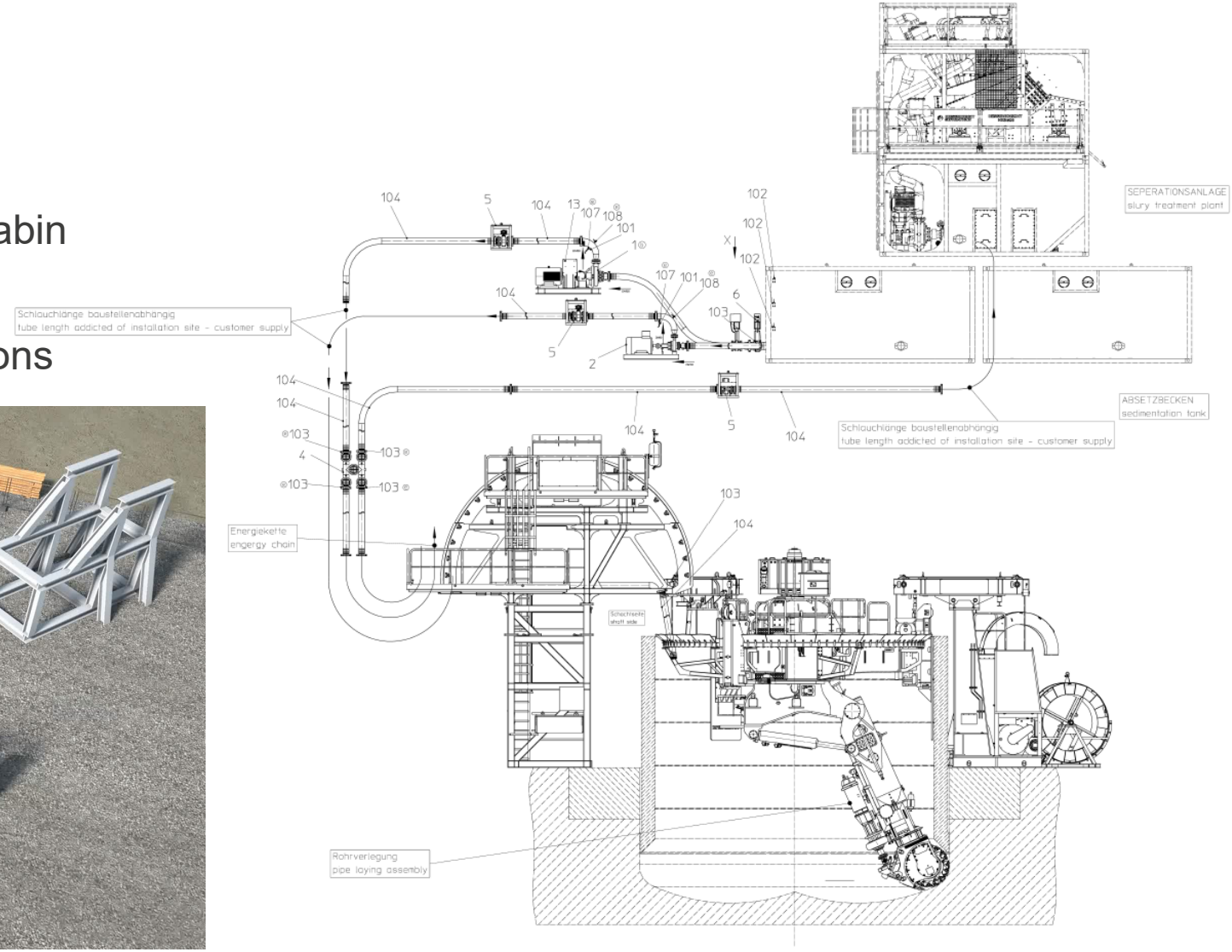


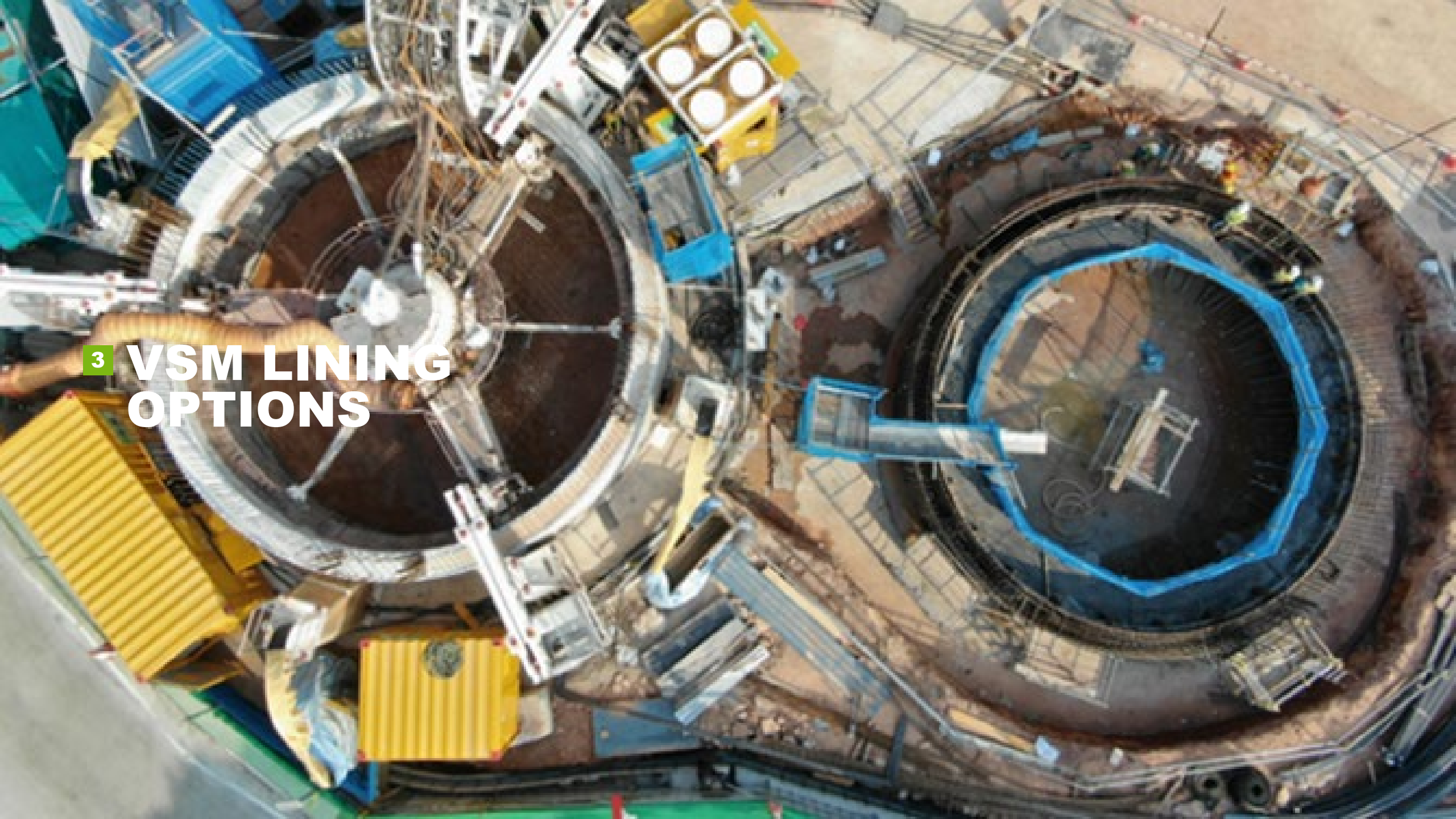
VSM Equipment Overview

VERTICAL SHAFT SINKING MACHINE
SLURRY CIRCUIT

- Slurry circuit volume: 0 – 800m³/h
- Slurry pipe size: ID150mm / ID200mm
- Fully automatic operation from control cabin
- Flow control in discharge and feed line
- Bypass gate valves for backwash functions



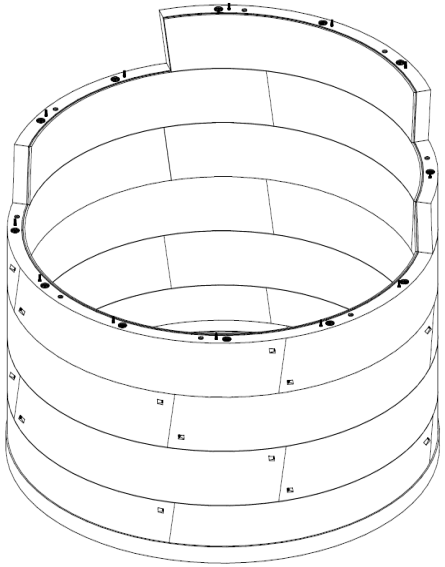


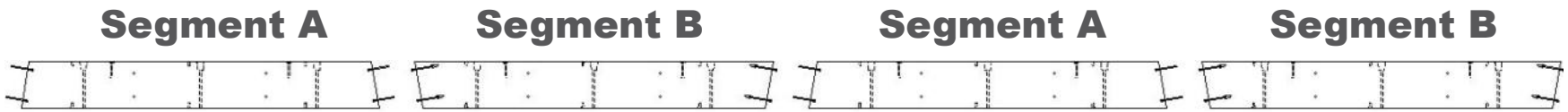


3 VSM LINING OPTIONS

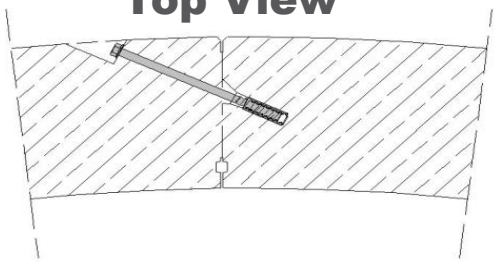
VSM Lining Options

SHAFT LINING OPTIONS
PRE-CAST SEGMENTS

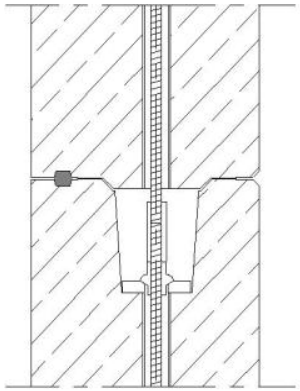




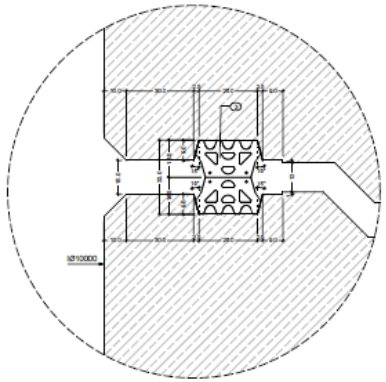
Top View



Side View



Detail Gasket





- Permanent water tightness right from the start
- One single shaft lining w/o secondary lining
- pre-cast segments with high accurancy

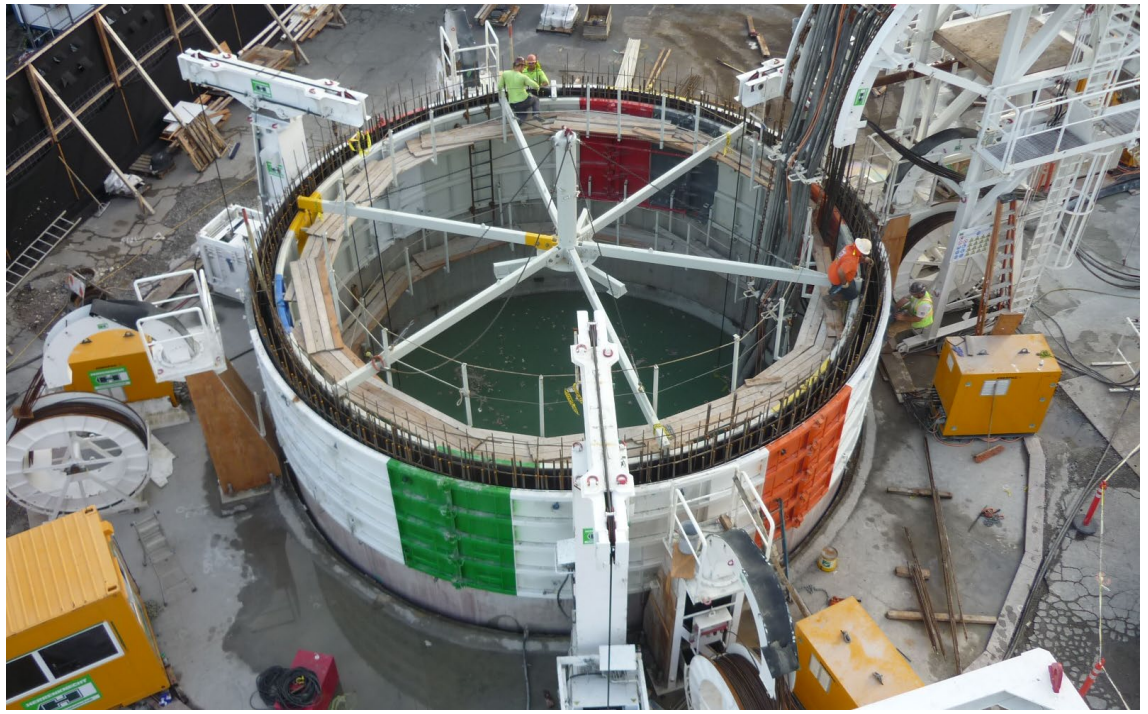
VSM Lining Options

SHAFT LINING OPTIONS
IN-SITU LINING

- In-situ lining = final lining
- Mould design in accordance to VSM siniking process
- High quality and precision
- Permanent water tightness right from the start
- Sinking speed of up to 1.2 m in 24h
- Easy integration of „soft-eye“, e.g. for tunnel conections
- Rebar or fibre reinforcement concrete







SHAFT LINING OPTIONS
SEGMENTS OR IN-SITU CASTING





Ring construction time	⧗	⧗ ⧗ ⧗
Lining costs	$ $ $	$ $
Personnel required	⛑ ⛑ ⛑ ⛑	⛑ ⛑ ⛑ ⛑ ⛑ ⛑ ⛑
Lining strength	+ +	+ + +
More factors	✓ High geometric precision ✓ Fewer working steps and components in motion ✓ Cleaner operation	✓ Easy integration of the tunnel connection into the lining ✓ Less joints

4

VSM JOBSITE SETUP

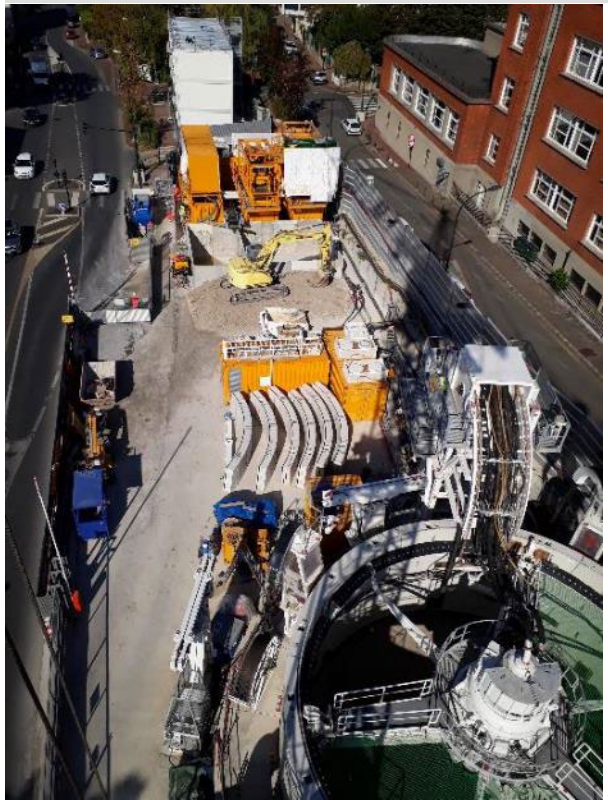


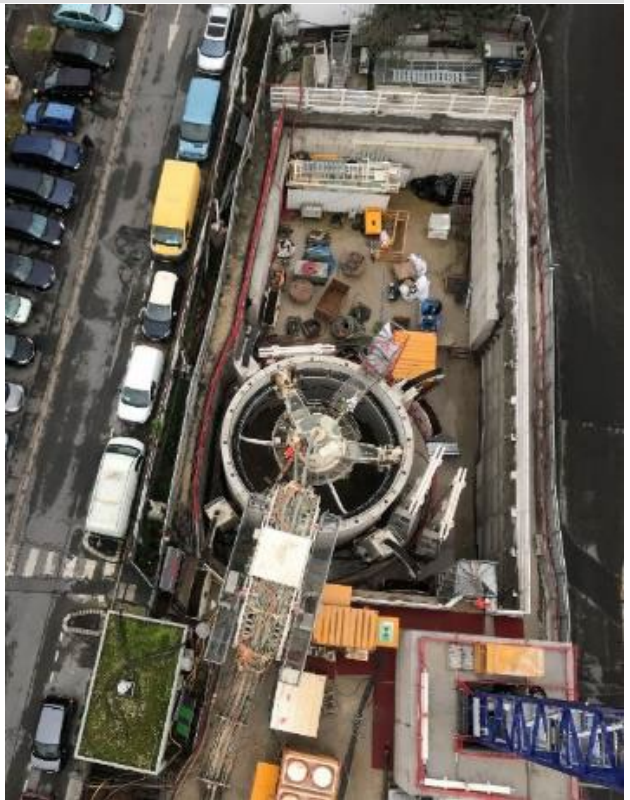
Mechanized Shaft Sinking

VSM JOBSITE SETUP
IN INNER-CITY CONDITIONS

› Compact and flexible setup of equipment, according to surface conditions

Setup example: GPE – Paris, France
Shaft OD up to 12,800 mm





Setup example: Girona, Spain
Shaft OD up to 5,950 mm





5

RING FOUNDATION

Mechanized Shaft Sinking

RING FOUNDATION
POSSIBLE RING BEAM VARIANTS



Lost ring beam





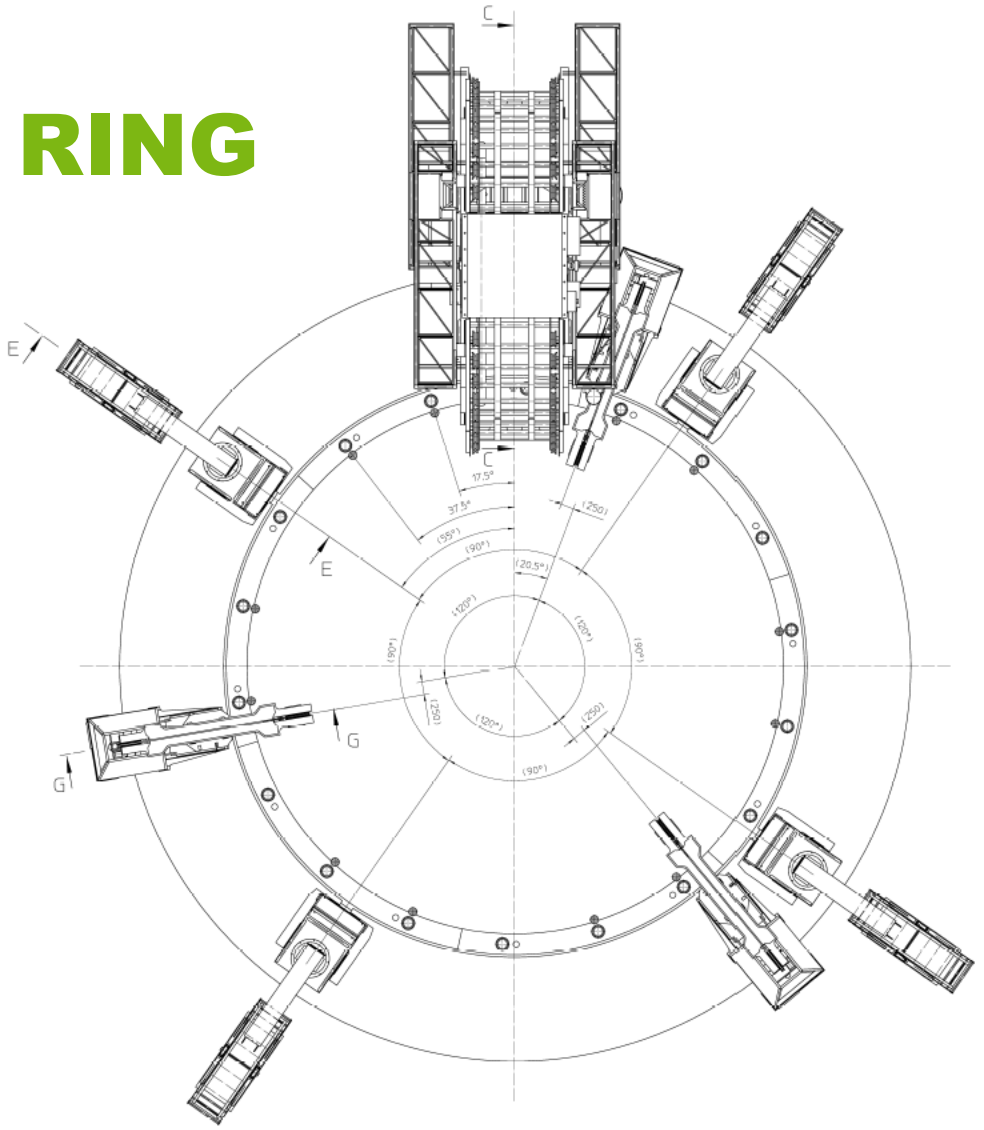
Reusable ring beam



Mechanized Shaft Sinking

RING FOUNDATION
INTERFACES FOR THE RING BEAM DESIGN

- Arrangement of the VSM machine equipment related to the jobsite and project conditions
- Reaction loads and load case of each machine component installed onto the ring beam
- Necessary pre-excavation height for the machine installation
- Specification of connection parts to fix the machine components onto the ring beam

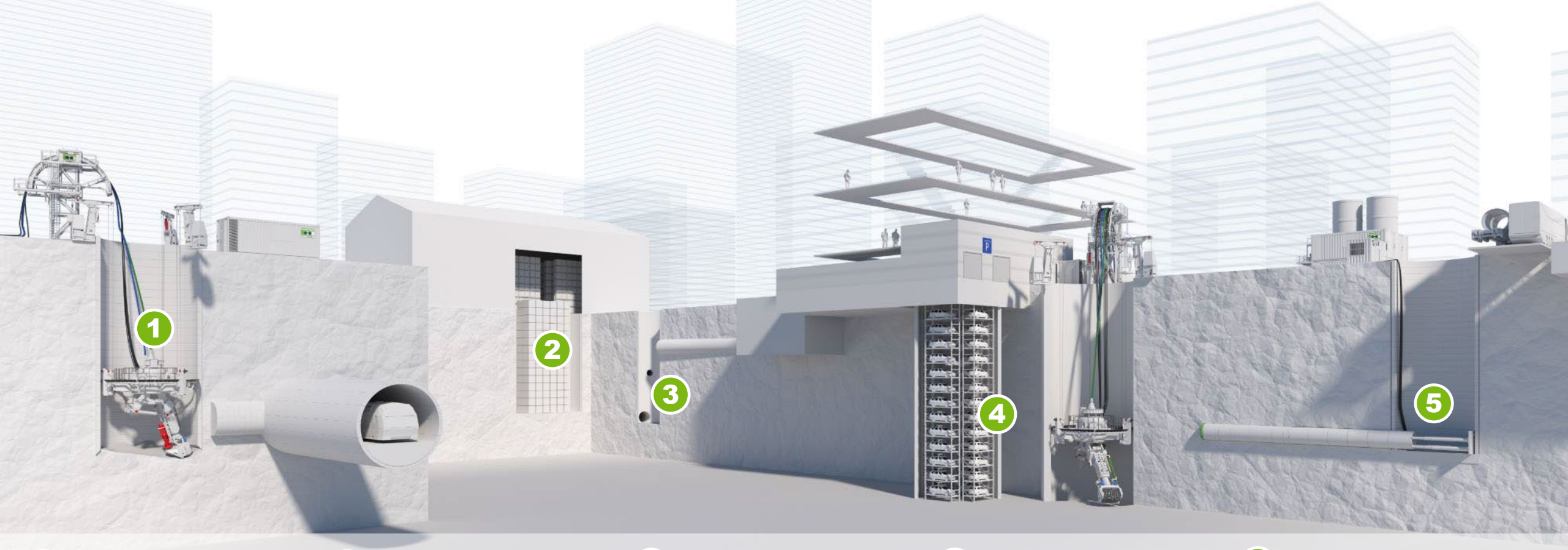




6 VSM APPLICATION FIELDS AND BENEFITS

Mechanized Shaft Sinking

VSM APPLICATION RANGE
INNER-CITY APPLICATION FIELDS



1. Metro ventilation and emergency shafts
2. Storage shafts
3. Sewage collector shafts
4. U-Park® systems
5. Tunnelling shafts

Mechanized Shaft Sinking

VSM APPLICATION RANGE
INNER-CITY CHALLENGES

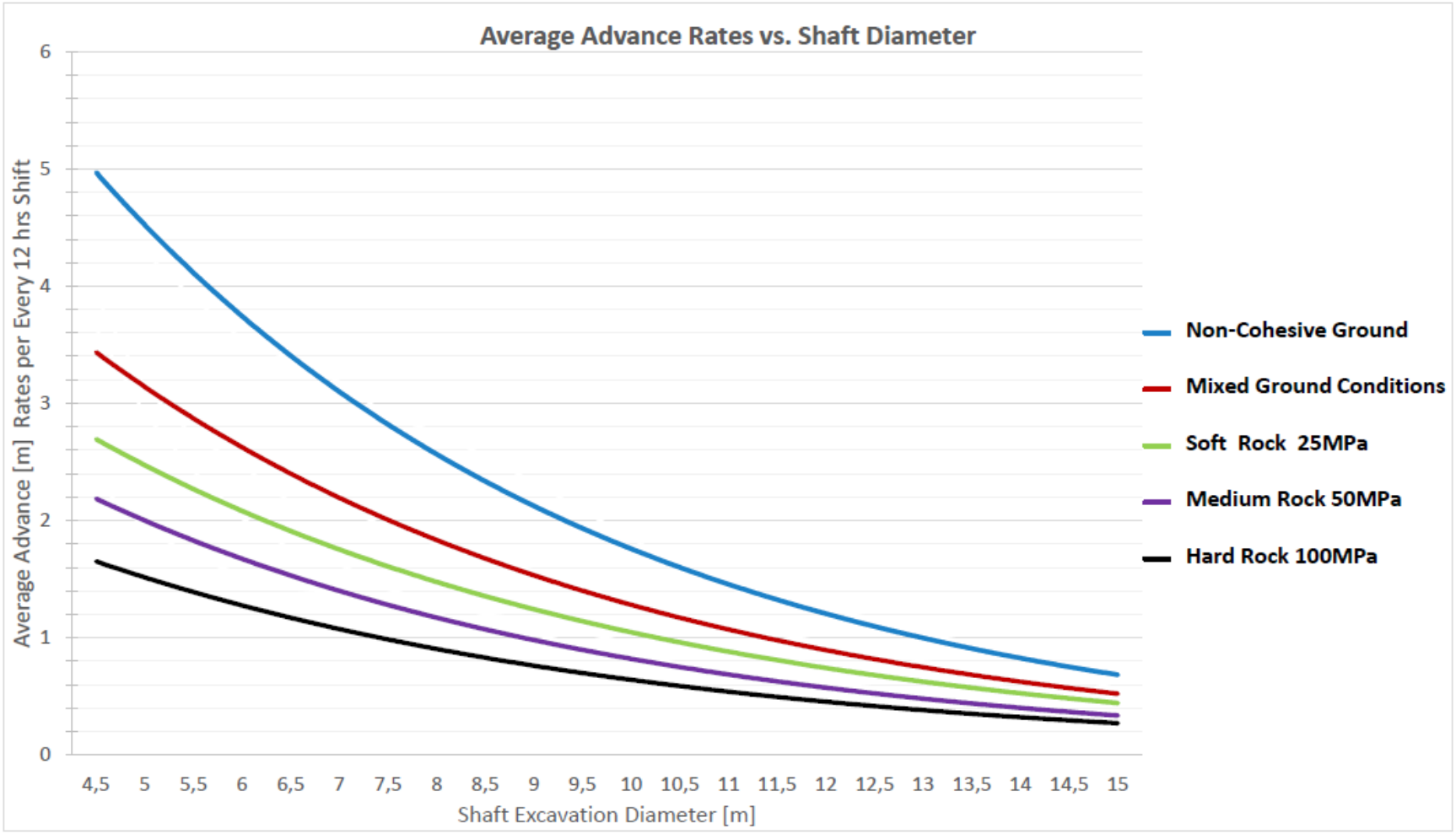
Challenges of inner-city jobsite surroundings:

- Confined space conditions
- Limited and therefore expensive availability of potential surface for jobsite installations
- Limited access for transport of deliveries and removal of excavated material
- Noise protection requirements
- Height restrictions of jobsite installations, e.g. due to air traffic regulations



Mechanized Shaft Sinking

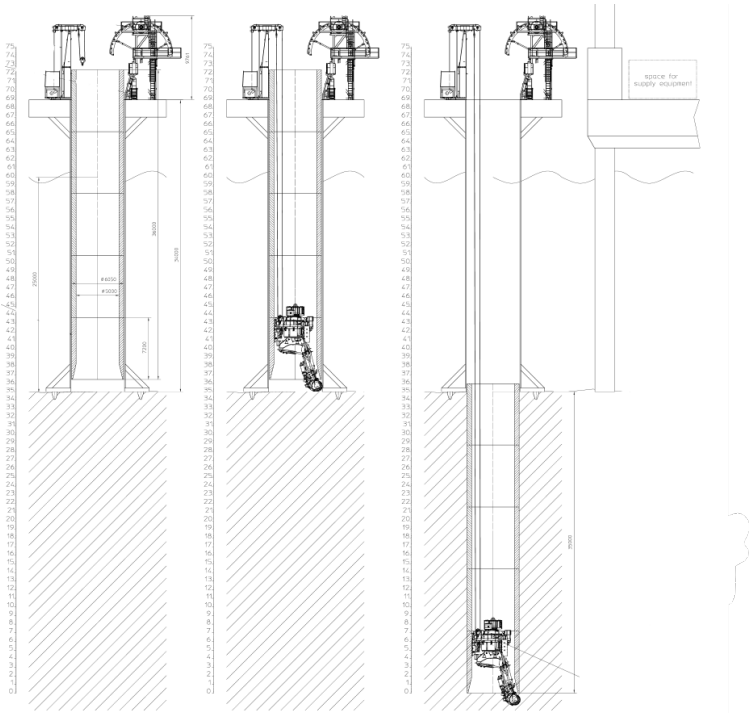
VSM APPLICATION RANGE
EXCAVATION RATES

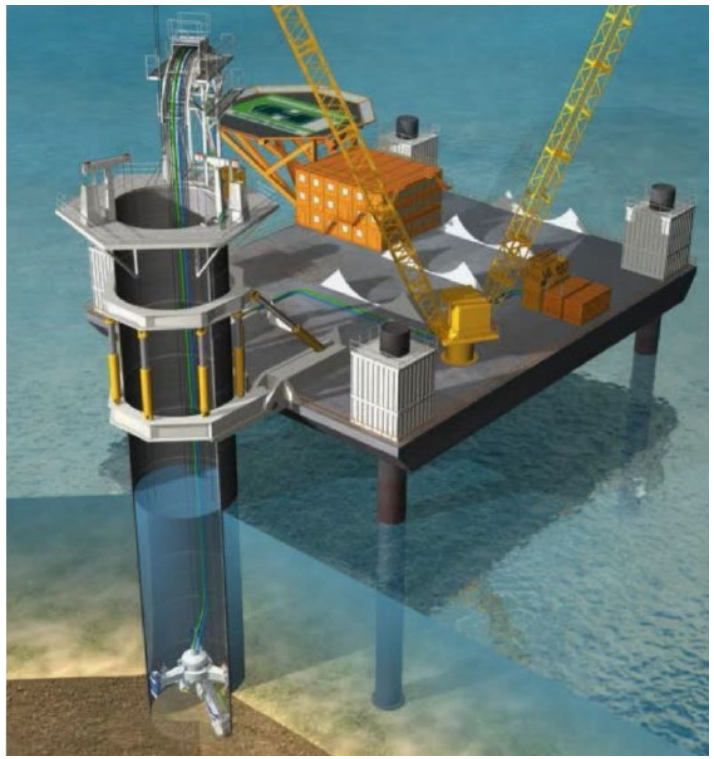


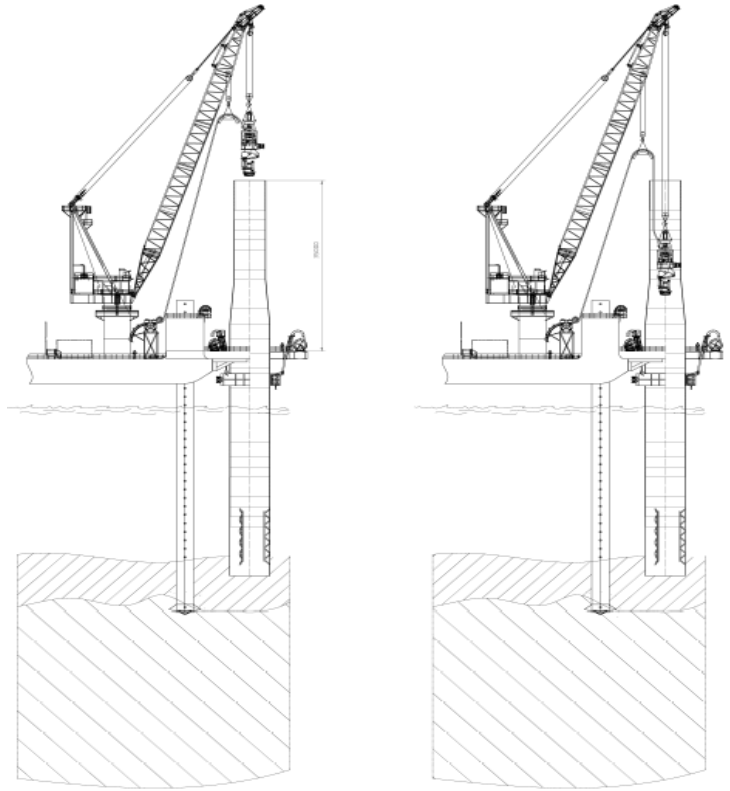
** The excavation rates in the diagram are net rates and based on a 12h working time*

VERTICAL SHAFT INSTALLATION METHODOLOGY

- Shaft holding structure mounted on the jack-up vessel.
- Shaft template for quick setup via hydraulic system.
- Shaft constructed in segmented sections.
- Monopile foundation installed using crane
- Monopile secured by a gripping unit mounted on the vessel
- Machine lifted and positioned inside the Monopile







Mechanized Shaft Sinking

GENERAL BENEFITS OF VSM
FASTER. GREENER. SAFER.

SAFETY FIRST
HIGH LEVEL OF SAFETY

no personnel in shaft during construction, less personnel in risk zones.



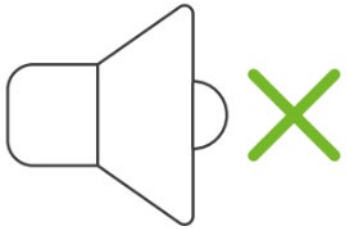


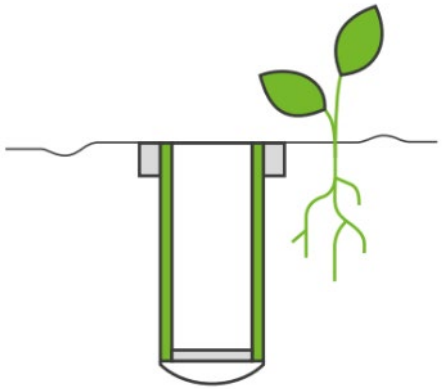
50 % FASTER
CONSTRUCTION TIME

compared to conventional methods.

LESS OVERALL
NOISE EMISSIONS

compared to conventional methods.





Compared to conventional methods,

40 %
LESS CONCRETE

is required for the shaft structure.

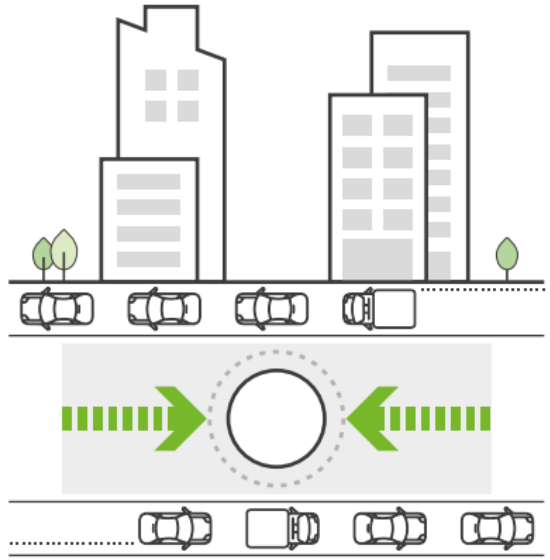
REDUCED OVERALL
CO_2 EMISSIONS

due to electrified operation of VSM equipment.



50 %
SMALLER JOBSITE

due to compact design and flexible arrangement compared to conventional methods.





7 VSM REFERENCE PROJECTS

VSM for tunnelling shafts

ALA MOANA HARBOUR CROSSING
HONOLULU, HAWAII, USA

- V-007, VSM10000 | Shaft Ø: ID 10000, OD 10800
- Shaft depth: 2 x 35m | 115ft.
- Ground conditions: sand, clay, gravel, high permeability
- Execution: 2012-2013
- Contractor: Kamloops Augering and Boring

- In-Situ concrete casting with glass-fibre reinforcement in the bottom







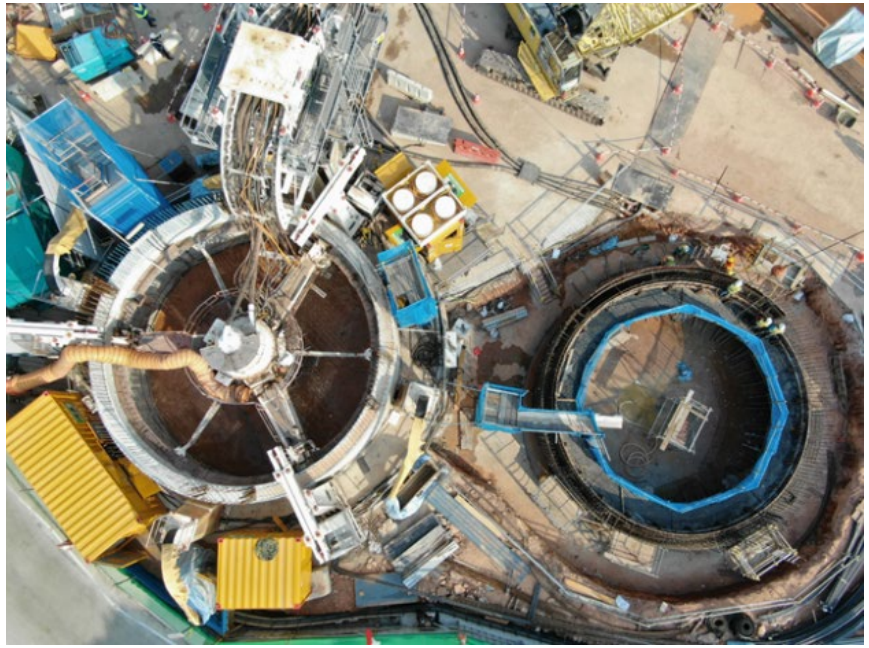
VSM for tunnelling shafts

SEWAGE SUPERHIGHWAY
DTSS2, SINGAPORE

- V-015, VSM12000 for Tunnelling Shafts
- Depth: 40m - 56 m,
- ID 10000/12000 mm | OD 11200/12800 mm

Project requirement:

- Shafts with less than 5 m distance
- Combination of Segment and In-situ lining to prepare bottom section of shaft for tunnelling operations





Laufende Projekte und Innovationen

VSM 15000 U-PARK
HYPERPARK GUOCAIXI, CHINA

- V-025, VSM 15000 (rental, with separation plant)
- For ID 10 – 15 m
- Location: Guangzhou, China
- 3 Shafts: ID 12 m | OD 12.9 m | Depth: **75 m**
- FAT: Jun 25th, 2025 @ HTE Guangzhou







HYPERPARK

- Car park in 3 shafts with 300 parking spaces

VSM for U-Park® shafts

AUTOMATED UNDERGROUND PARKING IN DEEP SHAFTS

› U-Park® general concept overview

				
Shaft inner diameter	**Ø 11 m**	**Ø 11-13 m**	**Ø 16 m**	**Ø 18 m**
Shaft depth	50 m / 70 m	50 m / 70 m	50 m / 70 m	50 m / 70 m
Total no. of parking spaces in shaft	75 / 105	100 / 140	150 / 210	200 / 280
No. of parking levels	25 / 35	25 / 35	25 / 35	25 / 35
Parking spaces per level	3	4	6	8

VSM for U-Park® shafts

AUTOMATED UNDERGROUND PARKING IN DEEP SHAFTS

- Conversion of outdated car parks to modern, automised underground parking systems
- Transfer of stationary traffic underground
- Creation of additional space on the surface for new potential uses in densely populated areas, where ground is extremely expensive
- Increase of quality of life and safety for drivers and residents



Innovative Secant Pile Excavation Support: Weapons Storage and Maintenance Facility – Malmstrom Air Force Base

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Prepared for



Prepared by

BRIERLEY ASSOCIATES
Creating Space Underground

In association with

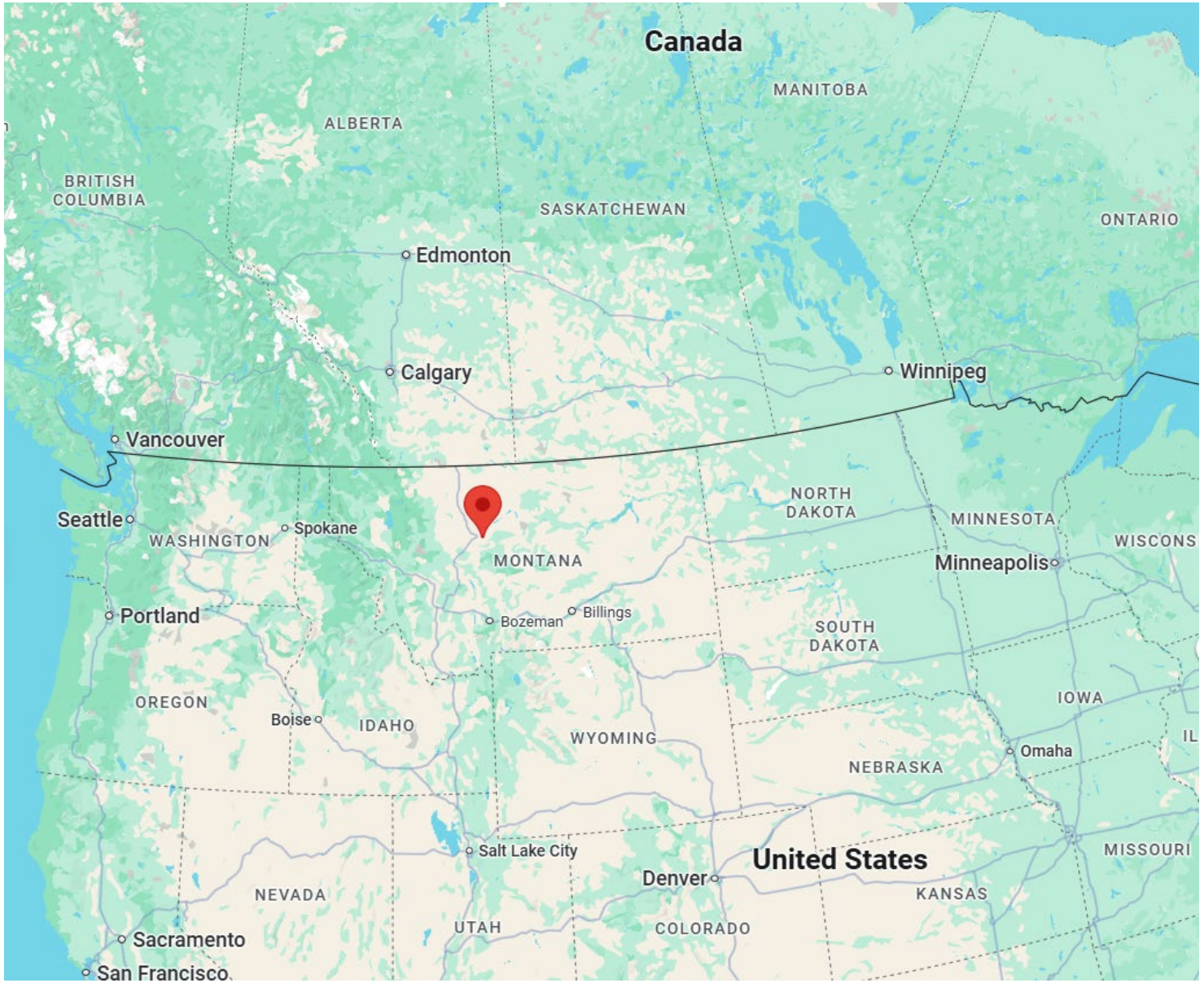
CONDON · JOHNSON & ASSOCIATES, INC.
CONTRACTORS AND ENGINEERS

October 22, 2025

Agenda

- Project Location and Geology
- Structure Configuration
- SOE Challenges and Constraints
- Options Considered
- Preferred Alternative
- Design Details
- Construction

Project Location



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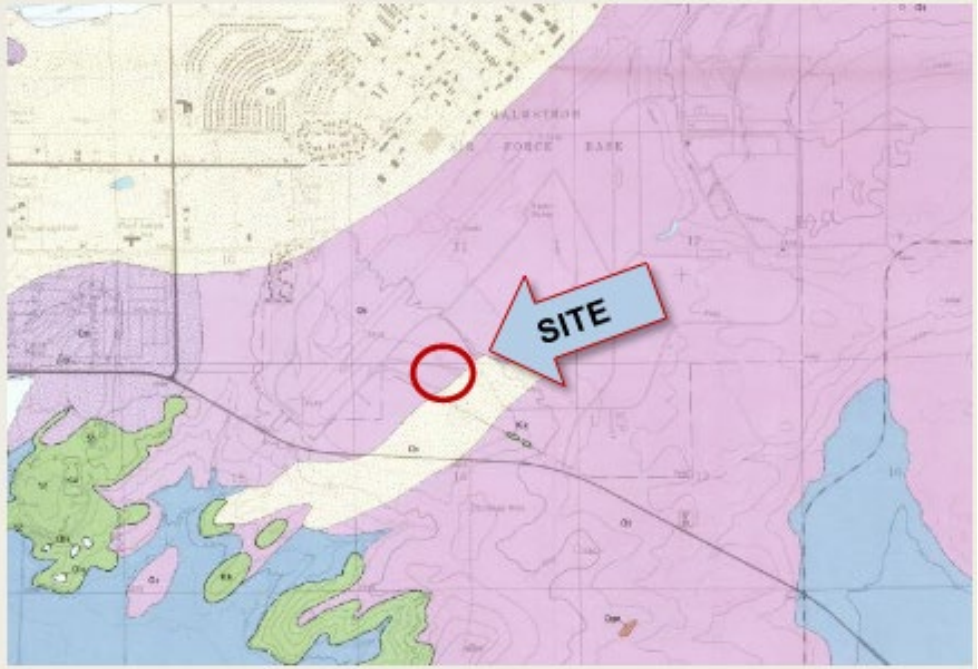
Site Geology and Soil Parameters

Layer No.	Depth	USCS Classification	Parameter	Recommended Value
1	0.0 to 73.0	CL, CH	Effective Unit Weight, γ	125 pcf
			Undrained Shear Strength, Su	1,875 psf
			K-value	1000 pci (static) 400 pci (cyclic)
			Strain, ε_{50}	0.005

Excerpts below from "Engineering Geology of the City of Great Falls ar Vicinity, Montana" by Richard W. Lemke and Edwin K. Maughan (1977)



Qs — DUNE SAND (HOLOCENE) — Mostly windblown fine sand, in places forming semiactive dunes 3–5 feet (1–2 m) high. Very loose, highly permeable, and highly subject to wind and water erosion. Generally less than 10 feet (3 m) thick; not mapped in most places where less than 2–3 feet (0.5–1 m) thick. Indefinite contact with Qls deposits in eastern part of map area

Qtl — TILL AND INTERCALATED GLACIAL LAKE DEPOSITS (PLEISTOCENE) — Intercalated layers of unsorted and unstratified clay, silt, and sand, resembling unit Qt in composition, and layers of glacial lake deposits of Lake Great Falls, resembling subunit Qlc. Commonly, individual layers are less than ¼ to 1 inch (0.6–2.5 cm) thick, but till layers may be considerably thinner or thicker. Plasticity, shrink-and-swell capacity, and strength generally lie between those of units Qlc and Qt (see table). Thicknesses generally range from less than 10 feet (3 m) along western margins of the unit to more than 50 feet (15 m) along eastern margins where the unit grades imperceptibly into Qt. The unit probably was formed along the terminal margin of the glacier ice front and represents ablation till, flow till, and ice-marginal lake deposits

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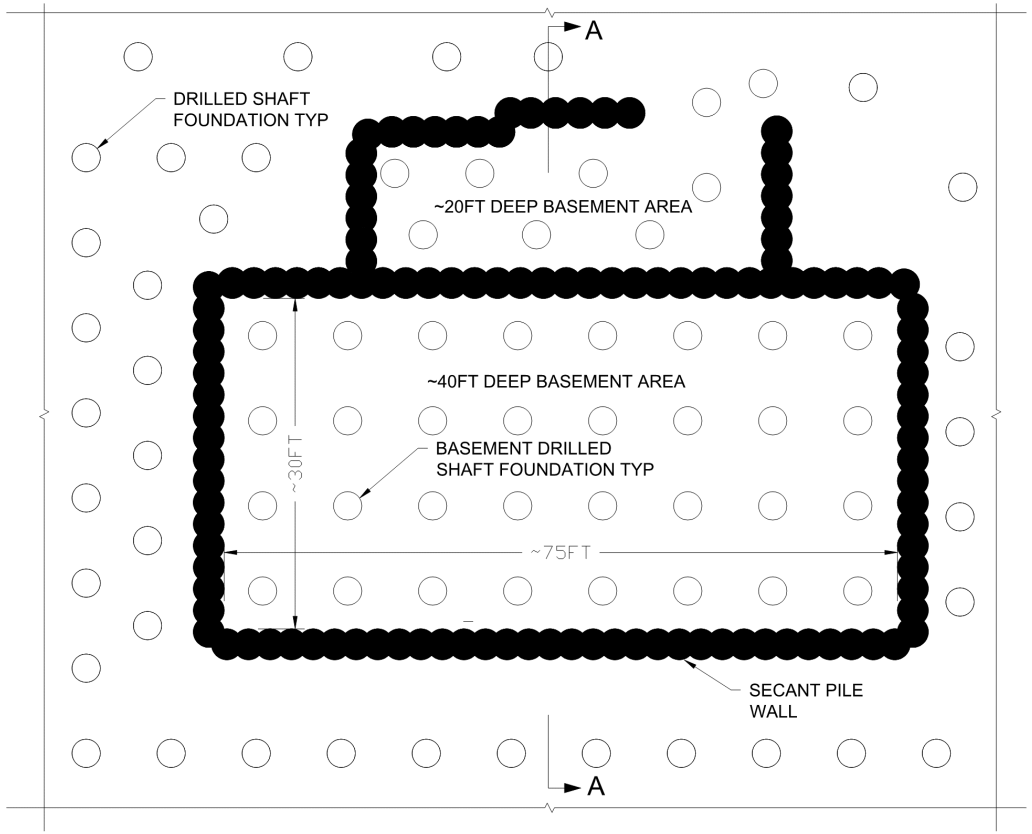
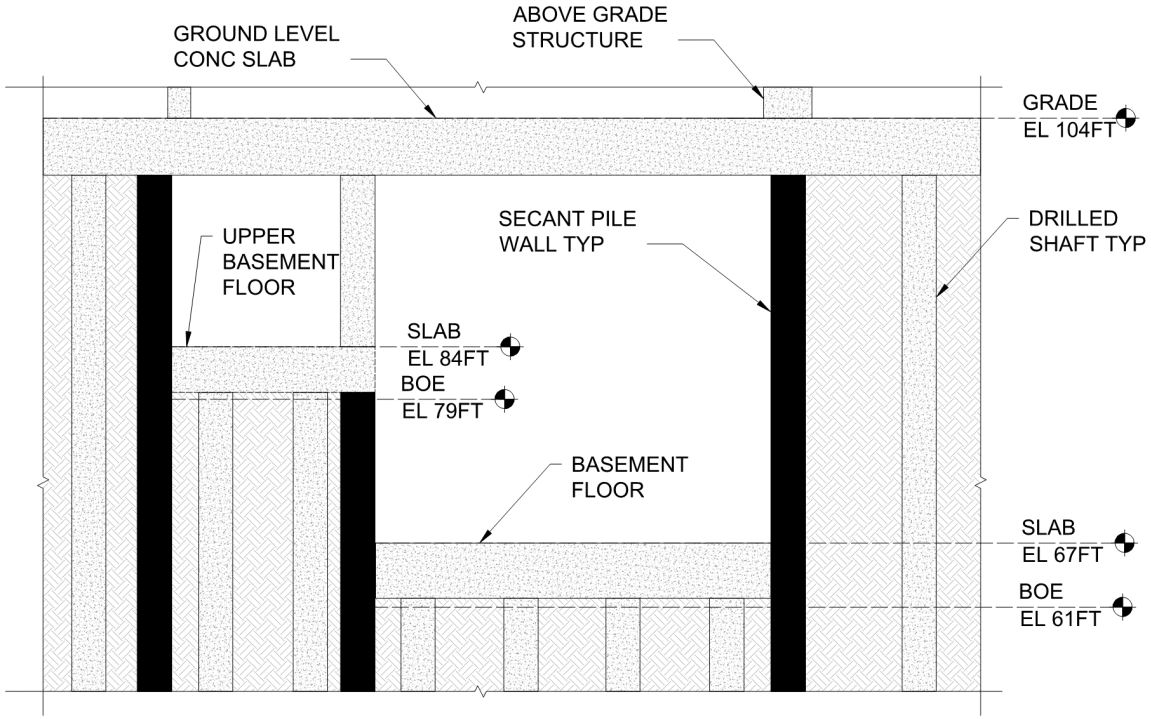
Basement Structure





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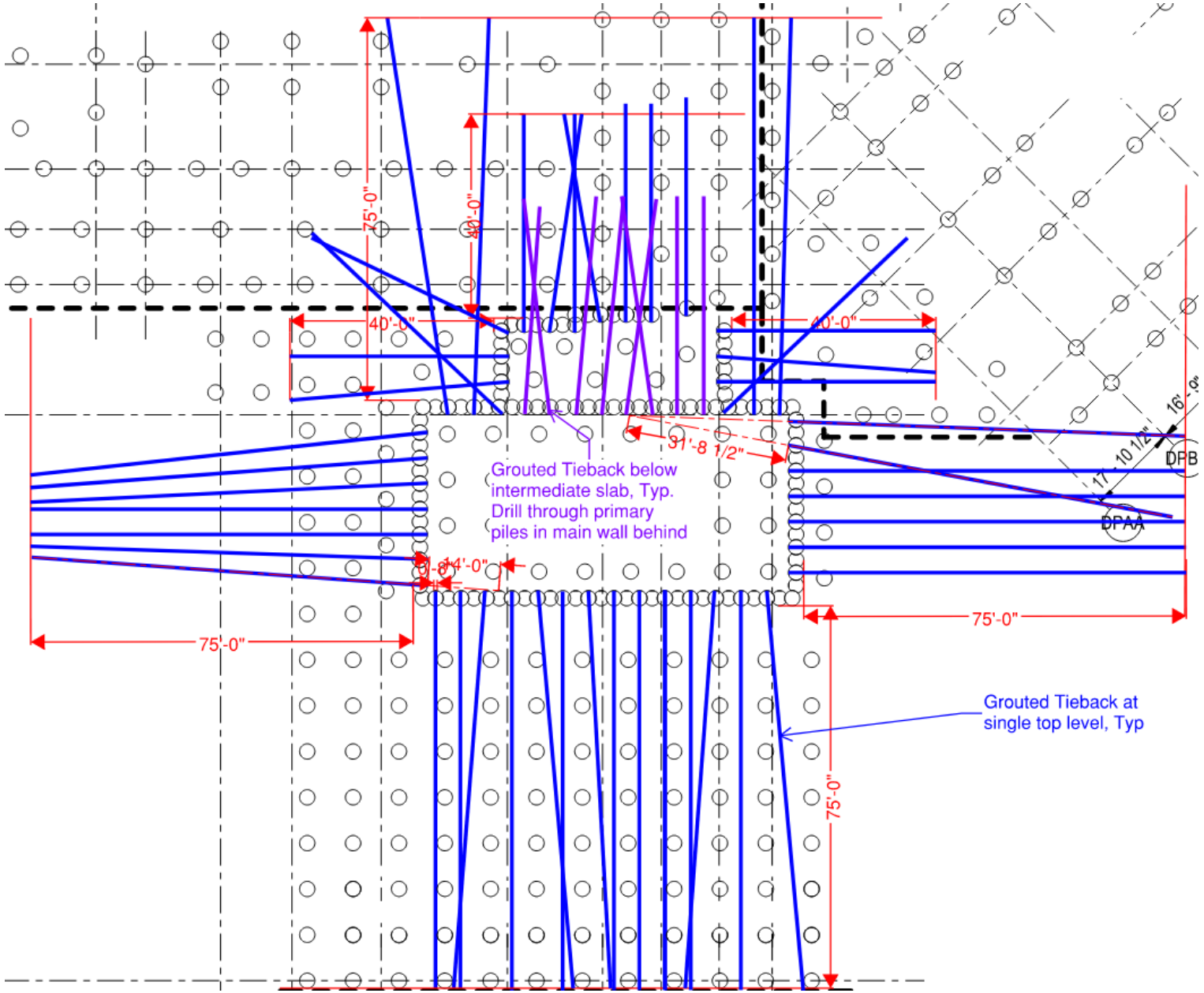
SOE Challenges and Constraints

- Congested site
- Dense surrounding drilled shafts
- Constraints on construction sequence
- Requirement for completion of interior work before top slab
- Low movement limits
  - ½” maximum

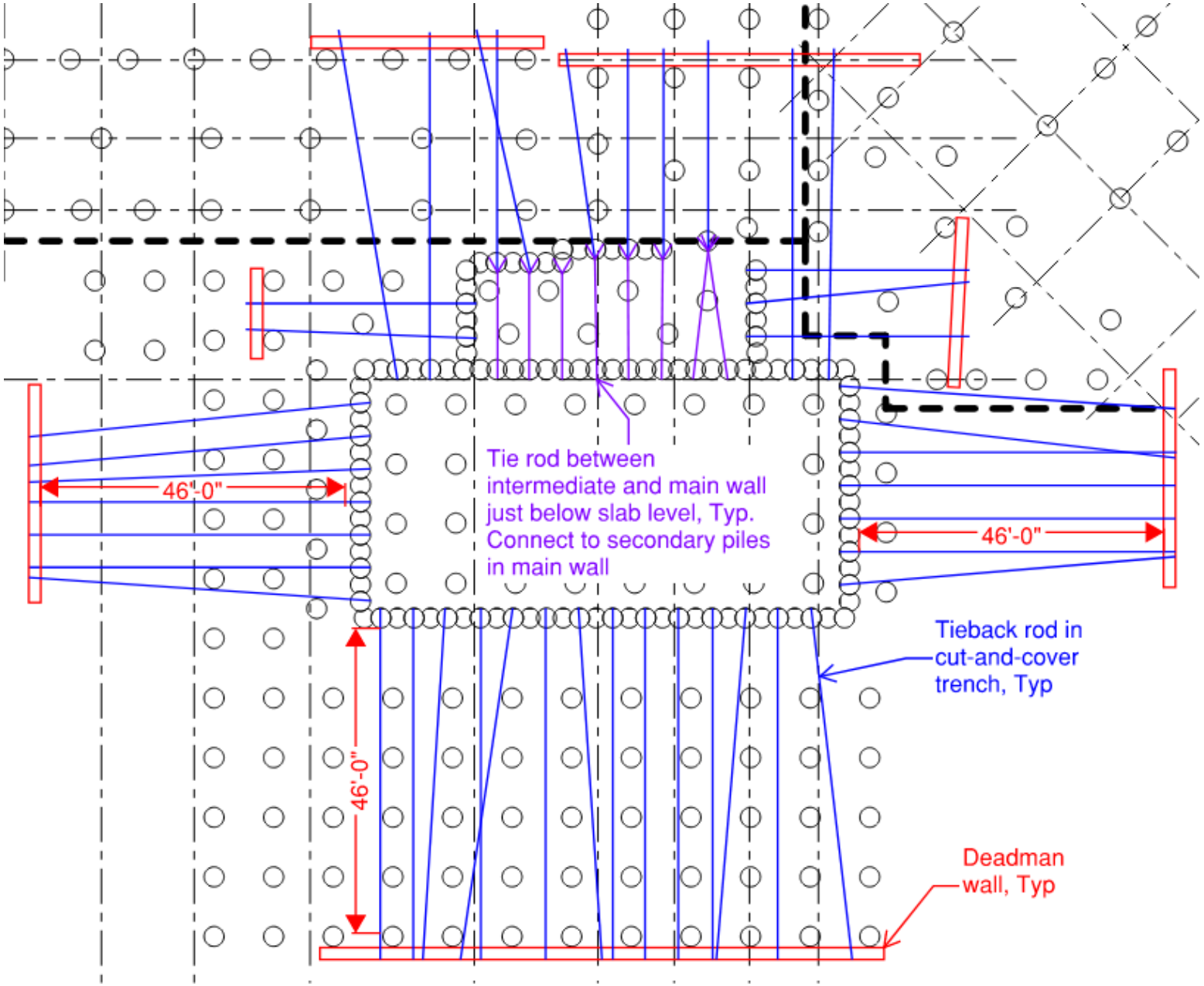
Options Considered

- Grouted Tiebacks
- Tie-Rods and Deadman Anchors
- External Wale and Deadman Anchor Shafts
- Internal Temporary Wale Frames
  - Below Top Slab
  - Above Top Slab
- Top-Down Construction

Grouted Tiebacks



Tie-Rods and Deadman Anchors



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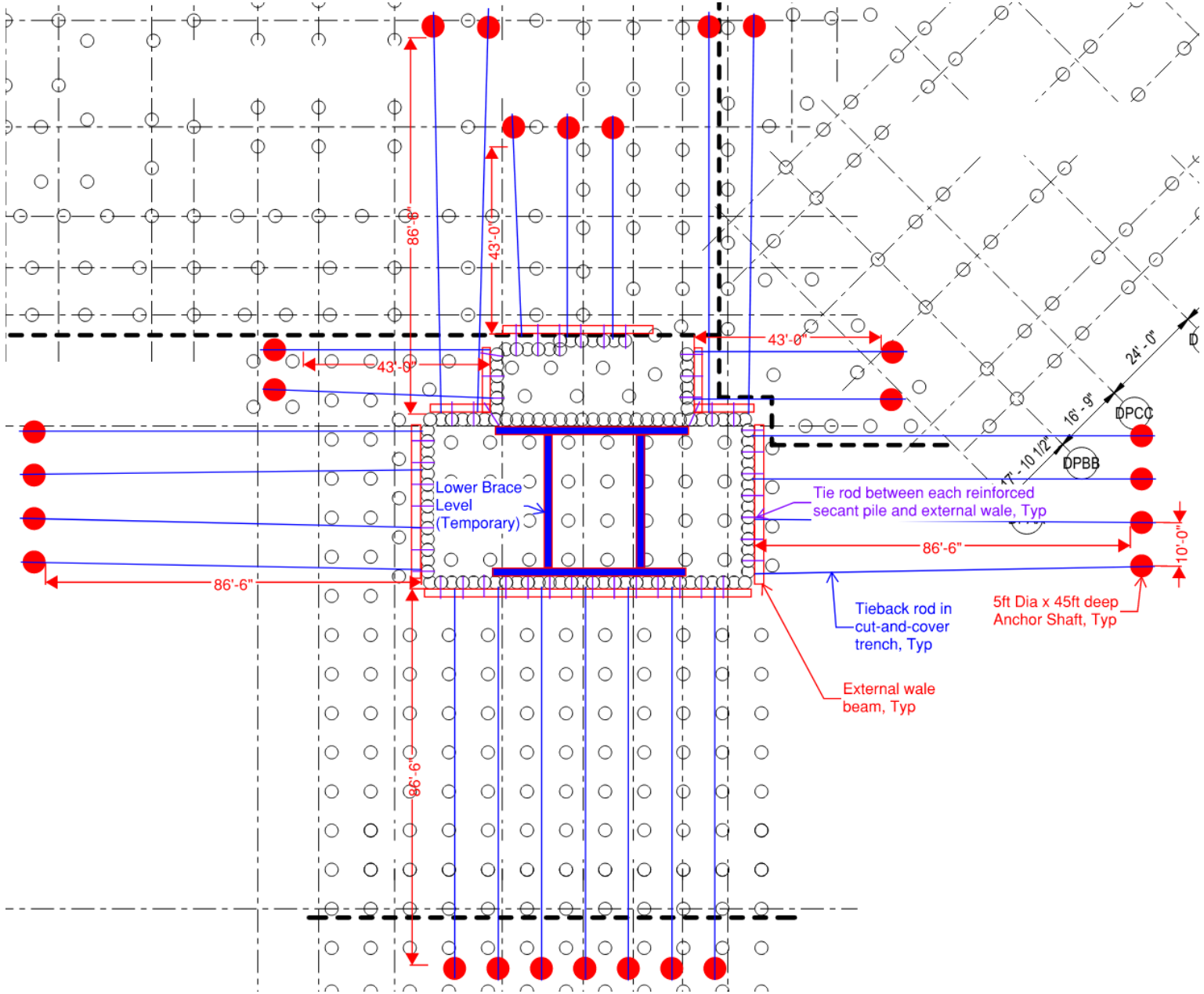
External Wale and Deadman Anchor Shafts



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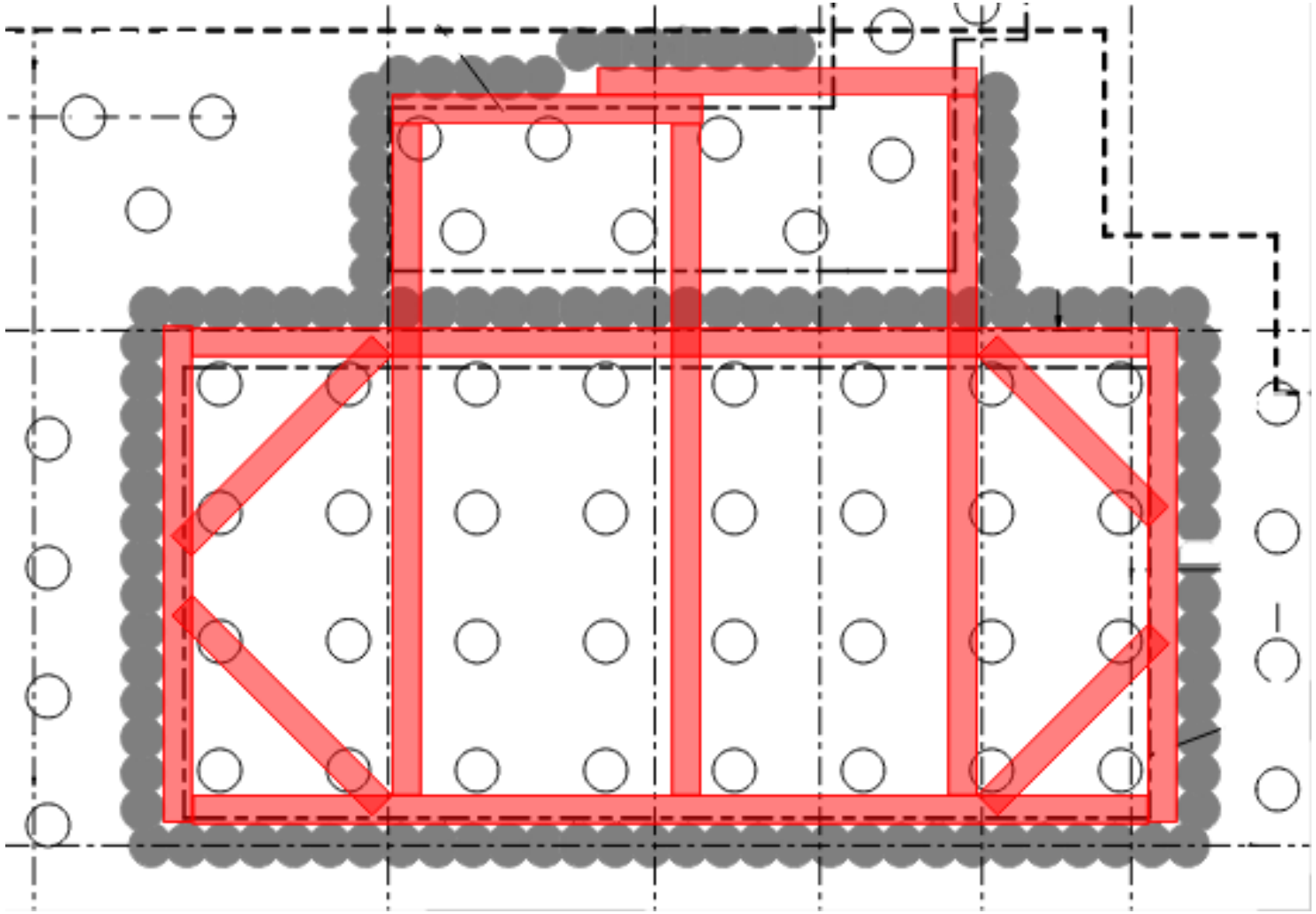
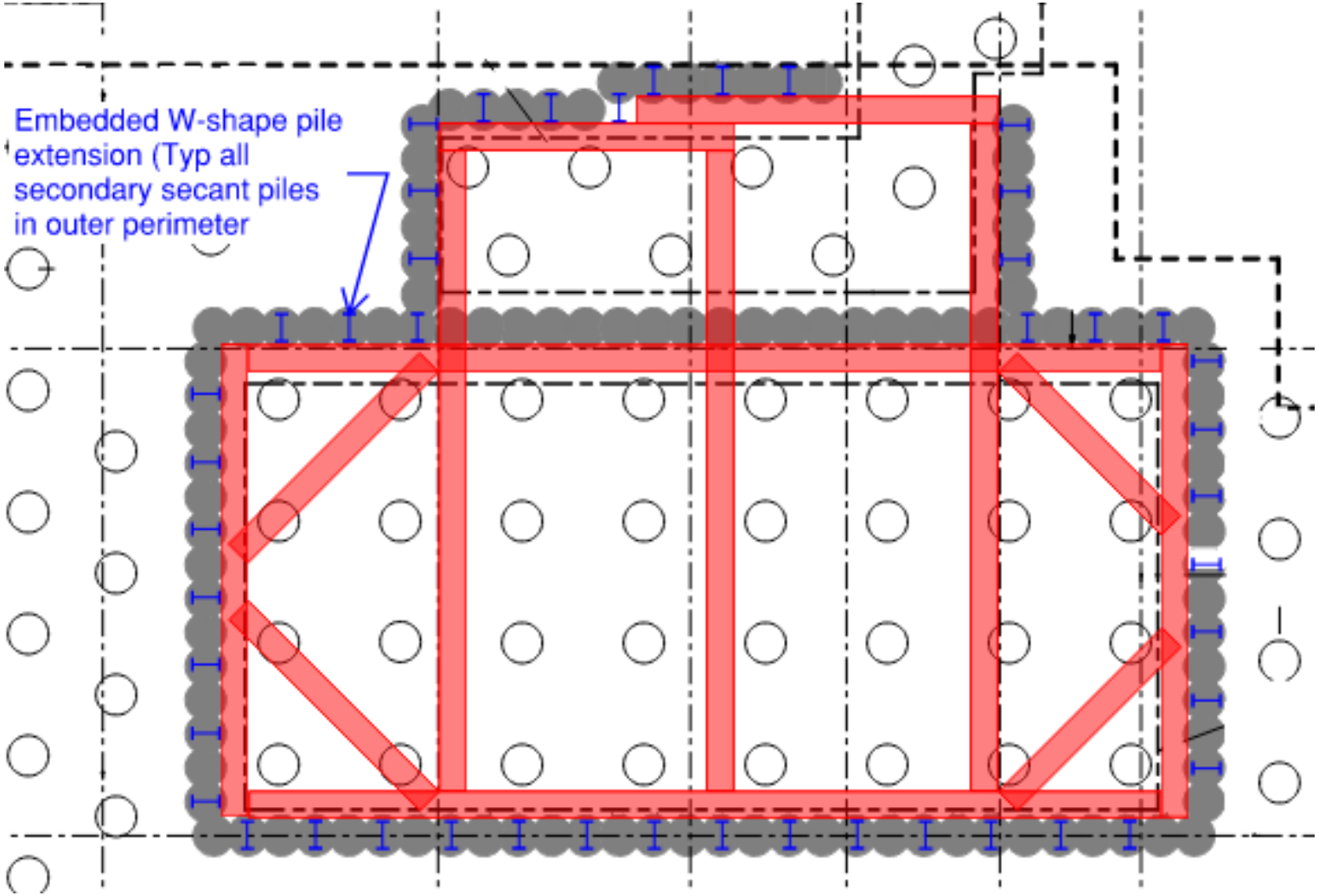
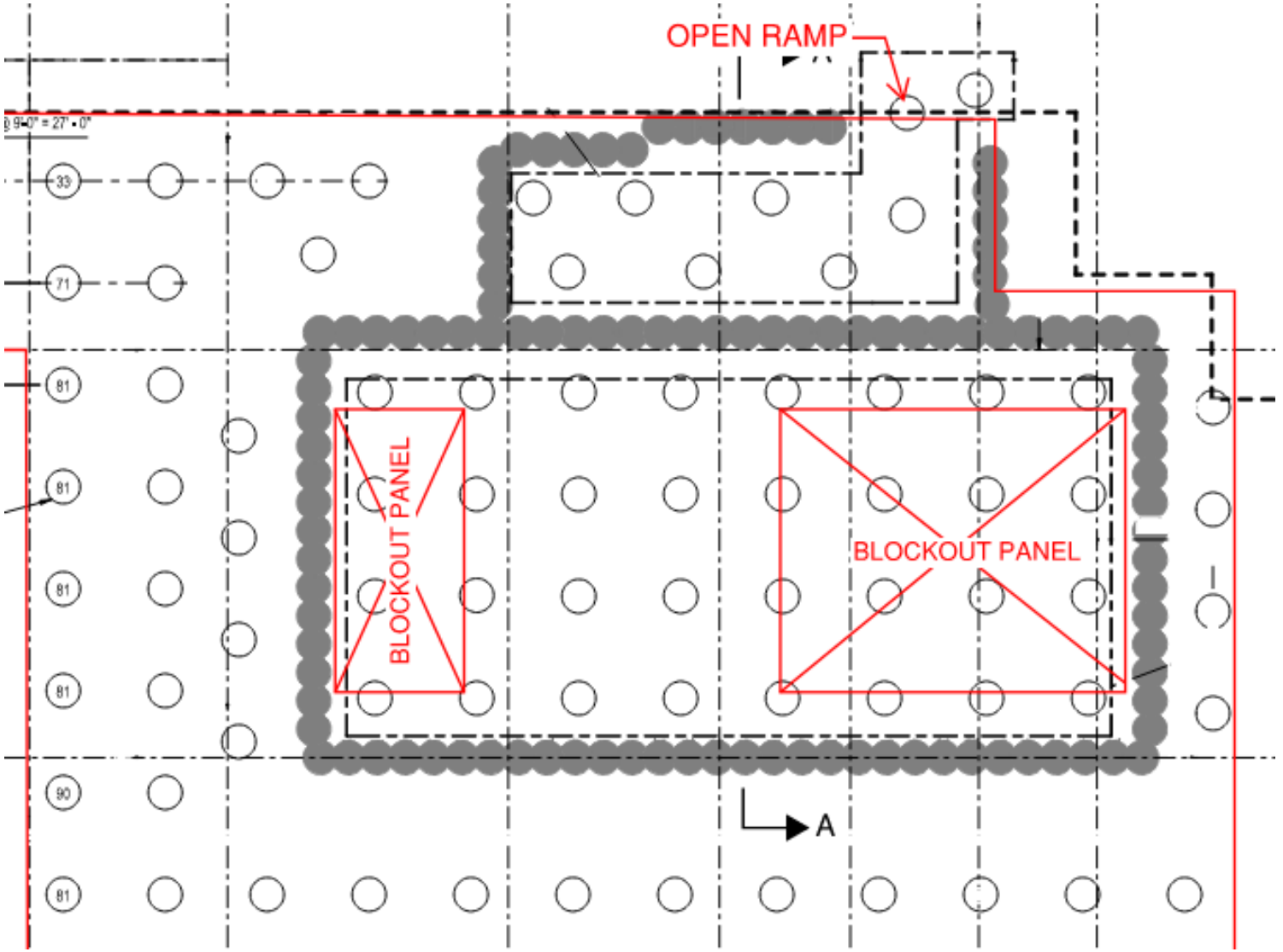
Internal Temporary Wale Frames – Below Top Slab



Internal Temporary Wale Frames – Above Top Slab

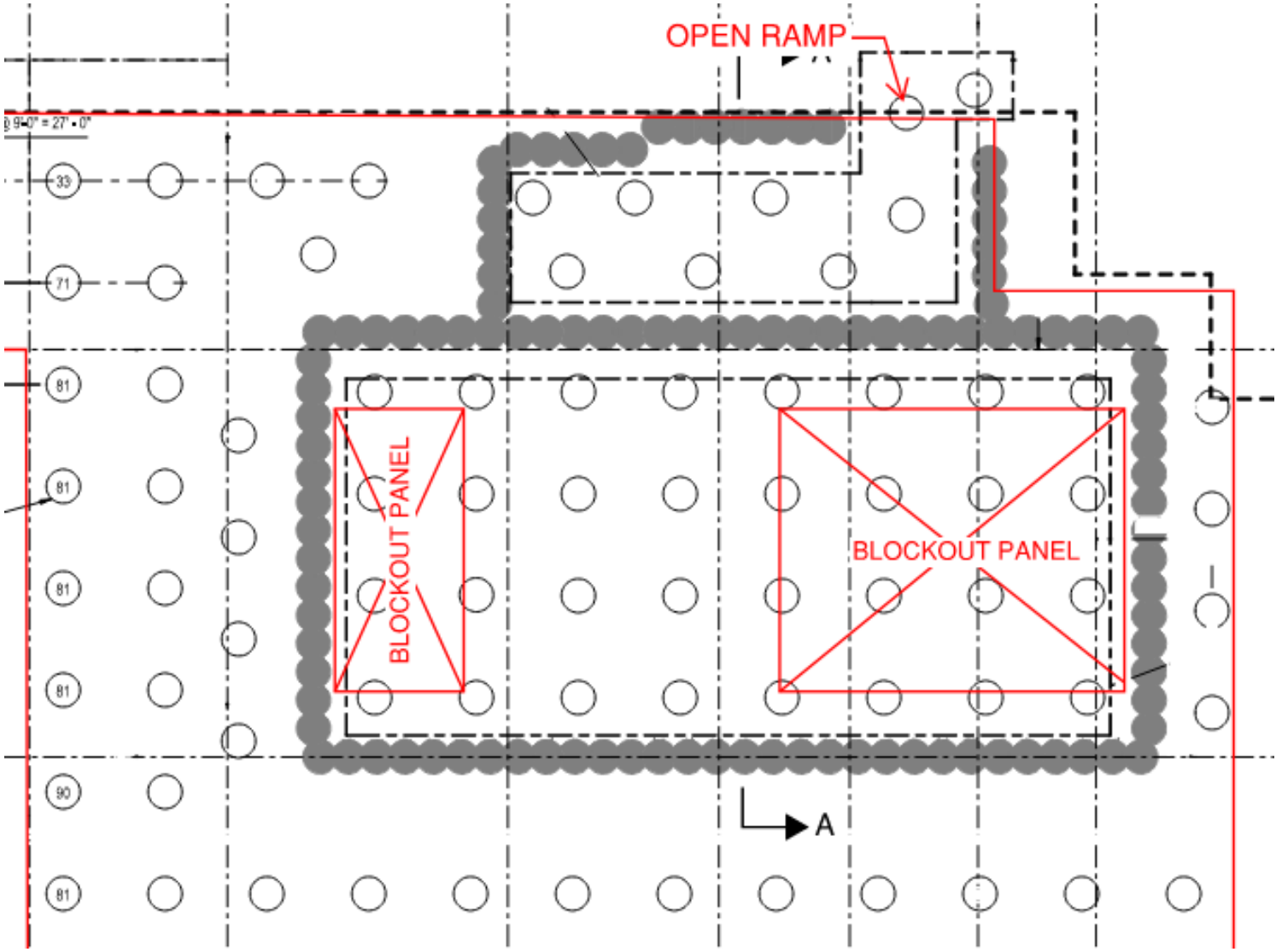


Top-Down Construction

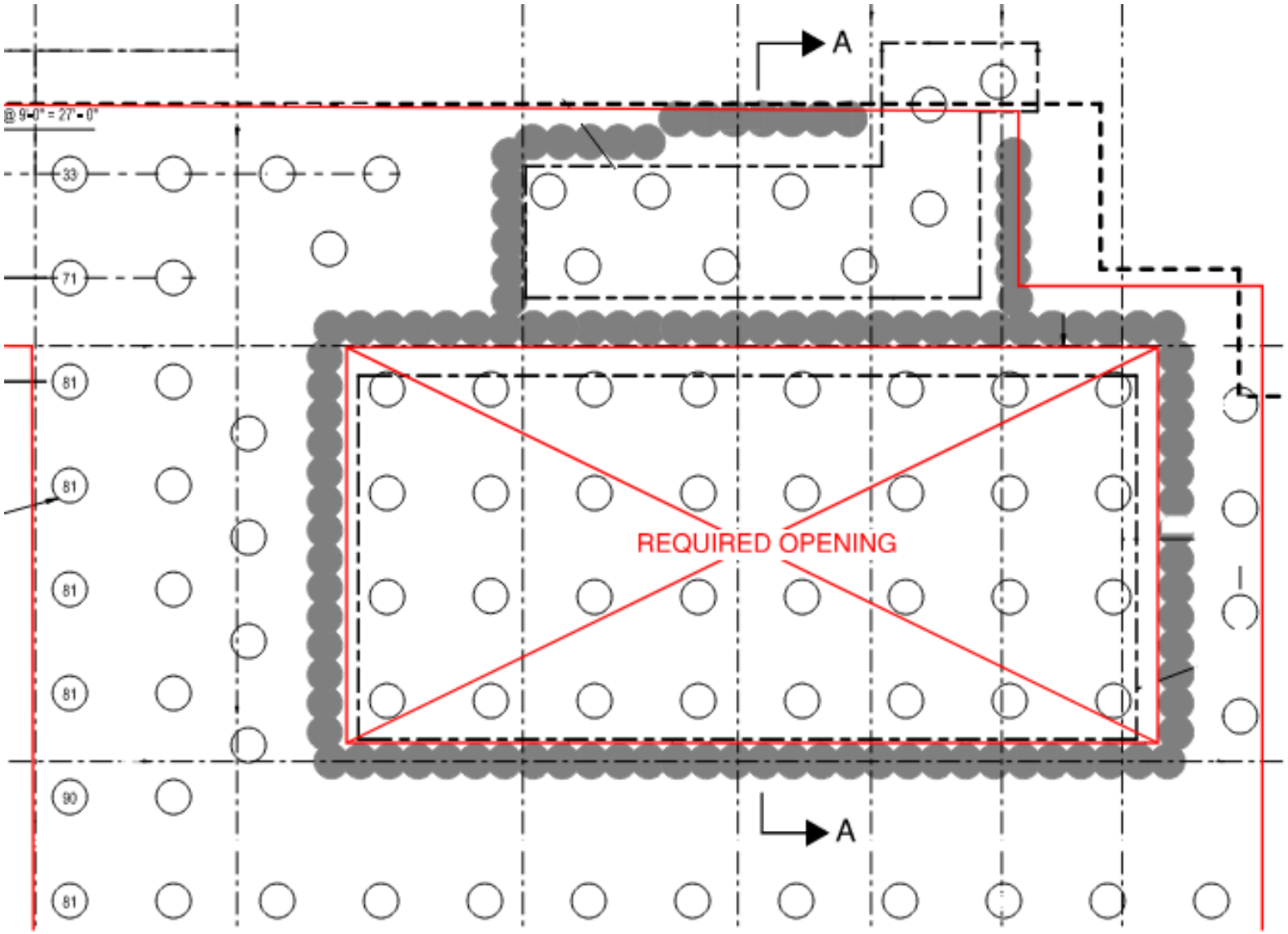


And the winner is…

Top-Down Construction!!



Not So Fast…

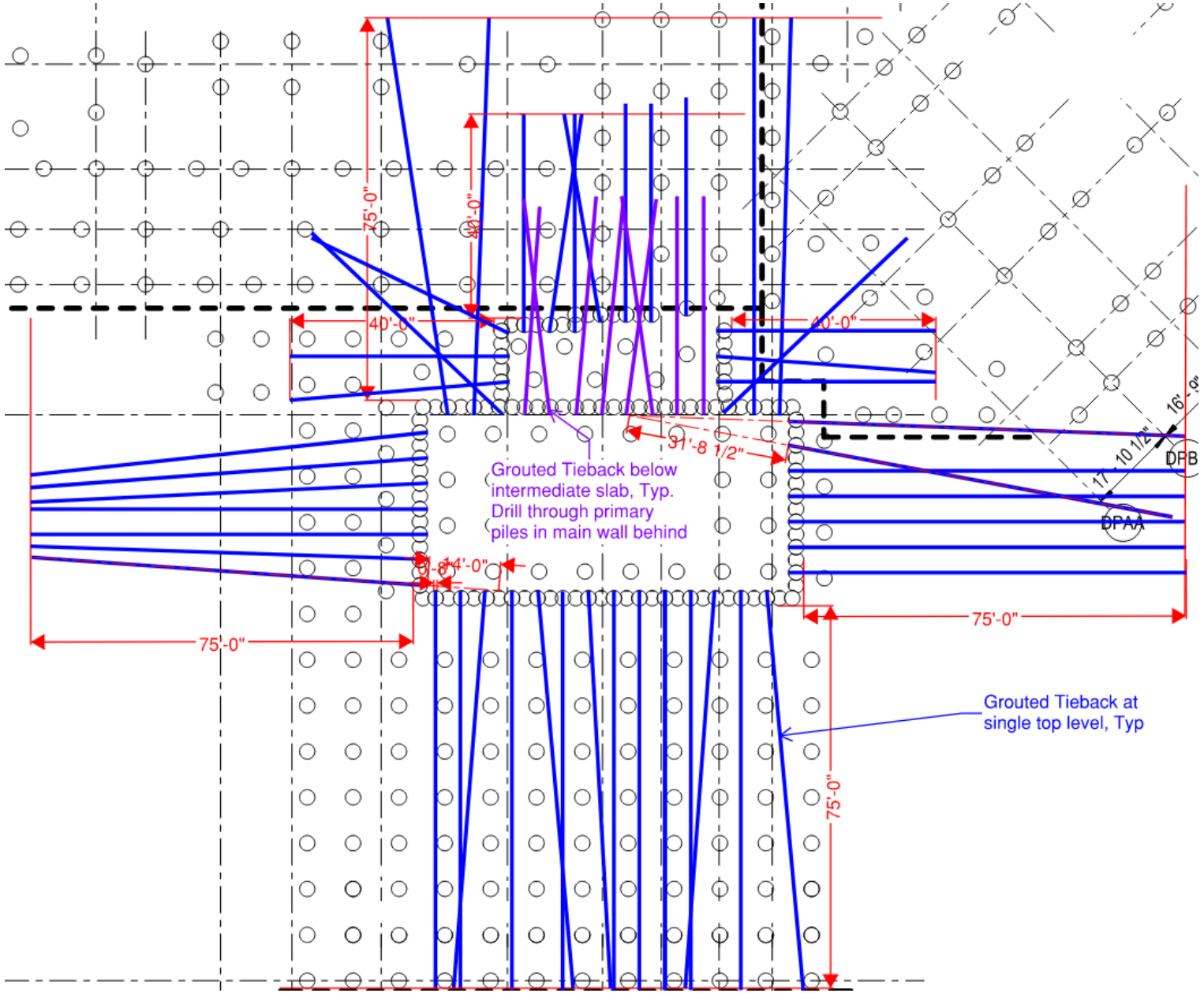


Options Considered

- Grouted Tiebacks
- Tie-Rods and Deadman Anchors
- External Wale and Deadman Anchor Shafts
- Internal Temporary Wale Frames
  - Below Top Slab
  - Above Top Slab
- ~~Top-Down Construction~~

Grouted Tiebacks

Impractical Geometry



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Options Considered

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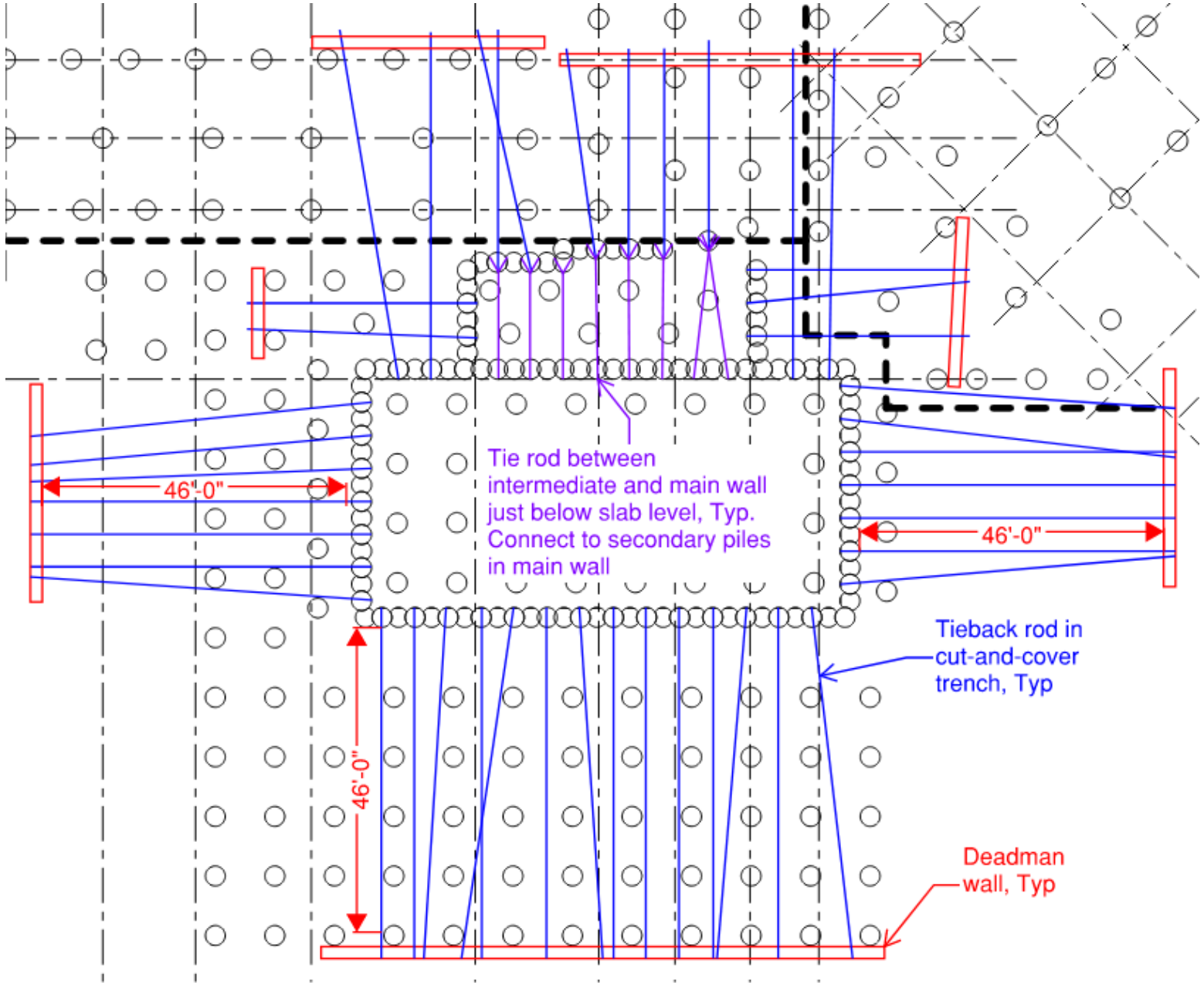
- ~~Top-Down Construction~~

Tie-Rods and Deadman Anchors

Better, but still Impractical



Options Considered

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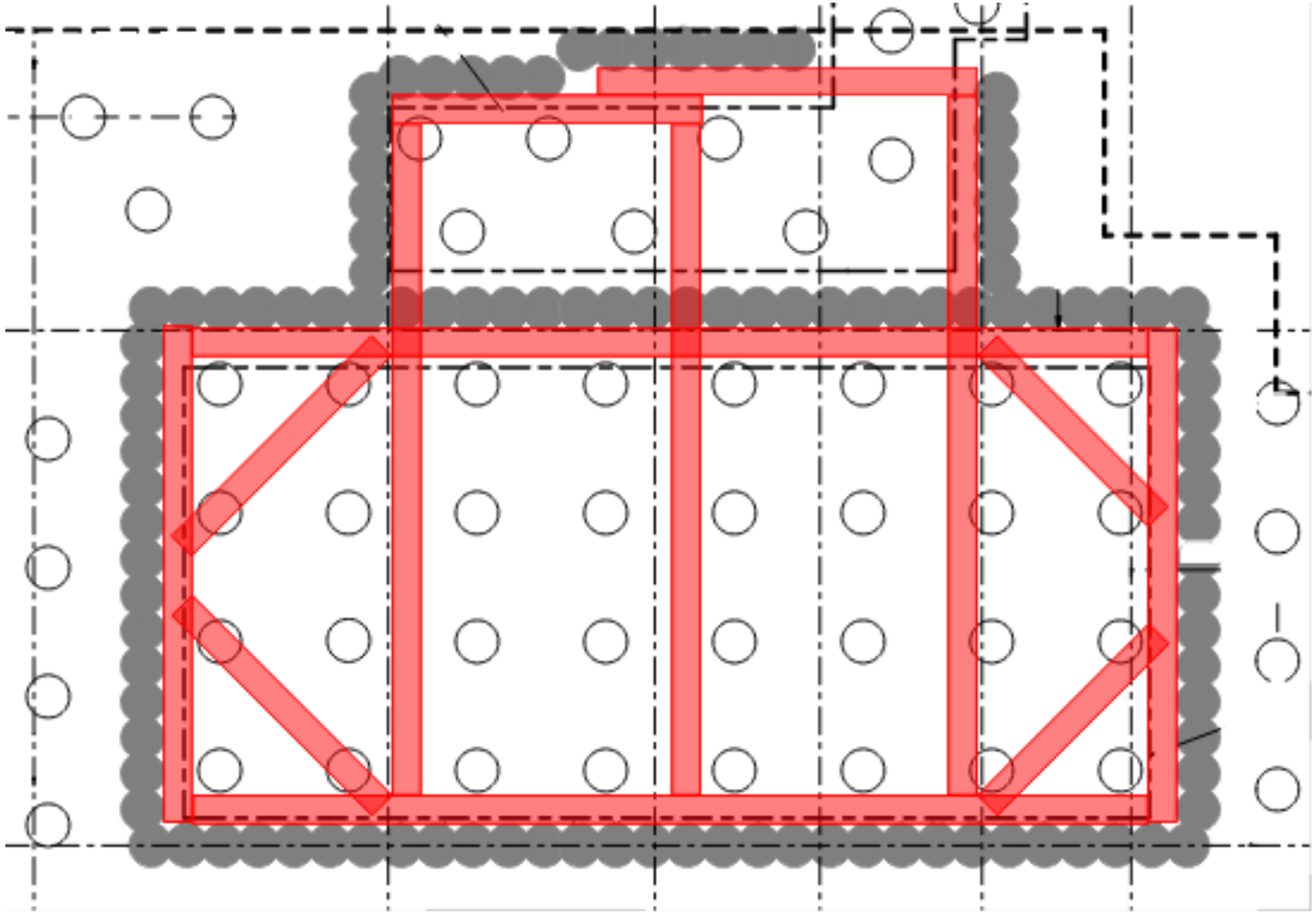
- External Wale and Deadman Anchor Shafts
- Internal Temporary Wale Frames
  - Below Top Slab
  - Above Top Slab
- ~~Top-Down Construction~~

Internal Temporary Wale Frames

- Below Top Slab
- Above Top Slab

Disqualified: Obstructed Opening



Options Considered

- ~~Grouted Tiebacks~~
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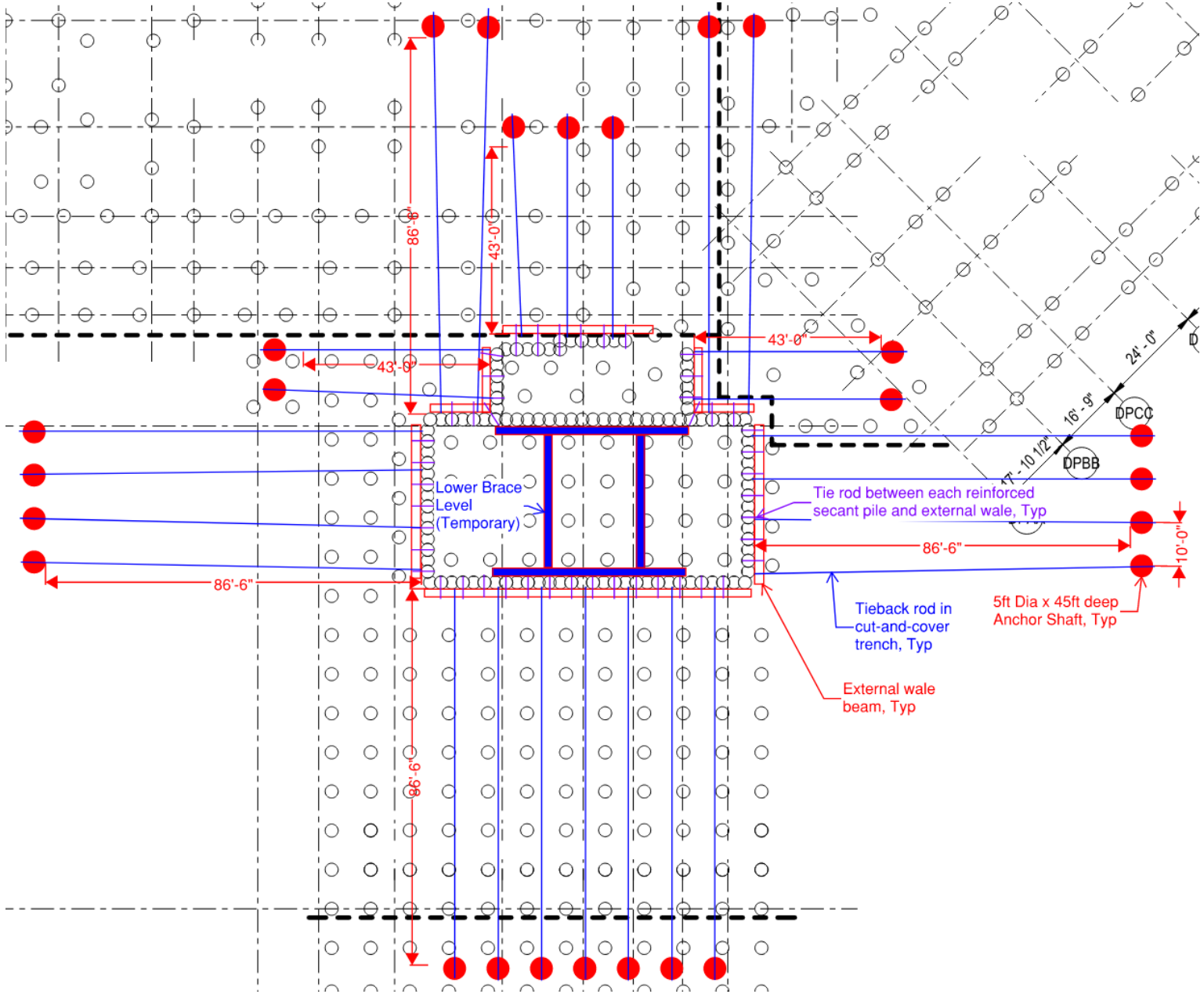
- External Wale and Deadman Anchor Shafts
- ~~Internal Temporary Wale Frames~~
  - ~~Below Top Slab~~
  - ~~Above Top Slab~~
- ~~Top-Down Construction~~

External Wale and Deadman Anchor Shafts

Feasible!

But maybe some improvements left…



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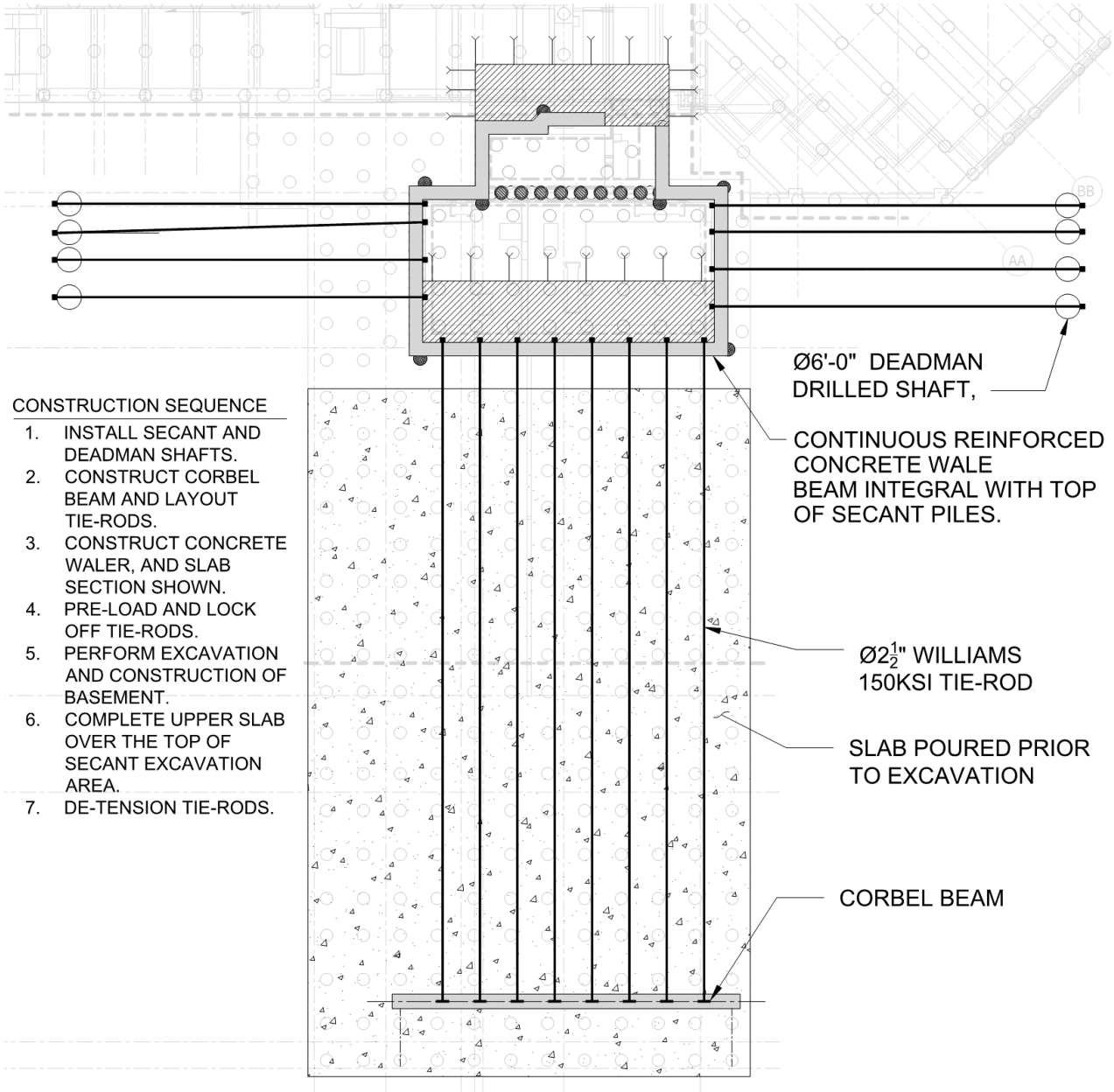
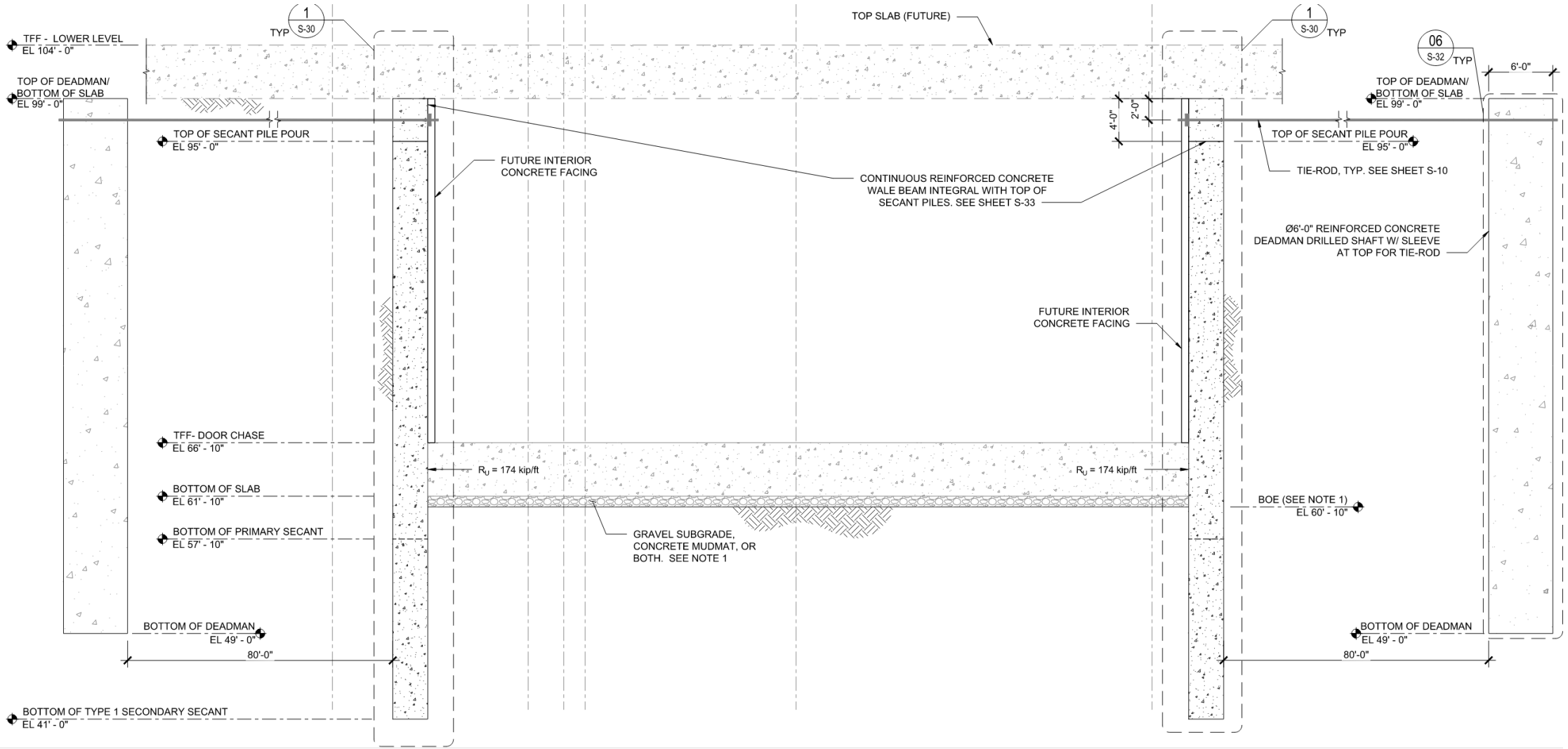
Selected Design Alternative

- Deadman anchor piles on East and West sides
- Utilize permanent mat slab and drilled shaft lateral resistance on South side
- Shallow basement piles designed as cantilever
- Integrated concrete wale



East-West Cross-Section



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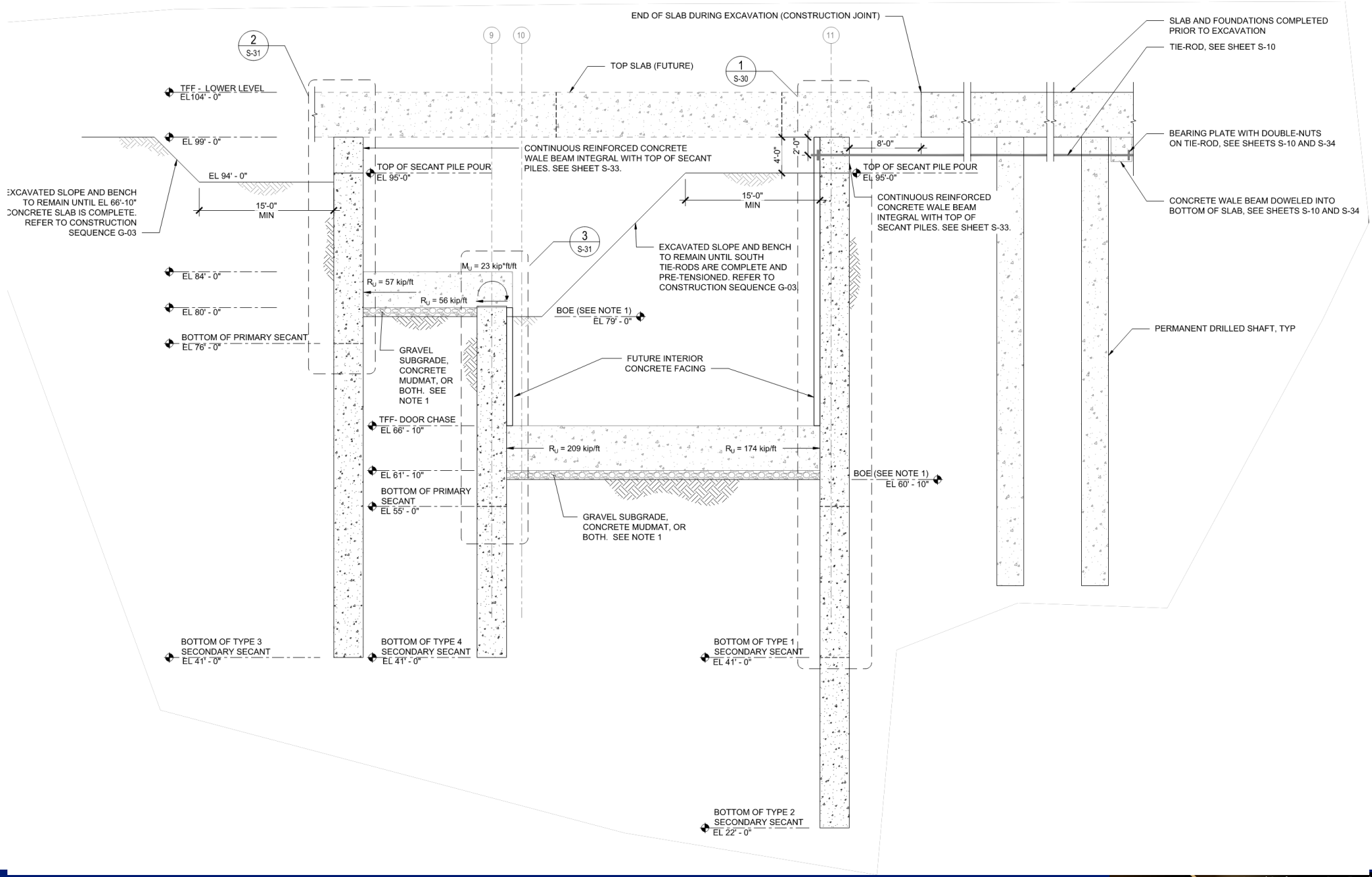
North-South Cross-Section



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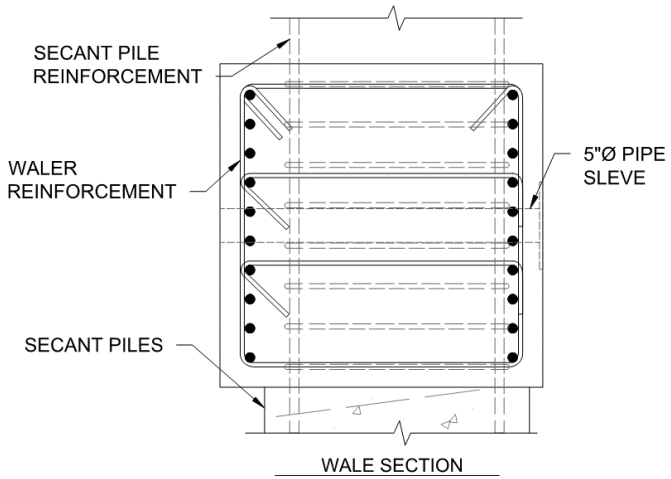
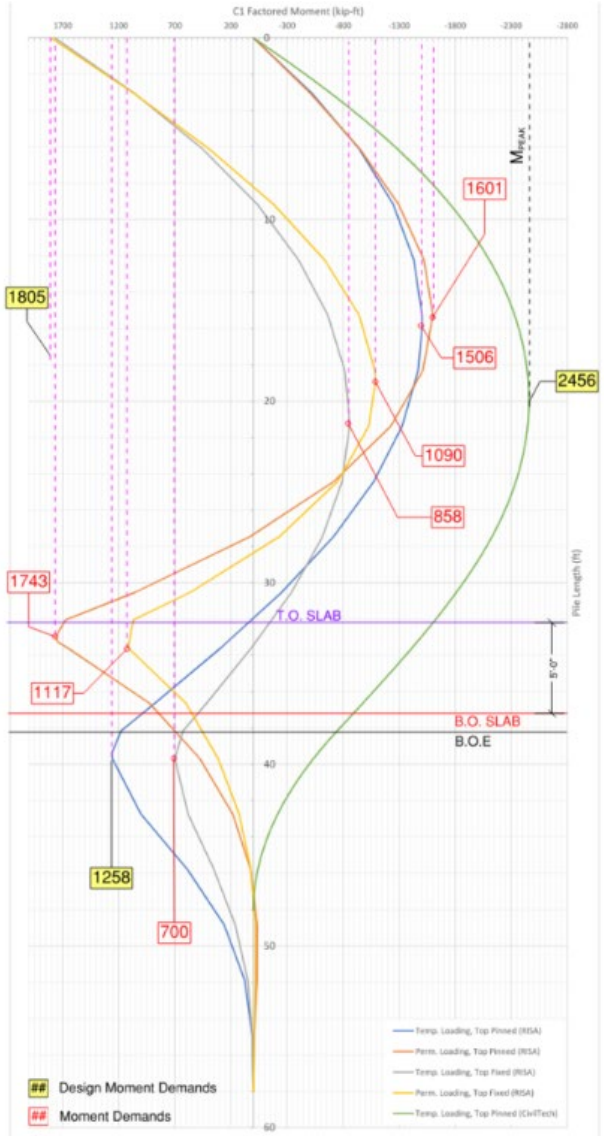
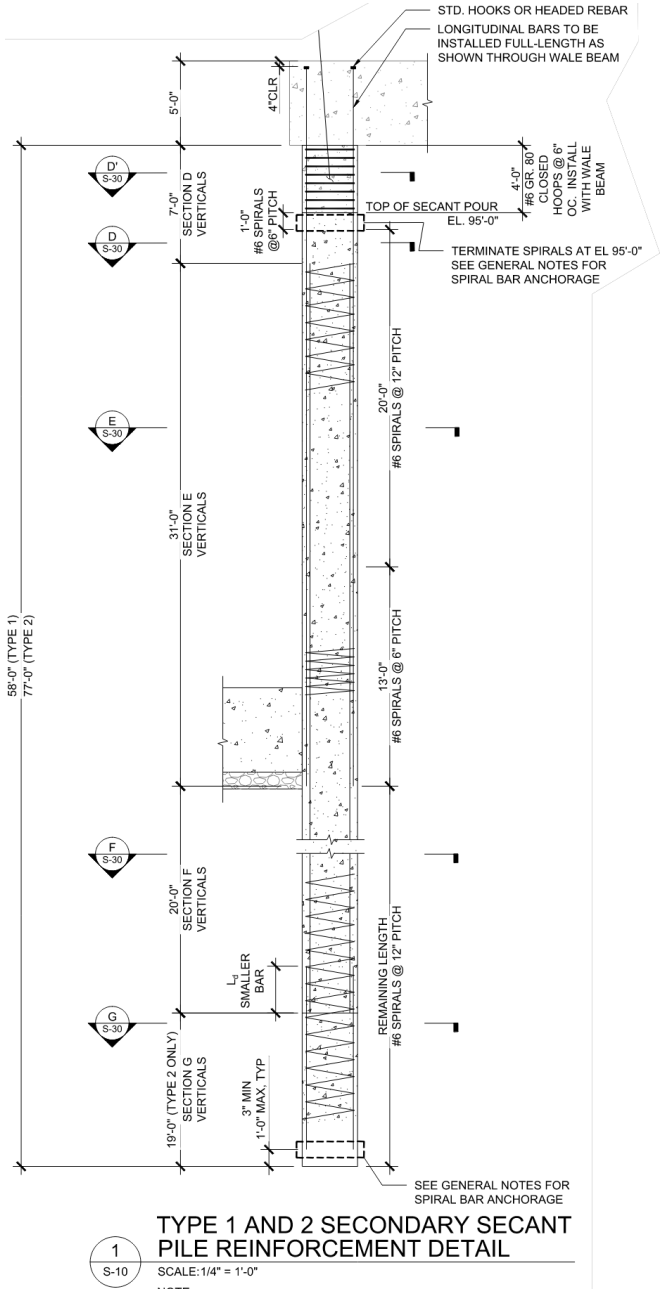
Secant Pile Design

- Permanent and temporary loads and boundary conditions
- Integrated wale







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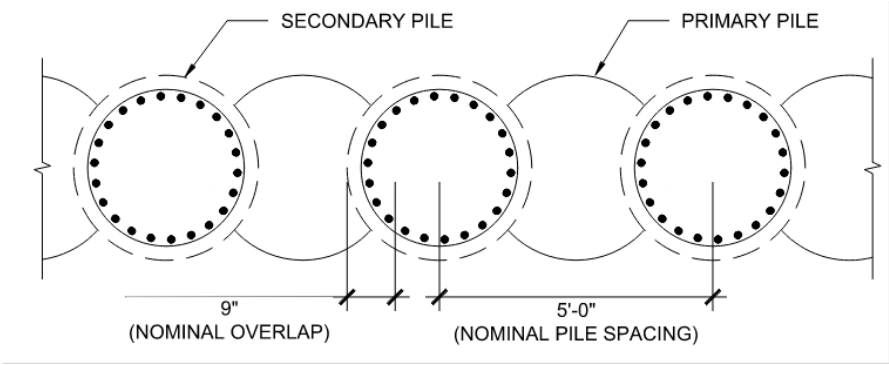
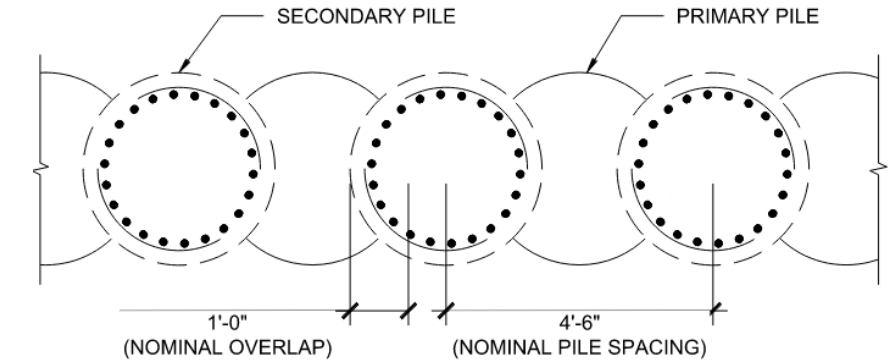
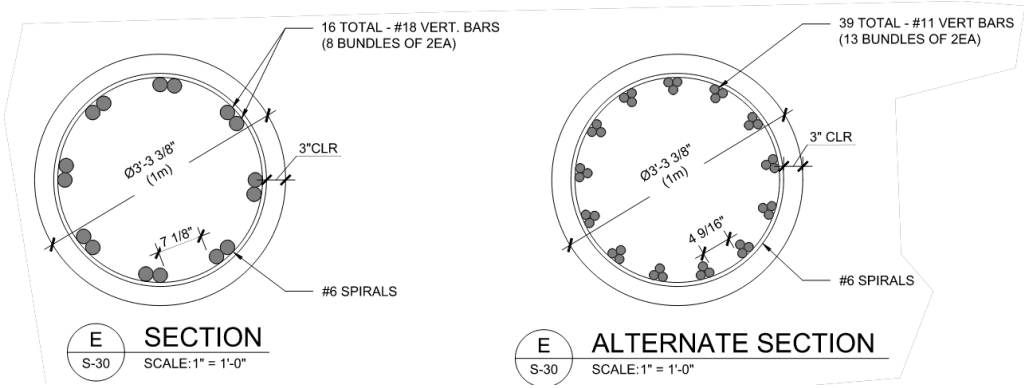
Secant Pile Details

- 3ft original secant diameter
  - Later upsized to 1m
- Tolerances
  - 1” plan location
  - 1% max out-of-plumb
- Pile Spacing
  - Shallow basement zone: 5’-0”
    - 9” nominal overlap
  - Deep basement zone: 4’-6”
    - 12” nominal overlap
- Multiple rebar options
  - Uncertain material availability







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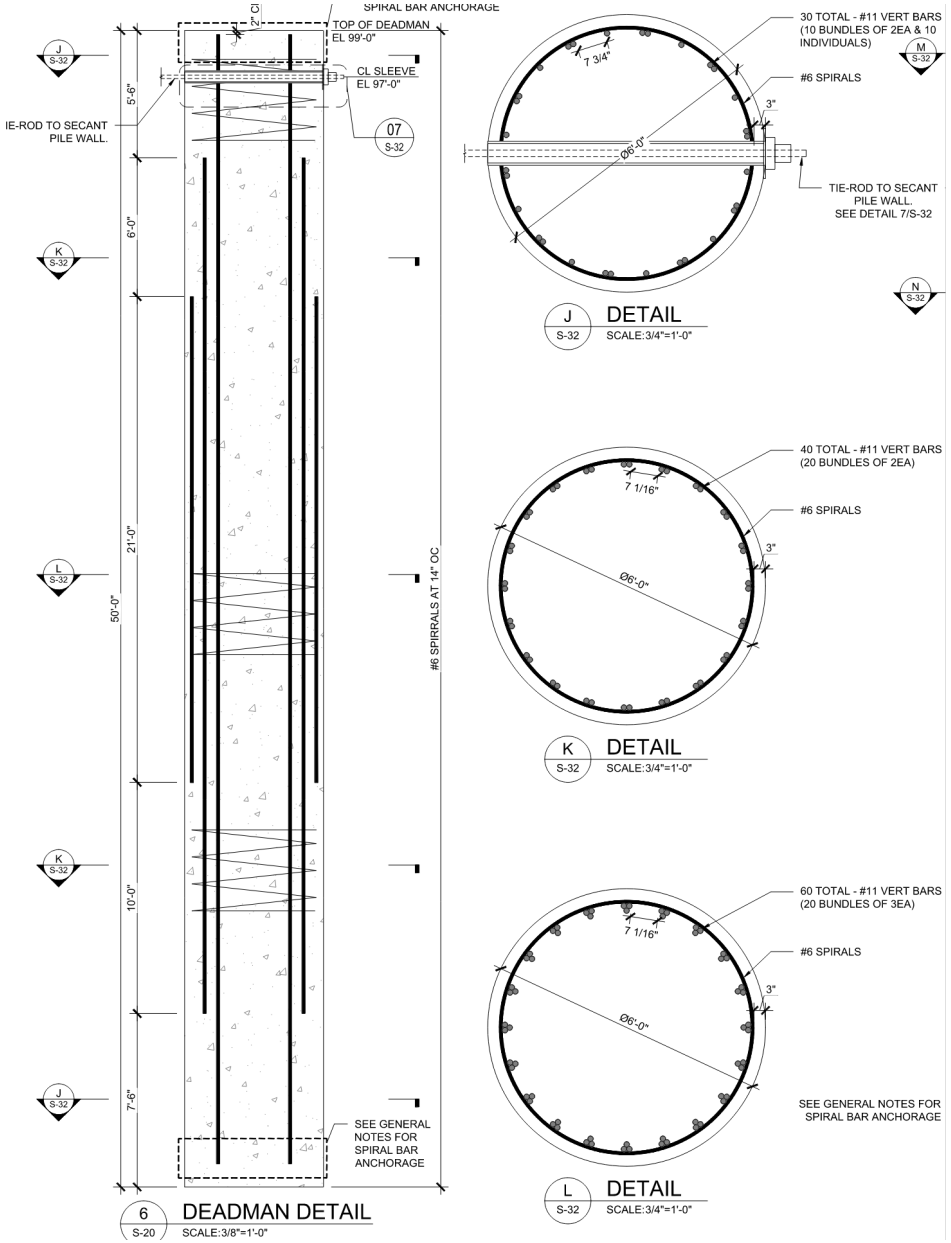
Deadman Pile Details

- 6ft diameter
- 50ft length
- Reinforcement varied for 3 cross-sections
  - Economize to flexural demand
- Integrated φ5” steel sleeve and bearing plate for tie-rods



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Construction

- Guide wall constructed to aid in alignment and ensure proper overlap.



Construction

- 900mm, 1M, and 1.2M diameter secants were explored during the design phase.
- 1M diameter secants were selected.



Construction

- Uniform stiff soils allowed for open hole drilling of the primary shafts.
- Secondary shafts were drilled using 1M sectional casing.



Construction

- Working within the swing radius of two tower cranes.
- Through planning and constant communication key to the success.



Construction

- With secant complete, top of wall was 4ft below working grade.
- 4ft was dug out for construction of waler.



Construction

- Tie-rods laid in excavated trenches while waler reinforcement was being placed.



Construction

- Trumpets installed prior to concrete form work.



Construction

- Deadman shafts installed using open hole drilling methods.
- PVC pipe was placed in the reinforcement cage to bock out for placement of the trumpet.



Construction

- Waler Concrete placed.
- Tie-rods ready to be extended through waler after form removal.



Construction

- Upper basement area did not require the tie-rods
- This section was excavated as soon as concrete reached strength.
  - Bench left to support south wall before tie-rod tensioning



Construction

- The upper basement area of secants excavated to final depth.



Construction

- Locking off tie-rods.
- Locked off at 250kips.
  - 400kip max design load



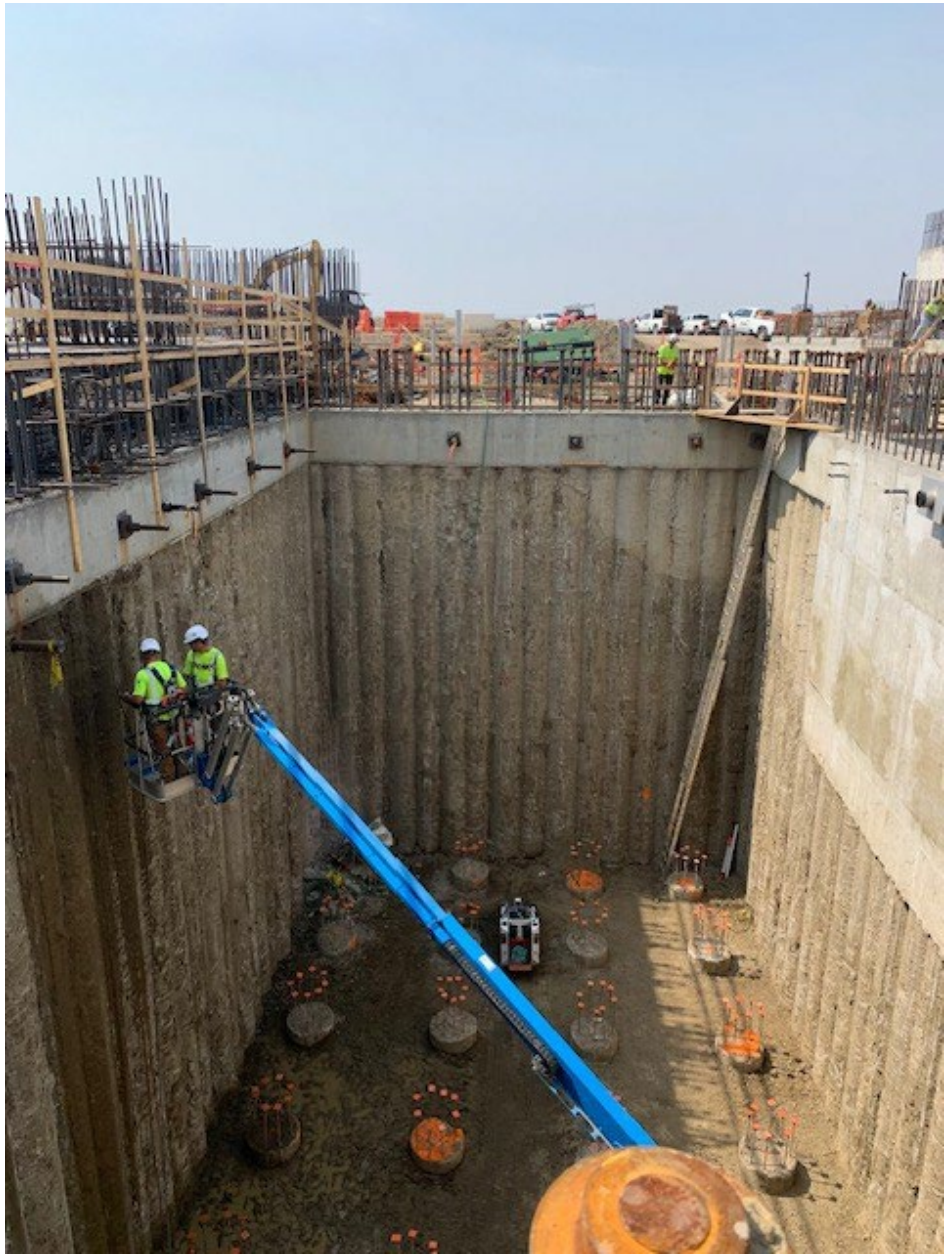
Construction

- Excavation to upper basement elevation.
- Construction of interior wall.



Construction

- Excavation to final elevation.



Construction

- Overhead look at completed secant walls.
- Tie-rods were de-tensioned once the slab was complete over the opening.



Conclusions

- Challenges overcome with innovative thinking
  - Considered many options
  - Best solution incorporated ideas from multiple options
- Best results when all stakeholders work together
- Unimpeded construction sequence and operations
- Minimal deflection and ground movements

Questions?



BRIERLEY ASSOCIATES
Creating Space Underground

CONDON · JOHNSON & ASSOCIATES, INC.
CONTRACTORS AND ENGINEERS

PARALLEL SESSION B4
Developing a Golden Rule

Live Audience Poll



Foundation Reuse Guide Specification Discussion

DFI Foundation Reuse Committee

DFI 50

October 22, 2025

The opinions expressed in this presentation and on the following slides are solely those of the presenter(s) and not necessarily those of DFI. DFI does not guarantee the accuracy or reliability of the information nor endorse the recommendations offered by the presenter(s).

Live Audience Poll

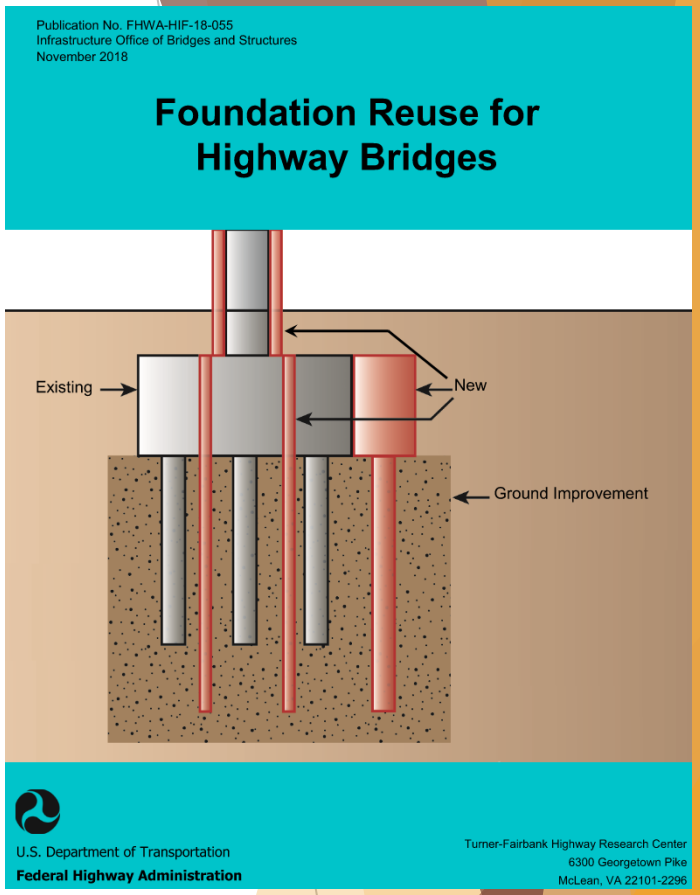




Nathan T. Davis, PhD, P.E.

McPhail Associates, LLC

PhD, 2018, Civil Engineering, Tufts University
B.S., 2010, Civil Engineering, University of Massachusetts Amherst



Nathan was a contributing author for FHWA-HIF-055 **Foundation Reuse for Highway Bridges.**

Nathan's professional experience includes design of reuse and repair of existing foundations including timber pile supported structures and adaptive reuse / retrofit of existing foundation walls for temporary earth support.

Nathan also has extensive experience designing geostructural systems such as underpinning, temporary and permanent earth support systems, and deep foundations.









Journal Publications

Davis, N.T., Hoomaan, E., Agrawal, A.K., Sanayei, M., Jalinoos, F. (2018) **Integrated Superstructure-Substructure Load Rating for Bridges with Foundation Movements**, Journal of Bridge Engineering, ASCE

Davis, N.T., Sanayei, M., Agrawal, A.K. Jalinoos, F. (2018) **Determining the Capacity of Reused Bridge Foundations from Limited Information**, Journal of Bridge Engineering, ASCE

Davis, N.T., Hoomaan, E., Agrawal, A.K., Sanayei, M., Jalinoos, F. (2019) **Foundation Reuse in Accelerated Bridge Construction**, Journal of Bridge Engineering , ASCE

Davis N.T., Sanayei, M. (2020) **Foundation Identification Using Dynamic Strain and Acceleration Measurements**, Engineering Structures, Elsevier

Hoomaan, E., Agrawal, A.K., Davis, N.T., Jalinoos, F. (2021) **Risks in Reuse of Highway Bridge Foundations**, Journal of Infrastructure Systems, ASCE

McPHAIL ASSOCIATES, LLC



Kenneth L. Fishman, Ph.D., P.E.
McMahon & Mann Consulting Engineering and Geology, P.C.

Ph.D., 1988, Civil Engineering, University of Arizona
M.S., 1983, Civil Engineering, University of Arizona
B.S., 1979, Civil Engineering, University of Maine at Orono



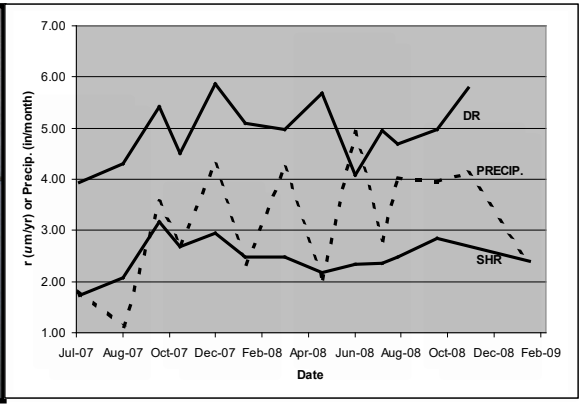
Condition Assessment of Rock Bolts and Ground Anchors

Dr. Fishman is a Principal at McMahon & Mann Consulting Engineering and Geology, P.C. where he has been a geotechnical engineering consultant since 1994. He serves as leader of the Earth Reinforcement Testing Division (ERTesting) to provide testing and field services related to service life design, condition assessment, corrosion monitoring, durability studies and corrosion mitigation or retrofit strategies for buried structures including foundation elements, ground anchors, earth reinforcements, tanks, pipelines and culverts.



ERTESTING





EC Testing & Temporal Variations

McMahon & Mann

1. NCHRP Report 958: **Electrochemical Test Methods to Evaluate the Corrosion Potential of Earthen Materials**, 2021.
2. NCHRP Report 675 : **LRFD Metal Loss and Service- Life Strength Reduction Factors for Metal Reinforced Systems in Geotechnical Applications**, 2011
3. FHWA-NHI-09-087: **Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes,** 2009.
4. FHWA-NH-RD-14282C, **Phase II: Condition Assessment and Evaluation of Rock Reinforcement Along I-93 Barron Mountain Rock Cut, Woodstock, NH,** 2005.
5. FHWA-NH-RD-13733L **Phase I: Condition Assessment and Evaluation of Rock Reinforcement Along I-93,** 2004.
6. FHWA-HRT-05-067: **Seismic Retrofitting Manual for Highway Structures: Part 2 – Retaining Structures, Slopes, Tunnels, Culverts and Roadways,** 2004.
7. NCHRP Report 477 : **Recommended Practice for Evaluation of Metal-Tensioned Systems in Geotechnical Applications**, 2002.
8. NCHRP Web Document 27: **Evaluation of Metal Tensioned Systems in Geotechnical Applications, Phase I**, 2001.

ERTESTING

McMahon & Mann

Joseph R. Nietfeld, PE



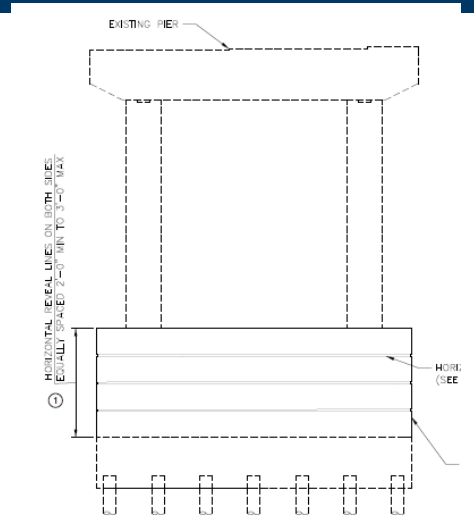
BS, MS University of Minnesota

Joe Nietfeld is a principal geotechnical engineer in the Minnesota DOT central office. Joe has over 20 years of geotechnical and structural engineering experience for a major geotechnical consultant and for MnDOT. Joe has been the engineer of record for several major pile, drilled shaft and embankment projects and served in bridge and Geotech oversight roles for major design build, CMGC and Light rail and Bus rapid transit projects. Joe is currently a member of the DFI Foundation Reuse Committee, TRB AKH19 Geomaterials Behavior Committee and a proxy member of the AASHTO Soil Structures Subcommittee.

DEPARTMENT OF TRANSPORTATION

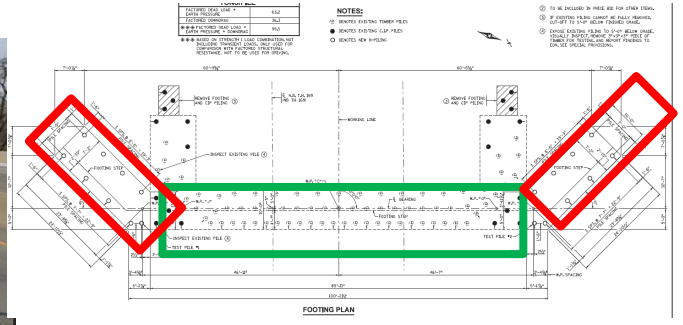
Foundation Reuse at MnDOT





Pier Protection

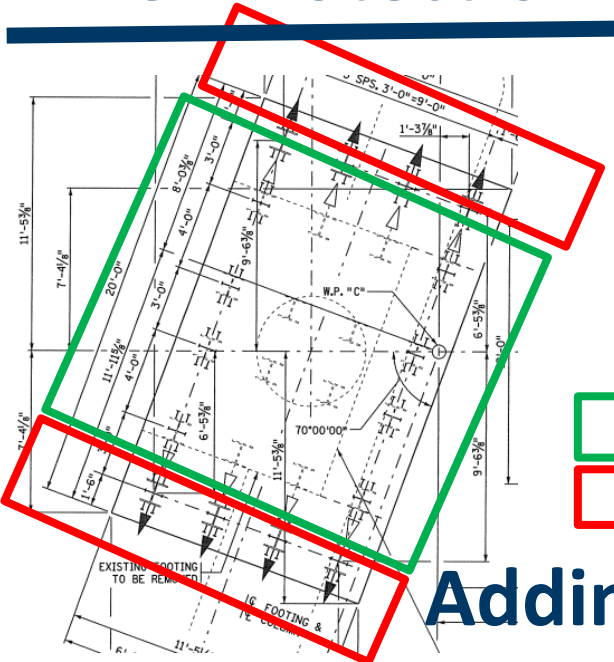




Reused substructure/ pile

New substructure/ pile

Bridge Widening

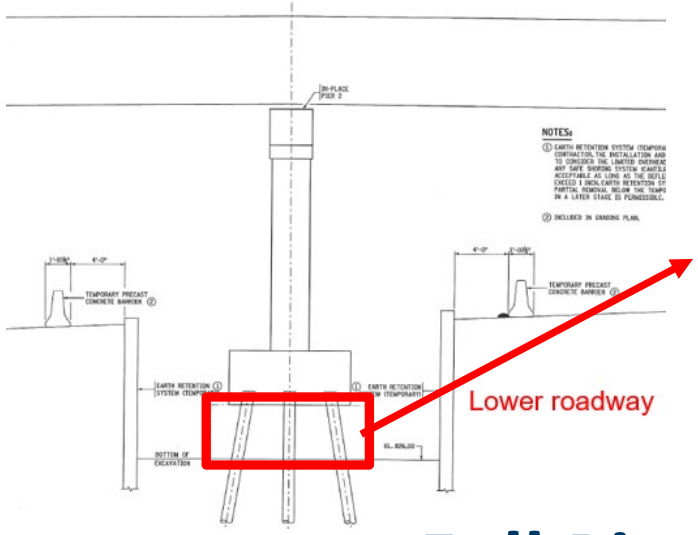




Reused pile

New pile

Adding Pile/New Cap





Full Pier/Pile Reuse

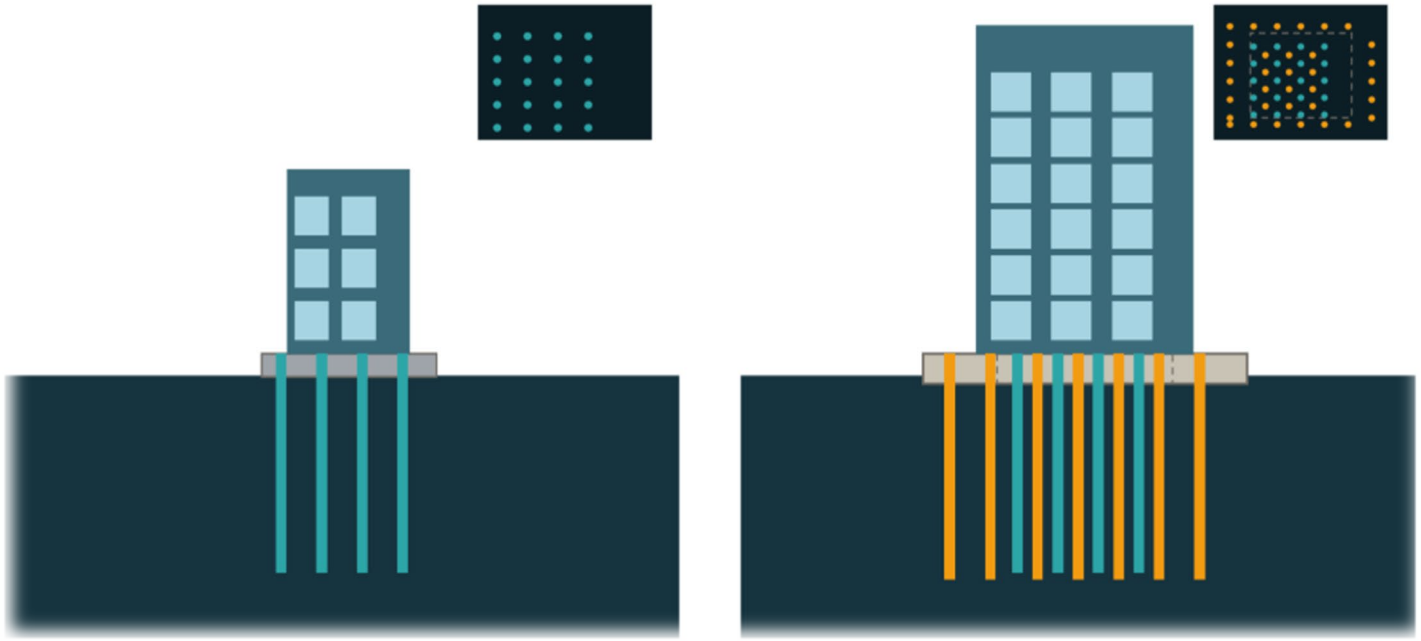


Navid Plaseied, P.E., G.E., PMP, DBIA Assoc.

MS Civil Geotechnical, University of Colorado at Boulder
MS Rock Mechanics, Amirkabir University of Technology
BS Mining Engineering, Isfahan University of Technology

Licensed in NY and CA

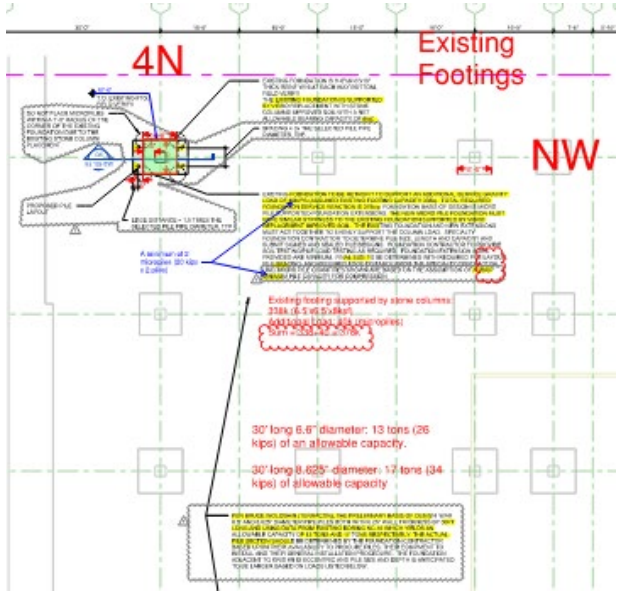
- Senior Geotechnical Design Specialist at Turner Engineering Group, focusing on design optimization, risk mitigation, and constructability reviews.
- With over 20 years of expertise across buildings, transportation, dams, and mining projects.

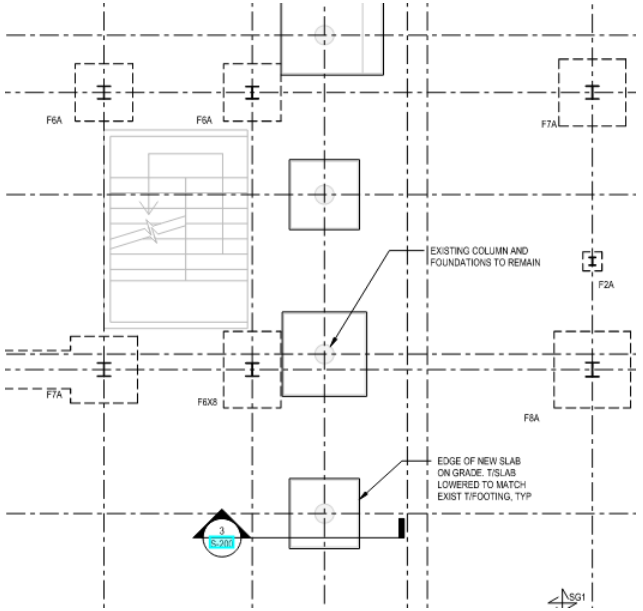


Foundation Reuse Illustration (generated by AI)

- Turner Engineering Group and our Sustainability team advocate early identification of foundation reuse opportunities, focusing on embodied carbon, cost, and schedule savings.
- Approach: reuse existing footings and deep foundation (e.g., piles) where practical, such as enlarging footings, adding micropiles, or tying into new pile caps to share demand with minimal disruption.
- Recent work, existing footings reused and modified to carry increased loads for renovations.
  - New Jersey project, existing footing exposed and enlarged during construction.
  - Florida project, aggregate pier supported spread footing, augmented with micropiles to raise column loads with minimal vibration.
  - NYC project, reuse existing spread footings where capacity and serviceability checks pass.







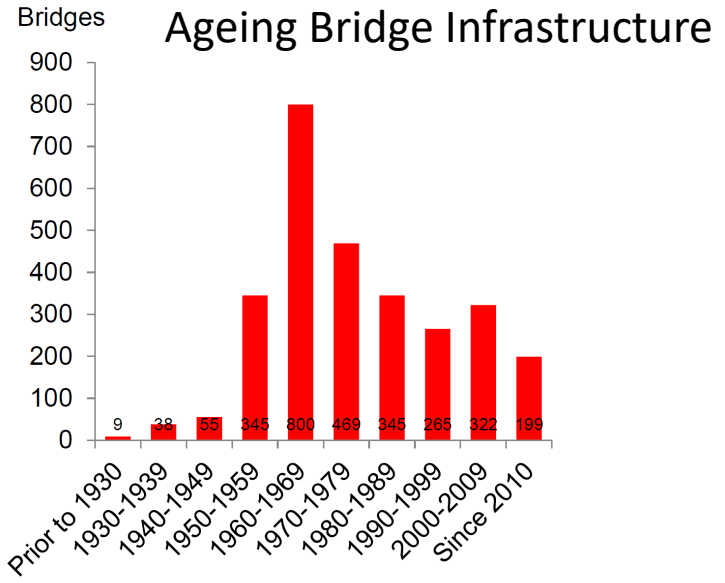


Tony Sangiuliano
Head of Foundations
Ministry of Transportation of Ontario
Reuse of Existing Foundations Guideline

- The reuse of existing foundations shall be considered as an alternative for any given replacement or rehabilitation project.
- A comprehensive process and procedure for assessing the risks and feasibility of reuse projects as warranted for any given project shall be carried to allow for informed decision-making on the potential benefits of foundation reuse.



Reuse of Existing Foundations Guideline

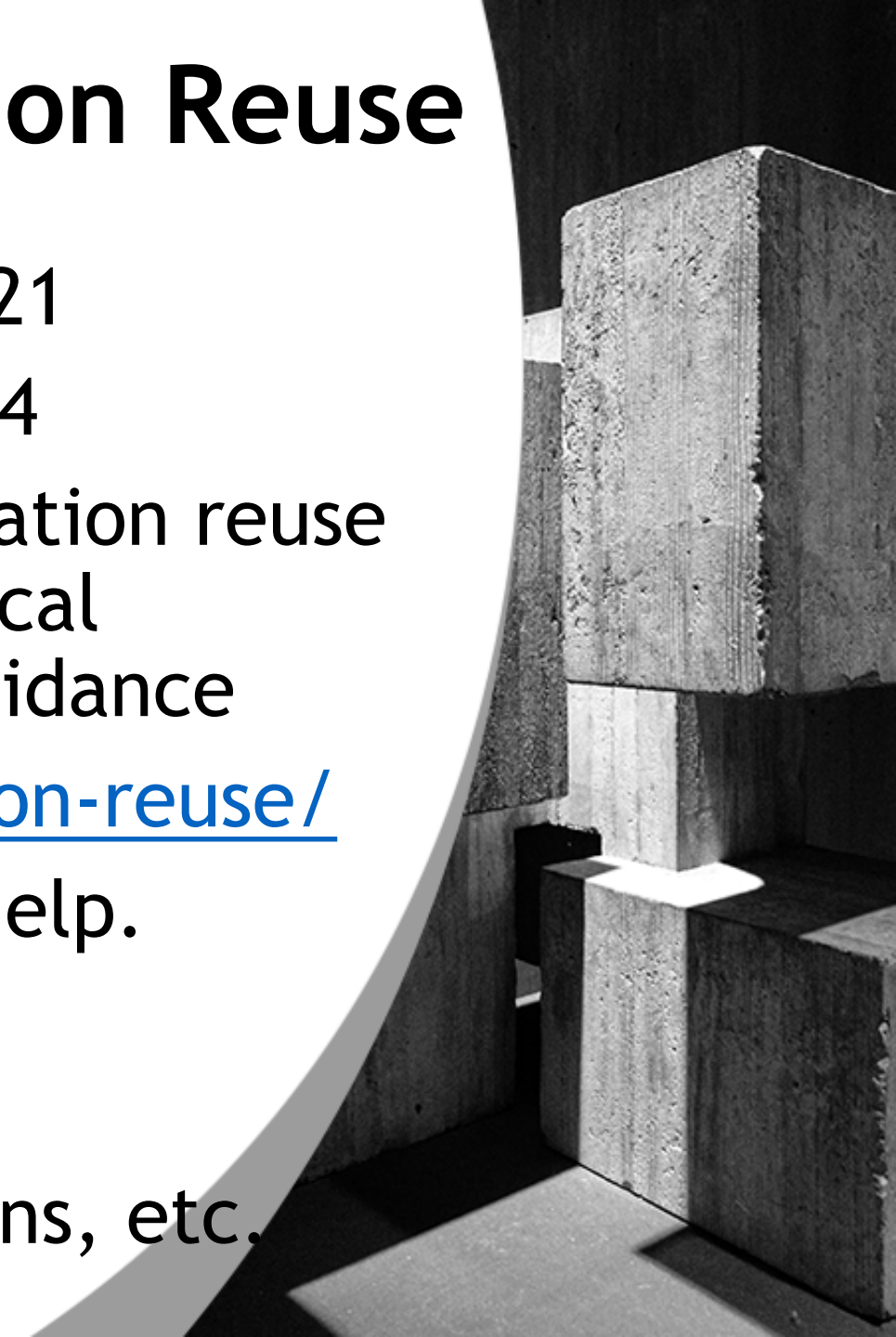


- The major steps of the decision-making process shall include:
  - Preliminary Assessment
  - Risk Analysis
  - Life Cycle Cost Assessment
  - Capacity Assessment
  - Estimating the remaining service life of the foundation
  - Environmental
  - Social



DFI Committee on Foundation Reuse

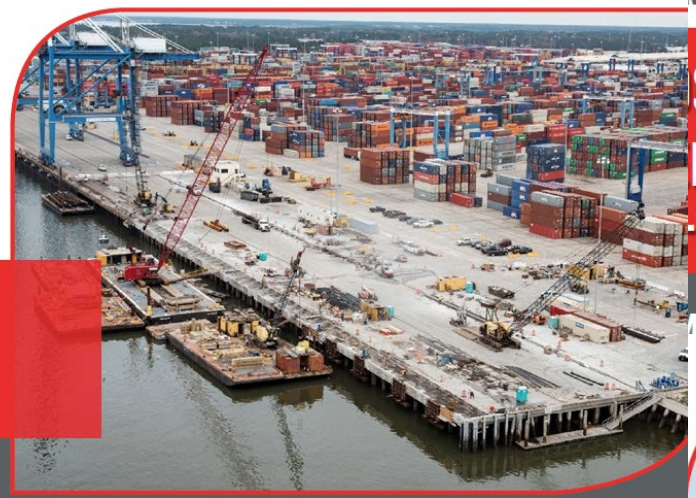
- Task force on reuse started in early 2021
- Officially a committee in February 2024
- Mission: Reduce impediments to foundation reuse through outreach, promotion of technical advancements, and development of guidance
- https://dfi.org/communities/foundation-reuse/
- Always welcoming new members and help.
- Alternating monthly meetings:
  - On-agenda: committee business
  - Off agenda: case histories, discussions, etc.



DFI Case Examples

DEEP FOUNDATIONS INSTITUTE
SUBSURFACE CHARACTERIZATION COMMITTEE PRESENTS

REUSE SAVES $9M
for Port Reconstruction



South Carolina Ports Authority
Charleston, SC

SCOPE OF FOUNDATION WORK

DEEP FOUNDATIONS INSTITUTE
SUBSURFACE CHARACTERIZATION
MMITTEE PRESENTS

STON WATERFRONT DEVELOPMENT:
-Use of Existing Pier Piles



Massachusetts Port Authority

COMMITTEE PRESENTS

MAJOR MOBILITY MITIGATION FOR A MAJOR RIVER BRIDGE REPLACEMENT:
Milton-Madison Bridge Foundation Reuse



Indiana DOT, Kentucky
Transportation Cabinet, USDOT
Madison, Indiana and Milton, Kentucky (Ohio River Crossing)

Mobility





93 Fast 14

- 14 structures
- ADT: 200,000 NB/SB
- 10 consecutive 55-hour work weekends over summer
- No impact to weekday rush hour traffic

Slide credit:
Pete Connors, MassDOT

Subsurface **and Existing Foundation** Investigation





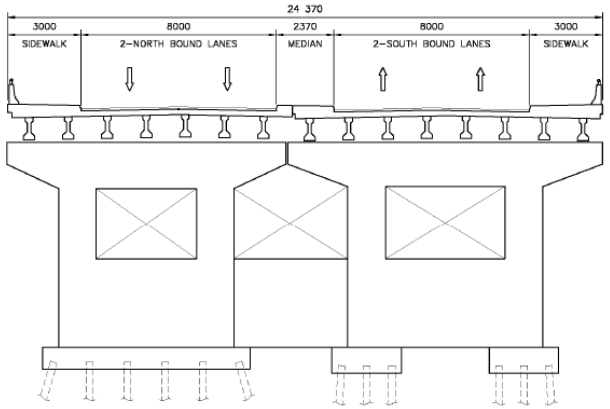




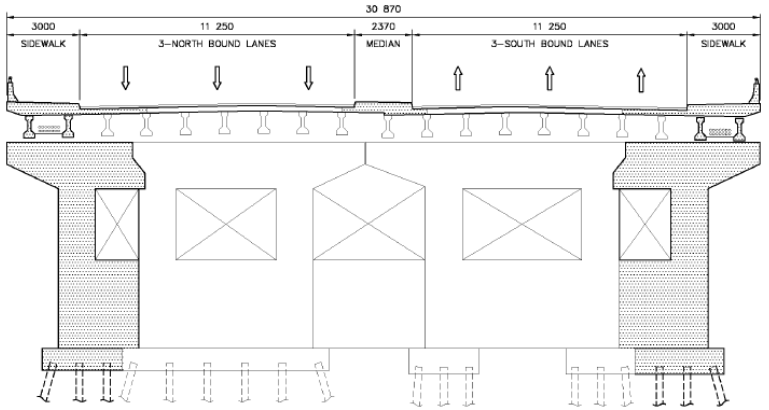
Photos courtesy of Haley & Aldrich

Creativity

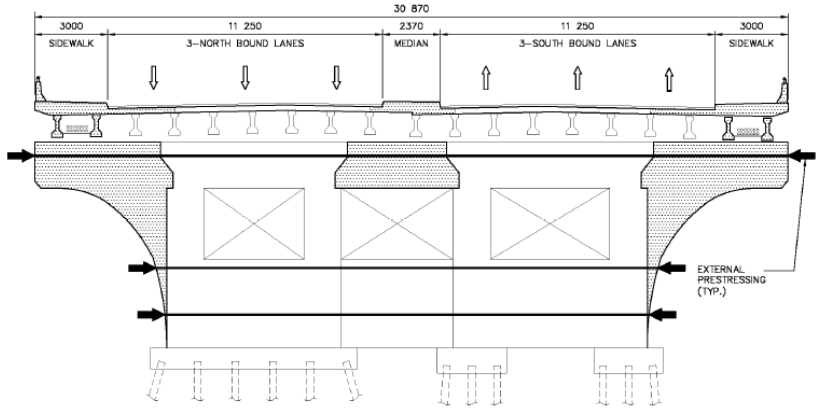
EXISTING BRIDGE



CONVENTIONAL WIDENING SCHEME



INNOVATIVE WIDENING SCHEME

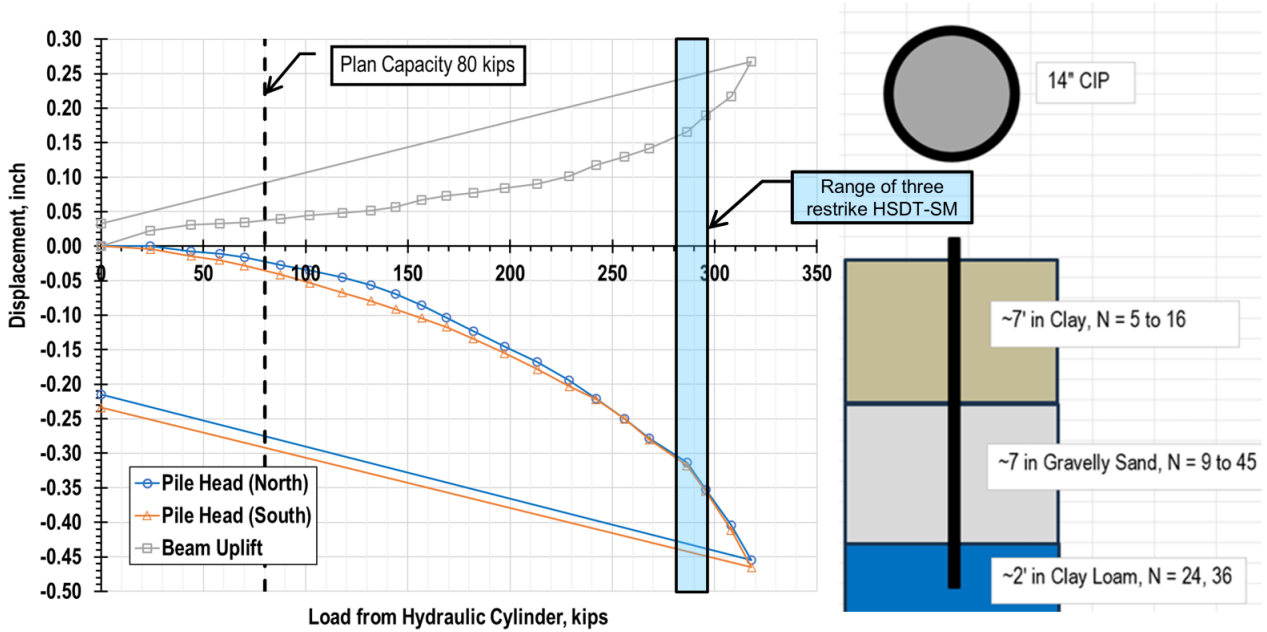


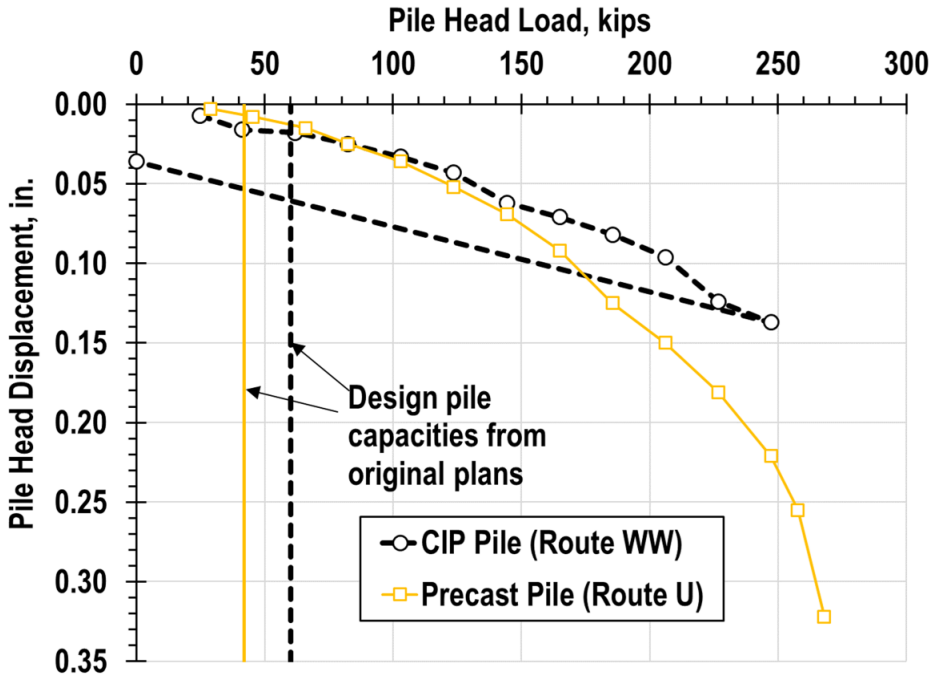
Li, Lee, Cholewa (2010), "An Innovative Solution to Integrate Existing Substructure to Effect Economy and Constructability in a Bridge Widening Project," Proceedings of the 8th International Conference on Short and Medium Span Bridges, Canada.

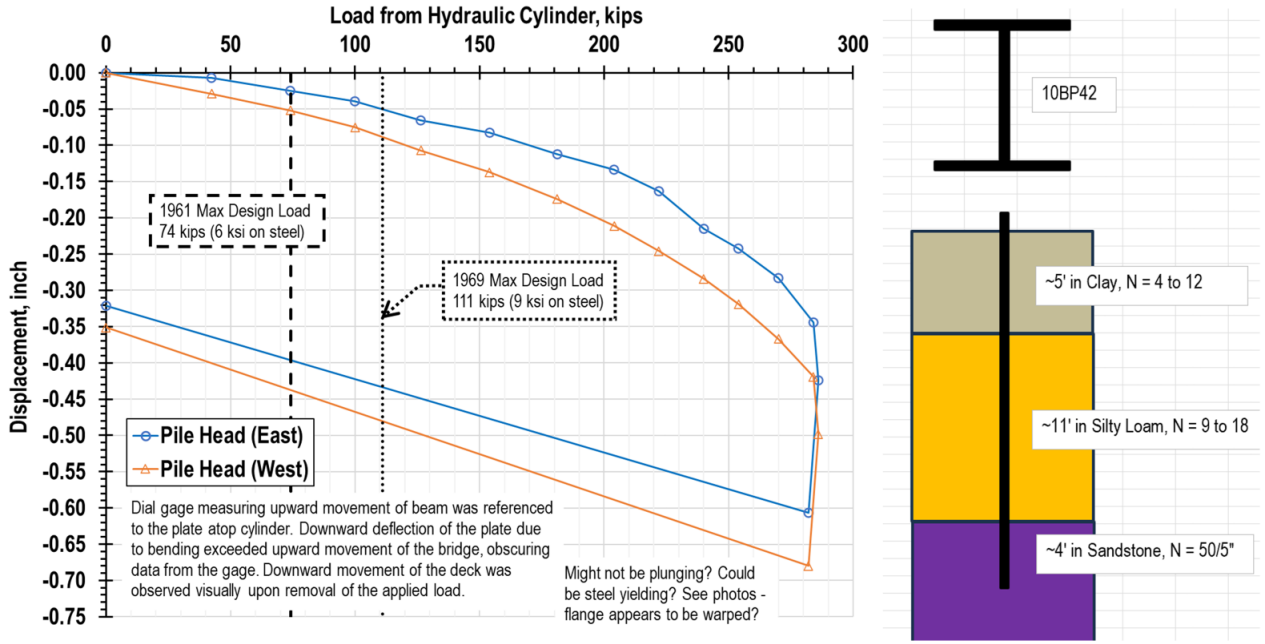
Capacity…

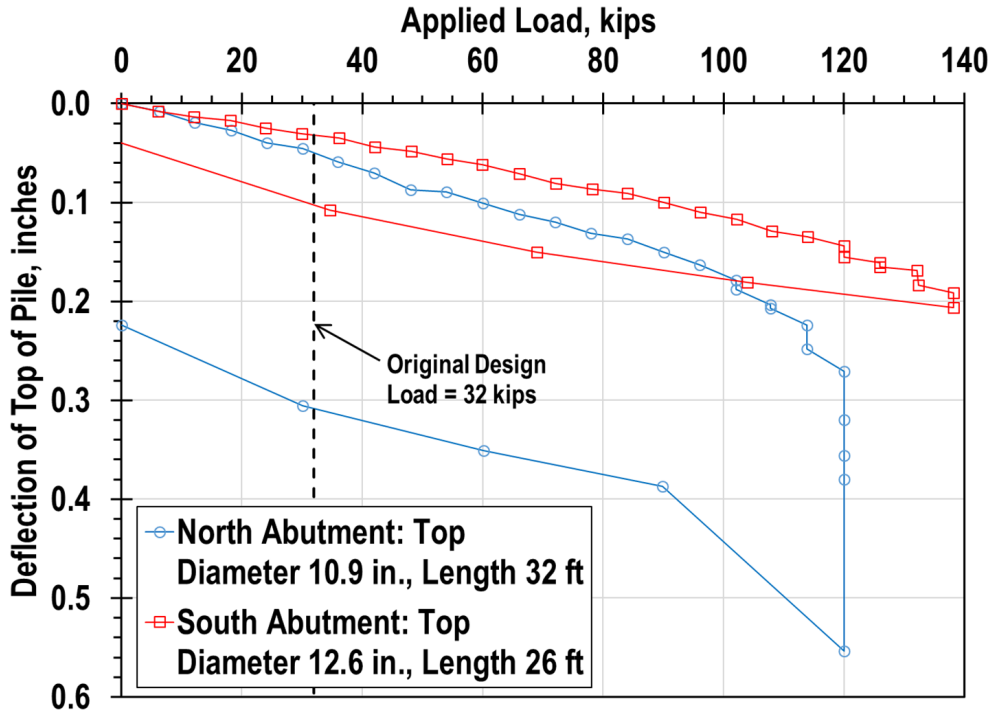


...in spades

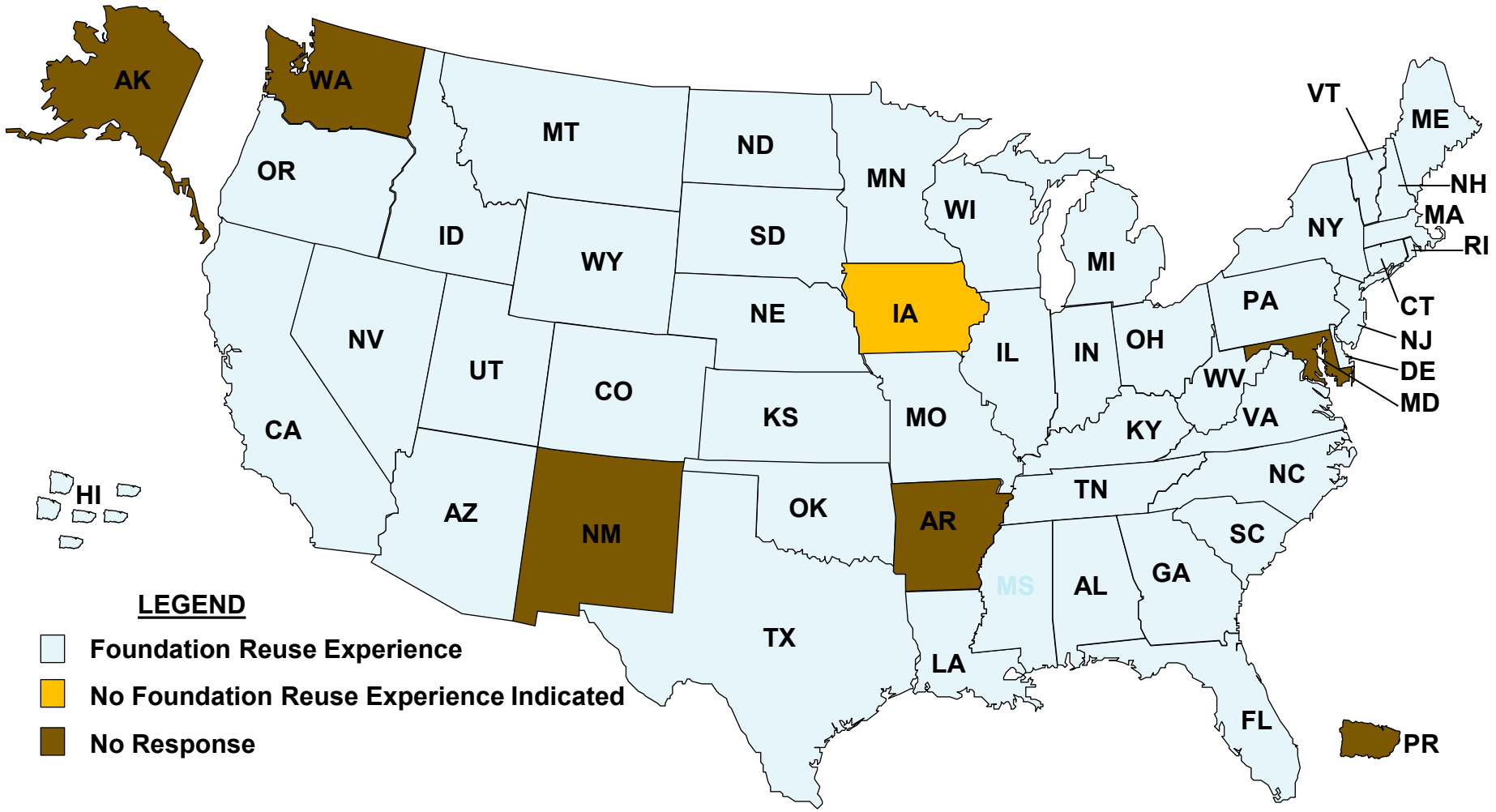

Plan Capacity 80 kips
Range of three restrike HSDT-SM
Displacement, inch
Load from Hydraulic Cylinder, kips
Pile Head (North)
Pile Head (South)
Beam Uplift
14" CIP
~7' in Clay, N = 5 to 16
~7 in Gravelly Sand, N = 9 to 45
~2' in Clay Loam, N = 24, 36


Pile Head Load, kips
Pile Head Displacement, in.
Design pile capacities from original plans
CIP Pile (Route WW)
Precast Pile (Route U)

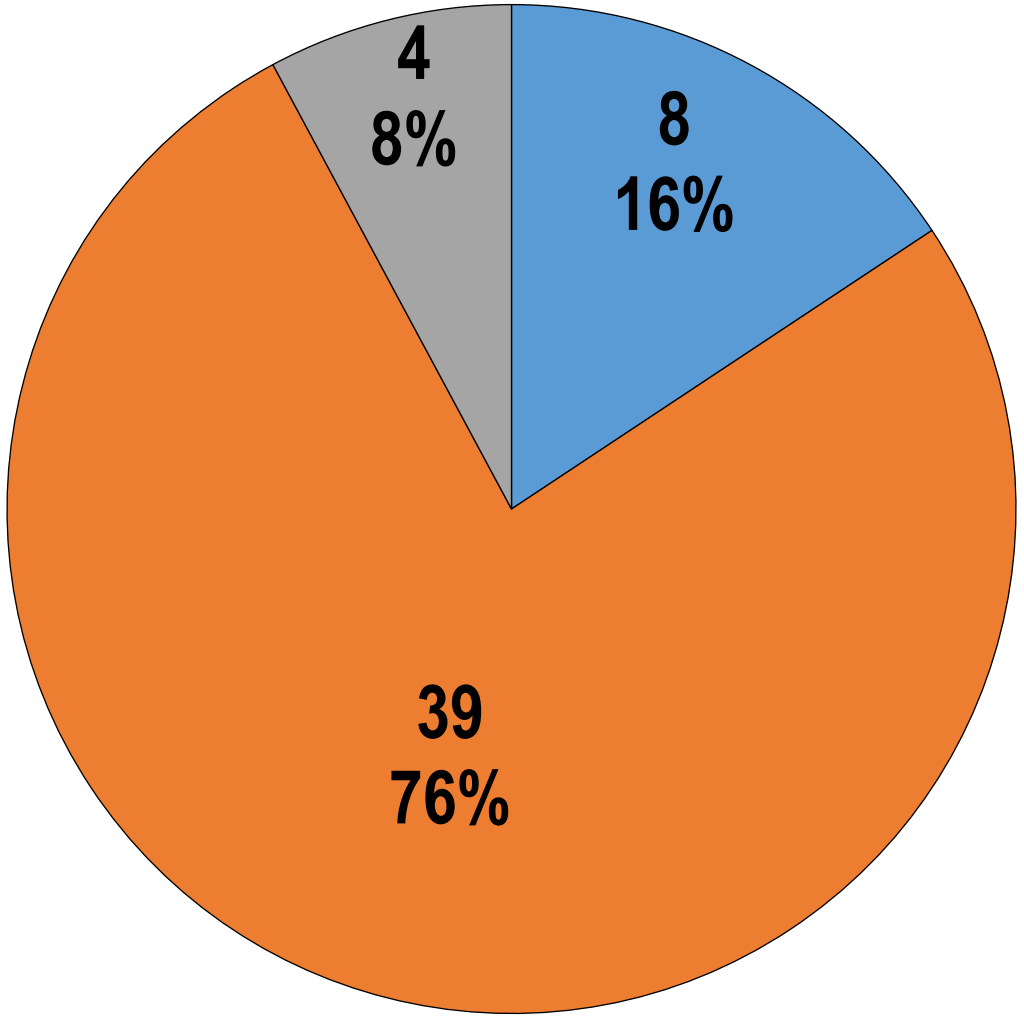

Load from Hydraulic Cylinder, kips
Displacement, inch
1961 Max Design Load
74 kips (6 ksi on steel)
1969 Max Design Load
111 kips (9 ksi on steel)
Pile Head (East)
Pile Head (West)
Dial gage measuring upward movement of beam was referenced to the plate atop cylinder. Downward deflection of the plate due to bending exceeded upward movement of the bridge, obscuring data from the gage. Downward movement of the deck was observed visually upon removal of the applied load.
Might not be plunging? Could be steel yielding? See photos - flange appears to be warped?
10BP42
~5' in Clay, N = 4 to 12
~11' in Silty Loam, N = 9 to 18
~4' in Sandstone, N = 50/5"


Applied Load, kips
Deflection of Top of Pile, inches
Original Design
Load = 32 kips
North Abutment: Top
Diameter 10.9 in., Length 32 ft
South Abutment: Top
Diameter 12.6 in., Length 26 ft

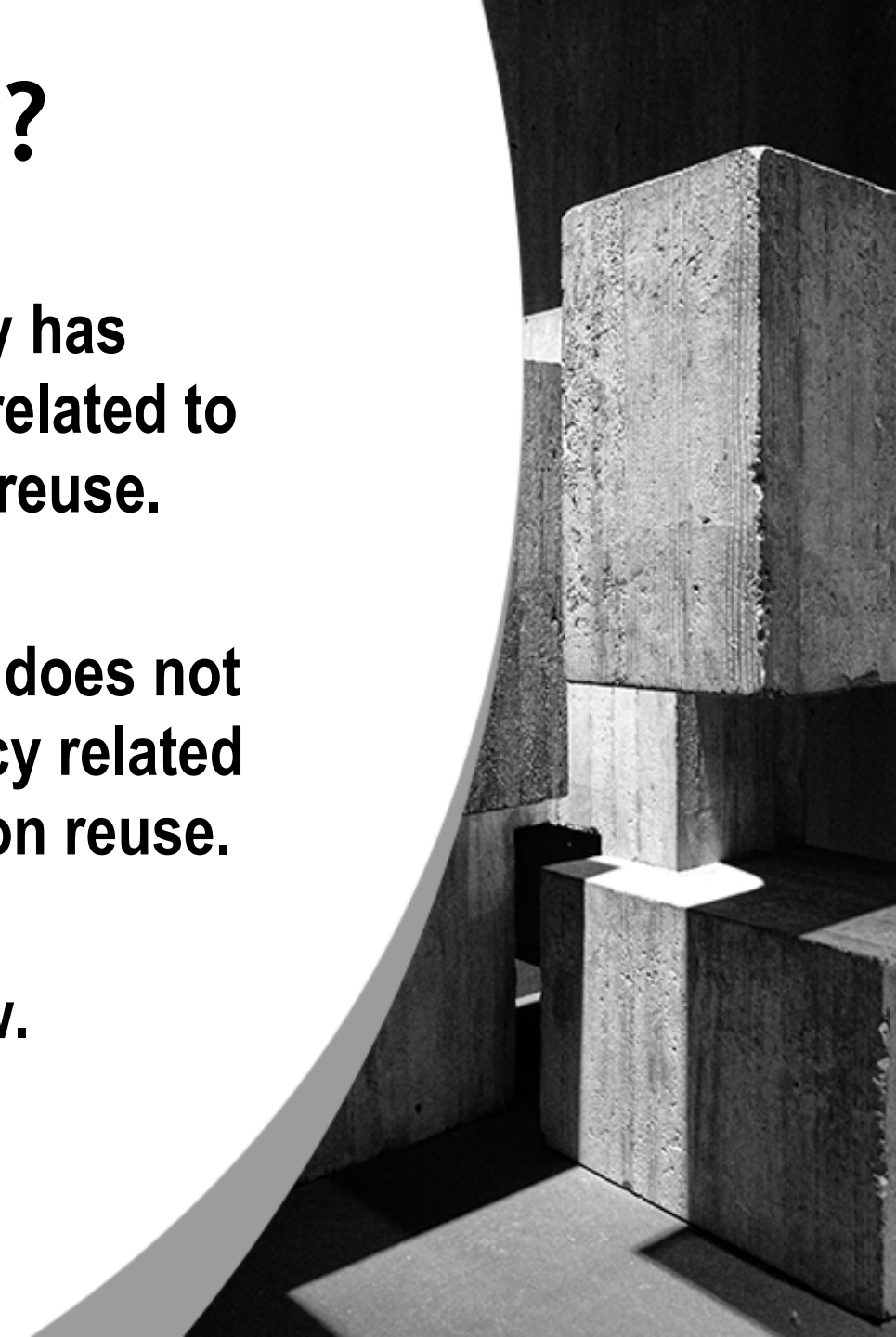
NCHRP Synthesis 47-03 Survey



Foundation Reuse Policy?



- Yes, agency has policy(ies) related to foundation reuse.
- No, agency does not have a policy related to foundation reuse.
- I don't know.



Guide Specification Outline

1. Description
2. Available Guidance
3. Definition of Reuse
4. Other Definitions
5. Design Requirements
6. Coordination & Risk Management
7. Use of Historical Docs
8. Field Investigation
9. Capacity Investigation
10. Remaining Service Life Evaluation
11. Observation during Construction

Poll Results

https://forms.office.com/Pages/DesignPageV2.aspx?subpage=design&FormId=Oe_m5pemUEqR-inUc_t7kJFhdfdTaPpCh_FS9_34s6JUMEsyRlpFUklZMkdUUFJMNUo4TFpLWUNWUy4u&Token=2dd89cb28fa54db5989fe763dd98e9e5

Prompts for Audience

- What is preventing reuse?
- What do you want in the guide specification?
- How could the outline be improved?



Thank you!

Additional comments:

Andy Boeckmann
Dan Brown & Associates
aboeckmann@dba.world



DAN BROWN AND ASSOCIATES

PARALLEL SESSION C4
MSSP Grout

Introduction to Drivecast™

Brandon Walter, Earth Contact Products

Matthew Conte, Conte Company

Disclaimer: “The opinions expressed in this presentation and on the following slides are solely those of the presenter(s) and not necessarily those of DFI. DFI does not guarantee the accuracy or reliability of the information nor endorse the recommendations offered by the presenter(s).”

DRIVECAST™

GROUTED DISPLACEMENT PILES

Grouted Displacement Piles

- Have existed for many years
- Several companies have their own means and methods, some proprietary
- A wide range in types
- Drivecast™ Grouted Displacement Piles combine the benefits of helical piles with the higher capacity of displacement piles.



Grouted Displacement Pile Capacity

Two mechanisms

- Side friction, Q_s (skin friction)
- Toe-bearing Resistance, Q_b (point-bearing resistance, end bearing Resistance, tip bearing resistance)

$$Q_{total} = Q_{bearing} + Q_{side}$$



Helical Installation Influence

- Low head clearance modular components
- No installation vibration
- No spoils
- Smaller equipment
- Project specific design options
- QA/QC Benefits





Low Site Impact



Clean Installation





Limited Access

Reach Equipment vs. Mast Equipment



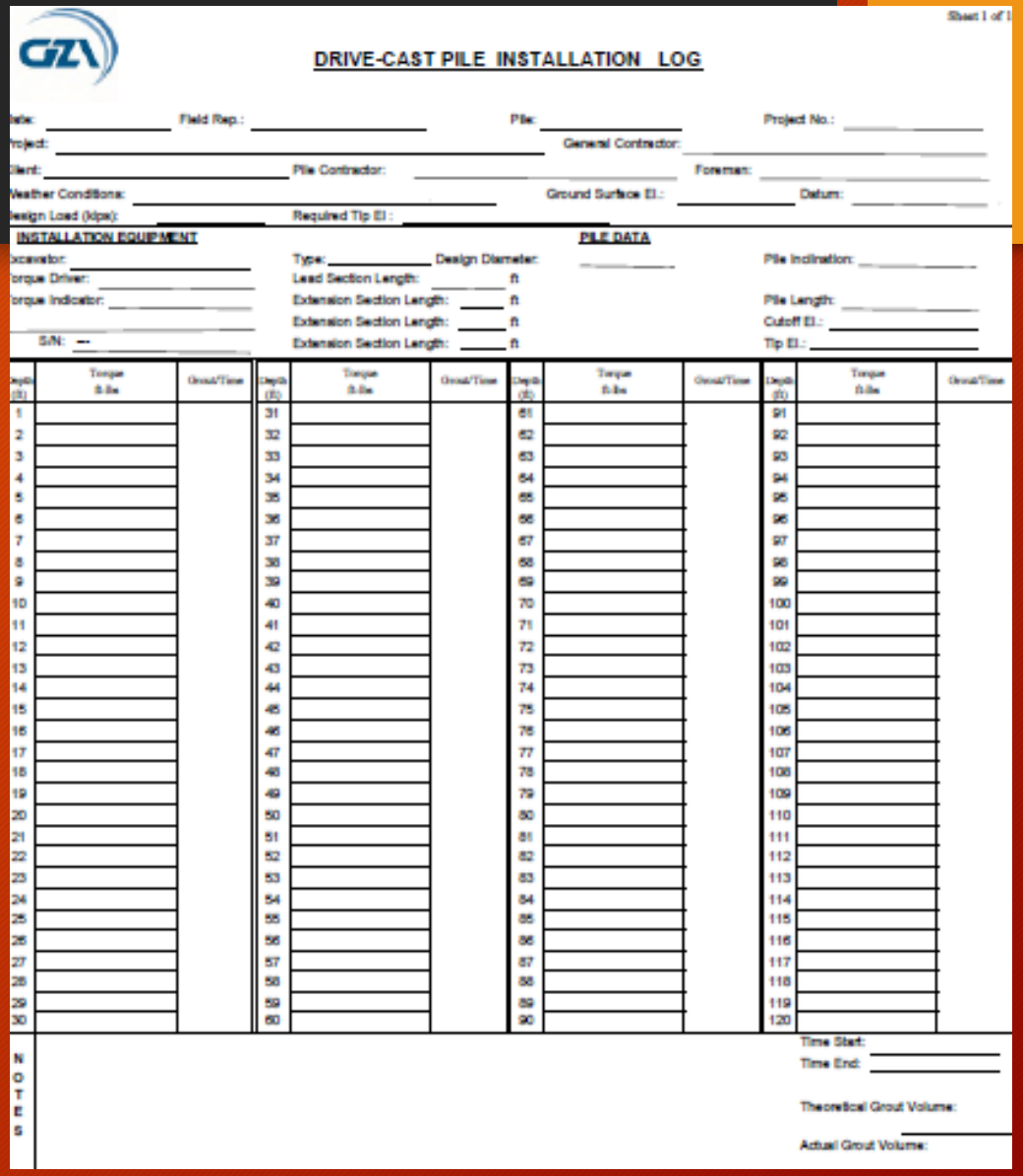
How do they work?

- Each section is placed on the ground surface and advanced into the soil using rotation
- Helices are used in each displacement assembly to advance into the soil
- Displacement paddles are used to ‘push’ the soils outward and create a cavity
- Grout is added from the ground surface and gravity fed into the cavity



Quality Control

- Torque is monitored in real time
- Actual soil conditions compared to soil borings through torque information
- Grout take monitored during installation
- All monitored items are logged in field report



DRIVE-CAST PILE INSTALLATION LOG

Field Rep.:
Pile:
Project No.:
Project:
General Contractor:
Client:
Pile Contractor:
Foreman:
Weather Conditions:
Ground Surface El.:
Datum:
Design Load (kips):
Required Tip El.:

INSTALLATION EQUIPMENT

Excavator:
Torque Driver:
Torque Indicator:
S/N:

PILE DATA

Type:
Design Diameter:
Pile Inclination:
Lead Section Length: ft
Extension Section Length: ft
Extension Section Length: ft
Extension Section Length: ft
Pile Length:
Cutoff El.:
Tip El.:

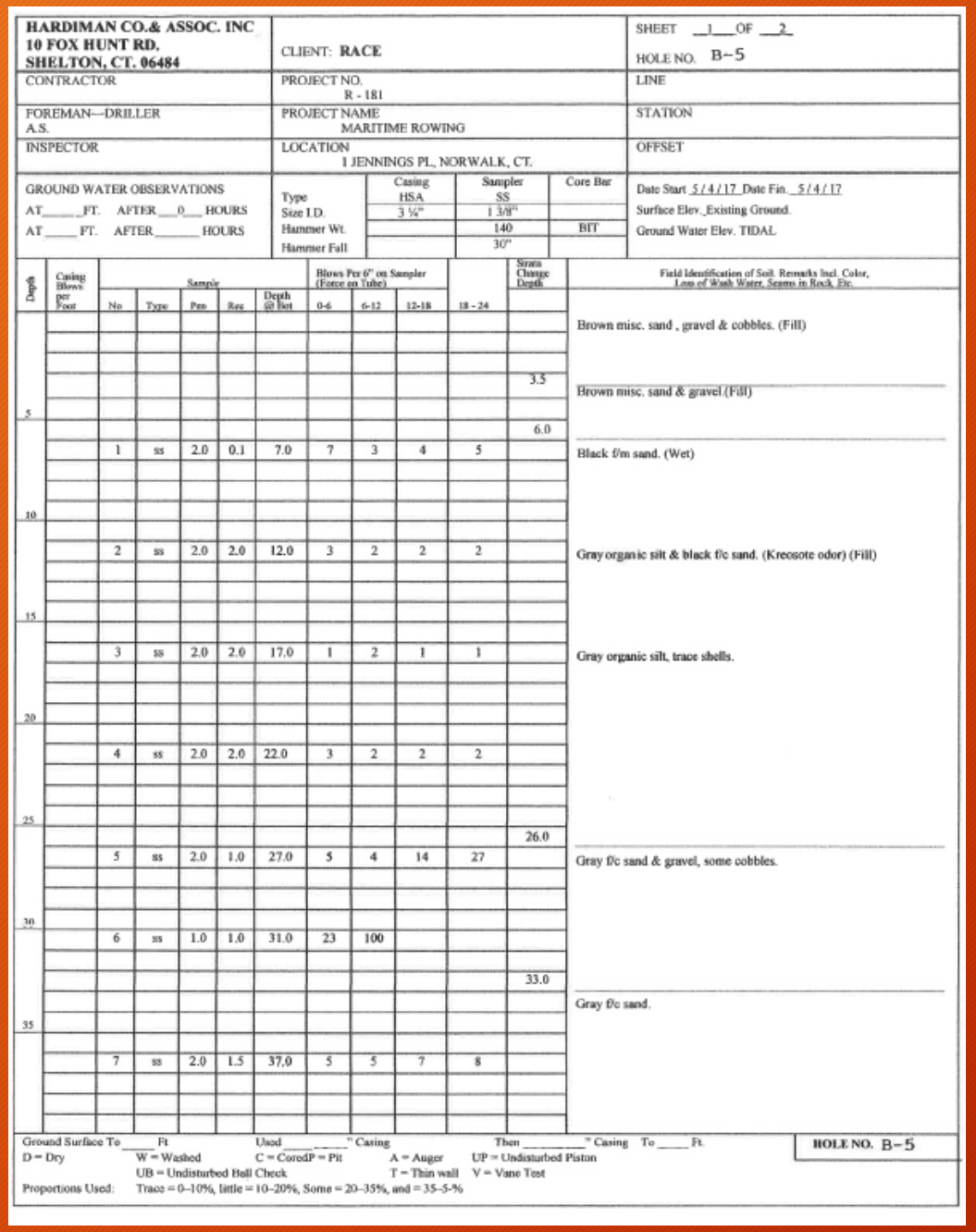
Depth (ft)	Torque ft-lbs	Grout/Time	Depth (ft)	Torque ft-lbs	Grout/Time	Depth (ft)	Torque ft-lbs	Grout/Time	Depth (ft)	Torque ft-lbs	Grout/Time
1			31			61			91		
2			32			62			92		
3			33			63			93		
4			34			64			94		
5			35			65			95		
6			36			66			96		
7			37			67			97		
8			38			68			98		
9			39			69			99		
10			40			70			100		
11			41			71			101		
12			42			72			102		
13			43			73			103		
14			44			74			104		
15			45			75			105		
16			46			76			106		
17			47			77			107		
18			48			78			108		
19			49			79			109		
20			50			80			110		
21			51			81			111		
22			52			82			112		
23			53			83			113		
24			54			84			114		
25			55			85			115		
26			56			86			116		
27			57			87			117		
28			58			88			118		
29			59			89			119		
30			60			90			120		

NOTES

Time Start:
Time End:
Theoretical Grout Volume:
Actual Grout Volume:

Testing Drivecast Piles
Compared to Other Pile Types





HARDIMAN CO.& ASSOC. INC
10 FOX HUNT RD.
SHELTON, CT. 06484

CLIENT: **RACE**

SHEET 1 OF 2
HOLE NO. B-5

CONTRACTOR

PROJECT NO. R - 181

LINE

FOREMAN---DRILLER A.S.

PROJECT NAME MARITIME ROWING

STATION

INSPECTOR

LOCATION 1 JENNINGS PL, NORWALK, CT.

OFFSET

GROUND WATER OBSERVATIONS
AT ____ FT. AFTER 0 HOURS
AT ____ FT. AFTER ____ HOURS

	Casing	Sampler	Core Bar
Type	HSA	SS	
Size I.D.	3 ¼"	1 3/8"	
Hammer Wt.		140	BIT
Hammer Fall		30"	

Date Start 5/4/17 Date Fin. 5/4/17
Surface Elev. Existing Ground.
Ground Water Elev. TIDAL

Depth	Casing Blows per Foot	Sample No	Sample Type	Sample Pen	Sample Rec	Sample Depth @ Bot	Blows Per 6" on Sampler (Force on Tube) 0-6	6-12	12-18	18-24	Strata Change Depth	Field Identification of Soil, Remarks Incl. Color, Loss of Wash Water, Seams in Rock, Etc.
5											3.5	Brown misc. sand , gravel & cobbles. (Fill)
											6.0	Brown misc. sand & gravel (Fill)
10		1	ss	2.0	0.1	7.0	7	3	4	5		Black f/m sand. (Wet)
15		2	ss	2.0	2.0	12.0	3	2	2	2		Gray organic silt & black f/c sand. (Kreosote odor) (Fill)
20		3	ss	2.0	2.0	17.0	1	2	1	1		Gray organic silt, trace shells.
25		4	ss	2.0	2.0	22.0	3	2	2	2		
											26.0	
30		5	ss	2.0	1.0	27.0	5	4	14	27		Gray f/c sand & gravel, some cobbles.
35		6	ss	1.0	1.0	31.0	23	100				
											33.0	Gray f/c sand.
		7	ss	2.0	1.5	37.0	5	5	7	8		

Ground Surface To ____ Ft Used ____ " Casing Then ____ " Casing To ____ Ft.
D = Dry W = Washed C = CoredP = Pit A = Auger UP = Undisturbed Piston
UB = Undisturbed Ball Check T = Thin wall V = Vane Test
Proportions Used: Trace = 0-10%, little = 10-20%, Some = 20-35%, and = 35-5-%

HOLE NO. B-5

Bore Log B-5
Closest Boring to Test Site

GZA Monitored Installation and Load Tests

Piles Installed and Tested by Conte Company

Test Pile Installation

TABLE 1- PILE INSTALLATION SUMMARY

Pile Type	Pile Dimensions	Depth to Pile Tip
H-Pile	12x53	27 feet
Pipe Pile	10.75” diameter x 0.375” Wall open-ended pipe	27 feet
Timber Pile	14” Butt, 8” Tip	27.5 feet
SS200 Drive-Cast	2” square shaft 10-inch diameter grout shaft	27 feet
RS3500 Drive-Cast	3.5” Diameter x 0.3” Wall Shaft 9-inch diameter grout shaft	27.6 feet
RS4500 Drive-Cast	4.5” Diameter x 0.337” Wall Shaft 9.6-inch diameter grout shaft	27.7 feet
SS150 Helical Piles	8-, 10-, and 12-inch diameter helices	30 feet
SS150 Pulldown Pile	8-, 10-, and 12-inch diameter helices 3.5-inch diameter grout shaft	31 feet

Test Results

TABLE 2- COMPRESSION LOAD TEST RESULTS

Pile Type	Test Load at 0.5" Deflection	Failure Load or Ultimate Load
H-Pile	17 kips	75 kips
Pipe Pile	40 kips	90 kips*
Timber Pile	78 kips	90 kips
SS200 Drive-Cast	120 kips	150 kips*
RS3500 Drive-Cast	90 kips	135 kips
RS4500 Drive-Cast	Deflection less than 0.5"	127.5 kips*
SS150 Helical Piles	22.5 kips	40 kips
SS150 Pulldown Pile	33 kips	40 kips*

* Pile did not fail

TABLE 3- LATERAL LOAD TEST RESULTS

Pile Type	Deflection at 8.5-kip Lateral Load	Deflection at 20-kip Lateral Load
H-Pile	0.338"	NA
Pipe Pile	0.471"	NA
Timber Pile	0.257"	NA
SS200 Drive-Cast	0.801"	NA
RS3500 Drive-Cast	0.998"	NA
RS4500 Drive-Cast	0.179"	0.9"

Project Case Studies



Electric Substation Project Site



- Power Generation Substation
- Boston, MA
- Two separate substations on the same property
- Active site, access to be maintained at all times
- 805 total piles

Project Design Details

Drivecast Pile Design

- RS3500 10/14 (3.5-inch diameter x 0.3-inch thick central steel shaft)
- 8”-10” Design Grout Shaft around the central shaft
- Calculated 20’ minimum embedment into Glaciodeltaic Deposits
- Calculated 23’ minimum embedment into Marine Deposits
- Test piles to be load tested in tension and laterally

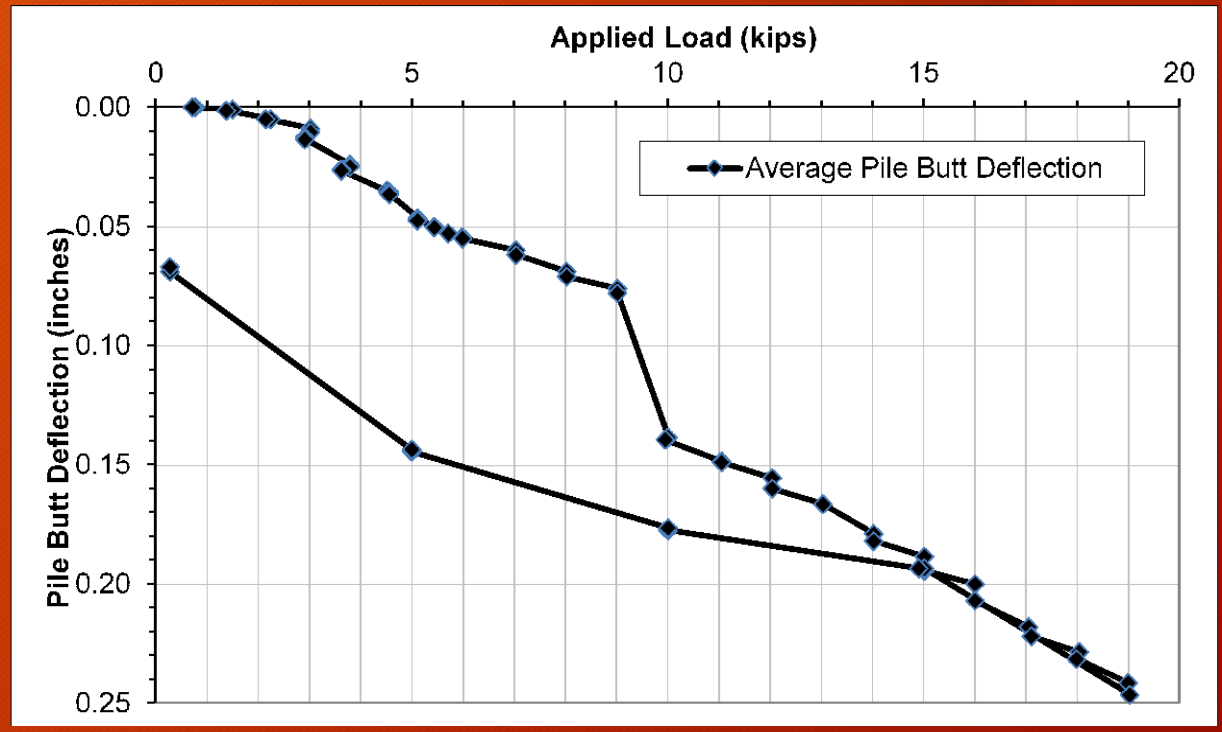
Micropile Design

- 7-inch diameter x 0.408 -inch thick, Grade 80 ksi casing
- Full length #8 bar, grade 75 ksi
- Calculated 12’ to 29’ embedment below casing, depending on pile cap
- Micropile test pile to be load tested in tension

Drivecast Load Testing – Lateral #1: 37’ Pile



Drivecast Lateral Test Setup



Drivecast Lateral Test: Load-Deflection Graph

Drivecast Load Testing – Tension #2

INSTALLATION EQUIPMENT

Excavator: CAT 314

Torque Driver: Pengo RS12

Torque Indicator: Marian Technologies

Model 1013 RF (1)

S/N:

PILE DATA

Type: RS3500 Diameter: 3.5" Pile Inclination: Vertical

Lead Section Length: 7 ft No. Helices: 4 Helix Sizes: 10,14

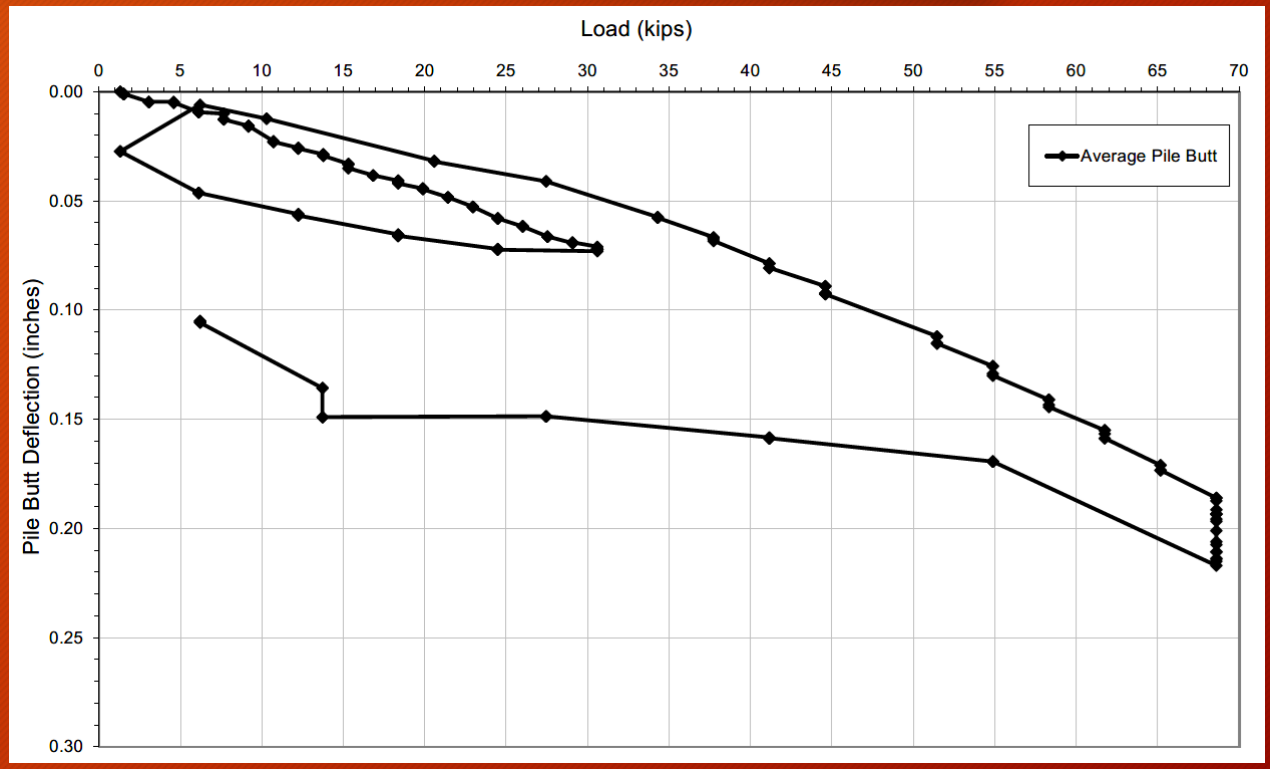
Extension Section Length: 7 ft No. Used: 3 Pile Length: 27 ft

Extension Section Length: ft No. Used: Cutoff El.: ft.

Extension Section Length: ft No. Used: Tip El.: ft

Depth (ft)	Torque ft-lbs	Grout/Time	Depth (ft)	Torque ft-lbs	Grout/Time	Depth (ft)	Torque ft-lbs	Grout/Time	Depth (ft)	Torque ft-lbs	Grout/Time
1	Vaccum		31			61			91		
2	Vaccum		32			62			92		
3	Vaccum		33			63			93		
4	Vaccum		34			64			94		
5	Vaccum		35			65			95		
6	1800-1900		36			66			96		
7	1964-2000		37			67			97		
8	1900-2000		38			68			98		
9	1800-1956		39			69			99		
10	2800-3000		40			70			100		
11	2200-2300		41			71			101		
12	1800-1900		42			72			102		
13	2000-2200		43			73			103		
14	2300-2345		44			74			104		
15	2000-2100		45			75			105		
16	2700-3200		46			76			106		
17	6600-6700		47			77			107		
18	7700-7800		48			78			108		
19	7400-7500		49			79			109		
20	6428-6450		50			80			110		
21	6300-6400		51			81			111		
22	8000-8060		52			82			112		
23	9300-9600		53			83			113		
24	10500-10553		54			84			114		
25	9700-9800		55			85			115		
26	11000-11100		56			86			116		
27	11900-12200		57			87			117		
28			58			88			118		
29			59			89			119		
30			60			90			120		

Test Pile Log



Drivecast Tension Test #2: Load-Deflection Graph

Commercial Underpinning Project: Cambridge, MA

- 49 Exterior Piles
- 115 Interior Piles
- 78 kip ultimate compression
- Stacked granite foundation
- Brick Exterior Walls / Steel & Wood Frame
- Helical piles could not show consistent results across site
- 24 +/- helical probes of various type attempted
- Thin bearing layer allowed for only 3-4’ of bond zone below organics
- All Drivecast piles installed matched load test results
- Some interior piles installed using 6k drivehead & 2 ton machine
- Project increase of 21% with change to Drivecast



Private Commercial Building Mystic, CT

- Original design for micropiles
- Helicals not preferred by EOR
  - A solid grout shaft to weather rock was required
  - Larger shaft for stability in loose soils required
- RS3500 10/14
- 8”-10” Design Grout Shaft around the central shaft
- Pile tip to weathered rock
- Test piles to be load tested in compression
- 103 total piles
  - Over $100k saved in direct price to micropiles
  - 4-5 weeks in time saved to micropiles
  - Eliminated contaminated spoils / large indirect cost



Rockaway Project
Pile Exhumation



Rockaway Project: 56’ Pile

Test Pile Log

INSTALLATION EQUIPMENT

Excavator: Komatsu PC 228 US LC

Torque Driver: DIGGA SD95

Torque Indicator: Digga Torque Logic

S/N:

PILE DATA

Reinforcement: 4.5" OD x 0.337" thick pipe — Pile Inclination: Vertical

Displacement Assembly: Spacing: 5' — No. Helices: 16 — Helix Sizes: 12",16"

Design Grout Diameter: 10" — Specific Gravity: 1.8

Reinforcement Section Length: 10 ft — No. Used: 4 — Cutoff El.: —

Reinforcement Section 5 ft — No. Used: 2 — Tip El.: -37.7 ft.

Depth (ft)	Torque ft-lbs	Grout/Time	Depth (ft)	Torque ft-lbs	Grout/Time	Depth (ft)	Torque ft-lbs	Grout/Time	Depth (ft)	Torque ft-lbs	Grout/Time
1			31	11,400	Start:-7.5"	61			91		
2		PVC Pipe	32		F. Meter: 27.5 gal	62			92		
3			33			63			93		
4		3.5 ft	34			64			94		
5		Start: -6.5"	35			65			95		
6			36		-16"	66			96		
7	4,000	-11.5"	37	12,000		67			97		
8			38			68			98		
9		F. Meter: 54 gal	39		-19.5"	69			99		
10	5,000		40	18k-19k		70			100		
11			41	18k-19k	End: -24"	71			101		
12		(54 gallons)	42	18k-19k	Start: -6.5"	72			102		
13		-20"	43	8,000	F. Meter: 27 gal	73			103		
14			44			74			104		
15	6,000		45			75			105		
16	8,000		46			76			106		
17		-25"	47	3,000	-17"	77			107		
18			48		-21.5"	78			108		
19		End: -28"	49			79			109		
20	13,000	Start: -7.5"	50		End: -25"	80			110		
21		F. Meter: 40.5 gal	51	11,600	Start: -6.25"	81			111		
22	8,000		52			82			112		
23		-12"	53		-13"	83			113		
24			54	23,000	-19.25"	84			114		
25	10,000	-16"	55	23,000		85			115		
26			56	25,000	End: -26"	86			116		
27			57		F. Meter: 28 gal	87			117		
28		-21"	58			88			118		
29			59			89			119		
30		End:-24.5"	60			90			120		

NOTES

1. Pile length measured from working grade
2. Contractor pre-augered about 33 vertical feet with 2-foot diameter auger (about 3 feet below proposed invert elevation).
3. Theoretical and actual grout volumes based on length of grouted pile below grout reservoir.
4. Grout mix consists of 94 lb bag to 5 gal of water.
5. Grout reservoir inner diameter was approximately 24" and was embeded into the ground about 42".
6. Torque values were provided by SECA during pile installation.
7. Pile encountered torque refusal at 25,000 ft-lbs. Tell Tales No. 1 and 2 installed 13 ft and 28 ft below working grade.

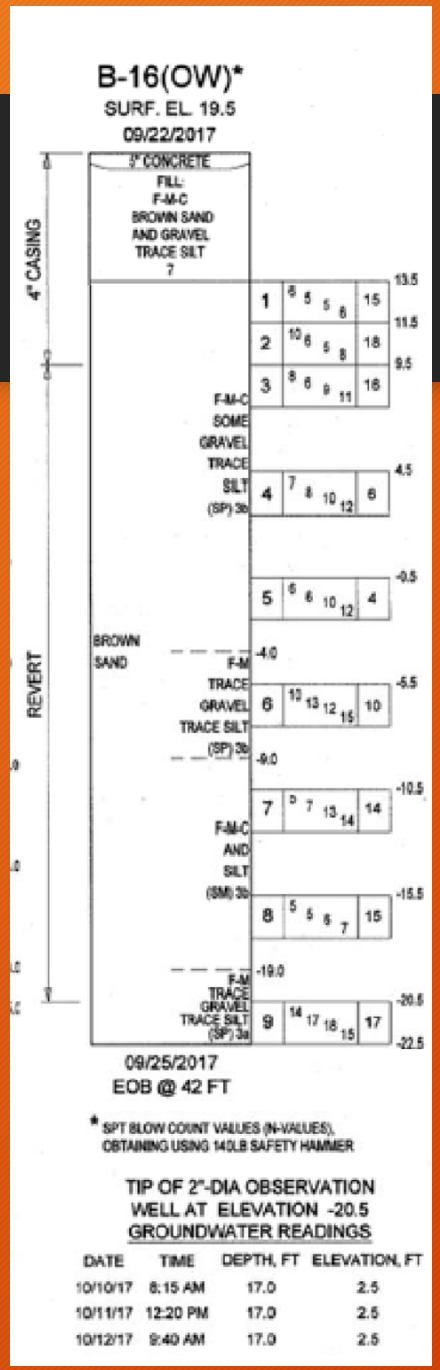
Time Start: 11:40

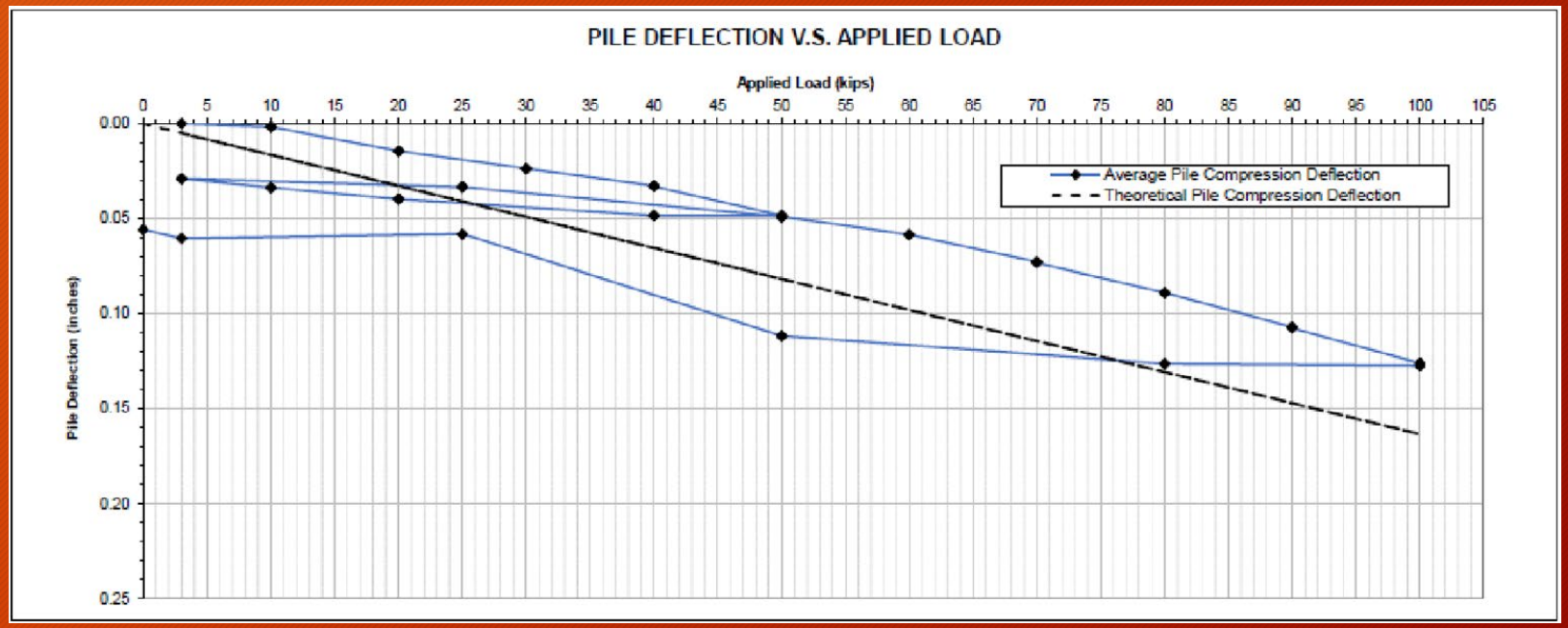
Time End: 12:40

Theoretical Grout Volume: 22.8 ft³

Actual Grout Volume: 24.3 ft³

Rockaway Project: 56’ Pile





Drivecast Compression Test: Load-Deflection Graph

Nearest Boring Log



Questions?

Thank You

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Bond Values of Screw-In Displacement Piles

Presentation By: Jason Herron, P.E., C.W.I.

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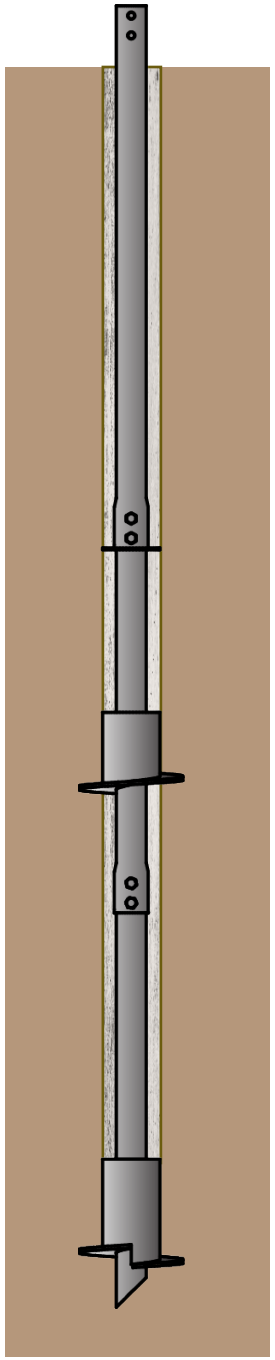
Definition

Screw-In Grouted Displacement Pile

- Foundation element installed by rotating a helical-shaped screw or flights into the ground.
- The process pushes or “displaces” the surround soil laterally and downward.
- The voids created by the “displacement elements is filled with structural grout or concrete.



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Advantages

Screw-In Grouted Displacement Pile

- Spoils Management
  - Minimal to No Spoils
  - Reduces Clean-Up
  - Reduces Disposal Costs
  - Reduces Environmental Impact
- Corrosion Resistance
- Medium to High Loading Capacity
  - Loads of 200 to 1000 kips
- Quick Installation
  - 10 to 20 Piles Per Crew/Per Day



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Installation and Test Considerations

- Installation
  - Length
  - Effective Torque
  - Grout Take Efficiency

$$\frac{\text{Measured Grout Volume}}{\text{Theoretical Grout Volume}} \text{ X } 100\%$$

- Axial Capacity
  - Max Applied Load
  - Load at PL/AE+0.4”, ½”, and 1”
  - Frictional Bond at PL/AE+0.4, ½, and 1”



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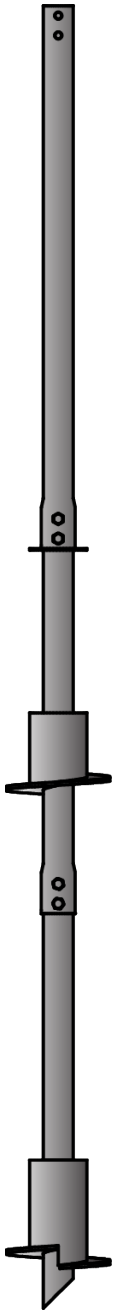
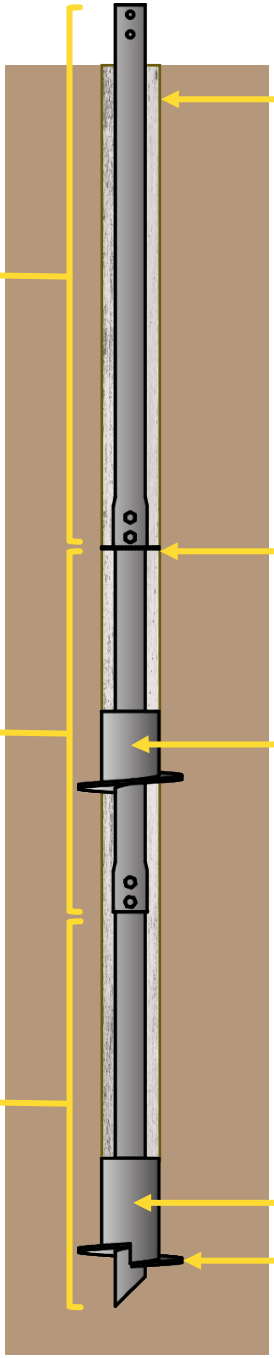
Pile Types and Configurations

- **4” Structural Tubing x 8.63” OD**
  - NPS Schedule 80
  - Lead Displacement Section – 5 FT Long x 8.63” OD Displacement W/ (1) 16” Screw
  - Extension Section – 5 FT Long x 8.63” OD Displacement W/ (1) 16” Screw
  - Plane Extensions with Centralizing Plates – To Depth
- **2-1/4” Round Corner Square (RCS) Bar x 8.63” OD**
  - 2-1/4” RCS Solid Steel Bar
  - Lead Displacement Section – 5 FT Long x 8.63” OD Displacement W/ (1) 16” Screw
  - Extension Section – 5 FT Long x 8.63” OD Displacement W/ (1) 16” Screw
  - Plane Extensions with Centralizing Plates – To Depth
- **6” Structural Tubing x 12.75” OD**
  - NPS Schedule 40
  - Lead Displacement Section – 5 FT Long x 12.75” OD Displacement W/ (1) 20” Screw
  - Extension Section – 5 FT Long x 12.75” OD Displacement W/ (1) 20” Screw
  - Plane Extensions with Centralizing Plates – To Depth





Grout Column

Plain Extension

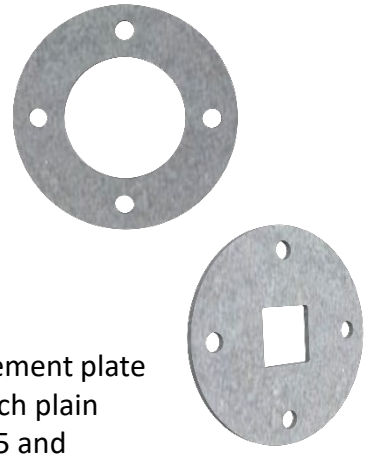
Extension Displacement Plate

Displacement Extension

Lead Section

Displacement Casings

Helix



Note: One displacement plate is added before each plain extension on SS225 and RS4500 piles

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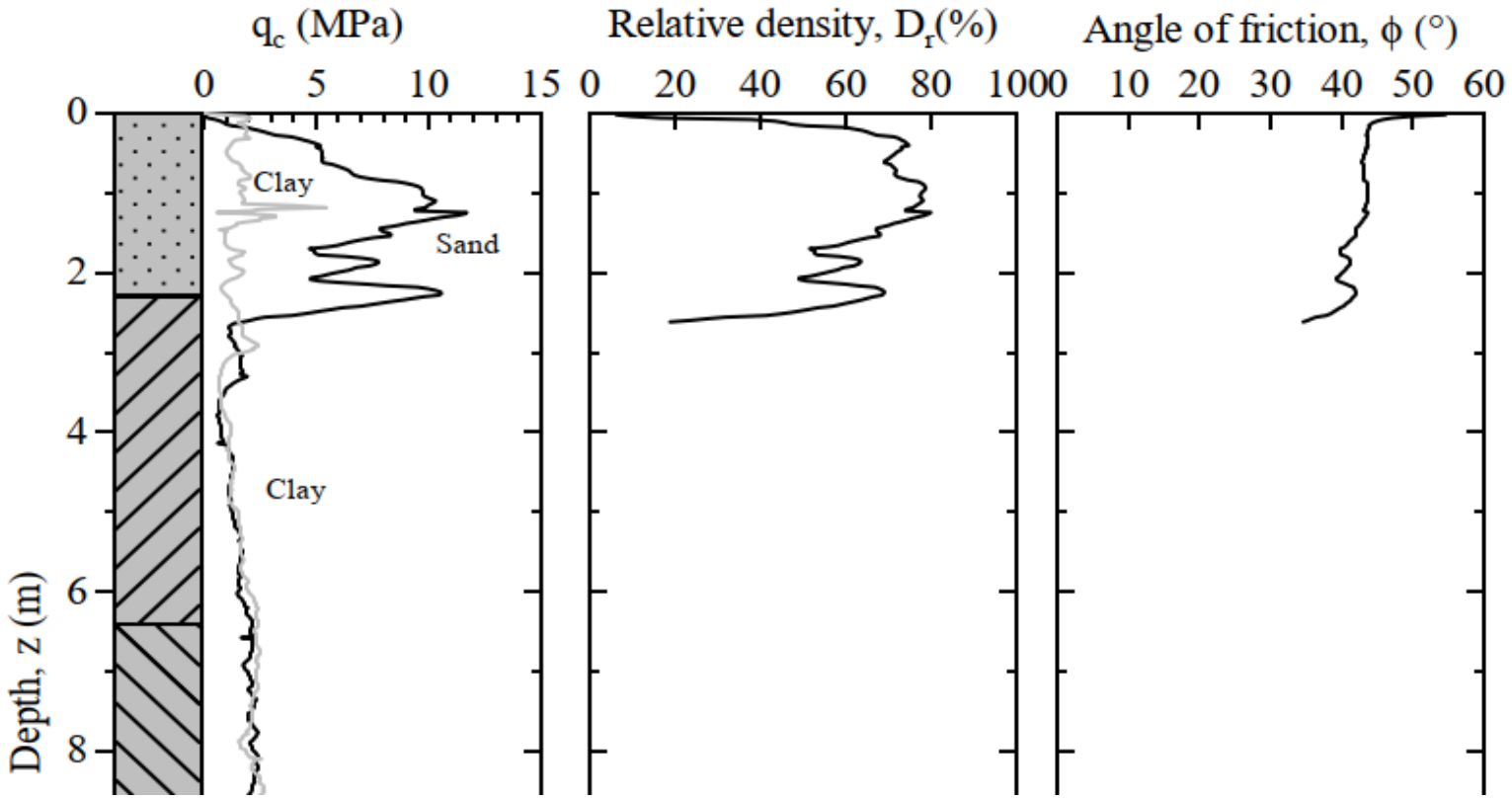
Test Locations: Non-Cohesive (Sand) Sites

- Location #1
  - Centralia, MO – “Sand Pit”
  - Test Date: May 5th, 2025
  - General Properties
    - Medium-Dense to Dense, Well Graded Sand Fill
- Location #2
  - Lakeville, MN
  - Test Date: October 4, 2024
  - General Properties
    - Medium-Dense, Poorly Graded Sand
    - Fill and Native



Location 1 - Centralia, MO Test Site

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STRATUM DEPTH		SAMPLE DEPTH		SAMPLE NUMBER	SAMPLE TYPE	FIELD RESULTS		STRATUM DESCRIPTION
FT.	ELEV.	FROM FT.	TO FT.			Rec"	N-Value	
0.0	0.0	0.0	1.5	1	Grab	18		18" Silty Sand with little Gravel, brown, dry (No topsoil)
2.5	-2.5	2.0	3.5	2	SPT	8	8	FILL, Clayey Sand, trace gravel, brown, moist Poorly Graded Sand, brown, moist
5.0	-5.0	4.5	6.0	3	SPT	9	10	FILL, Poorly Graded Sand with Clay, brown, moist
7.5	-7.5	7.0	8.5	4	SPT	7	10	FILL, Poorly Graded Sand with pieces of Clay, iron oxide staining, brown, moist
10.0	-10.0	9.5	11.0	5	SPT	11	11	(SP) POORLY GRADED SAND WITH TRACE GRAVEL, fine to medium grained, brown, wet, medium dense.
12.5	-12.5	12	13.5	6	SPT	11	12	(SP) POORLY GRADED SAND WITH TRACE GRAVEL, fine to medium grained, brown, wet, medium dense.
15.0	-15.0	14.5	16.0	7	SPT	14	13	(SP) POORLY GRADED SAND WITH TRACE GRAVEL, fine to medium grained, brown, wet, medium dense.

Location 1 – Lakeville, MN

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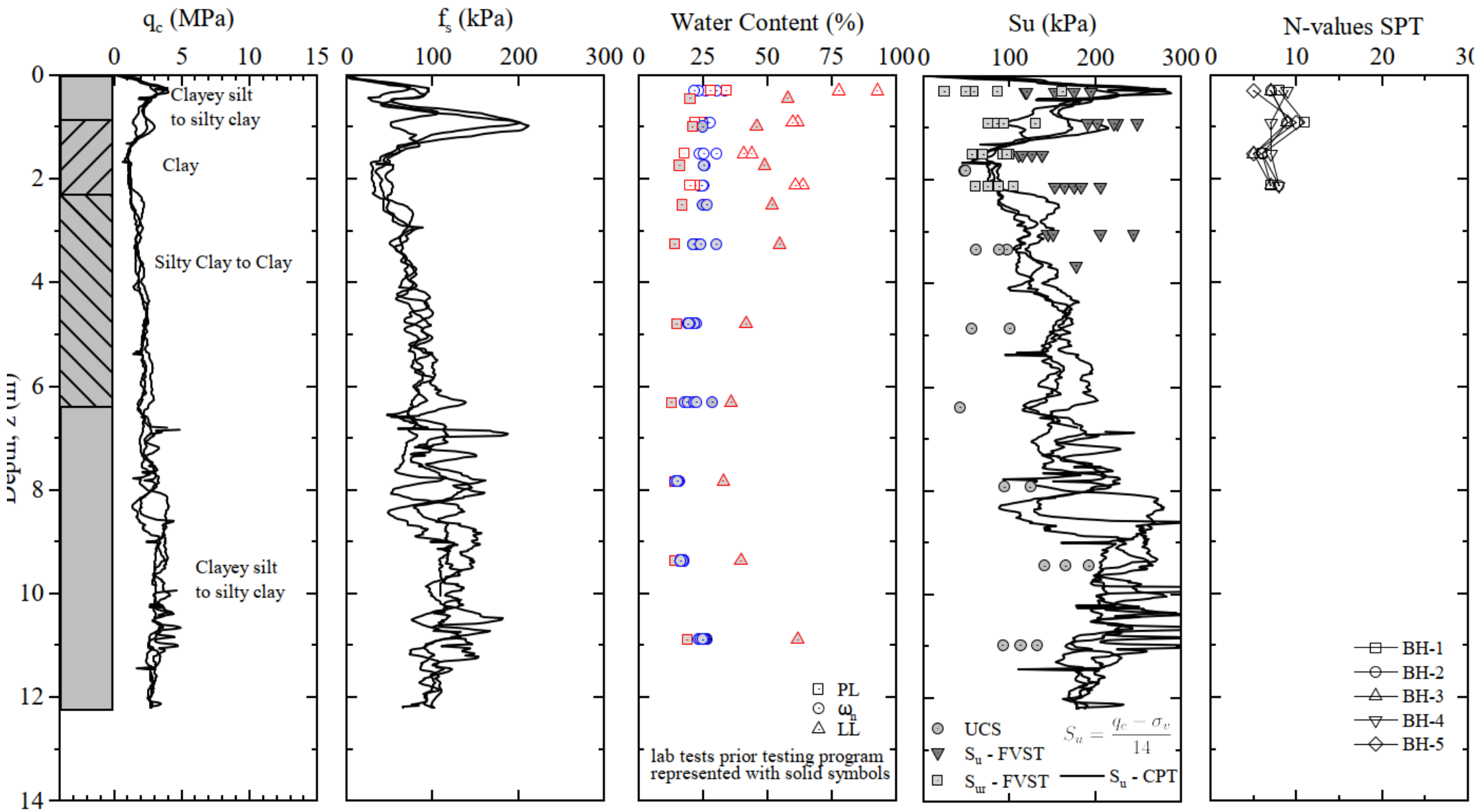
Test Locations: Cohesive Site

- Location
  - Centralia, MO
  - Test Dates: October 23, 2023 - June 24, 2025
  - General Properties:
    - Medium-Stiff Clay Transitioning to Stiff





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Test Procedure

- Tension Test
  - Loaded in accordance with ASTM D3689 – Standard Test Methods for Deep Foundations Under Static Axial Tensile Load.
    - Exception that wood cribbing spacing was closer than ASTM D3689 requires.
  - Loaded in a stepwise fashion with 4-minute hold times.
- Compression Tests
  - Loaded in accordance with ASTM D1143 Standard Test Methods for Deep Foundations Under Static Compressive Loads
  - Loaded in a stepwise fashion with 4-minute hold times.



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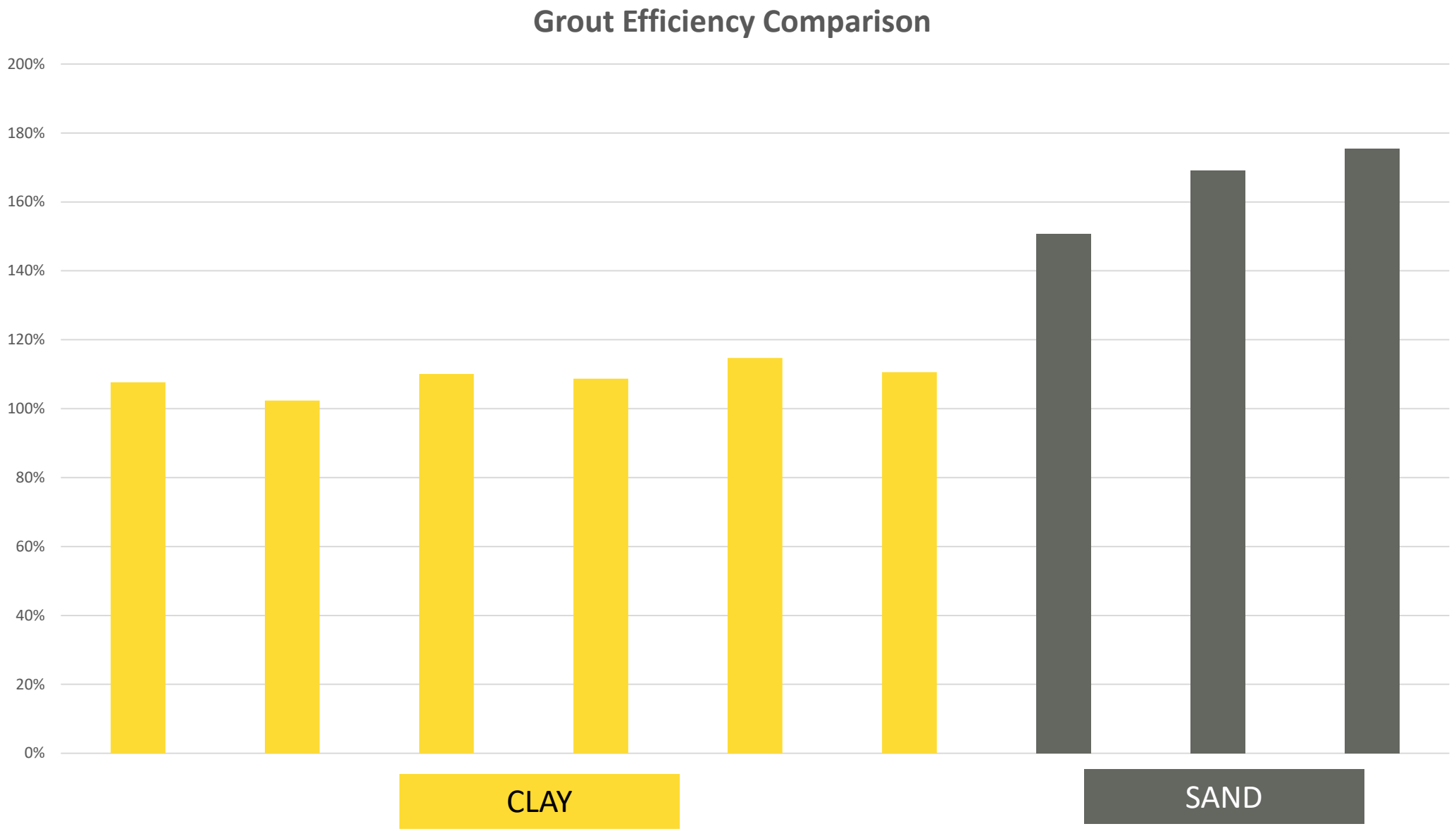
Summary of Piles Installed

Test Number	Sample Type	Test Location	Soil Type	Helix Diameters (in.)	Overall Length (ft)	Grouted Shaft Dia. (in.)
1	RS4500.337 x 8.63 O.D.	Centralia, MO	Clay	16,16	33	8.63
2	RS4500.337 x 8.63 O.D.	Centralia, MO	Clay	16,16	22	8.63
3	RS4500.337 x 8.63 O.D.	Lakeville, MN	Sand	16,16	16.5	8.63
4	RS4500.337 x 8.63 O.D.	Lakeville, MN	Sand	16,16	12	8.63
5	SS225 x 8.63 O.D.	Centralia, MO	Clay	16,16	20	8.63
6	SS225 x 8.63 O.D.	Centralia, MO	Clay	16,16	32.5	8.63
7	SS225 x 8.63 O.D.	Centralia, MO	Sand	16,16	6	8.63
8	RS6625.280 x 12.75 O.D.	Centralia, MO	Clay	20,20	25	12.75

Summary of Installation

Test Number	Sample Type	Installation Date	Grouted Shaft Length (ft)	Effective Torque (ft-lb)	Grout Efficiency	Average Installation Rotations Per Foot
1	RS4500.337 x 8.63 O.D.	9/15/2023	31.5	14,800	107%	NA
2	RS4500.337 x 8.63 O.D.	12/14/2023	20.5	10,800	102%	4.00
3	RS4500.337 x 8.63 O.D.	9/24/2024	15	24,200	150%	6.50
4	RS4500.337 x 8.63 O.D.	9/24/2024	10.5	20,100	169%	8.25
5	SS225 x 8.63 O.D.	4/16/2025	18.5	9,600	114%	4.17
6	SS225 x 8.63 O.D.	4/16/2025	31	13,200	110%	4.48
7	SS225 x 8.63 O.D.	4/16/2025	4.5	6,400	175%	4.69
8	RS6625.280 x 12.75 O.D.	9/8/2024	23	22,200	108%	2.68


Grout Efficiency Comparison
200%
180%
160%
140%
120%
100%
80%
60%
40%
20%
0%
CLAY
SAND

Summary of Testing

Test Number	Sample Type	Test Type	Load Max Applied (kips)	1/2" (kips)	Frictional Bond <1/2" (ksf)	PL/AE+0.4 (kips)	Frictional Bond PL/AE+0.4 (ksf)	1" (kips)
1	RS4500.337 x 8.63 O.D.	Compression Clay Centralia, MO	239	218	2.93	239	3.21	NA
2	RS4500.337 x 8.63 O.D.	Compression Clay Centralia, MO	194	183	3.68	186	3.74	191
3	RS4500.337 x 8.63 O.D.	Compression Sand Lakeville, MN	338	180	4.83	206	5.53	255
4	RS4500.337 x 8.63 O.D.	Compression Sand Lakeville, MN	300	142	5.24	145	5.35	197

Summary of Testing

Test Number	Sample Type	Test Type	Load Max Applied (kips)	1/2" (kips)	Frictional Bond <1/2" (ksf)	PL/AE+0.4 (kips)	Frictional Bond PL/AE+0.4 (ksf)	1" (kips)
5	SS225 x 8.63 O.D.	Compression Clay Centralia, MO	159	140	3.10	147	3.26	156
6	SS225 x 8.63 O.D.	Compression Clay Centralia, MO	293	220	3.00	270	3.68	270
7	SS225 x 8.63 O.D.	Tension Sand Pit Centralia, MO	28	28	2.07	NA	NA	NA
8	RS6625.280 x 12.75 O.D.	Compression Clay Centralia, MO	291	246	2.95	246	2.95	277

Bond Comparison

- Clay – Centralia, MO
  - Theoretical Bond Estimate
    - 1 ksf. to 2.6 ksf.
  - Bond from Testing
    - 2.95 ksf. to 3.74 ksf.
- Sand – Lakeville, MN
  - Theoretical Bond Estimate
    - 0.4 ksf. to 1 ksf.
  - Average Bond from Testing
    - 5.44 ksf.

No reduction for effective pile diameter



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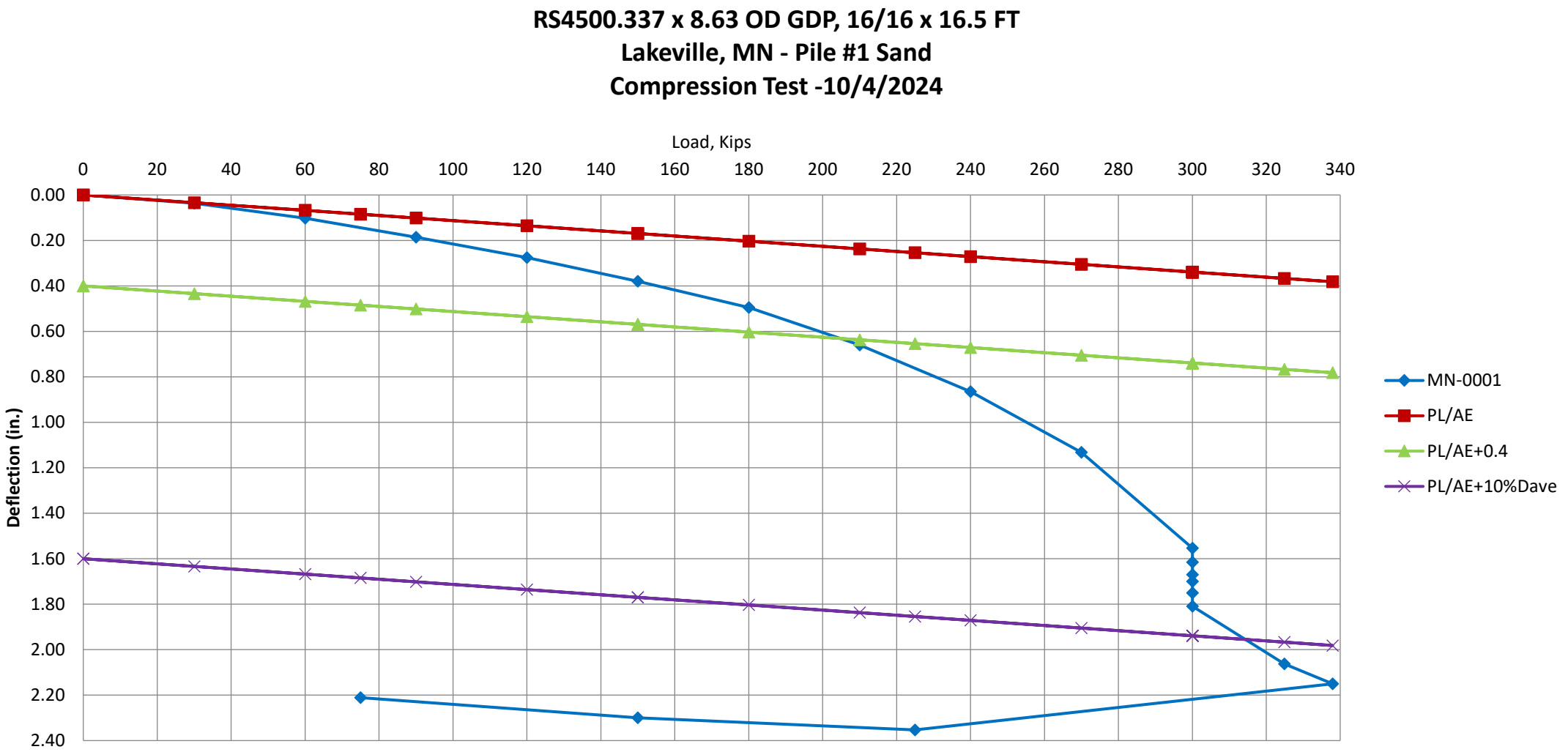
Sand - Sample Load vs. Deflection Plot



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Clay - Sample Load vs. Deflection Plot



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Conclusions

- Bond values are higher than theoretical.
- Grout take efficiency for cohesive soils 100% to 115%
- Grout take efficiency for non-cohesive soils is more than 150%.
- Non-cohesive soils are “mixing” with sand during install.
- Load vs. deflection charts shows most of the pile capacity is derived from shaft friction.



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Questions?



The new Lodos Mixers family: fully automated and easy to use mixers for your grouting project



October 2025

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Lorenzetto Loris Grouting Equipment

What we can supply:

- Mixing unit for cement and bentonite grout
- Agitators for store grout mixture
- Injectors and grout/transfer pumps
- Combi units: container or skid unit customized to customer needs


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AGITATORE L10000
made in Italy / www.lorenzetiloris.it


LS 38/15
LS 38/15


245751
477



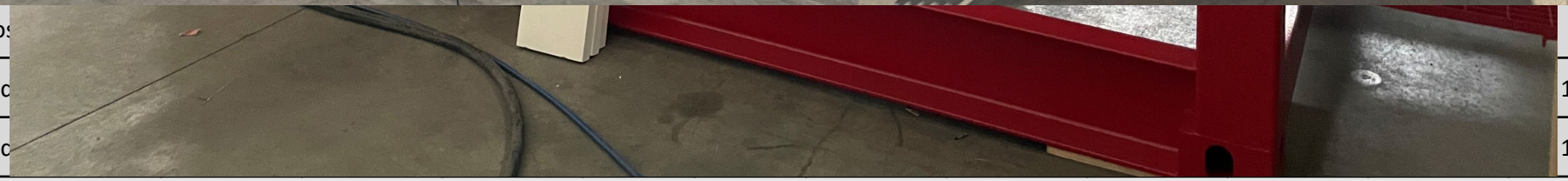



LODOS 15
made in Italy / www.lorenzettoloris.it
AGITATORE L1500
made in Italy / www.lorenzettoloris.it









Lodos		7"	1
Loc		12"	2
Loc		12"	2

The New Lodos Main Electric Panel:



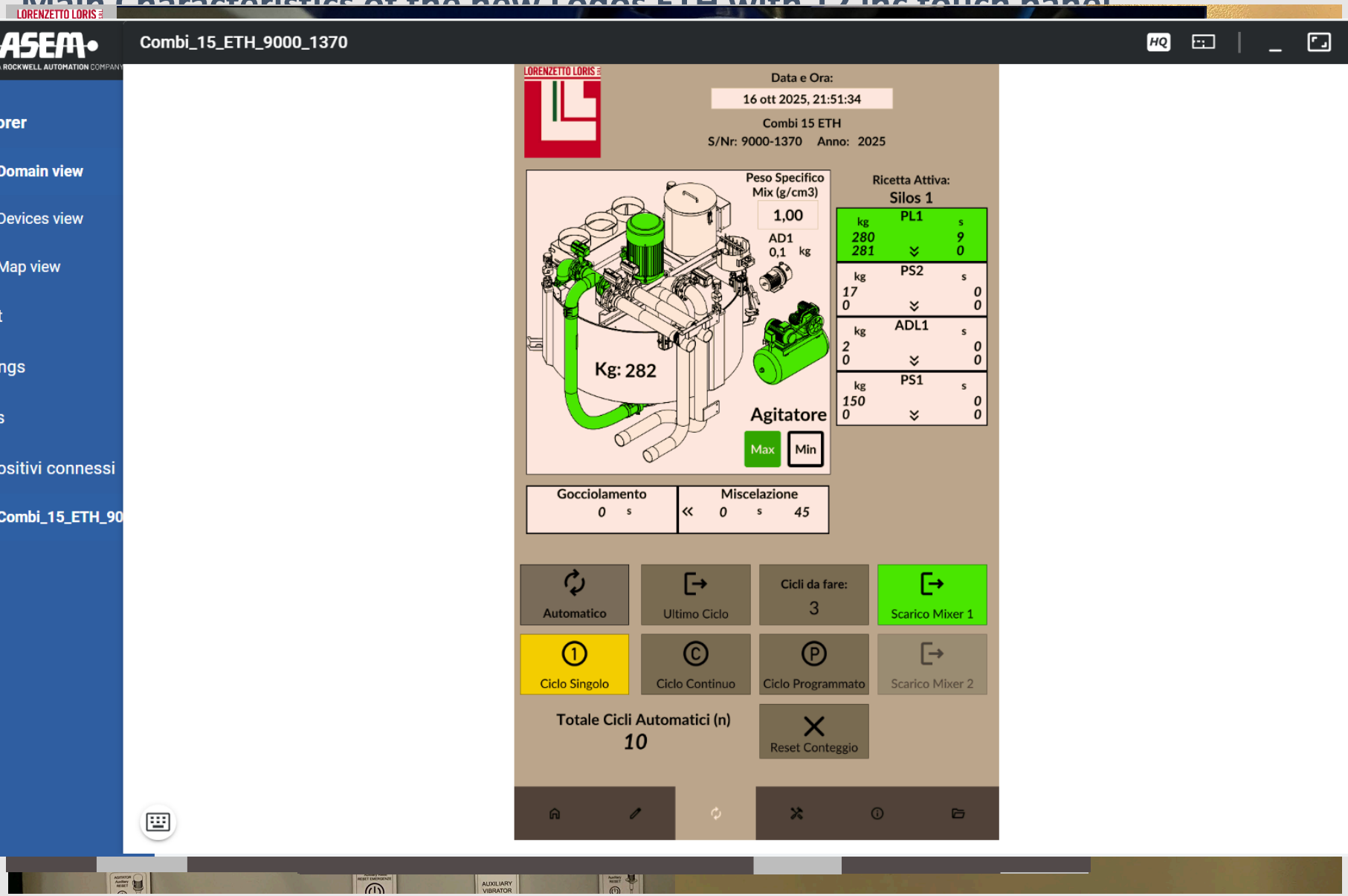
/IN 13

Lodos

displa



Main Characteristics of the new Lodos ETH with 12 inc touch panel



LORENZETTO LORIS S.R.L.

Thanks for your attention!
Come to Visit us at our boot #625!

PARALLEL SESSION A5
Gold Innovations IV

HILLIS-CARNES
ENGINEERING ASSOCIATES

CASE STUDY
EFFECTS OF UNDISCLOSED LARGE DIAMETER SINKHOLE ON CONSTRUCTION CLEARWATER (SW), FLORIDA

Presentation By:
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khiggins@hcea.com

Disclaimer: The opinions expressed in this presentation and on the following slides are solely those of the presenter(s) and not necessary those of DFI. DFI does not guarantee the accuracy of the information nor endorse the recommendations offered by the presenter(s).

CASE STUDY DESCRIPTION

The purpose of this presentation is to present a Case History of a recent Apartment complex project in Clearwater, SW FL where a previously undisclosed, large diameter sinkhole significantly impacted overlying construction. We will explain how the sinkhole was ultimately discovered during construction, what impacts the sinkhole had on the overlying construction and how we collaboratively dealt with the challenging problems.

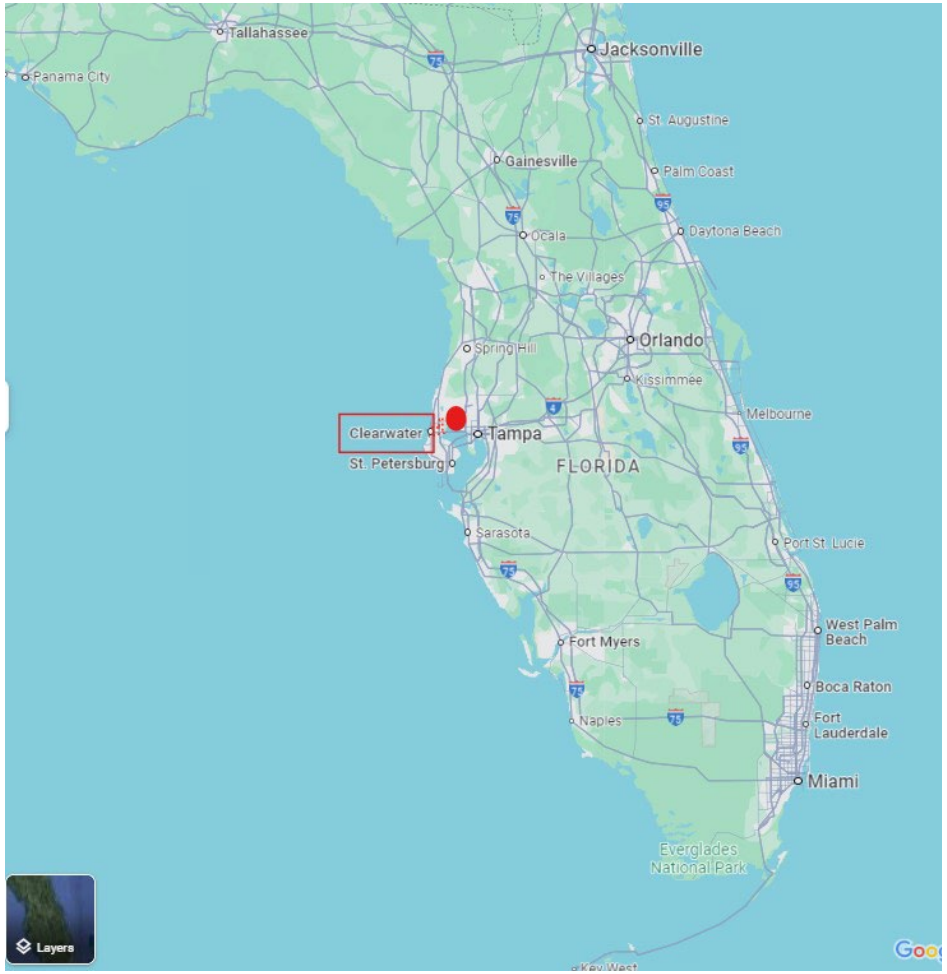
Disclaimer: Hillis Carnes Engineering Assoc. was not involved in this subject project. The case study was a project the Presenter worked on while employed with a different company.

OBJECTIVES

CHRONOLOGY

1. SHORT INTRODUCTION TO PROJECT
2. OVERVIEW OF INITIAL SUBSURFACE EXPLORATIONS
3. START OF CONSRUCTION/INITIAL CONCERNS ABOUT EXCESSIVE RETAINING WALL SETTLEMENT
4. DISCOVERY OF LARGE DIAMETER, COVER COLLAPSE SINKHOLE
5. RECONSTRUCTION OF MSE WALL (INITIAL REMEDIATION EFFORT)
6. INITIAL REMEDIATION EFFORTS FAILED – WHAT WE DID NEXT

SITE LOCATION – CLEARWATER FLORIDA THE SUNSHINE STATE!

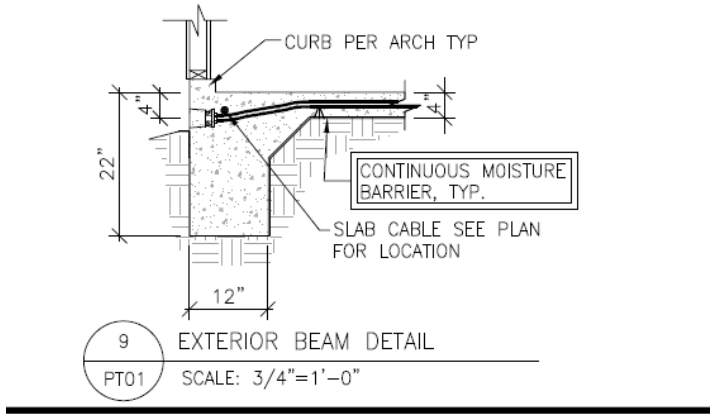


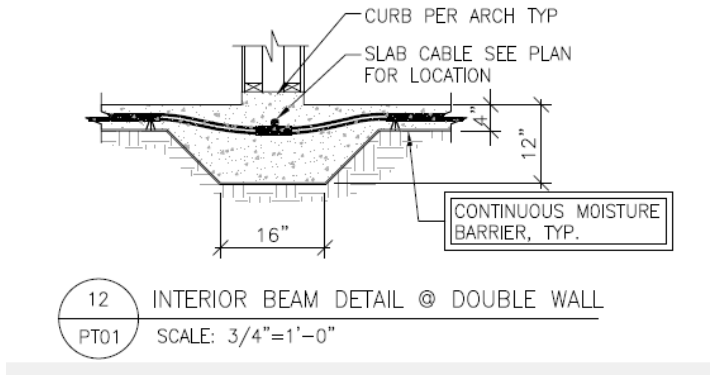


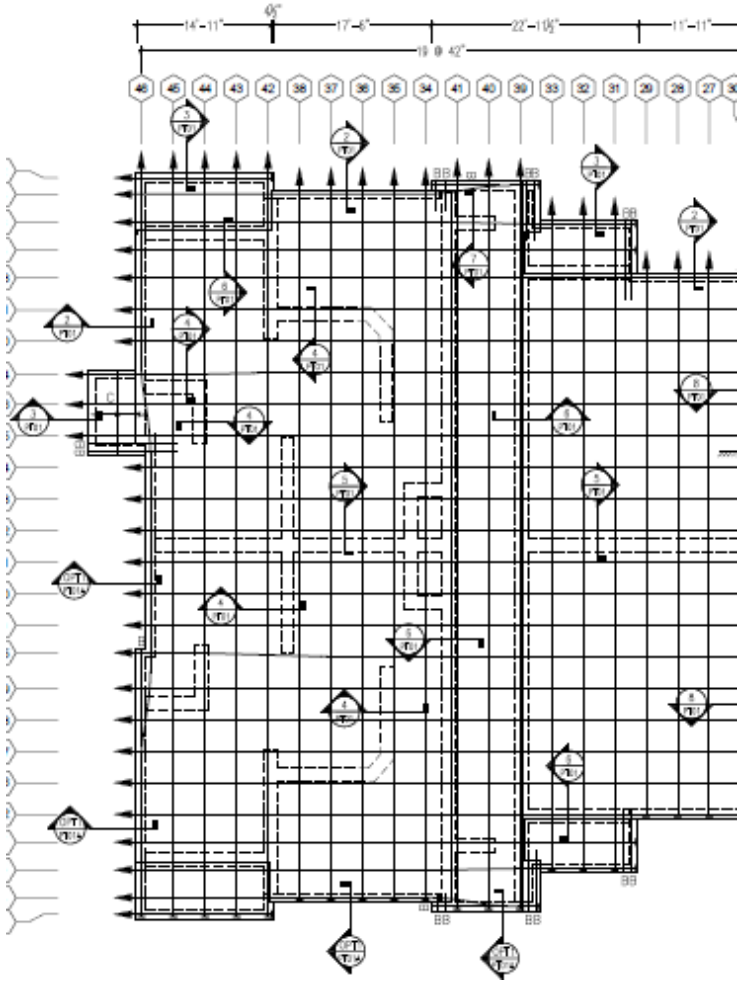
Fairly Typical SW FL Suburban/Urban Parcel

LARGE DIAMETER SINKHOLE SW FL

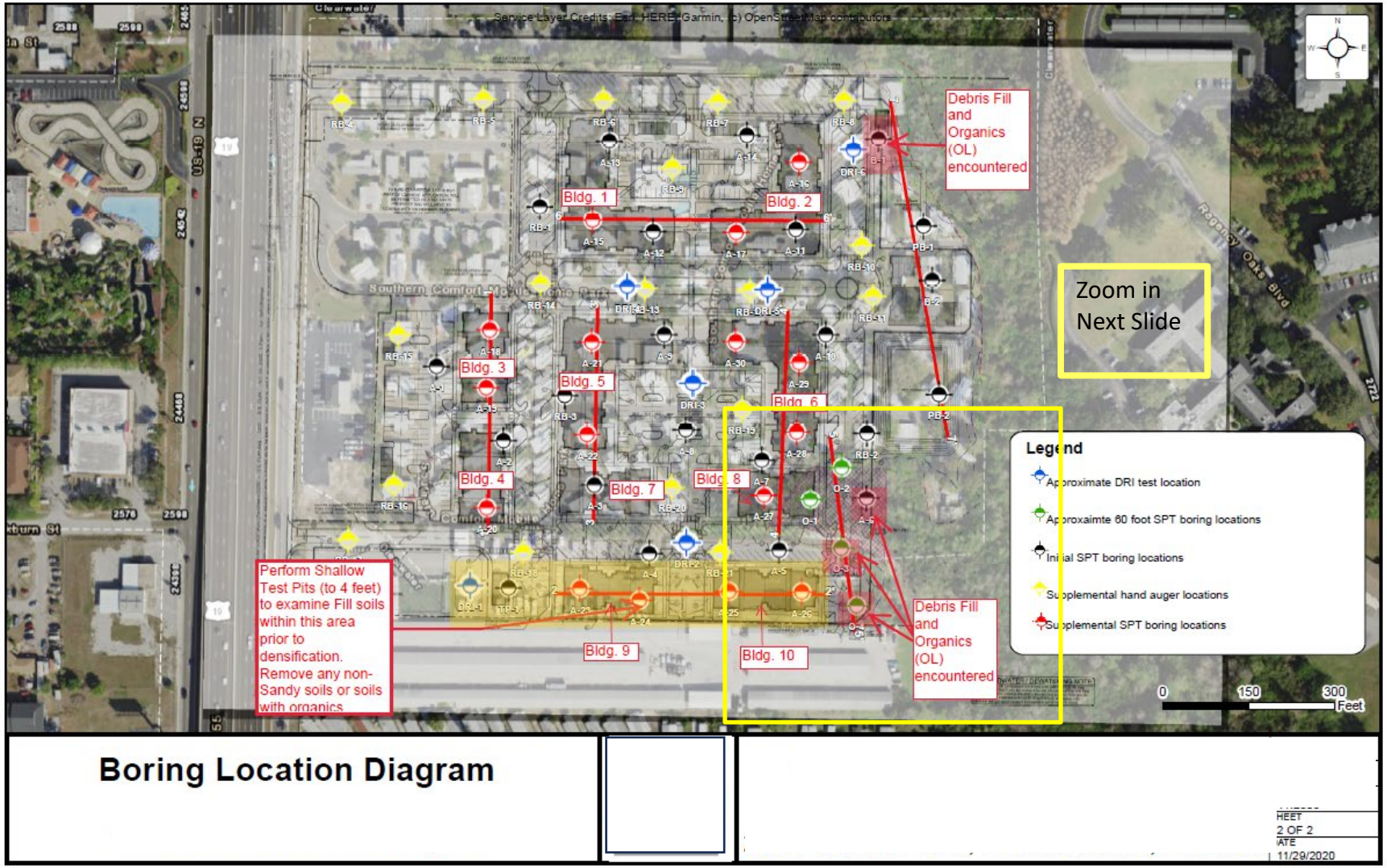
TYPICAL 3-STORY APARTMENT COMPLEX

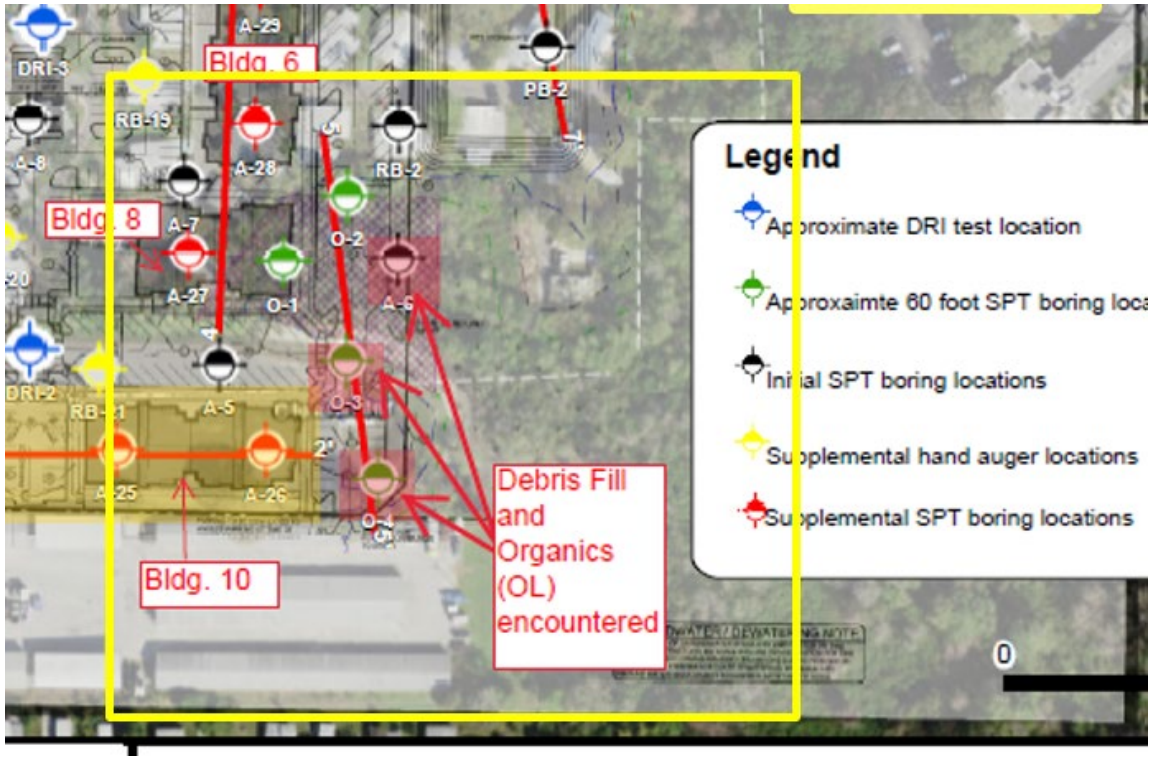






At-Grade Post Tensioned Shallow Foundation

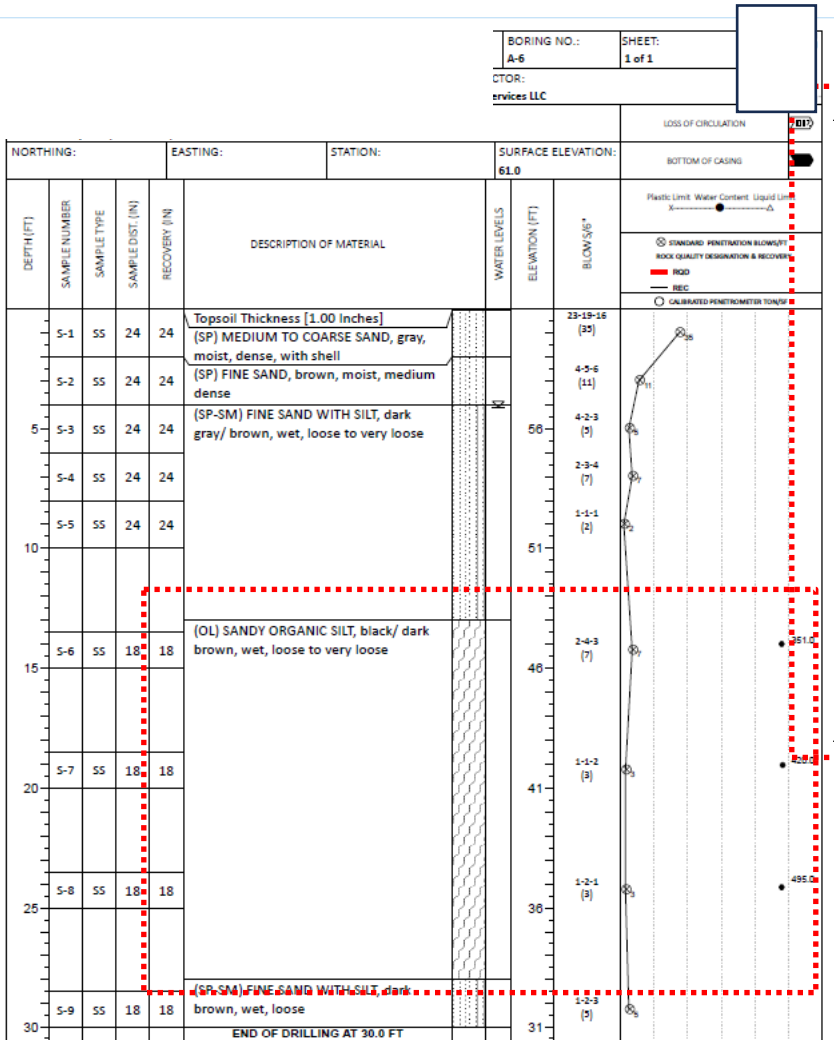

Debris Fill and Organics (OL) encountered
Bldg. 1
Bldg. 2
Zoom in Next Slide
Bldg. 3
Bldg. 5
Bldg. 6
Bldg. 4
Bldg. 7
Bldg. 8
Perform Shallow Test Pits (to 4 feet) to examine Fill soils within this area prior to densification. Remove any non-Sandy soils or soils with organics
Bldg. 9
Bldg. 10
Debris Fill and Organics (OL) encountered
Legend
Approximate DRI test location
Approximte 60 foot SPT boring locations
Initial SPT boring locations
Supplemental hand auger locations
Supplemental SPT boring locations
0
150
300
Feet
Boring Location Diagram
SHEET
2 OF 2
DATE
11/29/2020



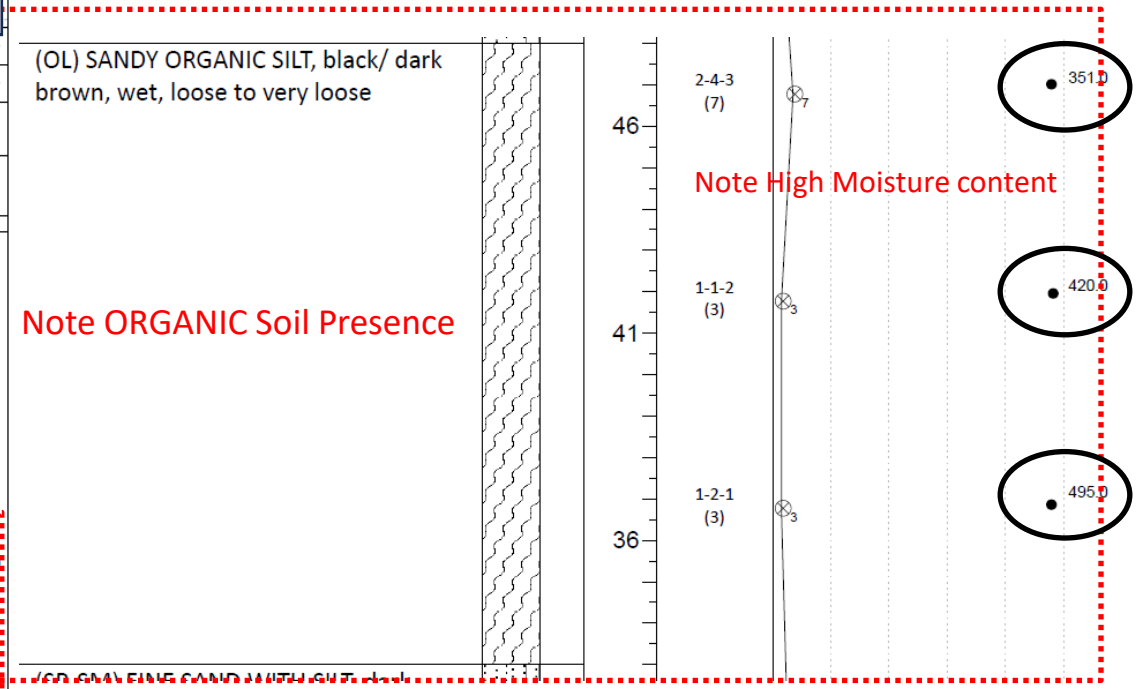
Focusing on SE corner of Parcel, Pads 6, 8, 10

- We did find SHALLOW and somewhat deeper organic soils and debris in this area, prompting a second round of borings and test pits and site reconfiguration.
- Building Pads were shifted westward to avoid organic soils and wetlands (away from Boring A-6, O-3).
- Shallow Debris and Organics were undercut in pad areas.

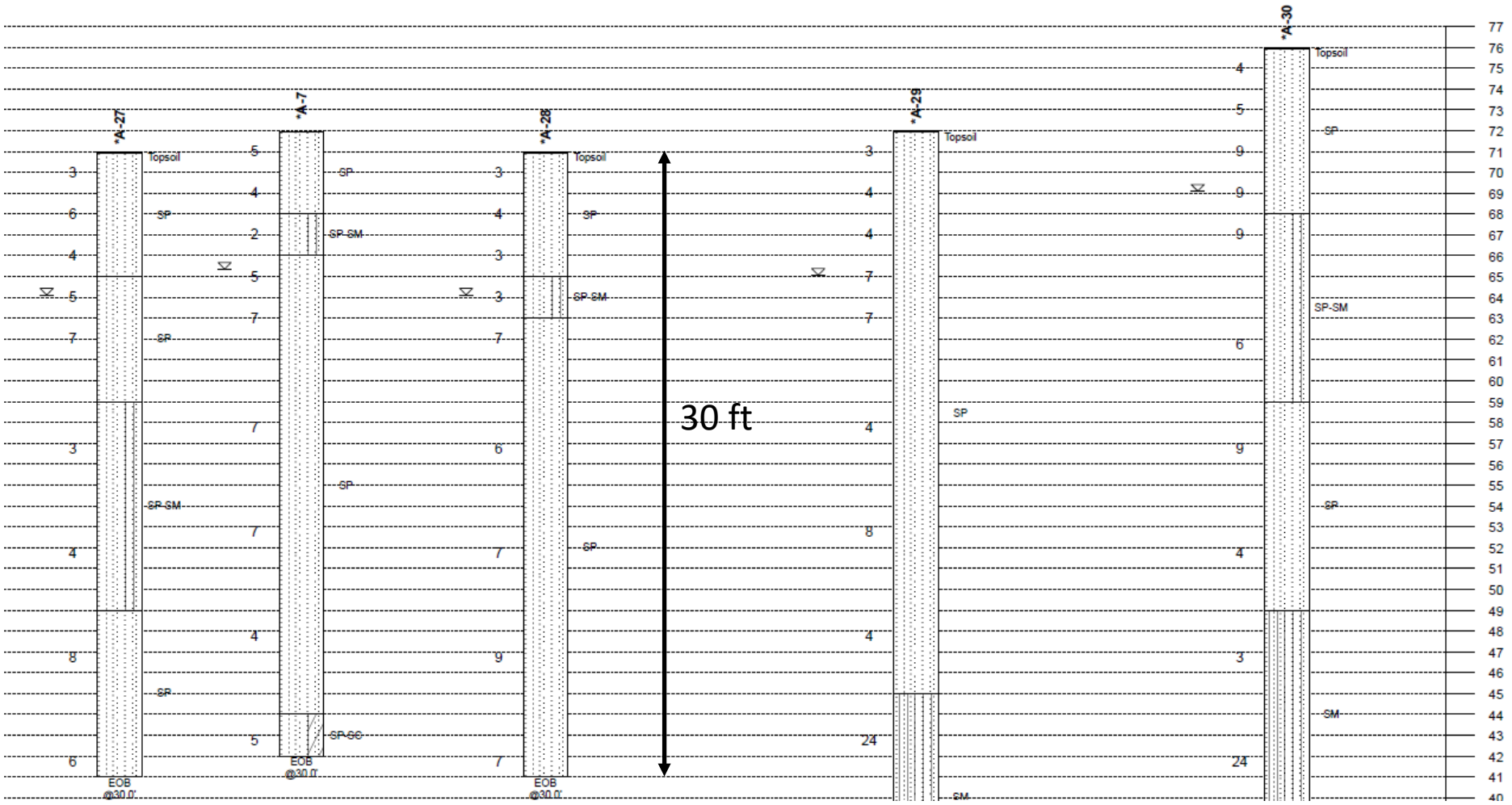
Boring A-6



Enlargement



Example Problematic Boring that prompted site plan shifting



Vast Majority of Borings showed only very loose to medium dense SAND which is a non-problem soil

FINAL GEOTECHNICAL RECOMMENDATIONS

1. SHIFT EASTWARD MOST BUILDINGS WESTWARD AWAY FROM DEEPER ORGANIC SOILS (A-6, O-3, O-4) - DONE
2. UNDERCUT SHALLOW FILL AND ORGANICS SOUTH PROPERTY AREA (Bldg. 9 and 10) - DONE
3. GEOTECHNICAL ENGINEER RECOMMENDED SHALLOW FOUNDATIONS SINCE NO KARST/SINHOLE CONCERNS WERE OBVIOUS

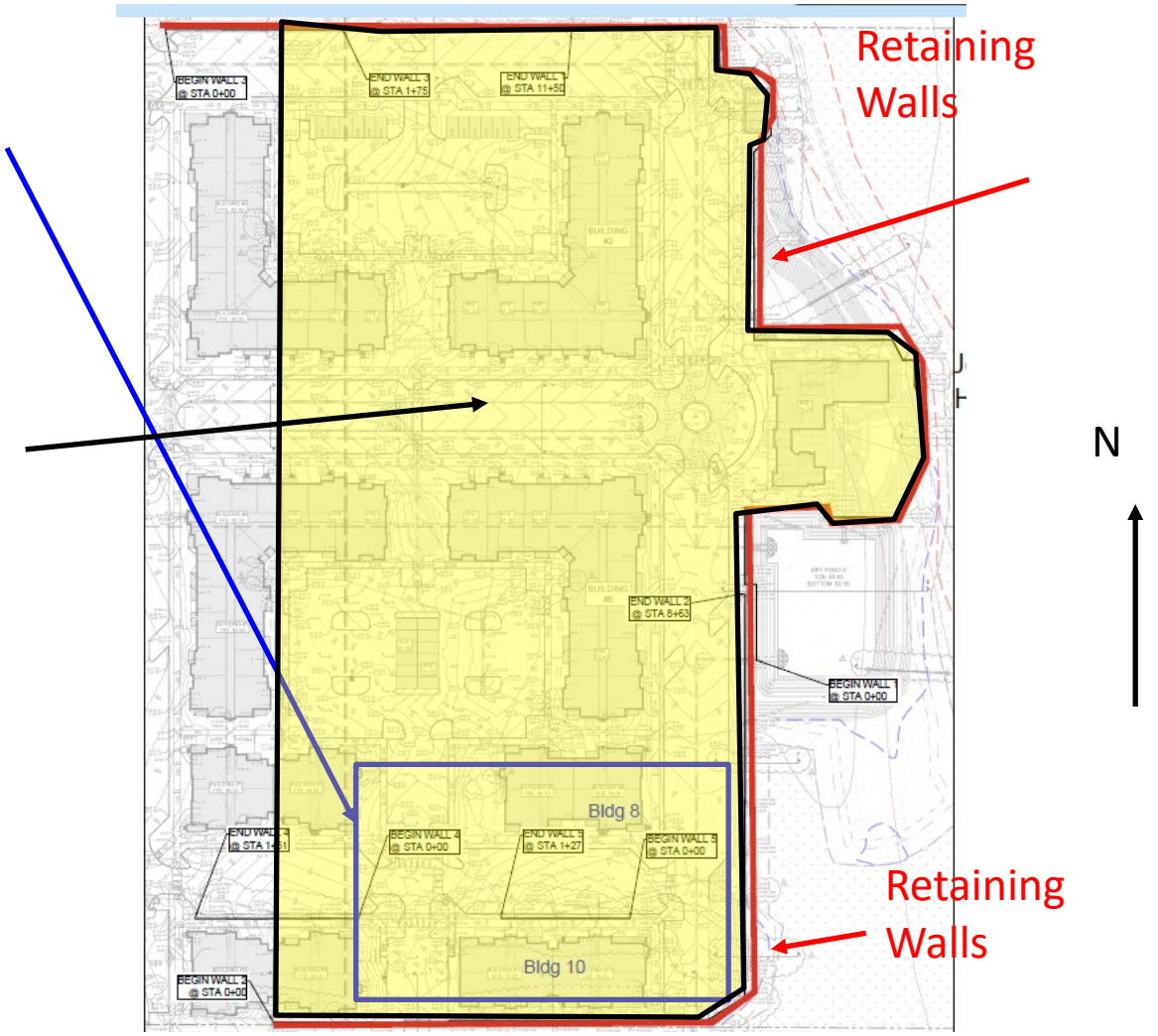
In summary, after drilling closely spaced borings, it appeared as a fairly typical SW Florida building site. Construction soon began in late 2021.

START OF CONSTRUCTION DEC. 2021

- Shallow fills and organics were removed during mass grading (southern area).
- What surprised the Geotechnical Engineer was the amount of new FILL (+2 to +18 ft) being brought to site to raise grades, most significantly on the west half of the parcel (Bldg. 8, 10 area).

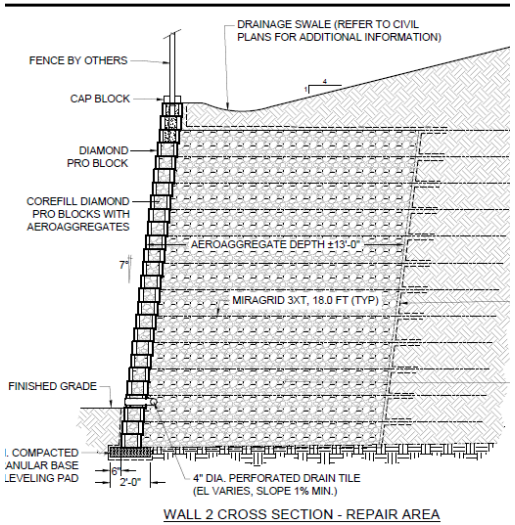
Pro tip: grading expectations in geotechnical reports is very important (it can significantly affect boring depth needs).

- This significant filling resulted in a need for a new MSE wall along the entire eastern border and significant southern and northern site border portions.

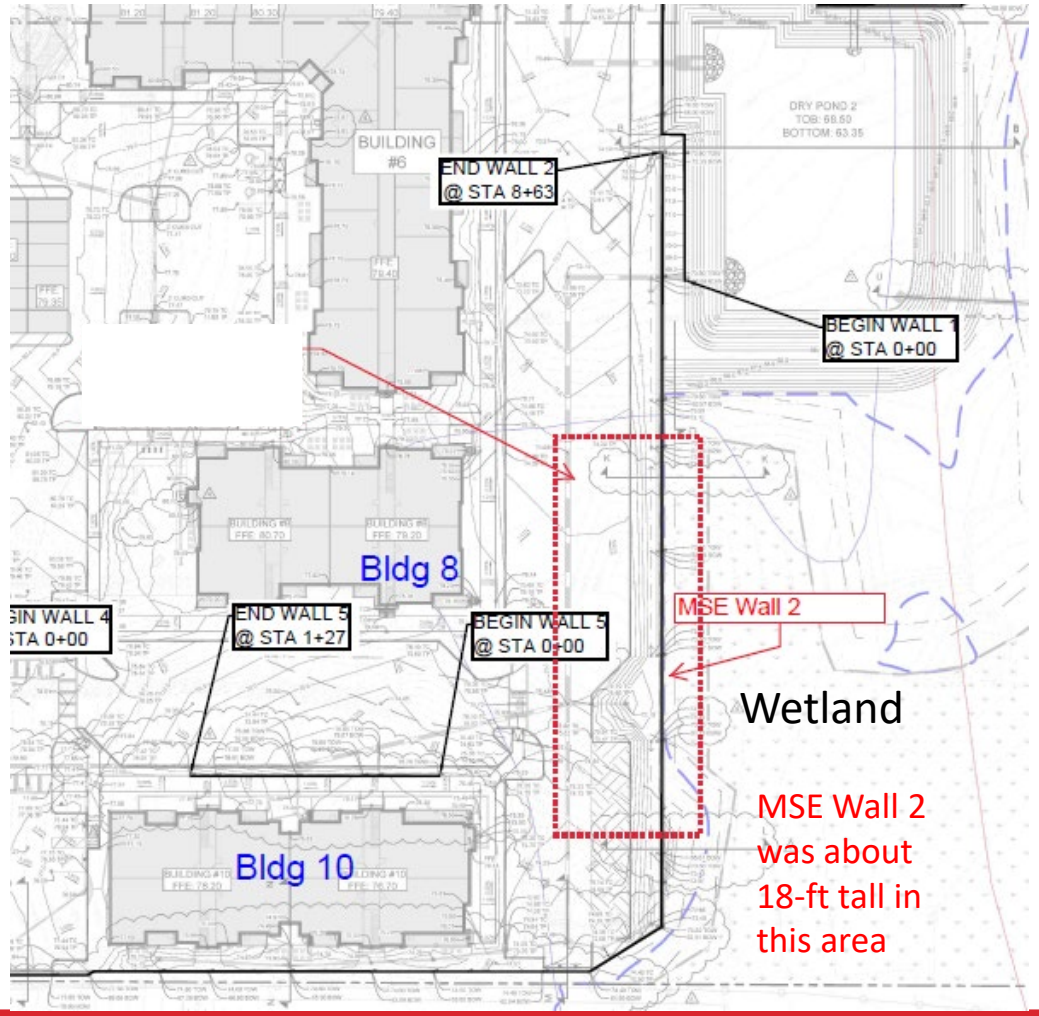


START OF CONSTRUCTION DEC. 2021

While Buildings 8 and 10 were shifted westward at the Engineer's recommendation, we were unaware of client's intent to build a large MSE wall in the area adjacent to the wetland and where we found deeper organic soils (Boring A-6). Walls were needed because of large grade changes.



WALL 2 CROSS SECTION - REPAIR AREA



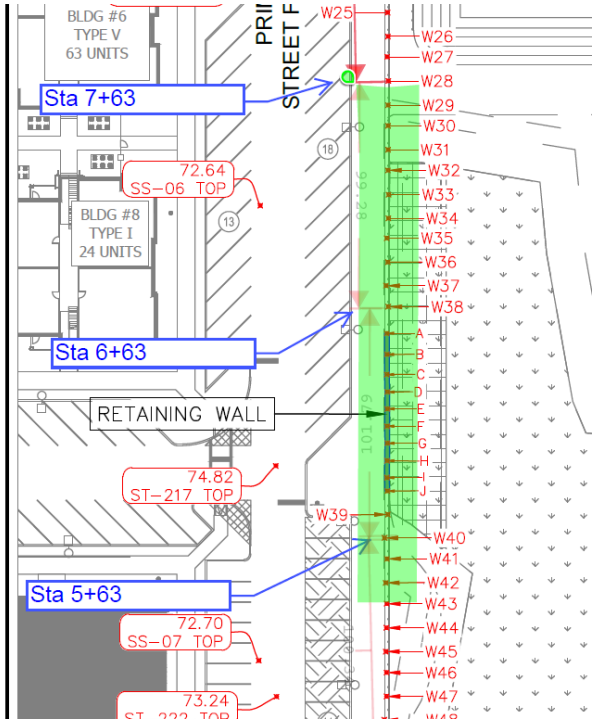
LARGE DIAMETER SINKHOLE SW FL

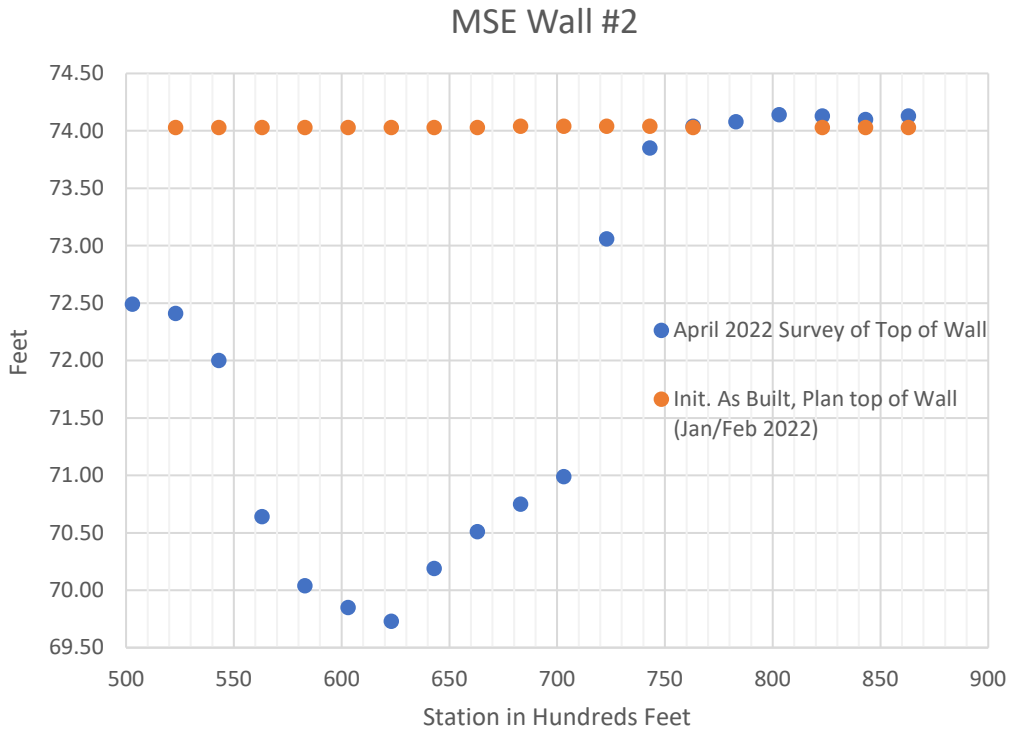
MARCH 2022

Client observed significant portion of MSE Wall 2 adjacent to Bldg. 8 had moved downward noticeably.

a) April 2022, we started periodic survey monitoring of MSE Wall 2. Three (3) to four (4) feet of downward movement was measured in affected area of Wall 2 from Jan. 2022 to April 2022.

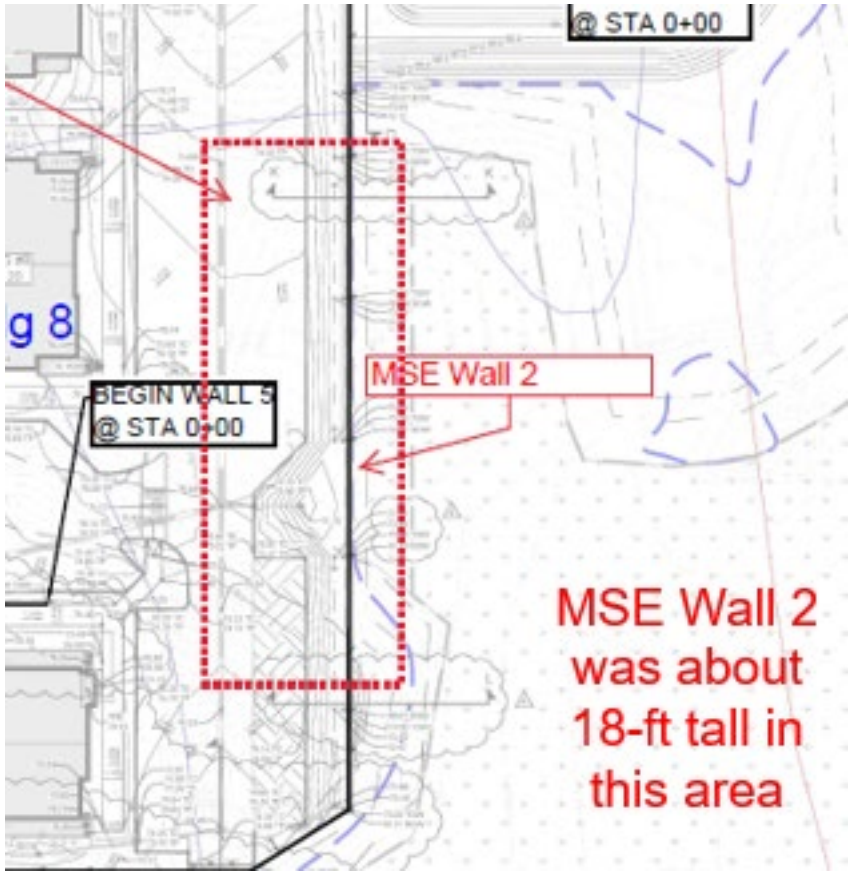
b) Between April 2022 and August 2022, downward movement continued now totaling 4 to 5.5 feet. Survey Plot below.





LARGE DIAMETER SINKHOLE SW FL

MSE WALL 2 APRIL 2022



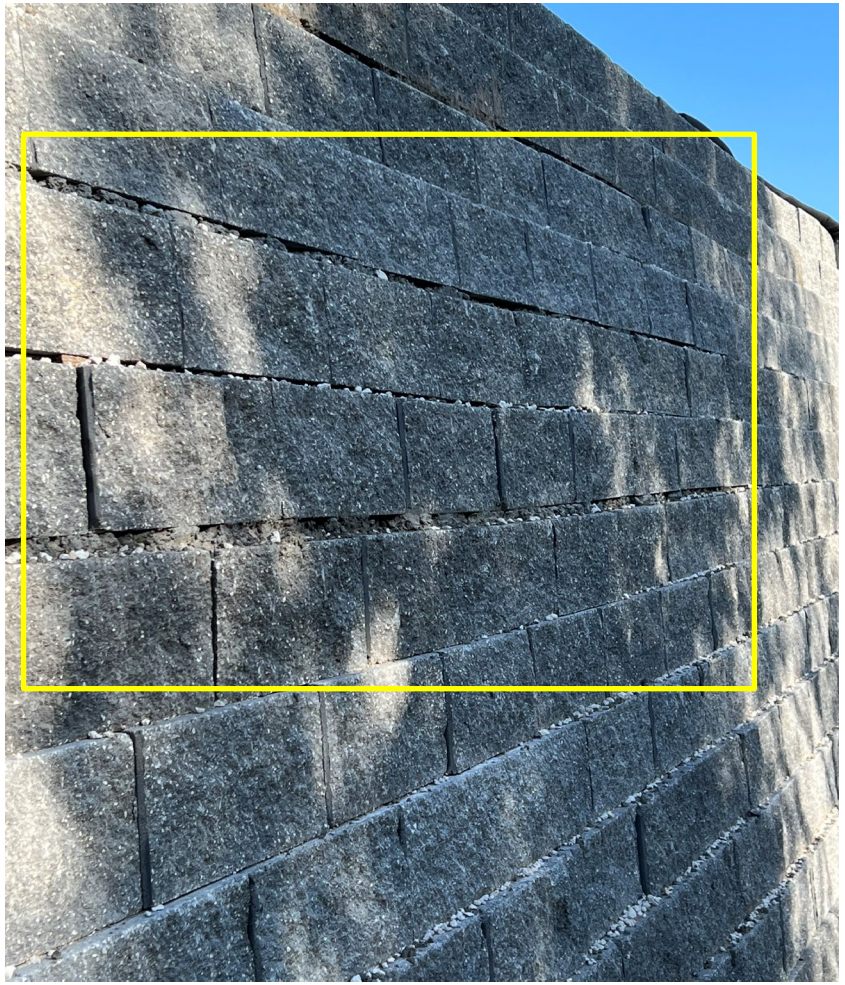


LARGE DIAMETER SINKHOLE SW FL

MSE WALL 2 APRIL 2022

Concerns Observed this date:

1. Separation of MSE blocks followed by soil infilling.
2. Loss of soil through the MSE face. A condition that will worsen.
3. Some blocks fractured/cracked due to excessive face of wall stress.
4. Eventually this wall would fail if not stabilized.



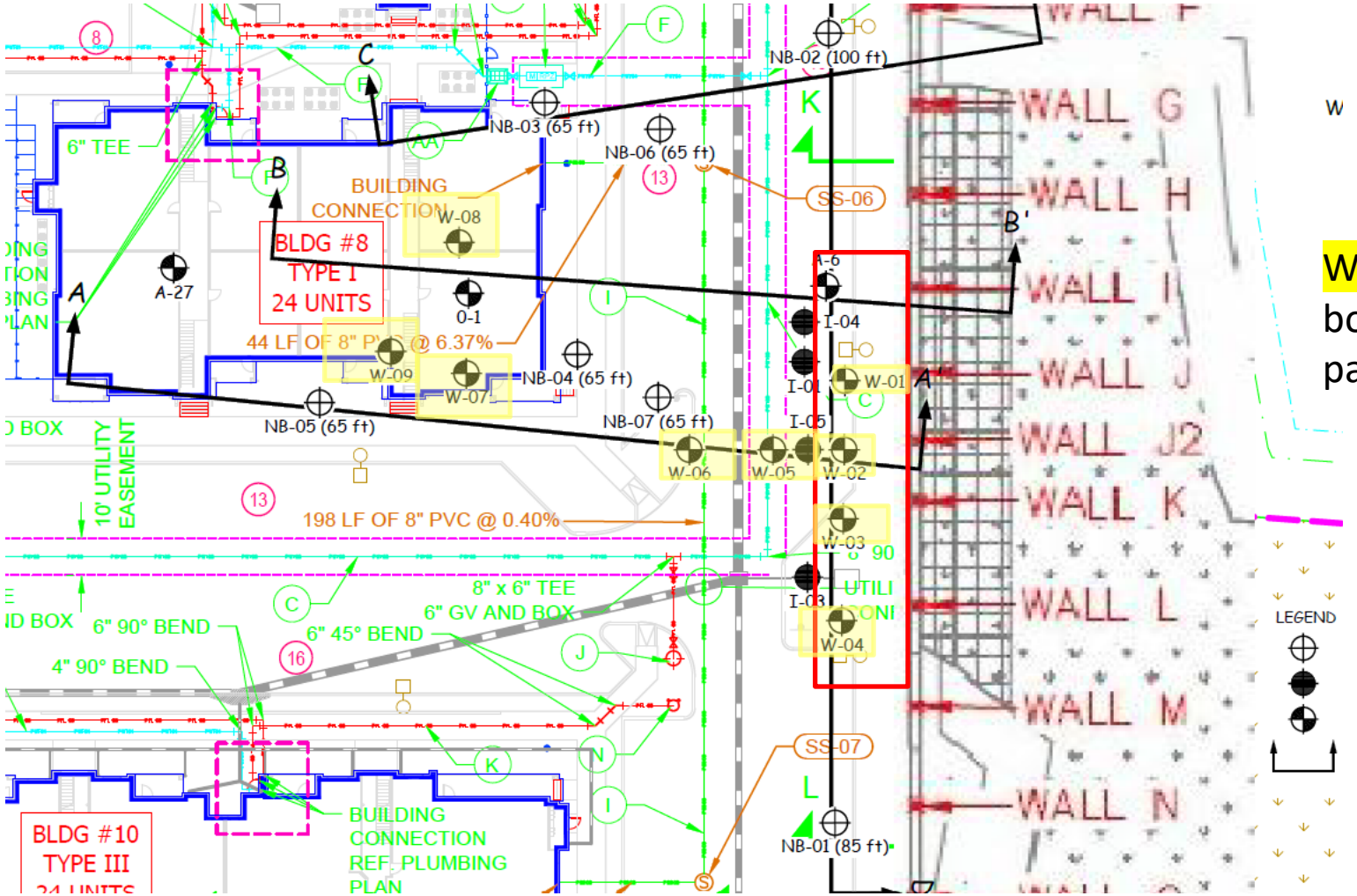
MAY 2022



Geotechnical Engineer Mobilizes to site and drills deep borings in the vicinity of MSE Wall 2 and Building 8 where settlement was so obvious – W-# series borings.

LARGE DIAMETER SINKHOLE SW FL

MAY 2022

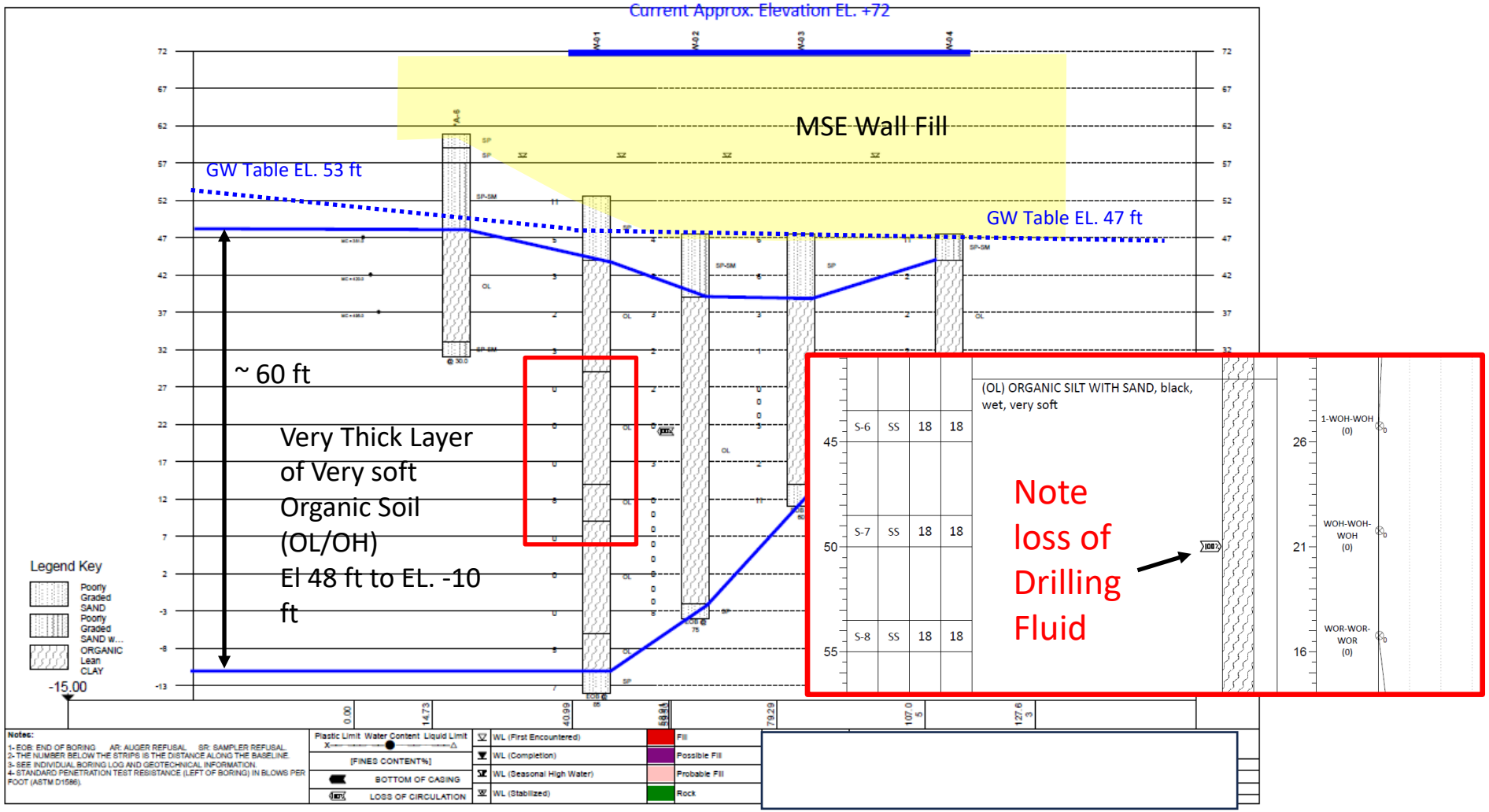


W-# series borings next page

Ultimately many borings were drilled in this problem area extending into 2023.

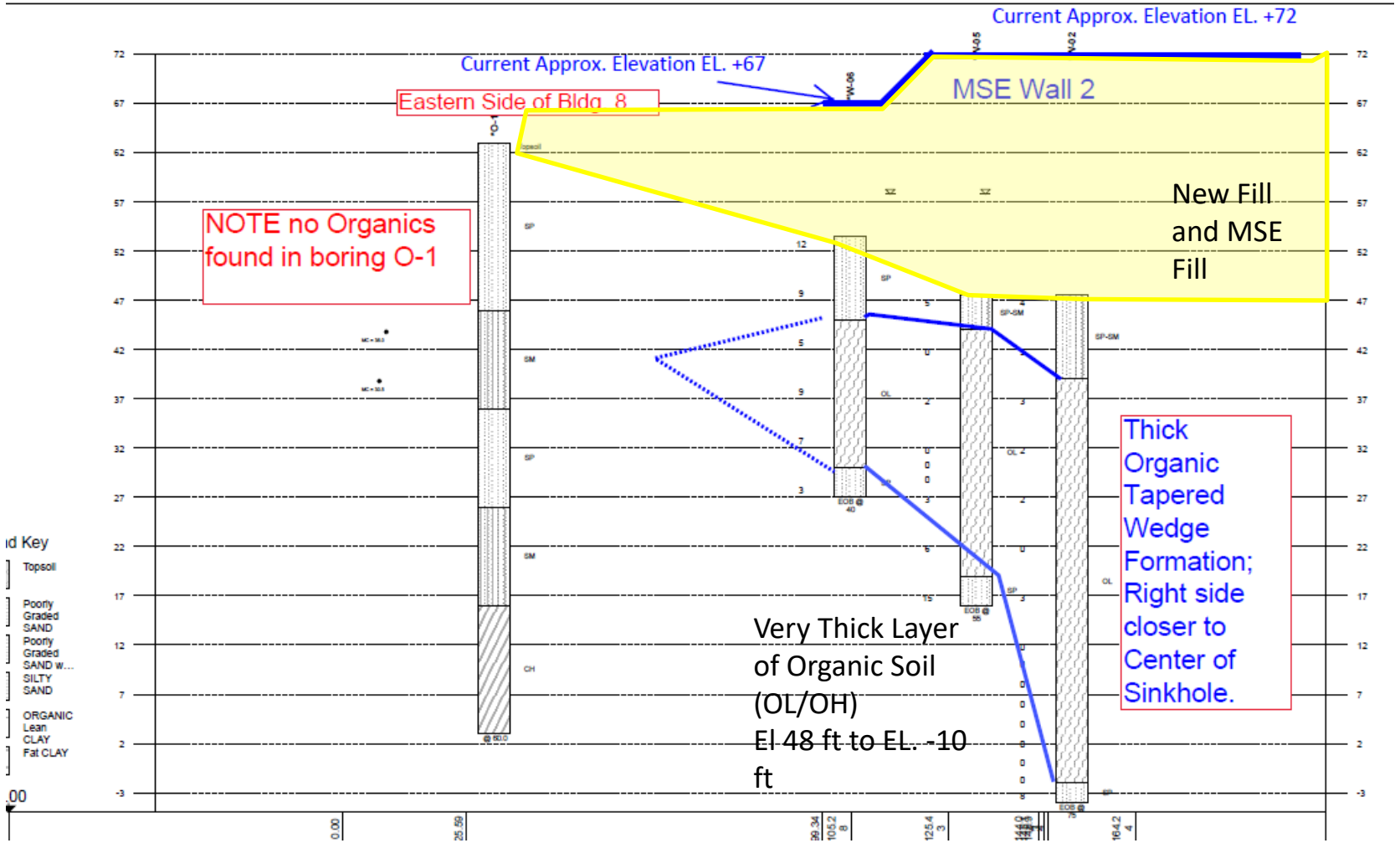
LARGE DIAMETER SINKHOLE SW FL

SUBSURFACE PROFILE ADJ. TO MSE WALL 2 MAY 2022 (PARALLEL TO WALL), W-# SERIES BORINGS



LARGE DIAMETER SINKHOLE SW FL

SUBSURFACE PROFILE ADJ. TO MSE WALL 2 MAY 2022 (PERPENDICULAR TO WALL), W-# SERIES BORINGS

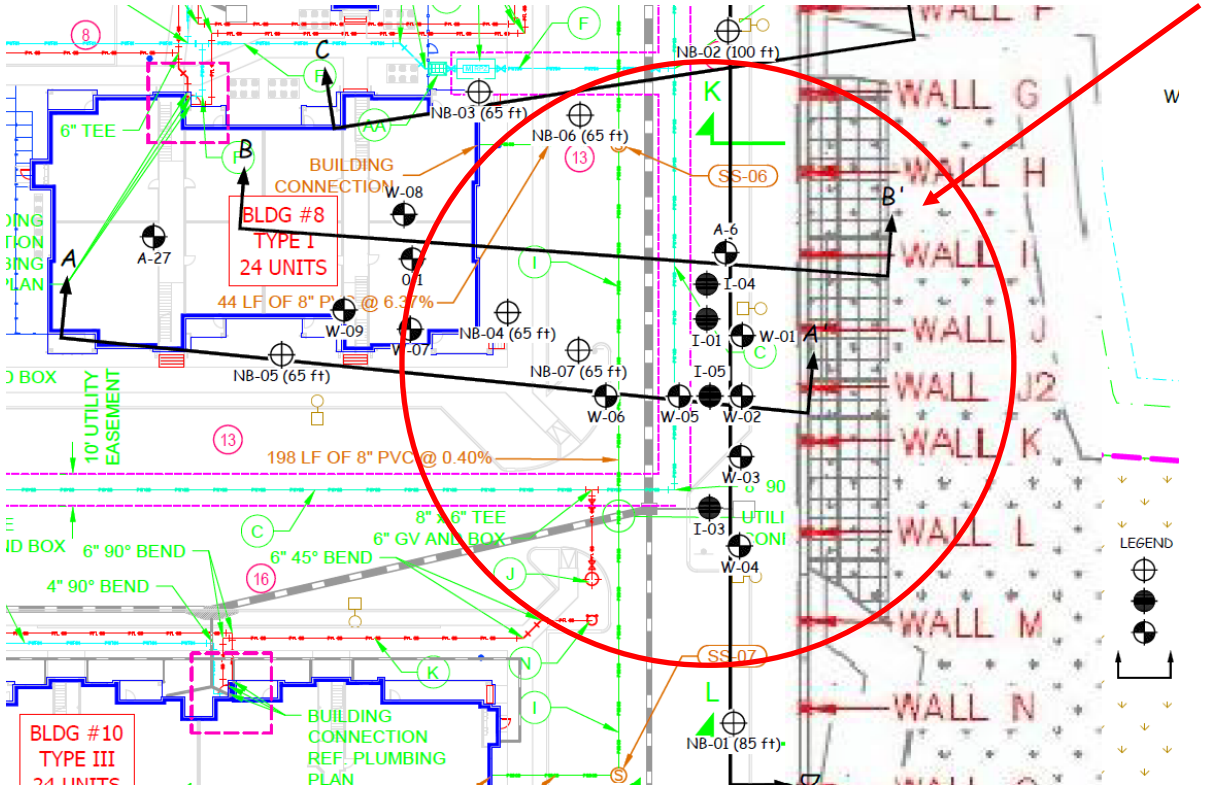


LARGE DIAMETER SINKHOLE SW FL

JUNE 2022

1. The Engineer issues report with results of W-# series borings identifying very deep organic soils not previously identified in SE portion of site, east of Buildings 8 and 10 and below MSE retaining wall #2.

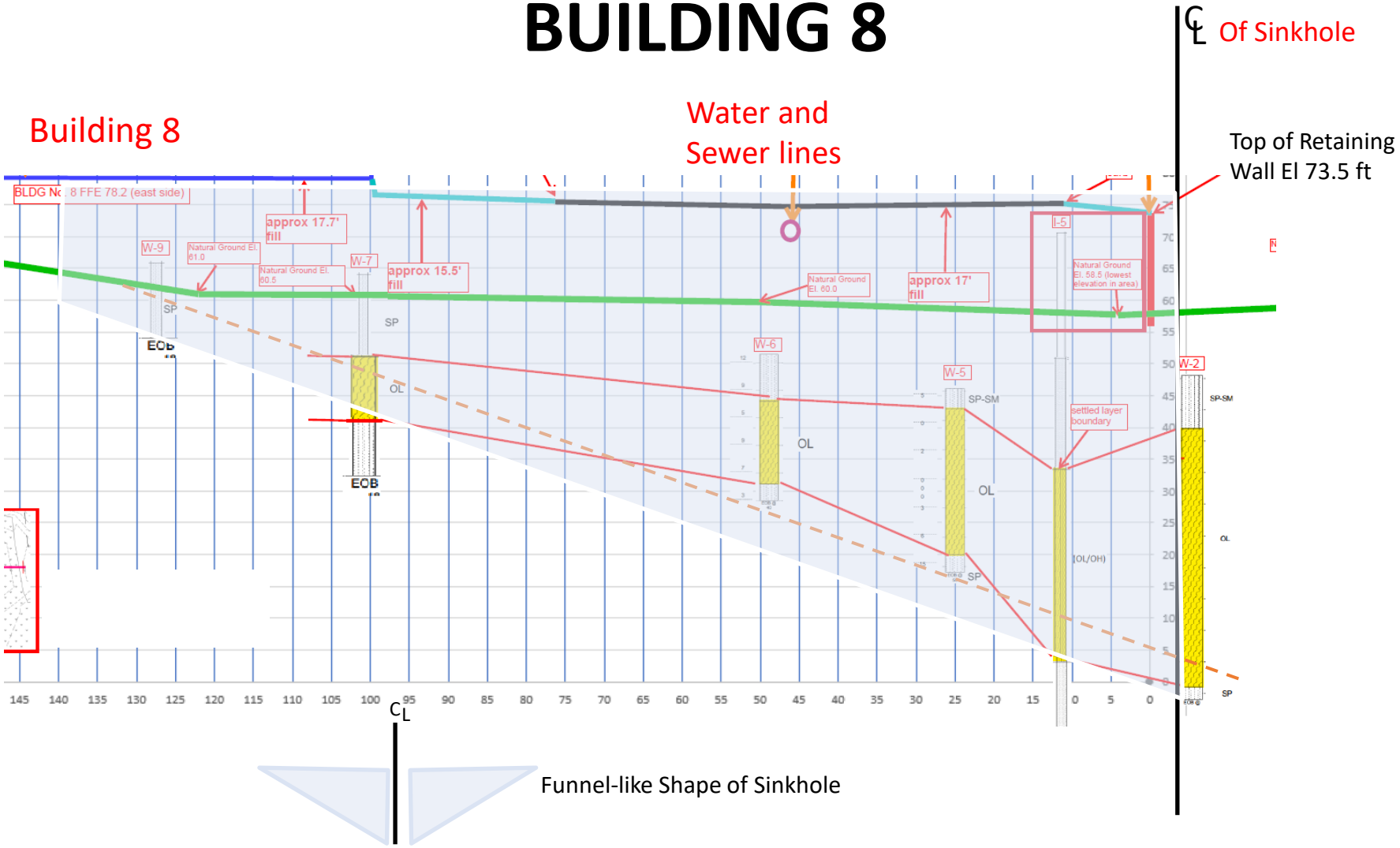
2. The Engineer characterized a deep, cover-collapse sinkhole in this area.



Difficult to characterize, but surface expression of sinkhole may be several hundred feet in diameter

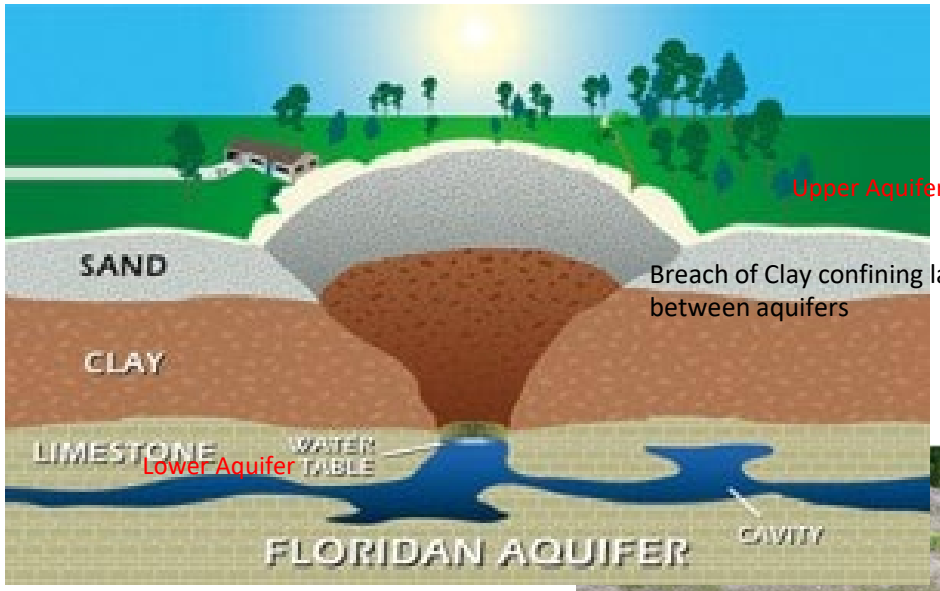
LARGE DIAMETER SINKHOLE SW FL

MAY 2022 – EAST-WEST PROFILE FROM MSE WALL #2 TO BUILDING 8



LARGE DIAMETER SINKHOLE SW FL

ANCIENT, COVER-COLLAPSE SINKHOLES IN FL EXPLAINED



For this project, the sinkhole likely happed eons ago. After the initial collapse, the void filled in with organic soils from surrounding areas via surface run off and drainage. Eventually the void filled and was later covered with an upper marine sand deposit.





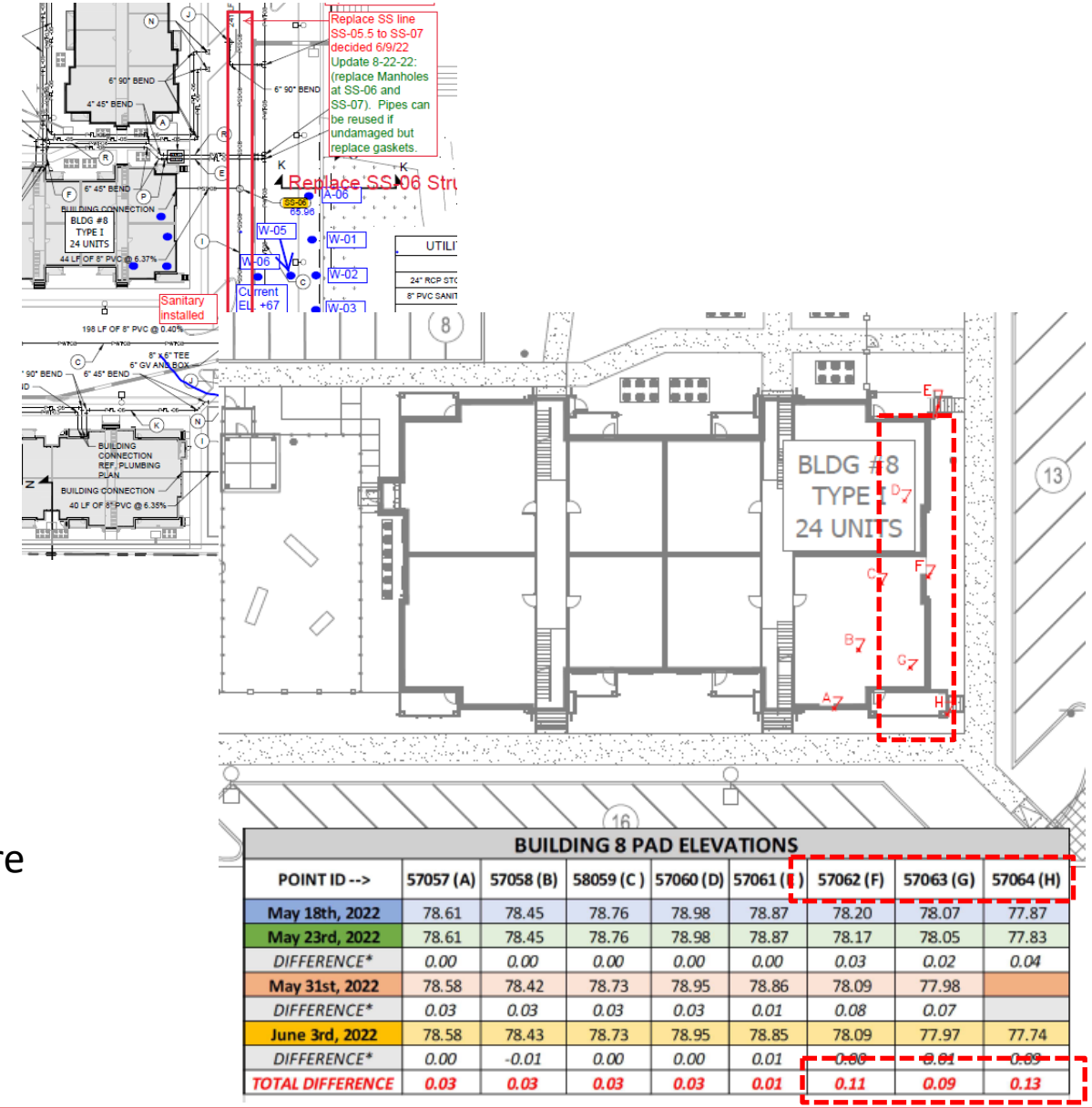
LARGE DIAMETER SINKHOLE SW FL

JUNE 2022

What did we do next?

In June of 2022, there were no indications that MSE Wall #2 Settlement had SIGNIFICANTLY affected adjacent Building 8 or 10.

1. We did know that adjacent Stormwater and Sanitary Sewer lines were significantly impacted needing repair/replacement.
2. East side of Building 8 had settled about 1 inch (0.1 ft) since when we started monitoring May 2022 (likely more between Jan. – April 2022) but building was still functioning well.



BUILDING 8 PAD ELEVATIONS

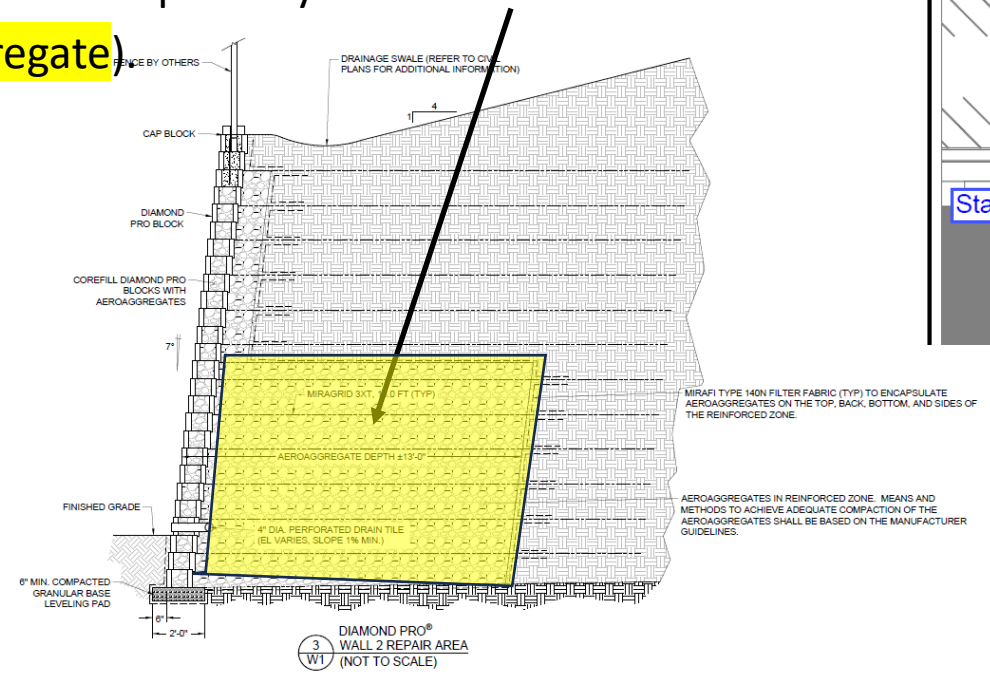
POINT ID -->	57057 (A)	57058 (B)	58059 (C)	57060 (D)	57061 (E)	57062 (F)	57063 (G)	57064 (H)
May 18th, 2022	78.61	78.45	78.76	78.98	78.87	78.20	78.07	77.87
May 23rd, 2022	78.61	78.45	78.76	78.98	78.87	78.17	78.05	77.83
*DIFFERENCE**	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.03*	*0.02*	*0.04*
May 31st, 2022	78.58	78.42	78.73	78.95	78.86	78.09	77.98	
*DIFFERENCE**	*0.03*	*0.03*	*0.03*	*0.03*	*0.01*	*0.08*	*0.07*	
June 3rd, 2022	78.58	78.43	78.73	78.95	78.85	78.09	77.97	77.74
*DIFFERENCE**	*0.00*	*-0.01*	*0.00*	*0.00*	*0.01*	*0.00*	*0.01*	*0.09*
TOTAL DIFFERENCE	***0.03***	***0.03***	***0.03***	***0.03***	***0.01***	***0.11***	***0.09***	***0.13***

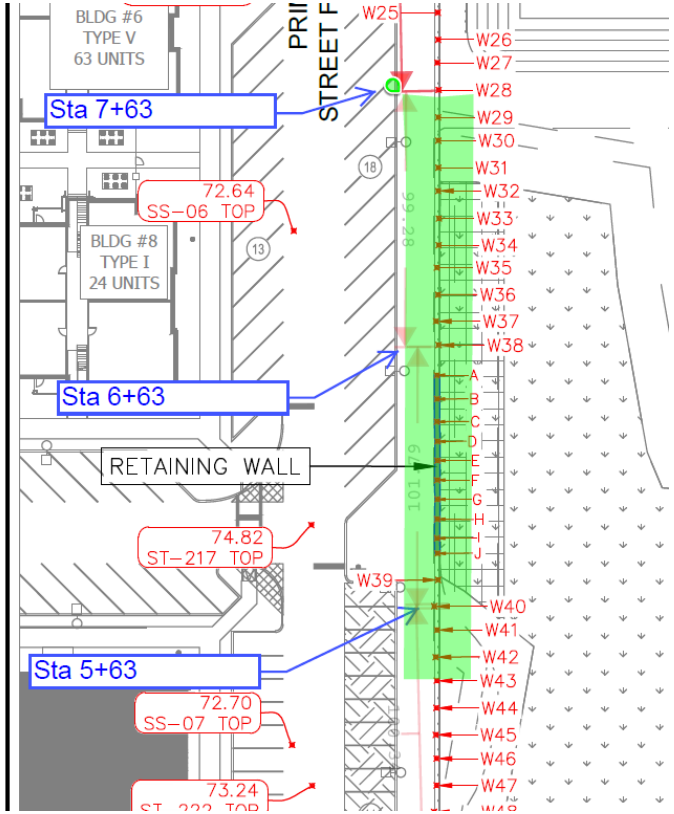
LARGE DIAMETER SINKHOLE SW FL

JUNE 2022

What did we do next?

3. We knew a portion of MSE wall #2 had to be replaced (shaded green)
4. The Team decided to remove and replace about 300 ft linear section of MSE wall #2, this time partially with super-lightweight fill (aero aggregate)





LARGE DIAMETER SINKHOLE SW FL

JUNE 2022

What did we do next?

5. Initially, entire MSE reinforced zone was to utilize aero aggregate, but for cost reasons it was reduced to half the height.
6. Net reduction in load at base of MSE of 1,000 psf.
7. With the “pre-loading” of the original wall fill and a 6+ month hold period prior to reconstruction, we expected the newly reconstructed wall to have much less post-construction settlement.
8. Client made all final decisions on remedial work with risks being identified. Insurance company had some say. More conservative remedial solutions identified and discussed. Lightweight fill to reconstruct wall chosen.

UL-FGA are ultra-lightweight aggregates produced from 100% post-consumer recycled glass. The aggregates have a highly frictional surface and combined with a low unit weight, inertness, high permeability, and insulating properties, foamed glass aggregate is ideal as a lightweight backfill.





15 pcf density

JULY 2022

MSE Wall #2 Reconstruction



Base of Wall adjacent to Wetland



Picture of Newly Constructed Portion of MSE Wall #2 July 2022



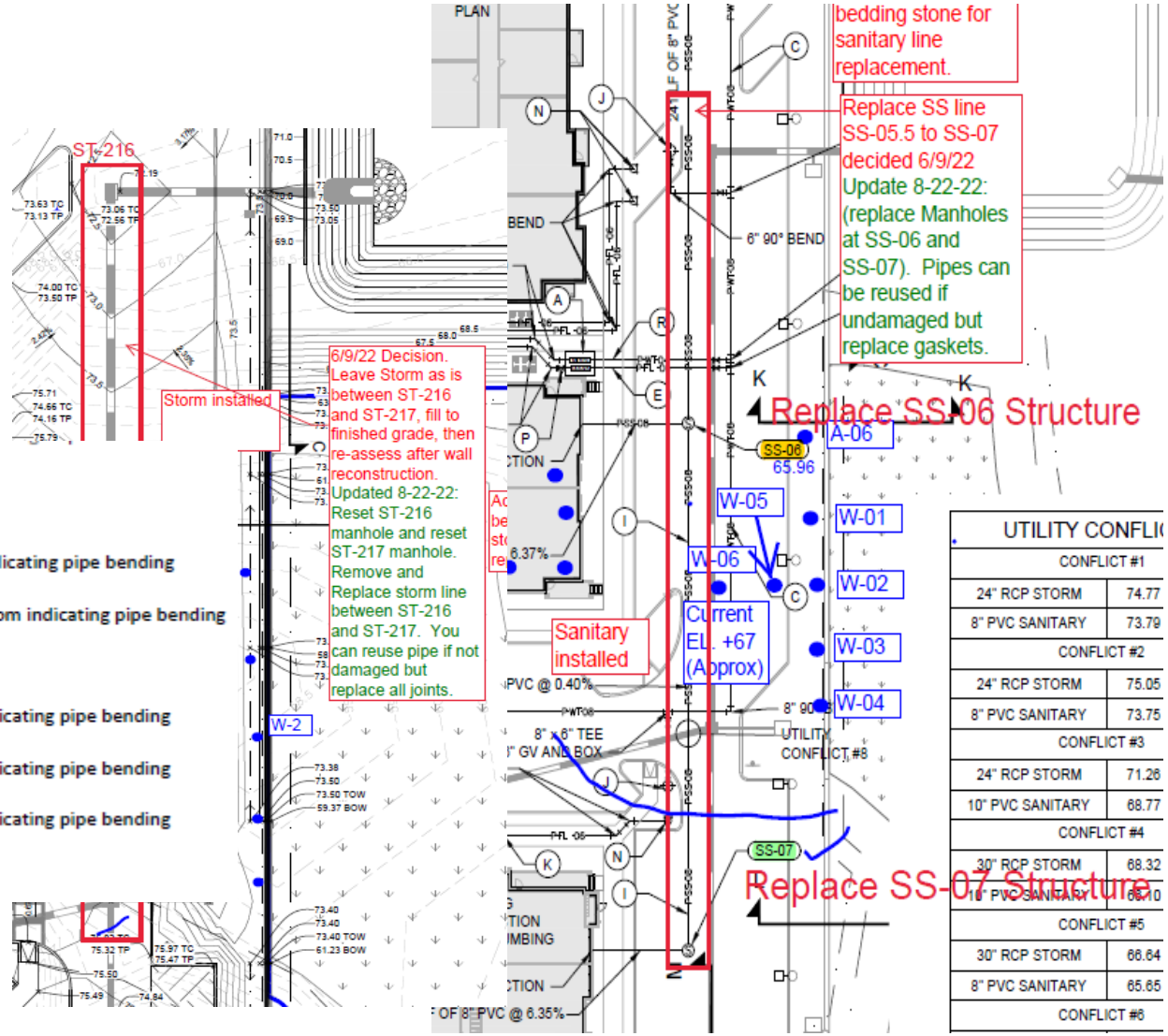
In-Progress Construction, Fabric over Aerolite

AUGUST 2022
Utility Reconstruction

Video Survey of Sanitary and Storm – very helpful to examine impacts of excessive settlement

ECS' observation of the video from ST-217 and 216 are summarized below:

Time	Comment
7:08	Pipe Joint Separation + significant soil accumulation, Joint sep. at bottom indicating pipe bending
9:59	More Pipe Joint Separation + significant soil accumulation, Joint sep. at bottom indicating pipe bending
12:00	Significant soil accumulation + standing water
14:10	Pipe Joint Separation + significant soil accumulation, Joint sep. at bottom indicating pipe bending
16:15	Pipe Joint Separation + significant soil accumulation, Joint sep. at bottom indicating pipe bending
18:30	Pipe Joint Separation + significant soil accumulation, Joint sep. at bottom indicating pipe bending
23:00	Pipe Joint Separation (no soil)



LARGE DIAMETER SINKHOLE SW FL

AUGUST 2022

In summary – Portion of MSE Wall #2 reconstructed, utility reconstruction done or planned, and we are ready to put pencils down. Insurance Carrier Paid for repairs. Moving on to the next project challenge.

PROJECT complete®

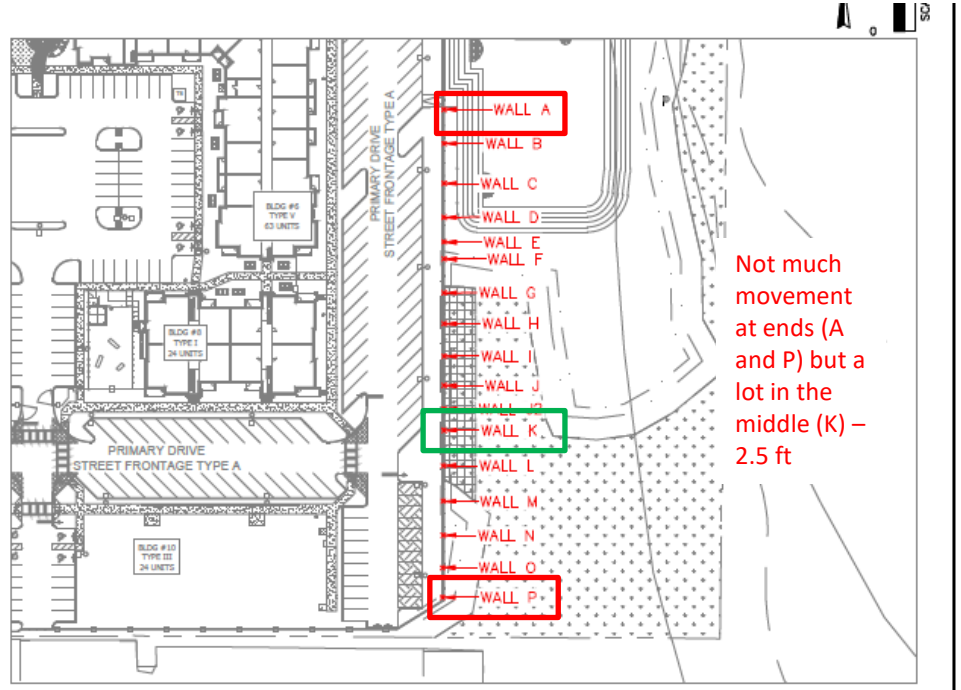
design consultancy and project management



JANUARY 2023

Not so fast!

- Nothing like starting the new year off with a bang.
- The Geotechnical Engineer got a call from the owner saying “hey, we are seeing retaining wall settlement AGAIN after reconstruction.
- Owner retained surveyor to get baseline top of wall elevation starting early January 2023
- Insurance carrier re-engaged, new case open.



RETAINING WALL ELEVATIONS

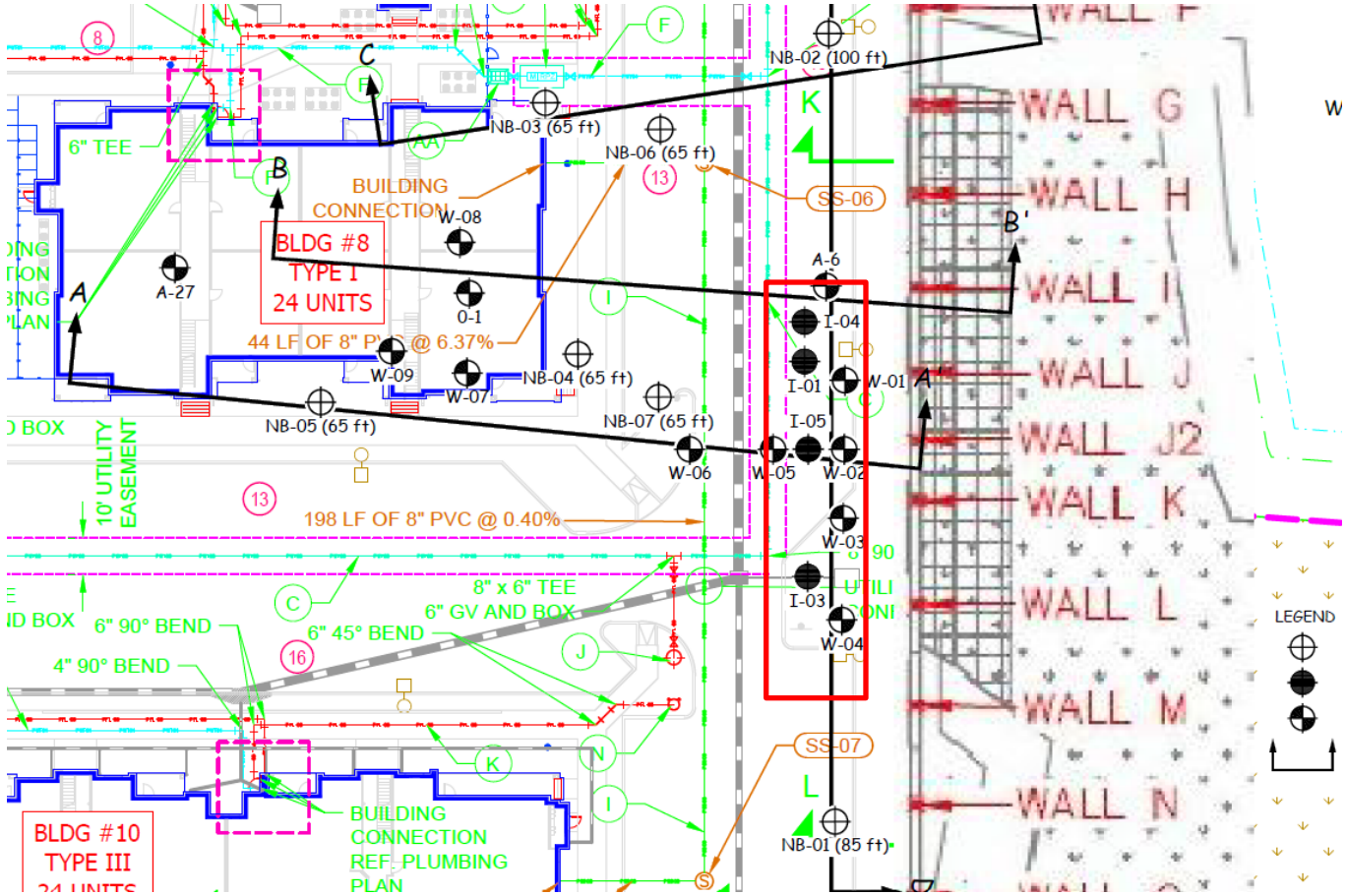
POINT ID-->	A	B	C	D	E	F	G	H	I	J	J2	K	L	M	N	O	P
Jan. 4th, 2023	73.87	73.82	73.80	73.74	73.62	73.26	72.68	72.22	71.94	71.69	71.56	71.57	72.09	72.66	73.01	73.2	73.26

Although not resurveyed after initial reconstruction, it is believed top of wall was built to EL. +74.03 ft as shown on the plans.

LARGE DIAMETER SINKHOLE SW FL

FEB. - MARCH 2023

- We're back on site drilling again. This time we are installing deep inclinometers behind the reinf. zone of Wall #2.
- The concern then was we have potential lateral wall movement in addition to vertical settlement (i.e., a global, circular base shear concern).
- See I-0# depicted to right.

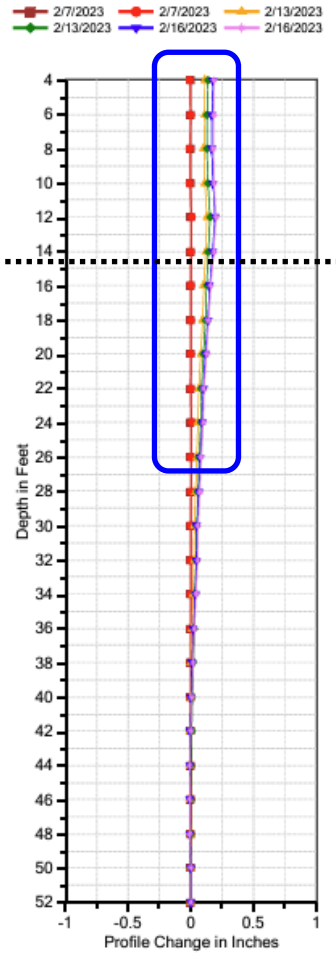


LARGE DIAMETER SINKHOLE SW FL

FEB. - MARCH 2023

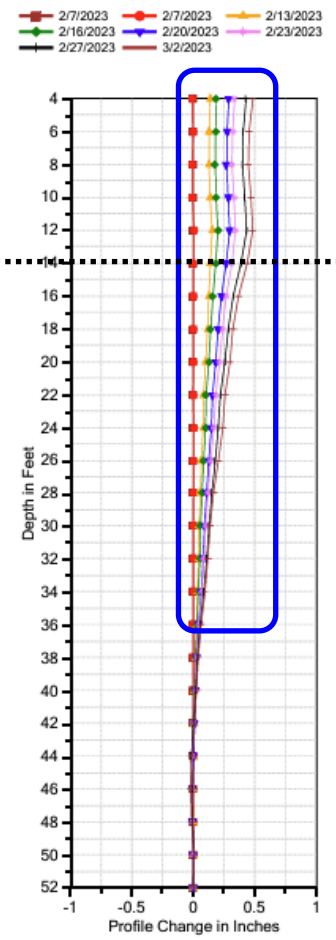
- Inclinometer Baselines: 2/7/23
- Pretty soon after baselines we are seeing lateral soil mass movement below grade
- Inclinometer monitoring continues weekly while we are working on the engineering solution.
- Vertical survey added for wall and Bldgs. 8 & 10.

I-3 A



Approx. Bottom of MSE Wall #2

I-3 A

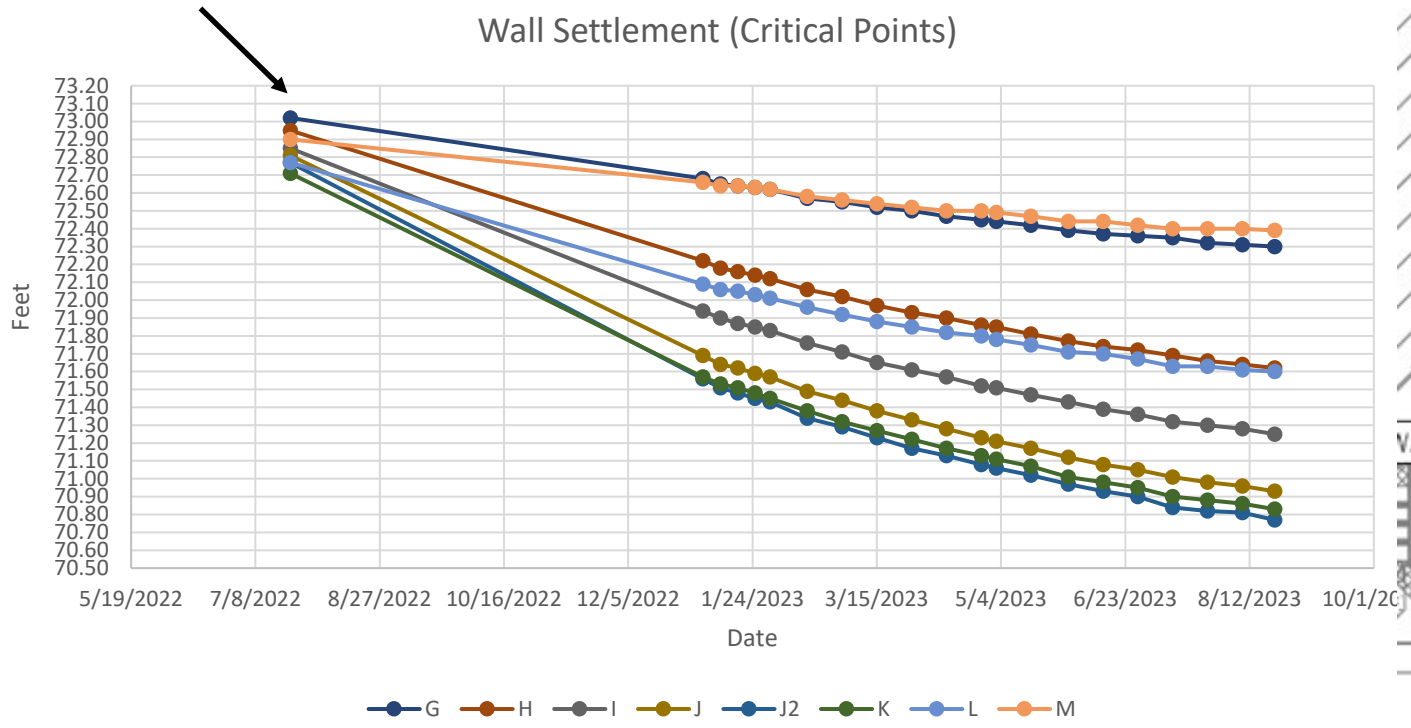


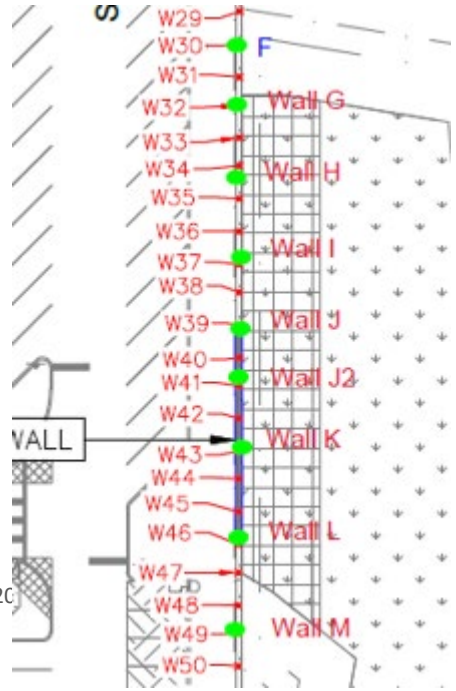
Lateral movement consistent for first three weeks. Wall is not Static but moving laterally west to east below grade.

JANUARY TO AUGUST 2023

Summer 2022 – Wall Reconstruction Done

MSE Wall Settlement Monitoring – Restarted Jan. 2023

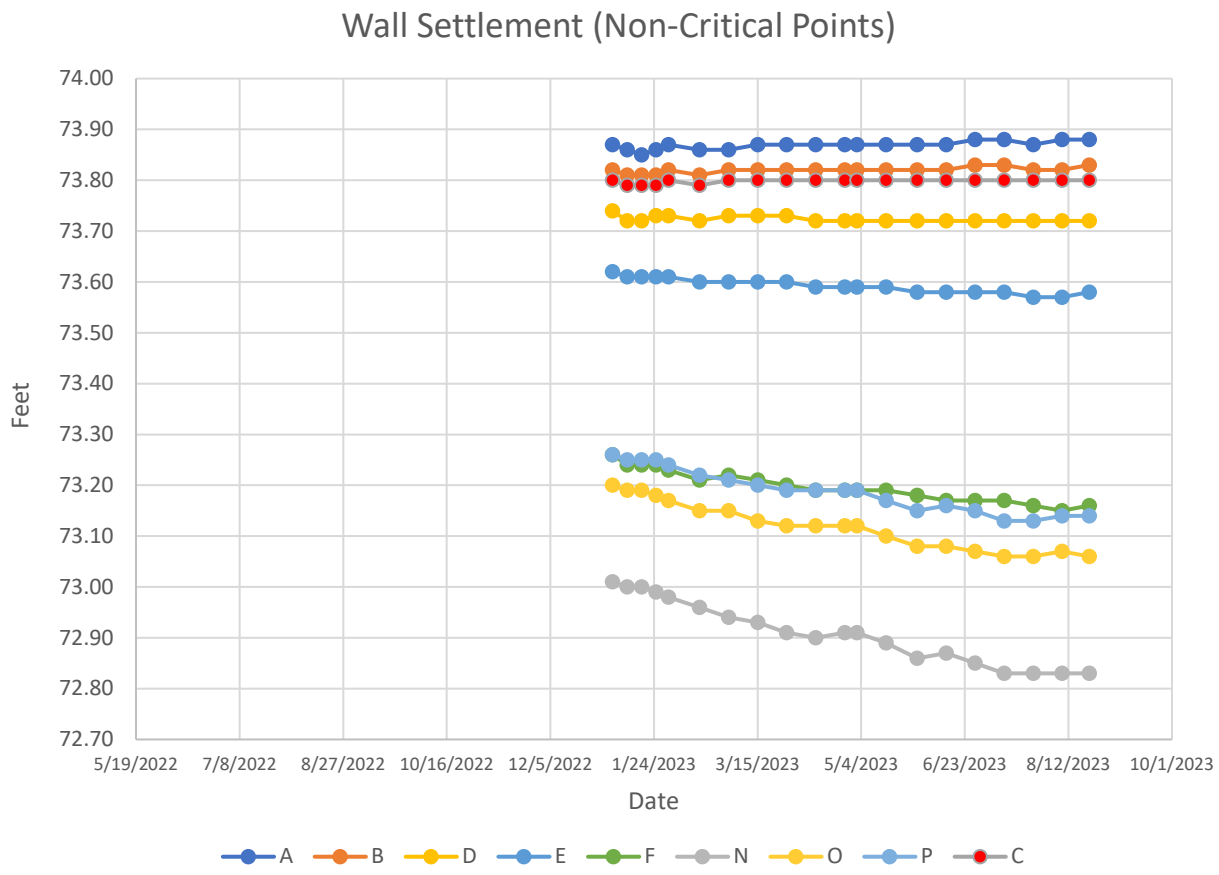


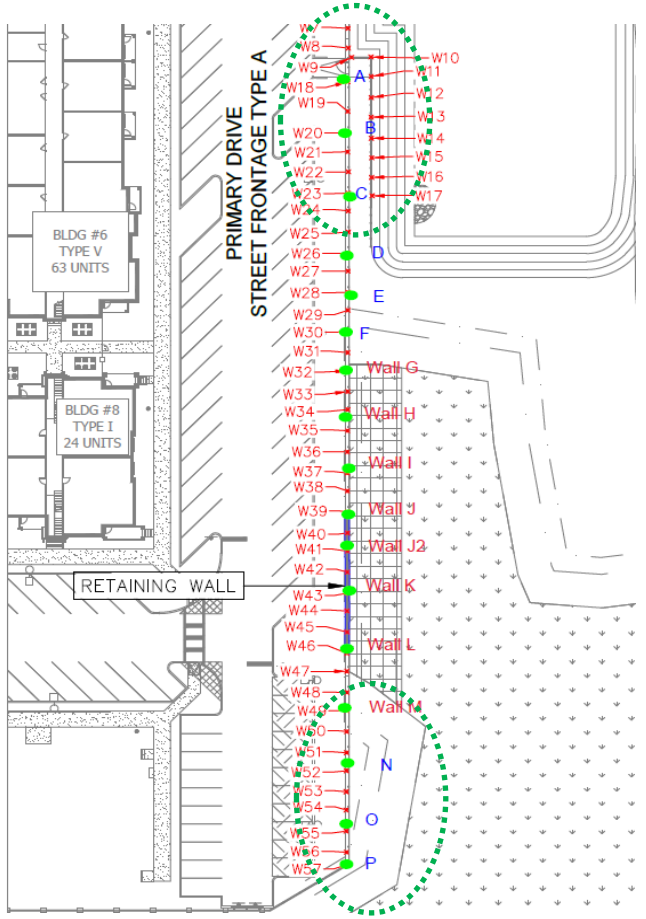


LARGE DIAMETER SINKHOLE SW FL

JANUARY TO AUGUST 2023

North and South of the Problem Area, MSE Wall #2 was stable.
Monitoring allowed us to quantify repair area.

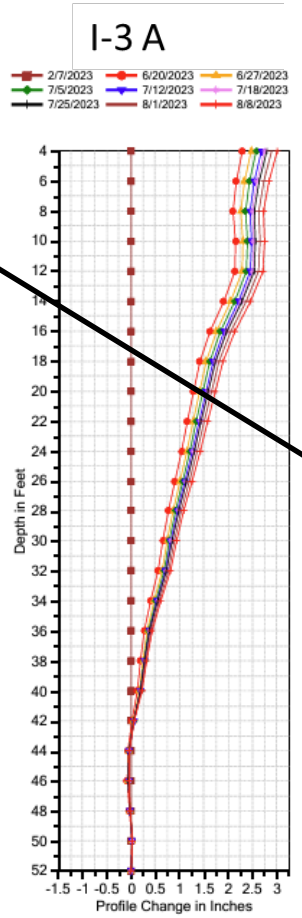




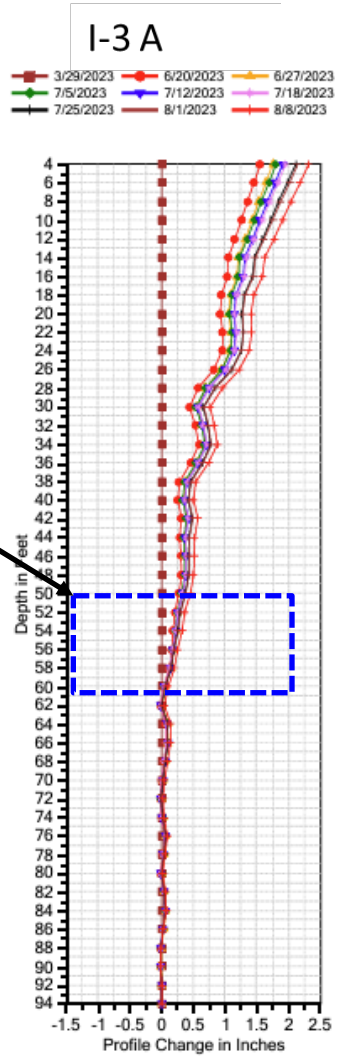
LARGE DIAMETER SINKHOLE SW FL

APRIL- AUGUST 2023

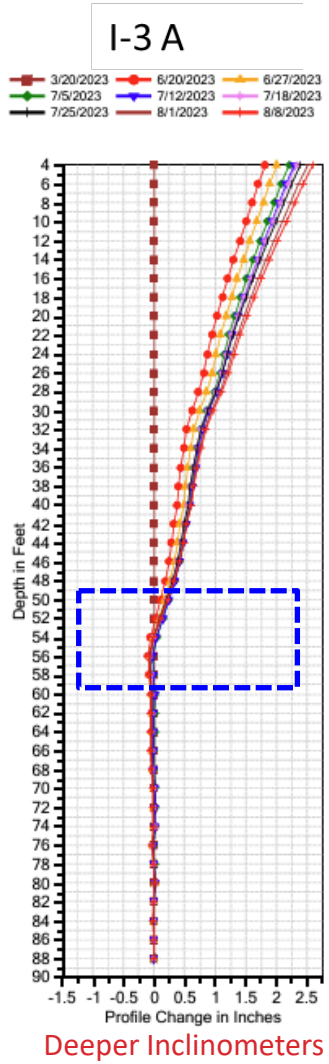
- Deeper Inclinometers Installed late March ‘23
- Movement as deep as 60 feet below top of wall (EL. +10) observed.
- With no indications the wall will not stop moving laterally, Geotechnical Engineer predicts wall failure in near future.
- Working early summer on engineering solution.



Note vertical scales different



Deeper Inclinometers



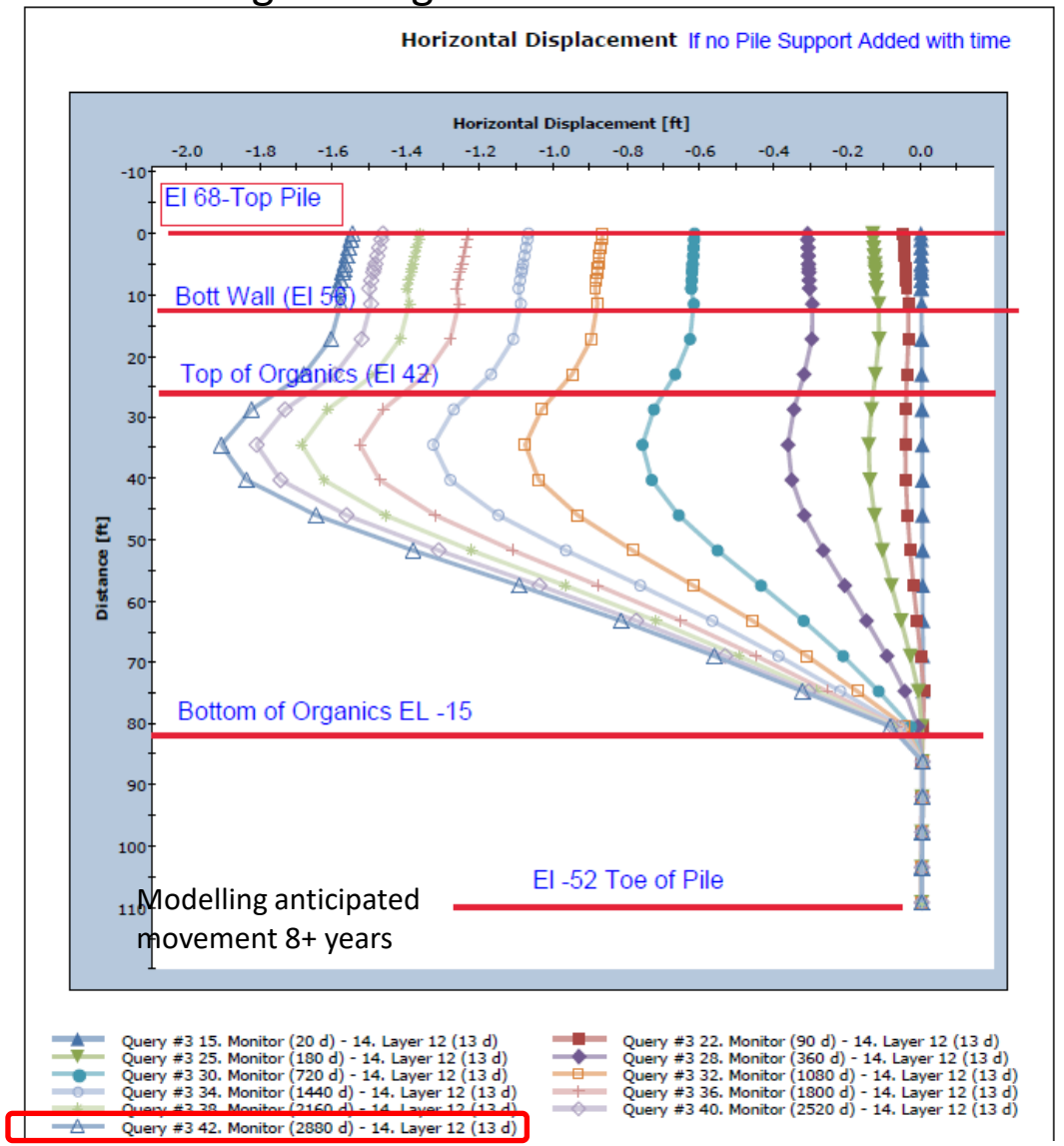
Deeper Inclinometers

LARGE DIAMETER SINKHOLE SW FL

SPRING – EARLY SUMMER 2023

- Geotechnical Engineer used Finite Element (FE) modeling to predict MSE Wall #2 movement if NO reinforcement is done: eminent failure.
- We also used FE to design reinforcement.
- FE approach chosen because of complexity of items in model (existing wall) and measured movement data (vertical and lateral) that help validate model parameters.
- Used first several months of inclinometer and settlement data to validate model parameters.

Engineering Solution – Round 2



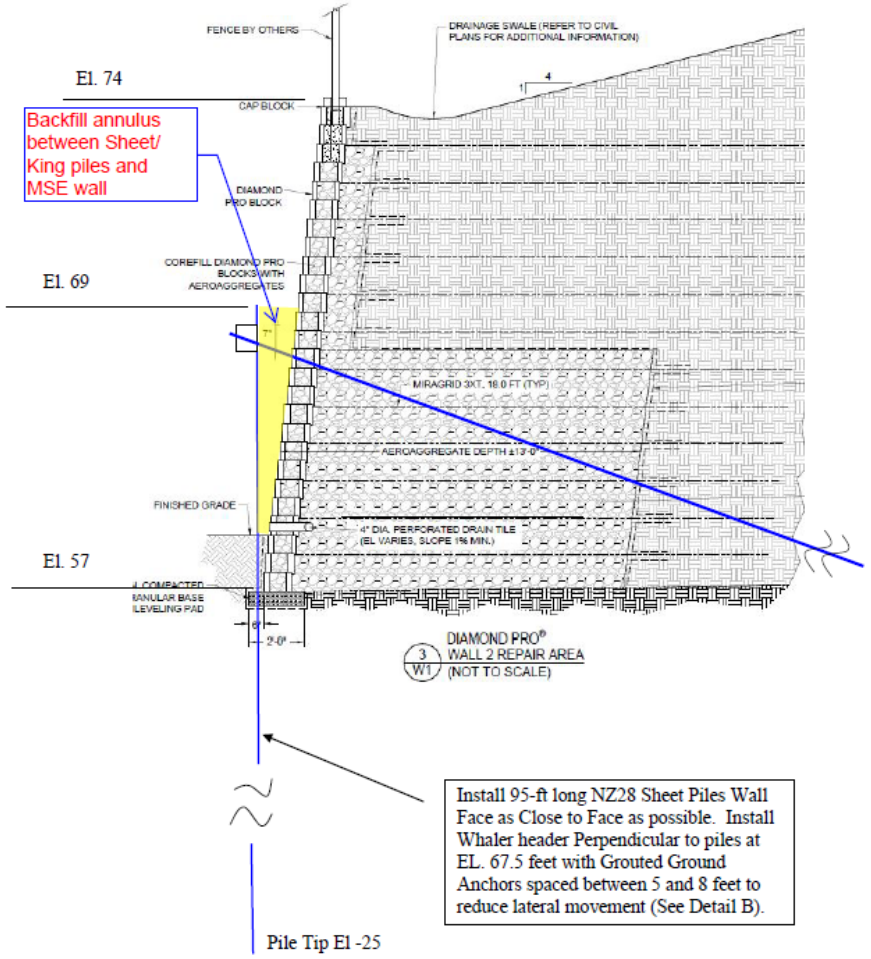
LARGE DIAMETER SINKHOLE SW FL

SPRING – EARLY SUMMER 2023

Option 1 – Steel Sheet Piles with Ground Anchors

- The Geotechnical Engineer jumped on engineering solutions once it was determined wall was moving laterally and likely to fail.
- Option 1 - Continuous Steel Sheets in front of Wall
- This option required ground anchors (horizontal support)
- Option 1 was the Engineer's Preference due to ground anchor lateral support

Engineering Solution – Round 2

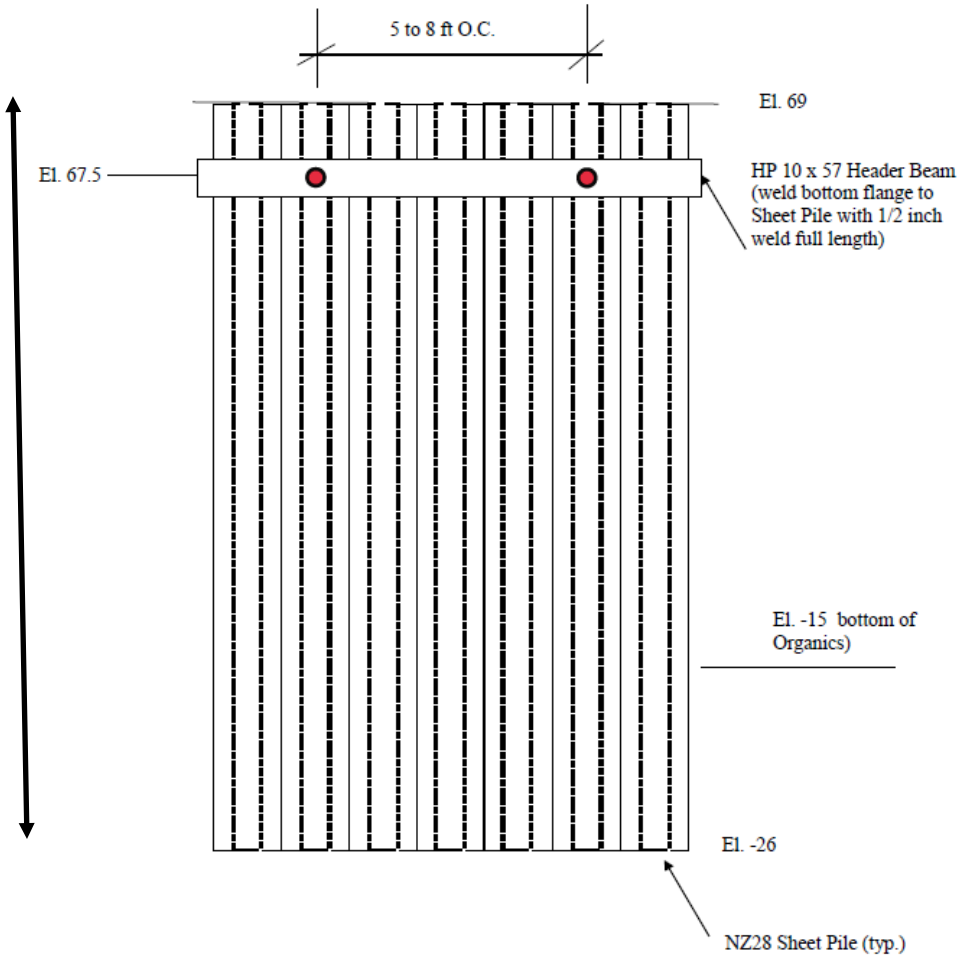


Option 1 - Detail A –Sheet Pile with Soil Anchors Profile

SPRING – EARLY SUMMER 2023

Option 1 – Steel Sheet Piles with Ground Anchors

95 ft long sheets



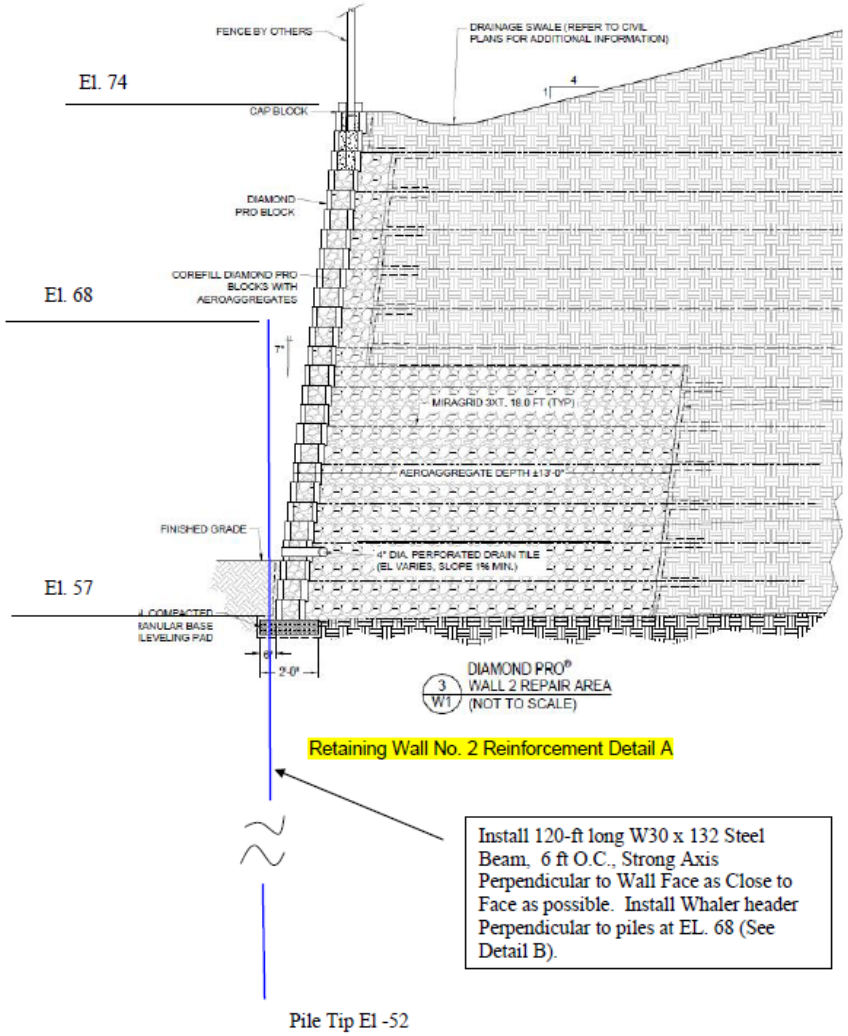
Option 1 - Detail B –Sheet Pile with Grouted Soil Anchors

LARGE DIAMETER SINKHOLE SW FL

SPRING – EARLY SUMMER 2023

Option 2 – King Pile Cantilevered

- This option did not require ground anchors, but very robust sized, closely spaced “King Piles”
- 120-foot-long Piles
- W30 x 132 in size

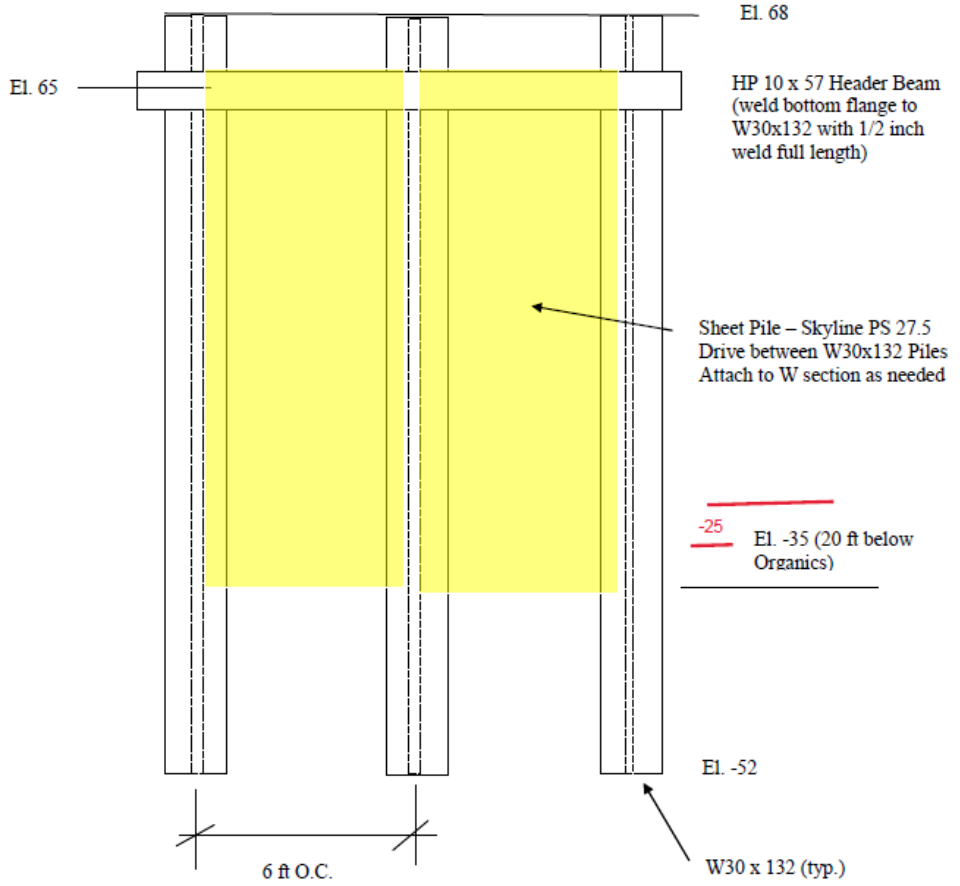


LARGE DIAMETER SINKHOLE SW FL

SPRING – EARLY SUMMER 2023

Option 1 – King Pile Cantilevered

- King Pile Concept
- Requires steel sheet piles between king piles.
- Long Piles (120 ft) needed because there is no lateral support via ground anchors.

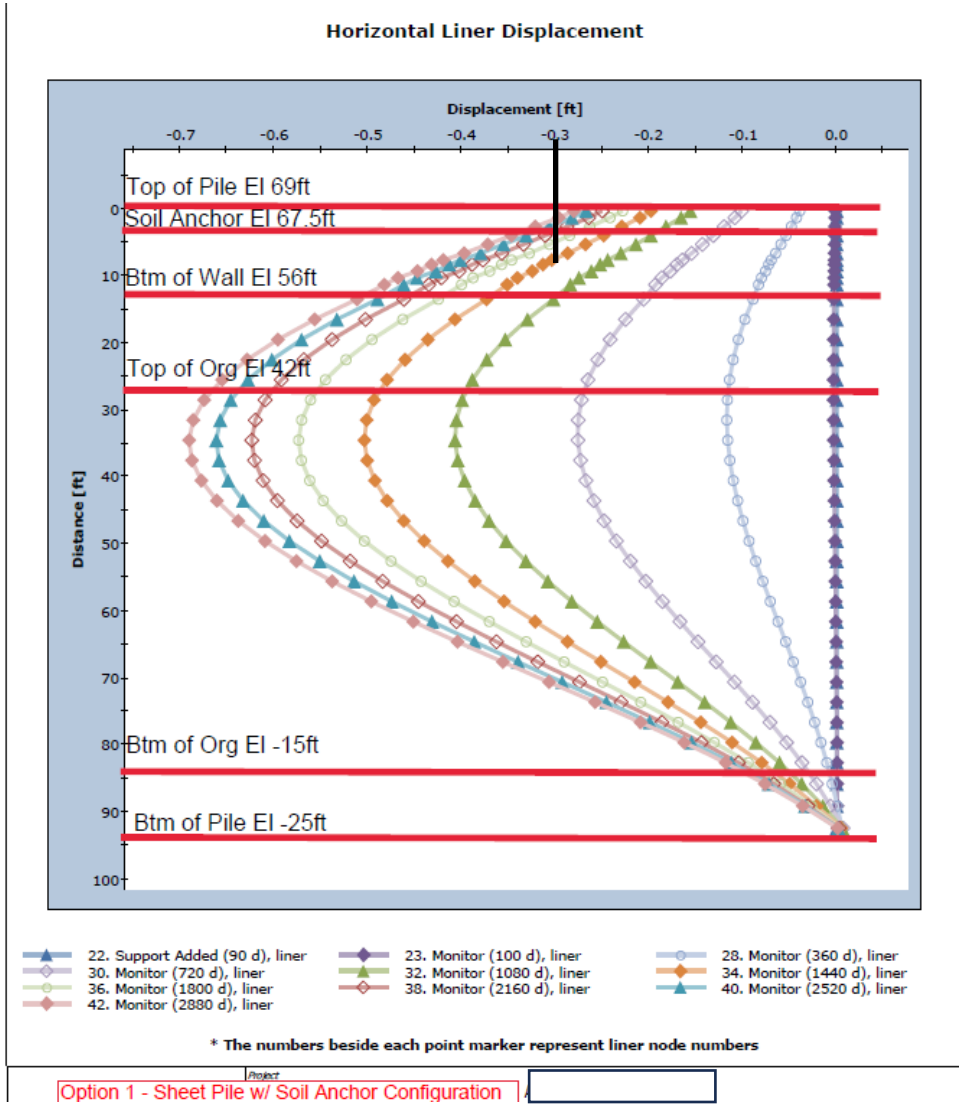


Detail B –Whaler Header

SPRING – EARLY SUMMER 2023

Option 1 – Sheet Pile with Ground Anchors

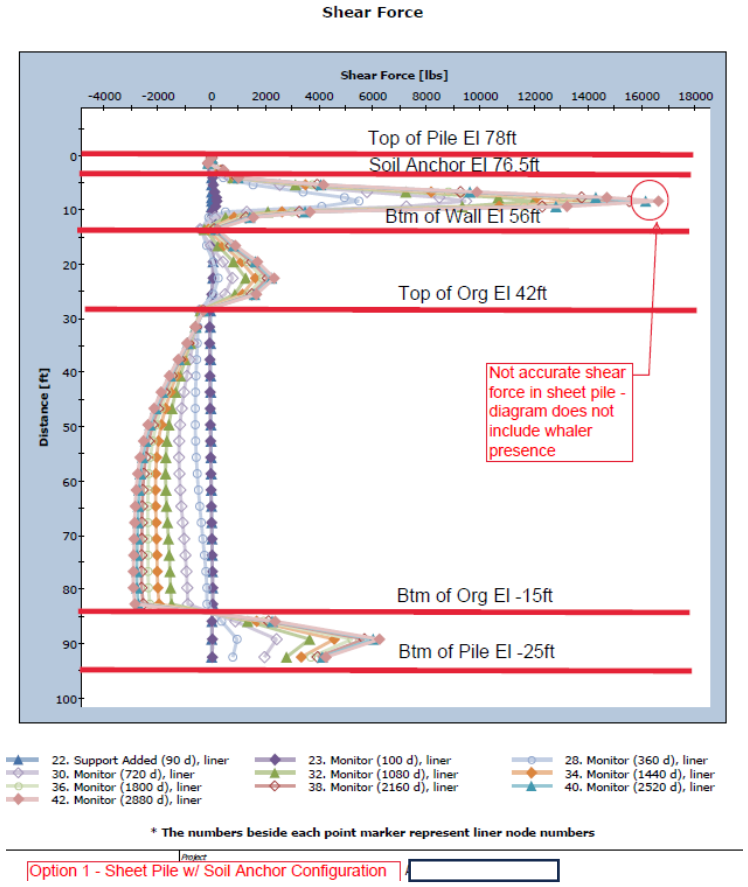
- FE model movement predictions
- Much less Top of Pile/MSE wall movement over time: 0.27 feet
- 0.7 ft of movement at Top of Organic Layer, but intercepted by steel elements.

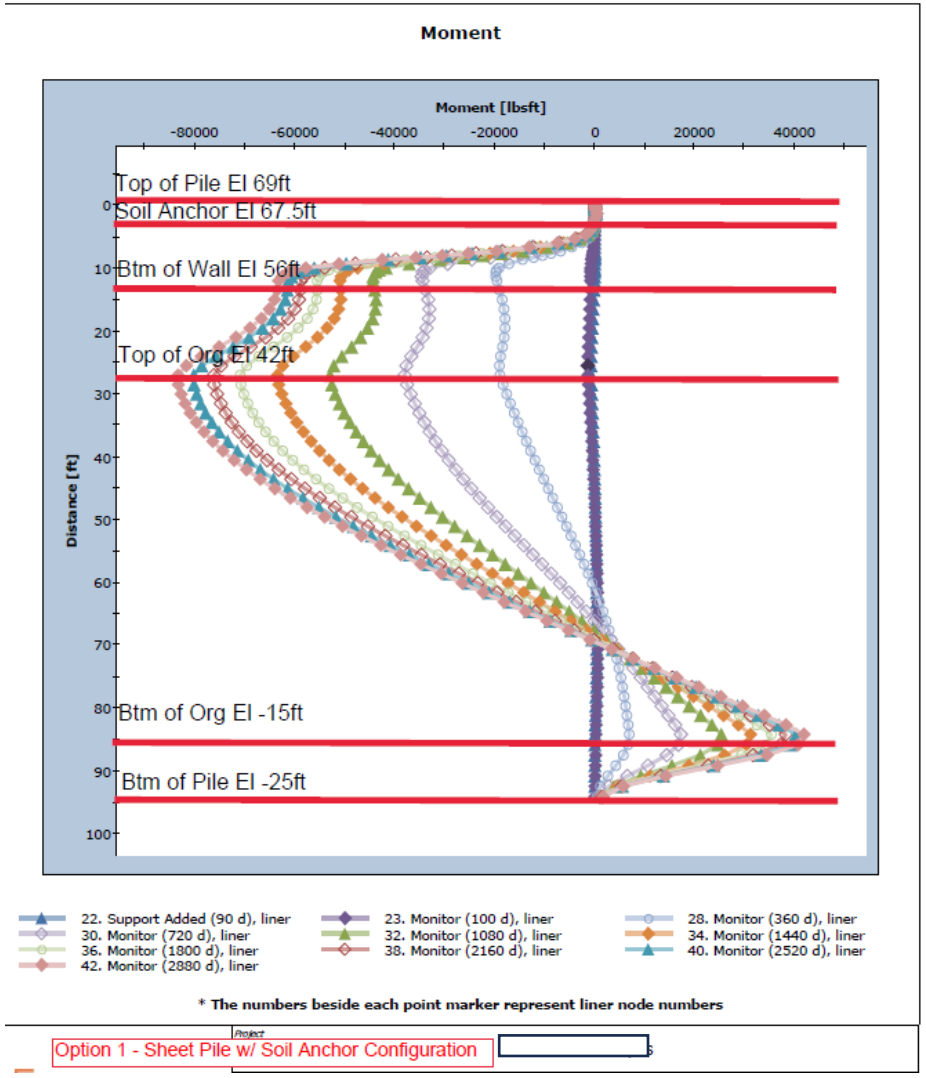


SPRING – EARLY SUMMER 2023

Option 1 – Sheet Pile with Ground Anchors

- FE Model can handle shear and moment calcs.

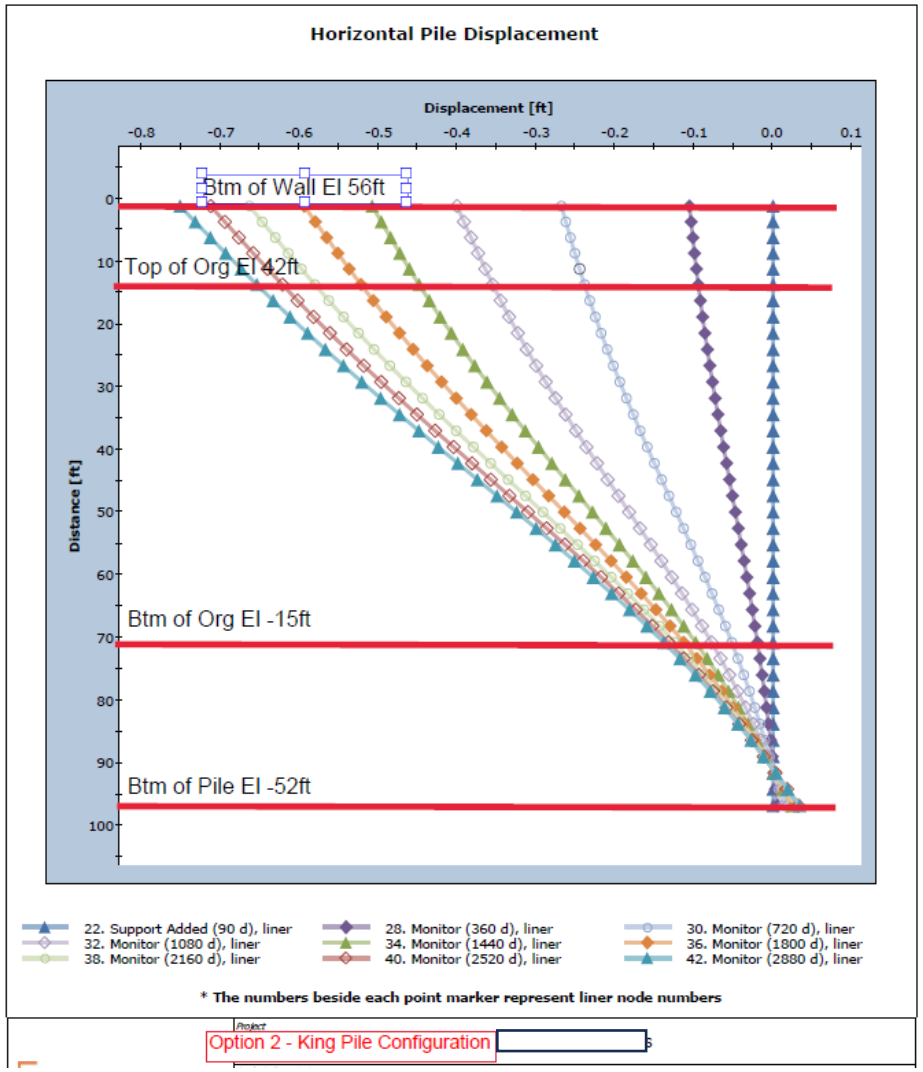




SPRING – EARLY SUMMER 2023

Option 2 – King Pile No Anchors

- FE model movement predictions
- More Movement than option 1:
  0.75 feet vs. 0.27 feet

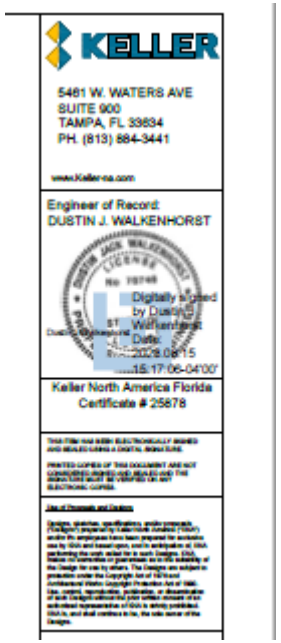


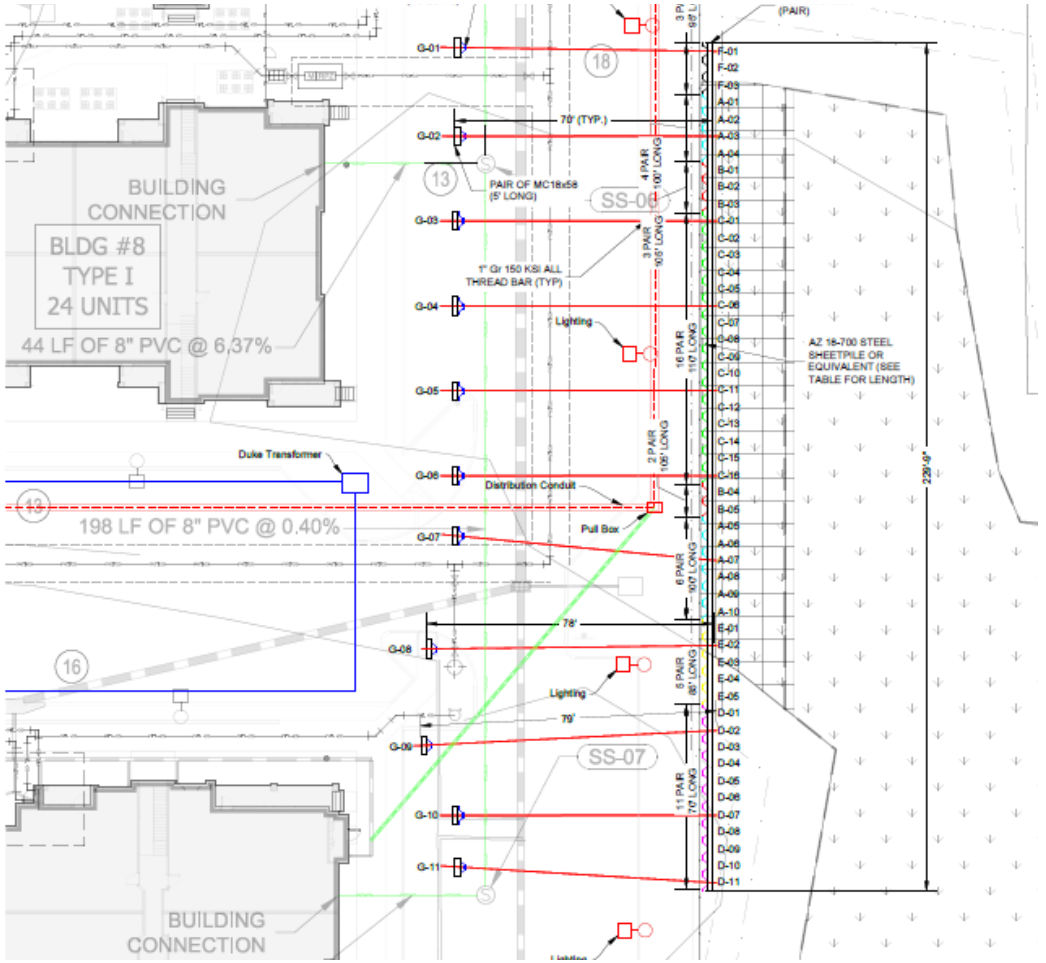
LARGE DIAMETER SINKHOLE SW FL

SPRING –SUMMER 2023

Option 1 – Chosen by Client

- Client elected to pursue Design-Build Contract with Keller in Florida.
- The Geotechnical Engineer worked collaboratively with Keller once client decided on Design-Build Approach.


KELLER
5461 W. WATERS AVE
SUITE 900
TAMPA, FL 33634
PH. (813) 884-3441
Engineer of Record:
DUSTIN J. WALKENHORST
Keller North America Florida
Certificate # 25878



LARGE DIAMETER SINKHOLE SW FL

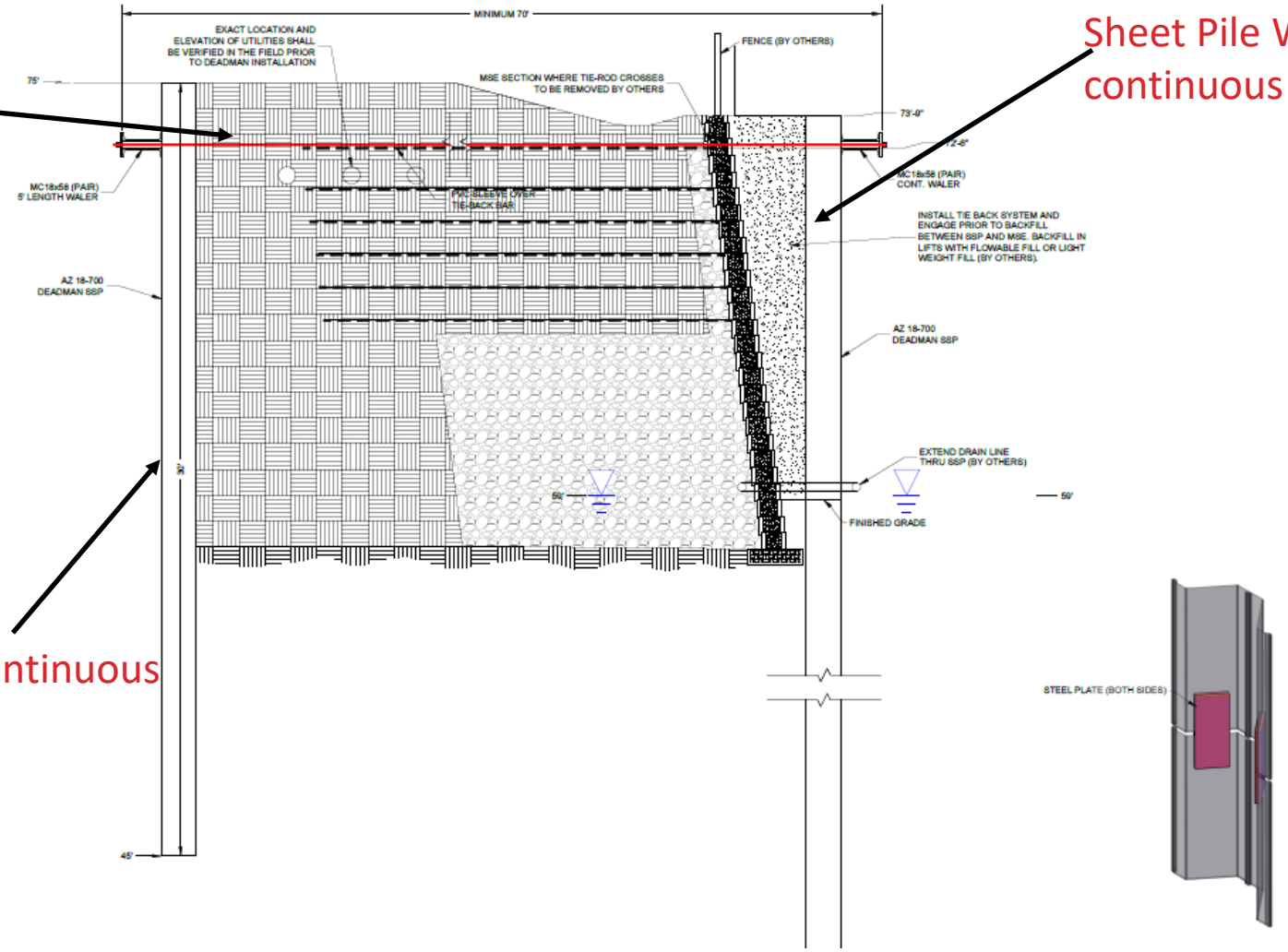
SPRING –SUMMER 2023

Option 1 – Chosen by Client

Threaded Rod Anchor

Sheet Pile Wall continuous

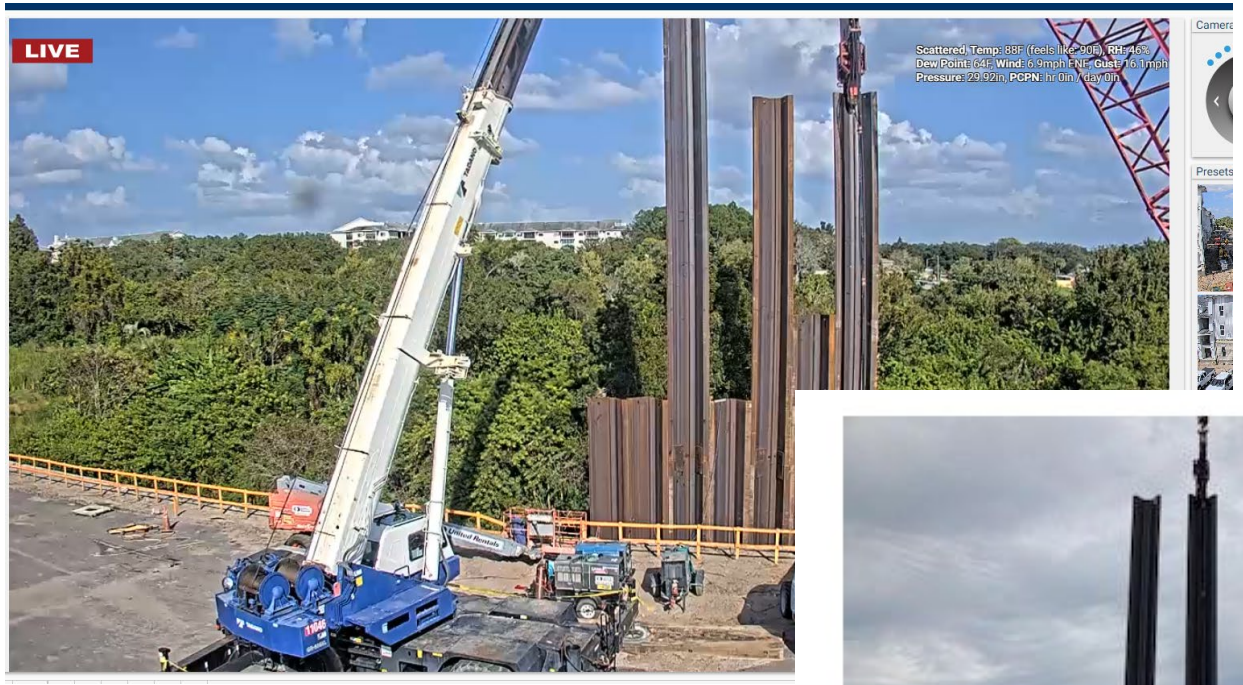
Deadman, discontinuous sheets



LARGE DIAMETER SINKHOLE SW FL

SPRING –SUMMER 2023

Option 1 – Chosen by Client



The center sheets were as long as 110 feet





Welding of the sheet piles

SPRING –SUMMER 2023



Finished Sheets adjacent to MSE wall





LARGE DIAMETER SINKHOLE SW FL

SPRING –SUMMER 2023



Deadman
Disc. Sheets



Sheet piles G-01, G-02, and G-03 installed



57 Stone Placement

Filling Annulus between Sheets and MSE wall Face

LARGE DIAMETER SINKHOLE SW FL

SPRING –SUMMER 2023

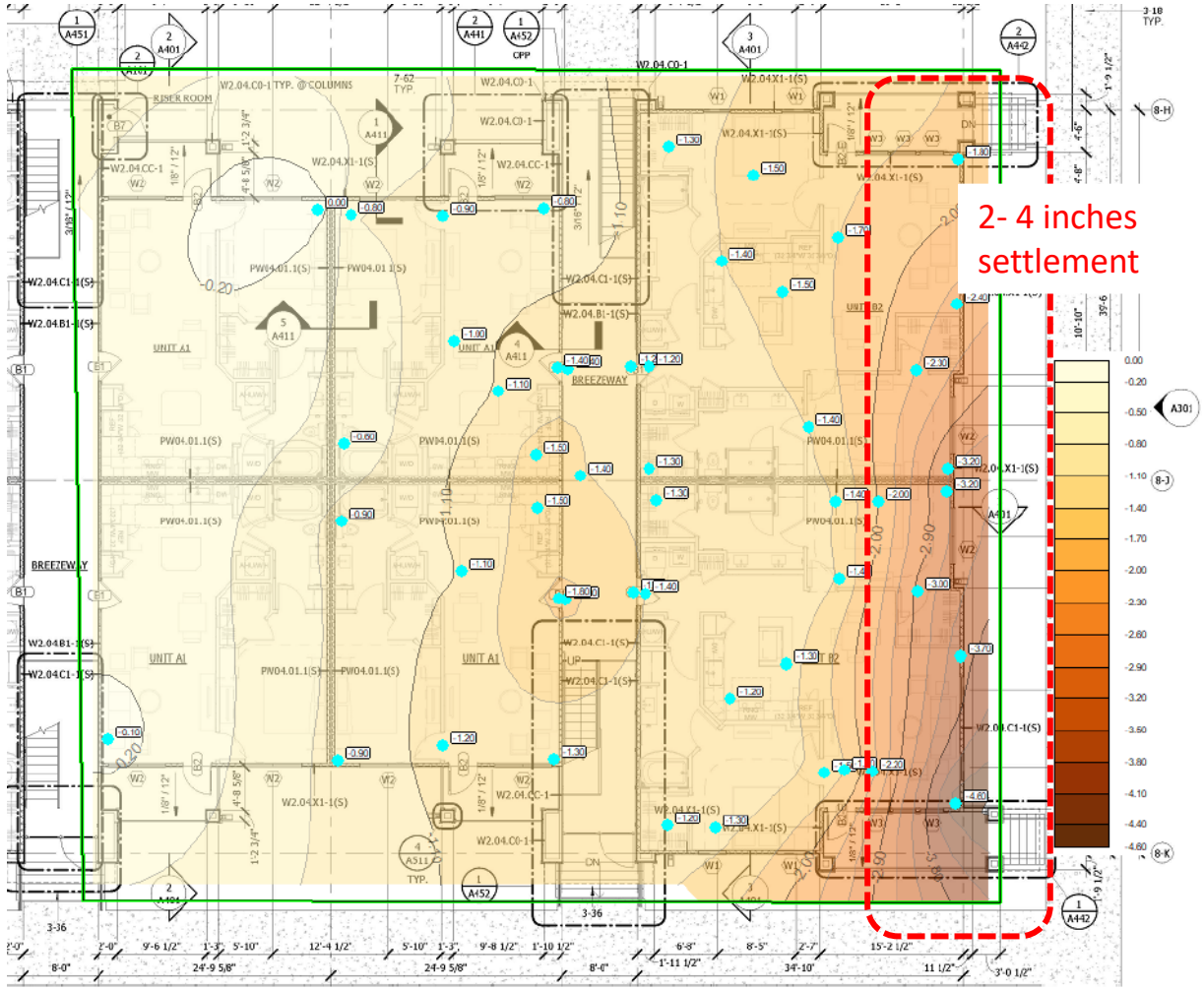




LARGE DIAMETER SINKHOLE SW FL

MAY 2023 – LET'S NOT FORGET ABOUT BLDG. 8

- We started monitoring the slab/foundation movements of Building 8 in the winter of 2023 once it was determined MSE wall #2 was moving laterally.
- Building 8, as of May 2023, had settled on the west side as much as 4 inches.
- Building 10 never really settled or moved laterally.

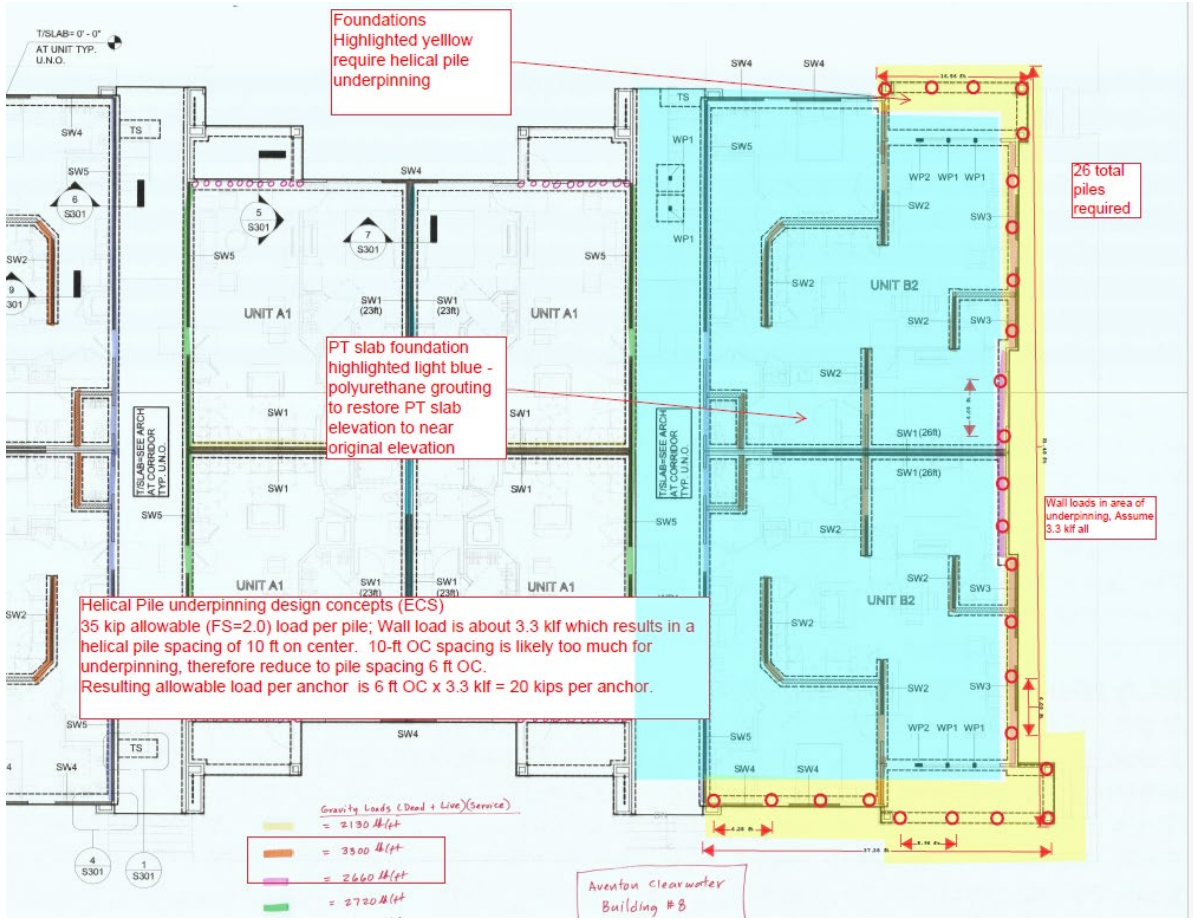


Slab Profilometer Survey – Bldg. 8

LARGE DIAMETER SINKHOLE SW FL

SUMMER OF 2023 – TWO ISSUES TO ADDRESS: RETAINING WALL #2 AND BLDG. 8

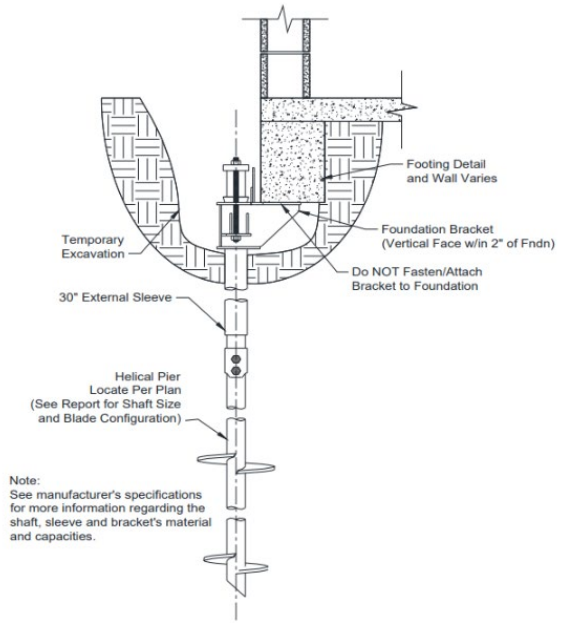
- Both are important, we focused on both simultaneously.
- Geotechnical Engineer and SER determined what the loads were on Bldg. 8 and an underpinning scheme of Helical Ground Anchors was conceived.
- 26-30 Helical Piles needed around eastern wall of Bldg. 8, loaded to about 35 kips each.



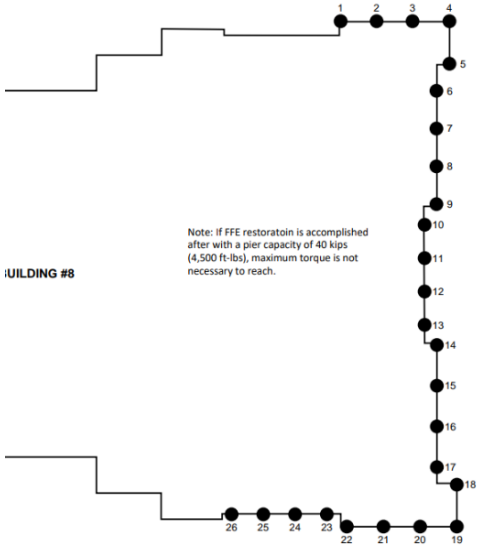
LARGE DIAMETER SINKHOLE SW FL

SUMMER OF 2023 – TWO ISSUES TO ADDRESS: RETAINING WALL #2 AND BLDG. 8

- The Geotechnical Engineer created a Scope of Work and preliminary engineering design to Helical Anchor Contractors for pricing.
- Plan was to jack off the helical anchors and restore foundation to NEAR its original design elevation. Hoping PT slab pulls up other settled areas beyond perimeter.
- With client and help from some industry contractors, we determined polyurethane grouting is needed below the PT slab AFTER we jack the perimeter walls with helical anchors, filling the “gap” between bottom of slab created during pile jacking. While this grouting technique can “lift” structural elements, the program was not designed to.



HELICAL PIER TO FOOTING DETAIL



LATE FALL 2023 –BLDG. 8 REMEDIATION



Rotary Hydraulic Equipment used to Screw Anchor into Ground. Foundation “Bracket” in Foreground.



Helical Bracket



Battered Helical Anchors Installed Around Bldg. 8 Perimeter

LATE FALL OF 2023 –BLDG. 8 REMEDIATION



Helical Pier P-15

Pier in Good Condition



Helical Pier P-18

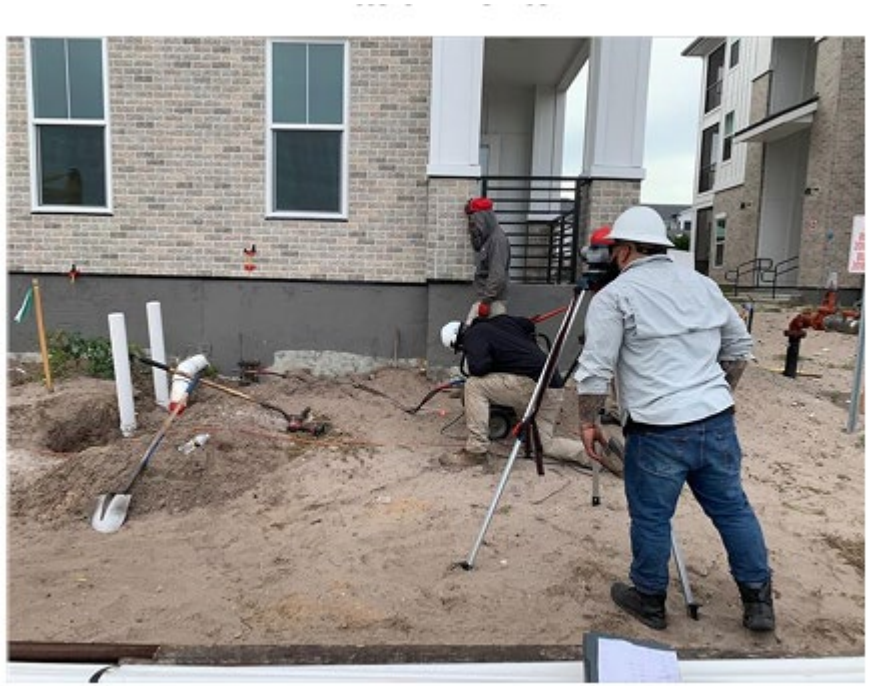
After jacking PT foundation upward, locked off load with screws designed to work with bracket.

- Hydraulic Jacking of Bracket attached to PT foundation
- After the piles were screwed in to plan depth in summer of 2023, there was a “wait” period until late Fall 2023 to jack foundation.
- Wait period was needed to allow the Retaining wall Repairs to proceed and wall to stabilize.
- We did not want to repair Bldg. 8 completely until repairs to retaining wall done.

LATE FALL OF 2023 –BLDG. 8 REMEDIATION



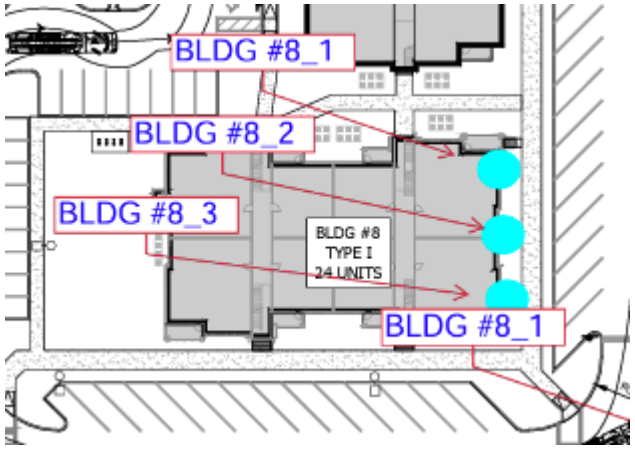
Two Hydraulic Compressors 12-20-23



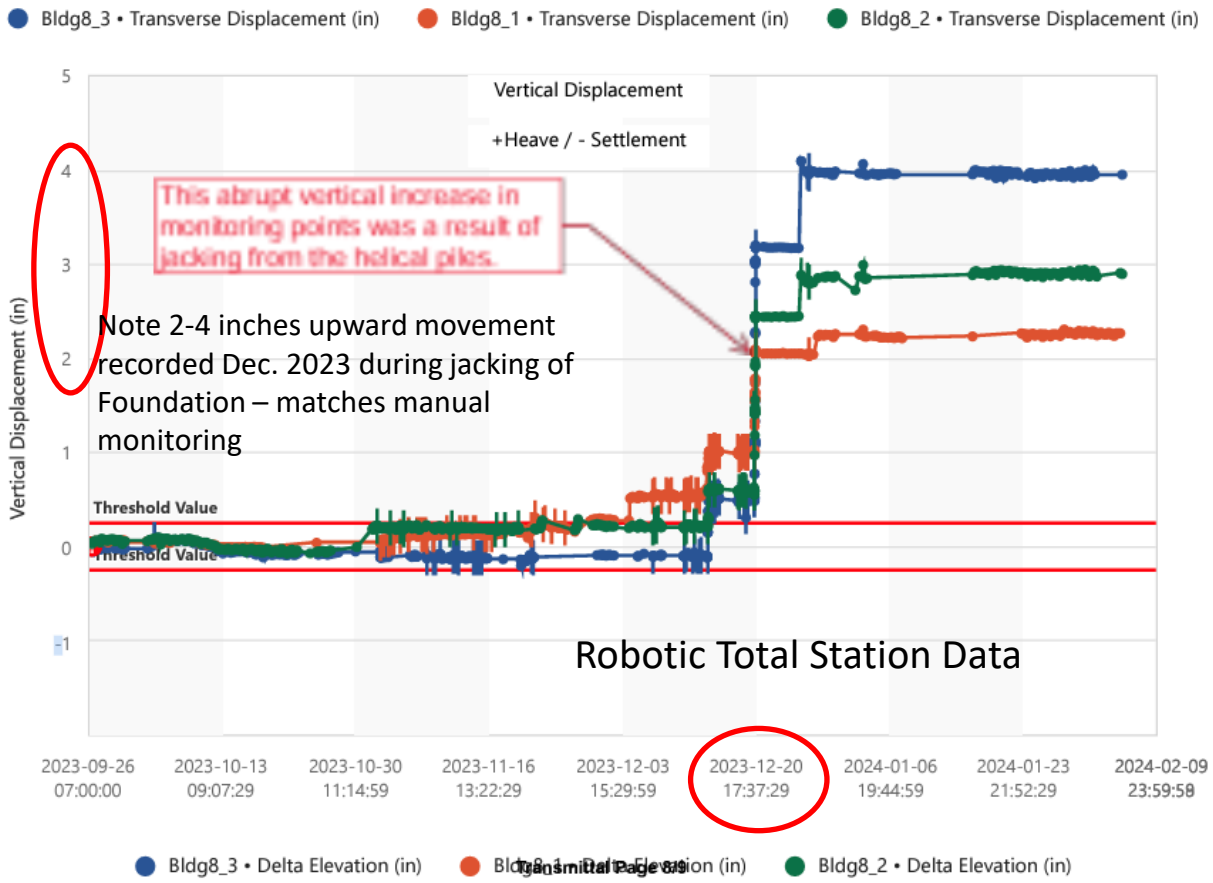
Lifting Building 8 12-14-23

- How we managed monitoring of Foundation Lifting (too much is bad)
- Manual Total Station / Level usage.
- Also Recorded/Documented Upward Movement with remote, robotic total station (explained later).

LATE FALL OF 2023 –BLDG. 8 REMEDIATION



- Bldg. 8 had 3-targets affixed to its roof line that were monitored with our robotic total station (near continuous monitoring).
- Monitoring started in Fall of 2023 for Bldgs. 8, 10 and Key parts of the newly reinforced MSE wall.



LARGE DIAMETER SINKHOLE SW FL

LATE FALL OF 2023 –BLDG. 8 MOVEMENT MONITORING



Robotic Total Station Setup to shoot targets on new Sheet Pile Wall and Building 8.

FALL OF 2023 –BLDG. 8 REMEDIATION

- Able to restore up to 4 inches cumulative settlement in SE Corner of Bldg. 8 where the settlements were more severe.
- Overall, the team was happy with the foundation jacking program.
- Some anchors were damaged during lifting, bending at the base of the bracket where the moment load is highest.
- Bent piles abandoned in place – new piles installed.



Helical Pier P-12

Bent Pier

Helical Pier	Lift Observed (in)	Total Cumulative Lift Observed (in)	Pressure (psi)	Date	Time
4	1/4	1/4	Off	12/21/2023	1:23 PM
5	7/16	7/16	Off	12/21/2023	1:23 PM
6	1/2	15/16	4200	12/21/2023	1:23 PM
6	1 1/8	1 5/8	3300	12/21/2023	3:24 PM
7	1/16	1/16	6000	12/14/2023	3:47 PM
7	9/16	5/8	4200	12/21/2023	1:23 PM
7	1 3/16	1 4/16	3300	12/21/2023	3:24 PM
8	5/16	5/16	6000	12/14/2023	3:10 PM
8	3/4	1 1/16	4200	12/21/2023	1:23 PM
8	1 3/8	1 11/16	3300	12/21/2023	3:24 PM
9	1/4	1/4	6000	12/14/2023	3:10 PM
9	3/4	1	4200	12/21/2023	1:23 PM
9	1 5/8	1 7/8	3300	12/21/2023	3:24 PM
10	3/16	3/16	6000	12/14/2023	3:10 PM
10	3/4	15/16	4200	12/21/2023	1:23 PM
10	1 5/8	1 13/16	3300	12/21/2023	3:24 PM
11	3/16	3/16	6000	12/14/2023	3:10 PM
11	7/8	1 1/16	4200	12/21/2023	1:23 PM
11	1 3/4	1 15/16	3300	12/21/2023	3:24 PM
12	1/4	1/4	6000	12/14/2023	3:10 PM
12	7/8	1 1/8	4200	12/21/2023	1:23 PM
12	1 15/16	2 3/16	3300	12/21/2023	3:24 PM
13	3/16	3/16	6000	12/14/2023	3:10 PM
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13	2	2 3/16	3300	12/21/2023	3:24 PM
14	3/16	3/16	6000	12/14/2023	2:05 PM
14	7/8	1 1/16	4200	12/21/2023	1:23 PM
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16	2 1/2	2 11/16	3600	12/21/2023	4:10 PM
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17	2 1/2	2 5/8	3300	12/21/2023	3:24 PM
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18	3/16	3/16	6000	12/14/2023	12:10 PM
18	1/2	11/16	Off	12/21/2023	1:23 PM
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19	1/2	1/2	6000	12/14/2023	12:10 PM
19	5/16	13/16	Off	12/21/2023	1:23 PM
19	3	3 1/2	3300	12/21/2023	3:19 PM
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21	1/4	3/8	3300	12/21/2023	2:44 PM
21	1/2	5/8	3400	12/21/2023	3:22 PM
21	1 1/2	1 5/8	3600	12/21/2023	4:10 PM
22	1/8	1/8	6000	12/14/2023	12:10 PM

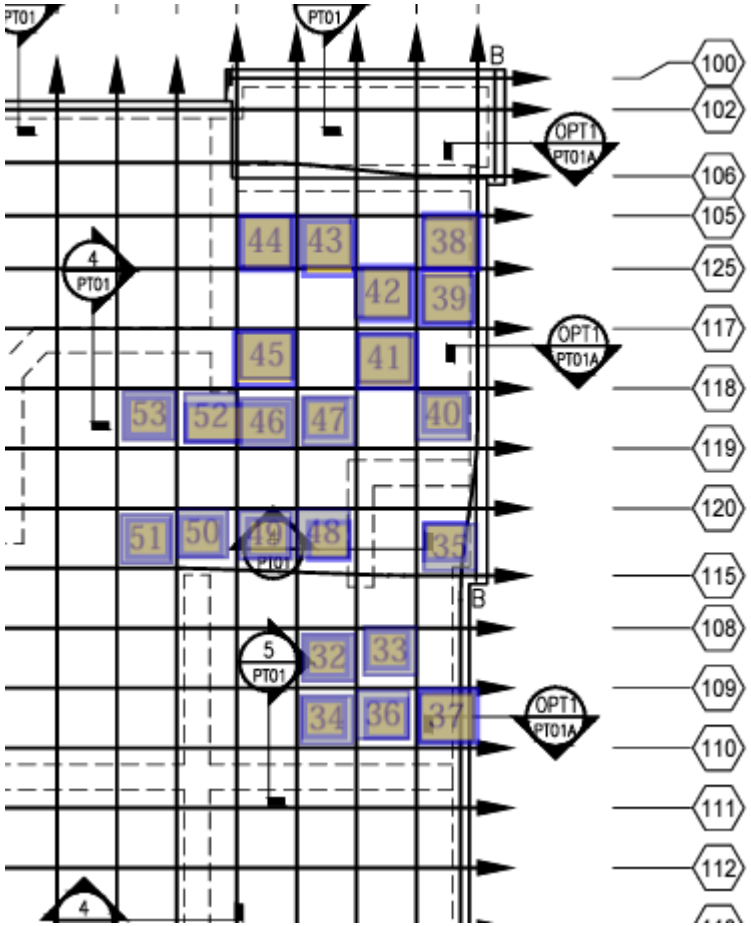
FALL OF 2023 –BLDG. 8 REMEDIATION





“Poly-Level” Urethane Injection,
Approx. 3 ft O.C. after jacking

FALL OF 2023 –BLDG. 8 REMEDIATION



Typical Poly Leveling Activity – one day

Foam Injection Log

[redacted]			Date:	12/28/2023
			Weather:	Cloudy
			Project Number:	41-3271-F
Start time:	9:11 AM	End Time:	12:00 PM	
Pump Calibration (lbs/stoke) 0.33	Pins	Depth (ft)	Strokes	Foam Injected (lbs)
	32	1	317	104.6
	33	1	21	6.9
Comments/Observations:	34	1	282	93.1
	35	1	1	0.3
Pump calabration was provided by LRE	36	1	51	16.8
	37	1	77	25.4
	38	1	538	177.5
LRE elected to stop foam injection at Pin 38 once foam was	39	1	15	5.0
observed protruding threw the adjacent crack in the slab. It	40	1	124	40.9
should be noted the foam caused the crack to expand and	41	1	51	16.8
separate vertically and horizontally.	42	1	15	5.0
	43	1	11	3.6
	44	1	66	21.8
LRE drilled holes within the slab about 8 inches deep within	45	1	58	19.1
the northwest bedroom and encoutered subgrade soils.	46	1	59	19.5
There for ELR did not inject any foam in this area.	47	1	17	5.6
	48	1	25	8.3
	49	1	57	18.8
	50	1	37	12.2
	51	1	12	4.0
	52	1	12	4.0
	53	1	10	3.3

Daily Record of LBS. Injected

Total injected foams	612.5

LARGE DIAMETER SINKHOLE SW FL

SUMMER 2024– DAMAGED BLDG. 8 REPAIRED

- Despite significant eastern wall settlement (3-4 inches), the foundation was still “intact” thanks to the Post-Tensioning Feature.
- PT Type II foundations are a GREAT idea in sinkhole prone areas.
- Some perimeter wall windows and doors had to be reconstructed due to settlement.



Gap at the siding by the balcony of Unit 8206



Cracks at the siding by the balcony of Unit 8206

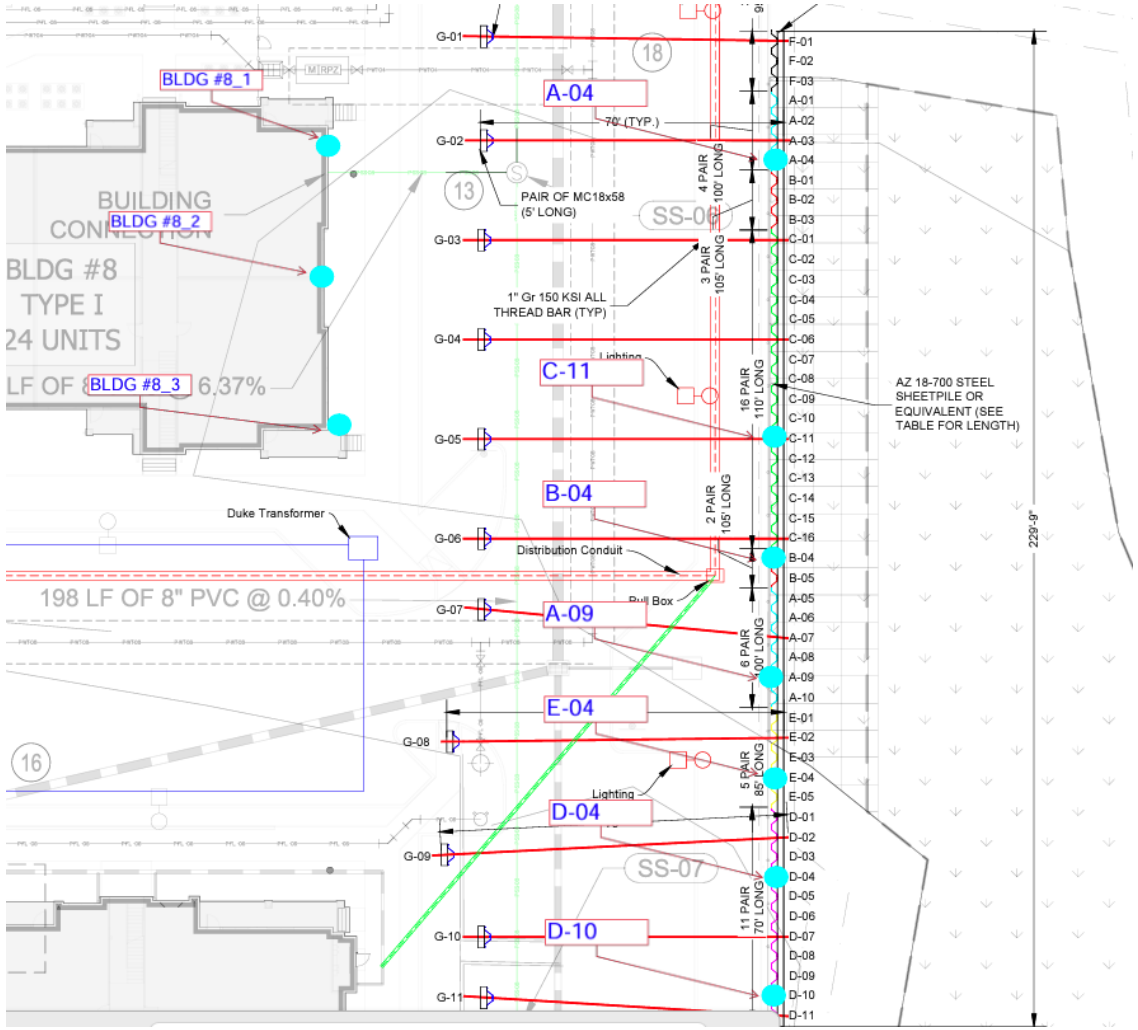


Bldg 8 Window Repairs.1

SUMMER 2024

Final Monitoring Data

- Needed stability before final Bldg. 8 and Hardscape, Utility repairs



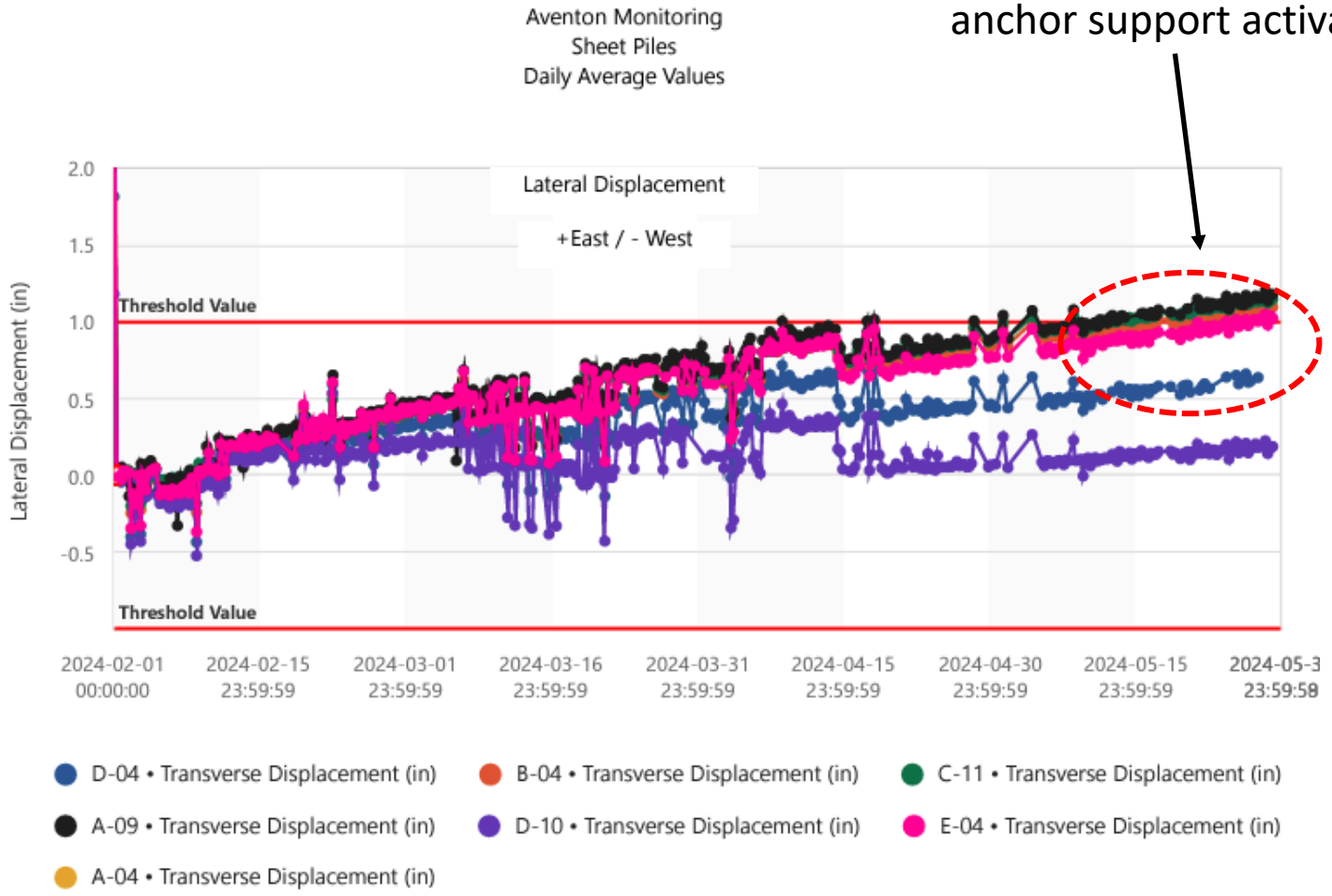
LARGE DIAMETER SINKHOLE SW FL

SUMMER 2024

Final Monitoring Data

- Sheet Piles

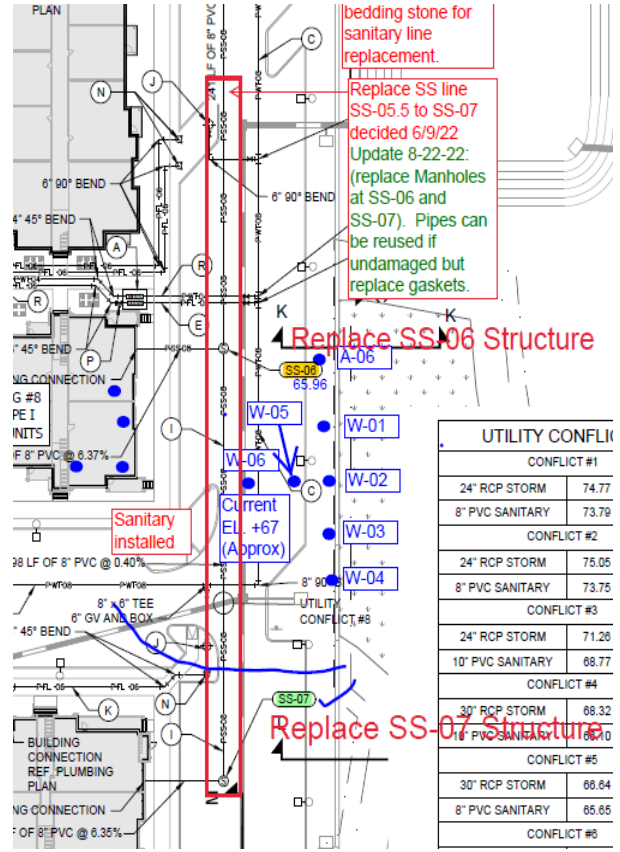
Still some minor lateral creep – anchor support activating



SUMMER 2024 – REPAIRS DONE



New Sheet Pile Wall in Front of MSE Wall #2



Hardscape/Asphalt repaired, Utilities Repaired, Fence installed

LARGE DIAMETER SINKHOLE SW FL

QUESTIONS?



HILLIS-CARNES
ENGINEERING ASSOCIATES

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ARUP

Enhancing structural performance of monopiles using hybrid concepts: a comparative study on lateral pile displacement and load capacity

D. Gondalia, F. Ciruela-Ochoa, F. Mirada

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Problem Statement

Monopiles are the most efficient foundation solution for offshore wind turbines in shallow to medium-depth waters (~30-40m), due to their ease of fabrication and installation.

Growing turbine sizes and offshore expansion demand larger monopiles to support increased loads and longer distances from shore.

Jacket foundations are suitable for deeper waters, but they generally come with higher costs despite being a proven technology.

Strategic goal: *Leverage the use of existing monopile fabrication and installation methods to accommodate deeper water deployments and heavier turbine loads.*

Outline

- Past work
- Proposed concept
- Geometry and Materials
- Results and discussions
- Load transfer mechanism
- Future scope of work
- Q&A

Past Work

- Studies on various hybrid features like reinforced concrete footing, steel footing, skirted steel footing, steel fins, rock armors
  - Complexity to fabrication and installation

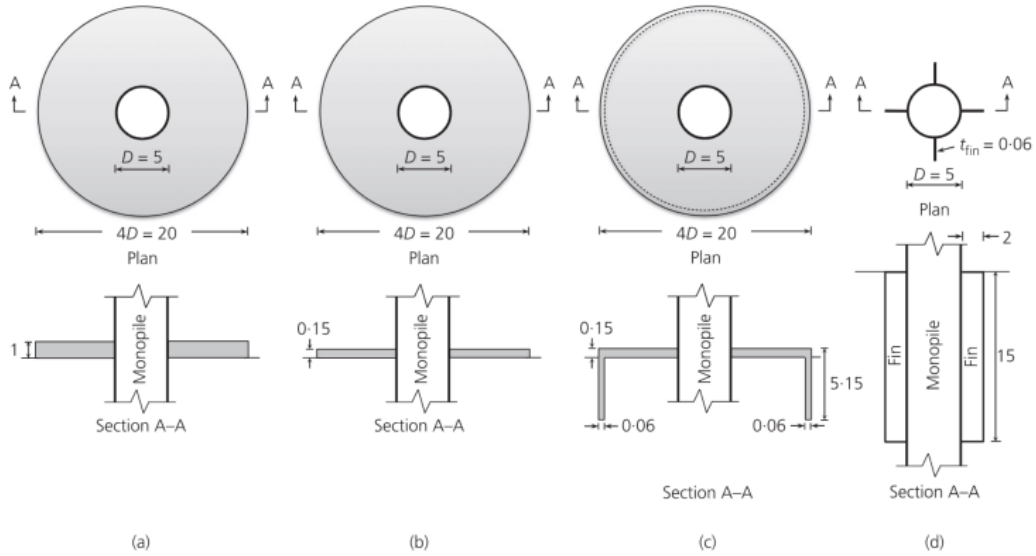
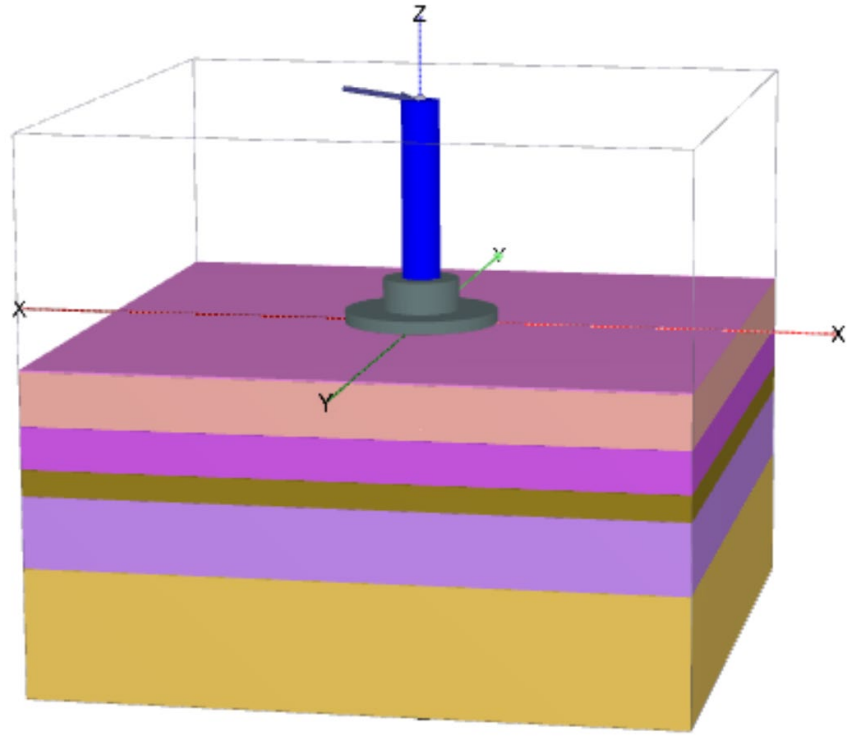


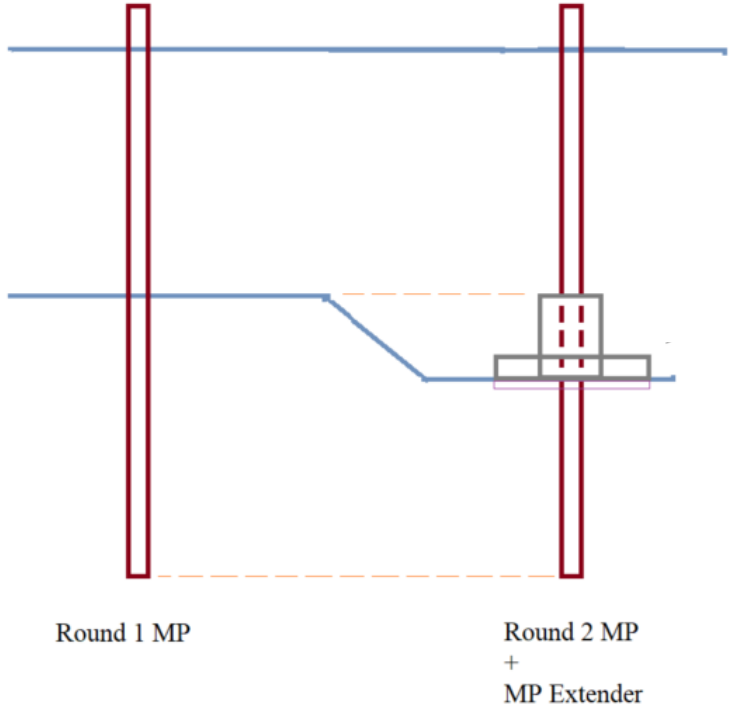
Figure 1. Geometry of hybrid features: (a) RC footing; (b) steel footing; (c) skirted steel footing; (d) steel fins (not to scale; all dimensions are in m)

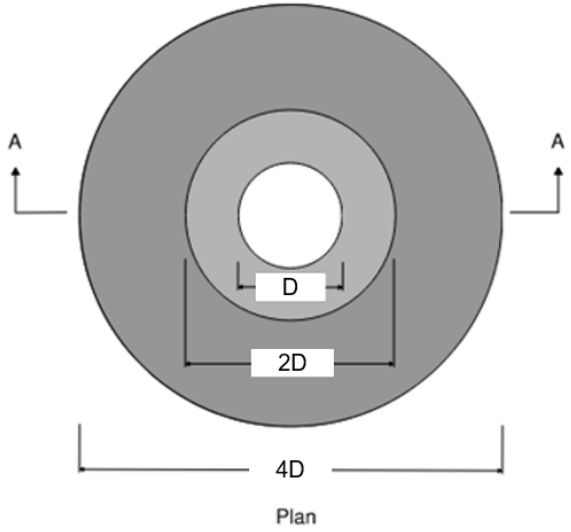
Reference: Haiderali, A., & Madabhushi, G. (2016). Improving the lateral capacity of monopiles in submarine clay. Proceedings of the Institution of Civil Engineers-Ground Improvement, 169(4), 239-252.

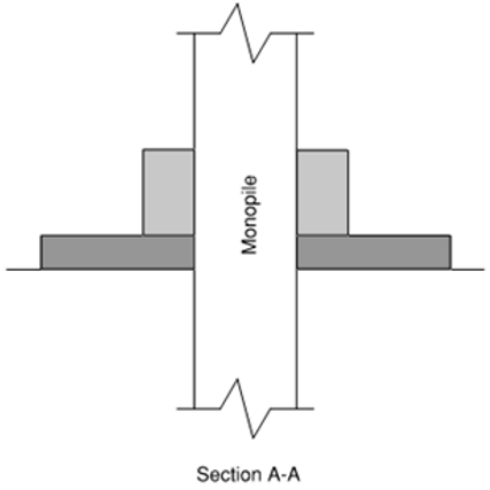
Proposed Concept

- Monopile – steel pipe pile
- Proposed concept – monopile with RC footings



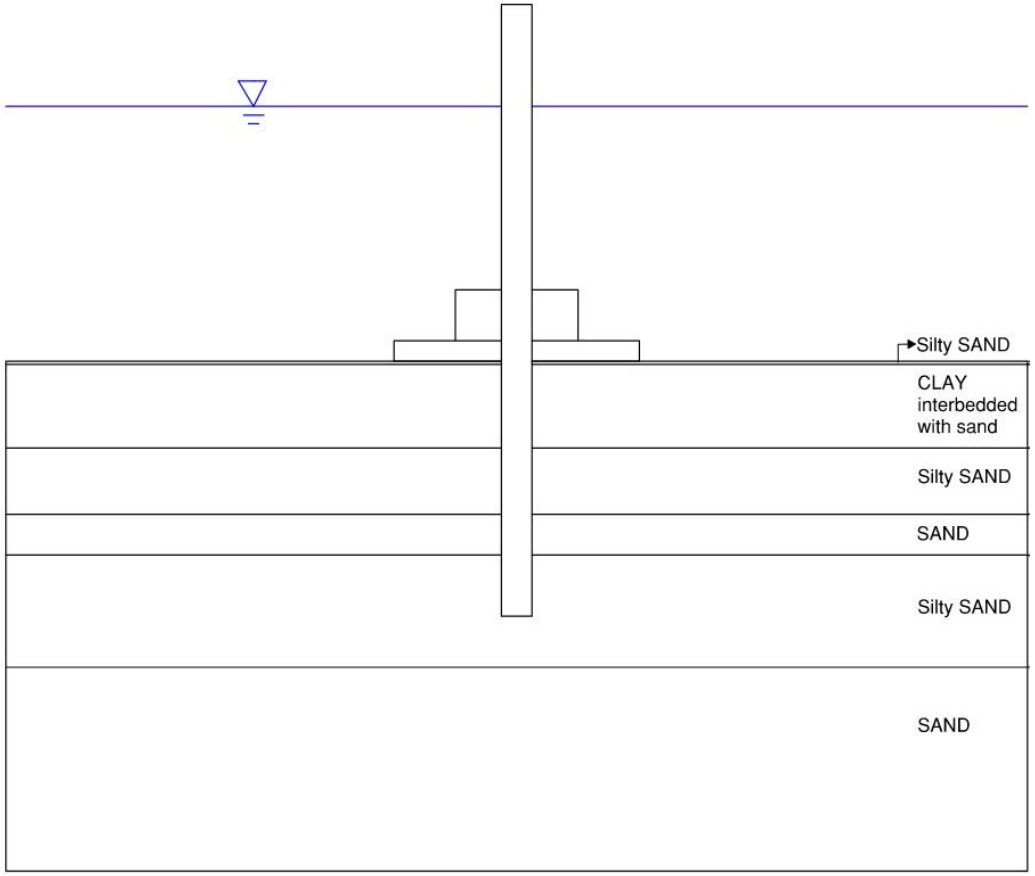






Geometry and Materials

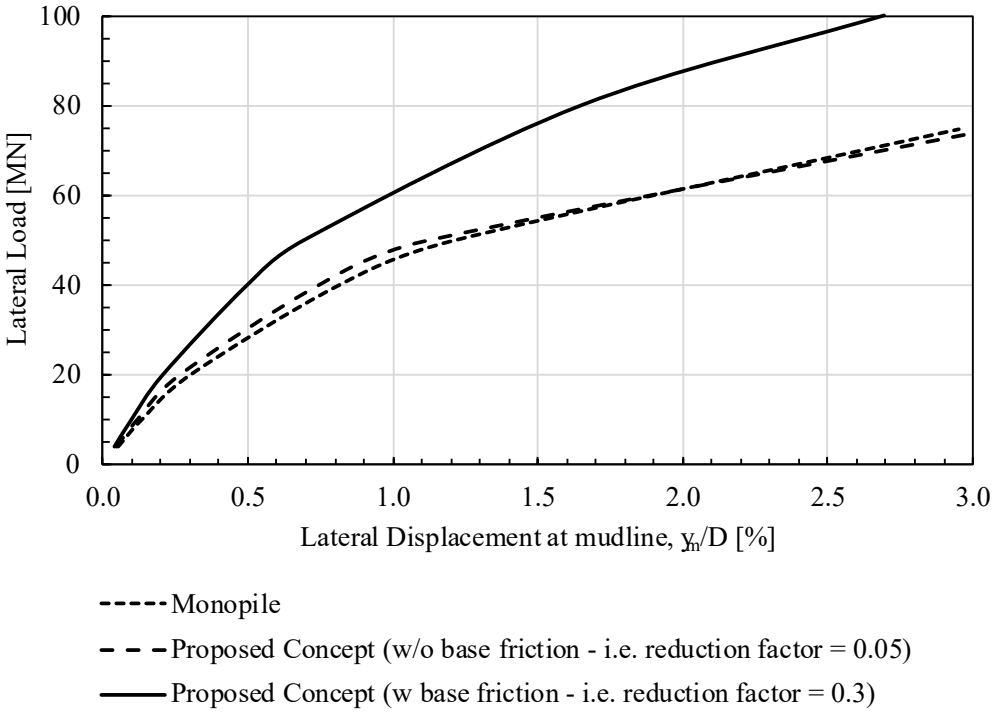
- Water depth – 50m
- Monopile – steel pipe pile
- Proposed concept – RC footings
- Soil (standard mix of clays and sands) – representative materials from the East Coast



Evaluation Scenarios

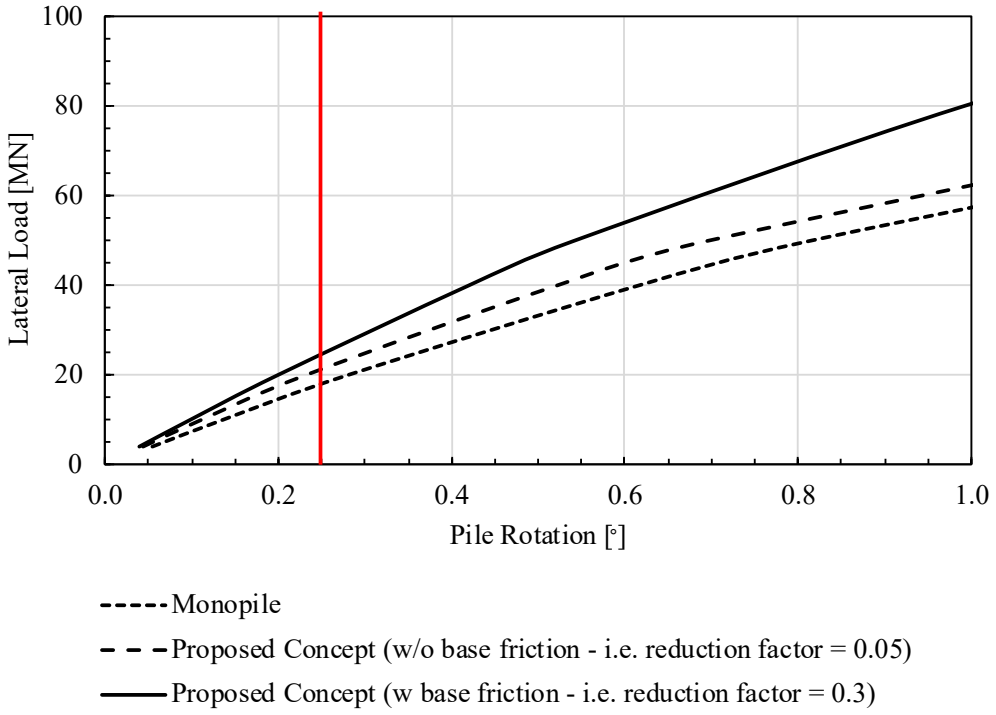
- Single monopile
- Proposed concept without base friction contribution (i.e., Plaxis interface reduction factor of 0.05)
- Proposed concept with base friction contribution (i.e., Plaxis interface reduction factor of 0.3)

Results



Lateral Response

Base friction is important for improvement of lateral capacity



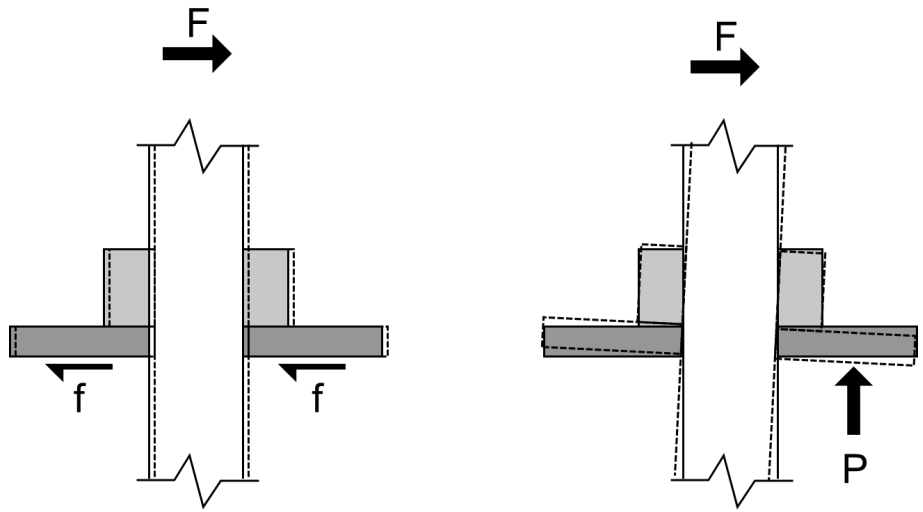
Rotational Response

Point of rotation shifts up with proposed concept resulting in increase in flexural rigidity

Load transfer mechanism

- Monopile:
  - Passive resistance from soil
- Proposed Concept
  - Passive resistance from soil
  - Friction from base of the footing
  - Rotation of footing acting against the soil

Shear stress contribute to increased lateral resistance



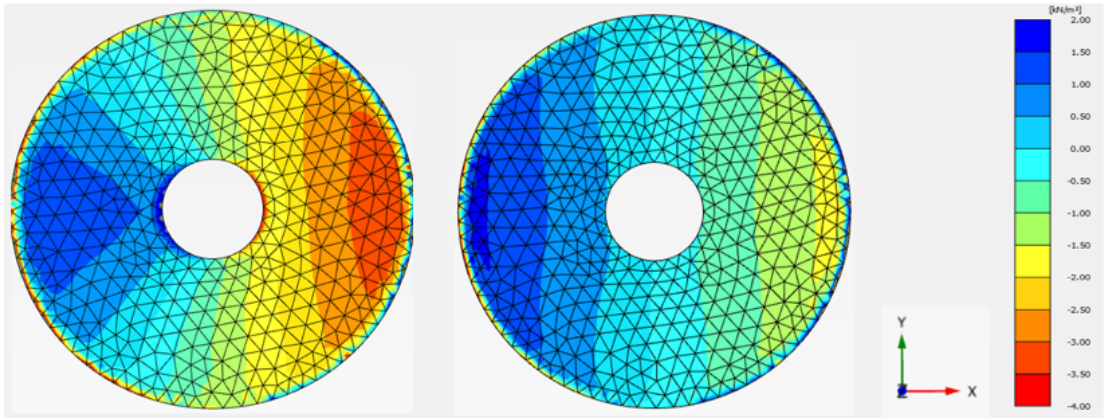


Fig. 5. Interface Shear Stress in the direction of loading for a lateral load of 20MN for (a) Hybrid Monopile w base friction – reduction factor = 0.3; and (b) Hybrid Monopile w/o base friction – reduction factor = 0.05

Future scope of work

- Understanding of benefits of structural performance
- Optimizing the dimensions and geometry of the footings
- Constructability of the proposed concept
  - Preparation of base of seabed
  - Placement of concrete footings

Thank you and questions

ARUP

Experimental Curvature and Moment Response of Pile Foundations: Comparison with *p-y* Predictions

Will Shaffer, P.E.
Rabie Farrag, P.E.
Carter Cox, P.E.
Anne Lemnitzer, Ph.D.

Does our current *p-y* analysis accurately describe pile response to lateral load?

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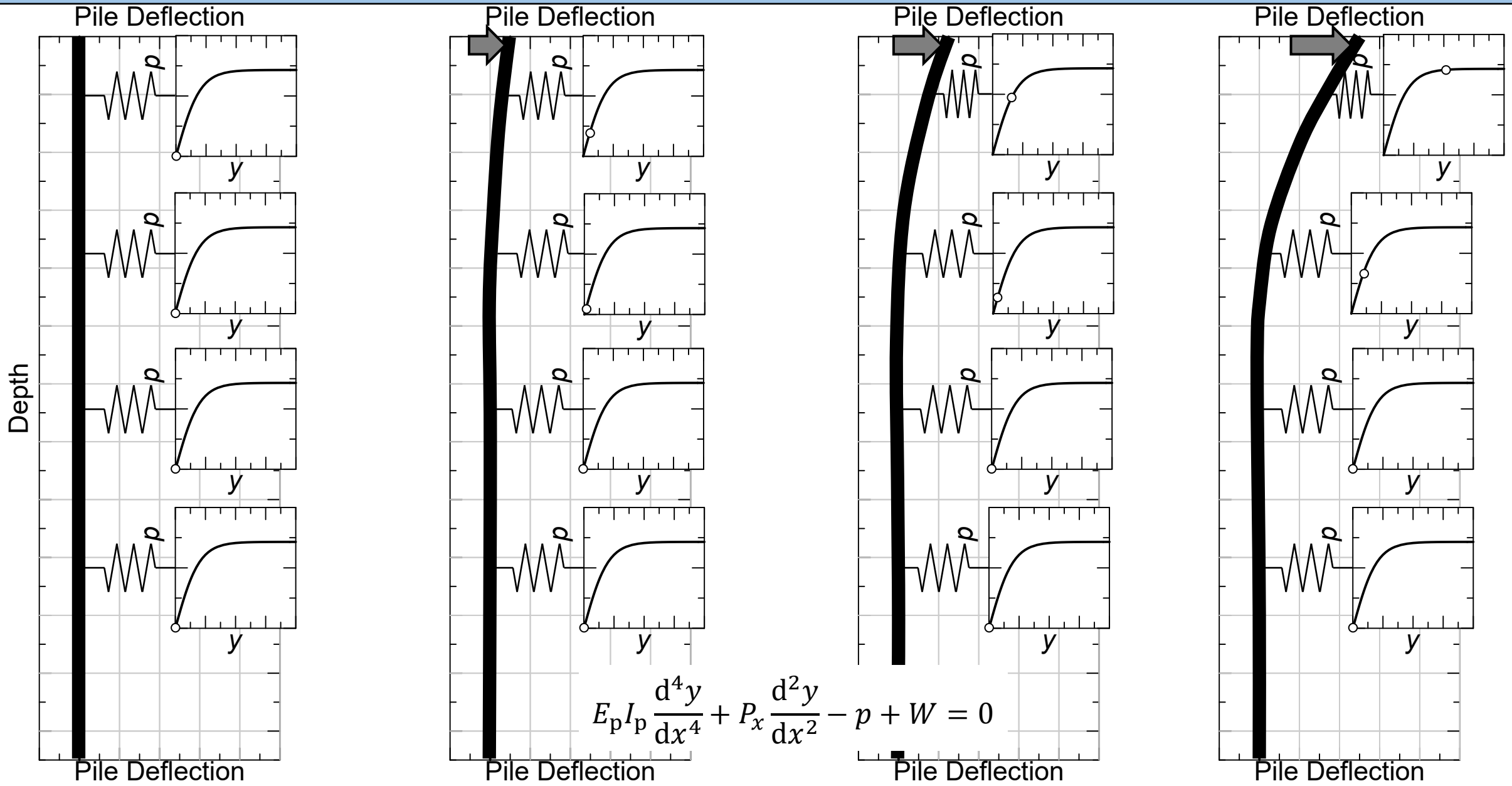
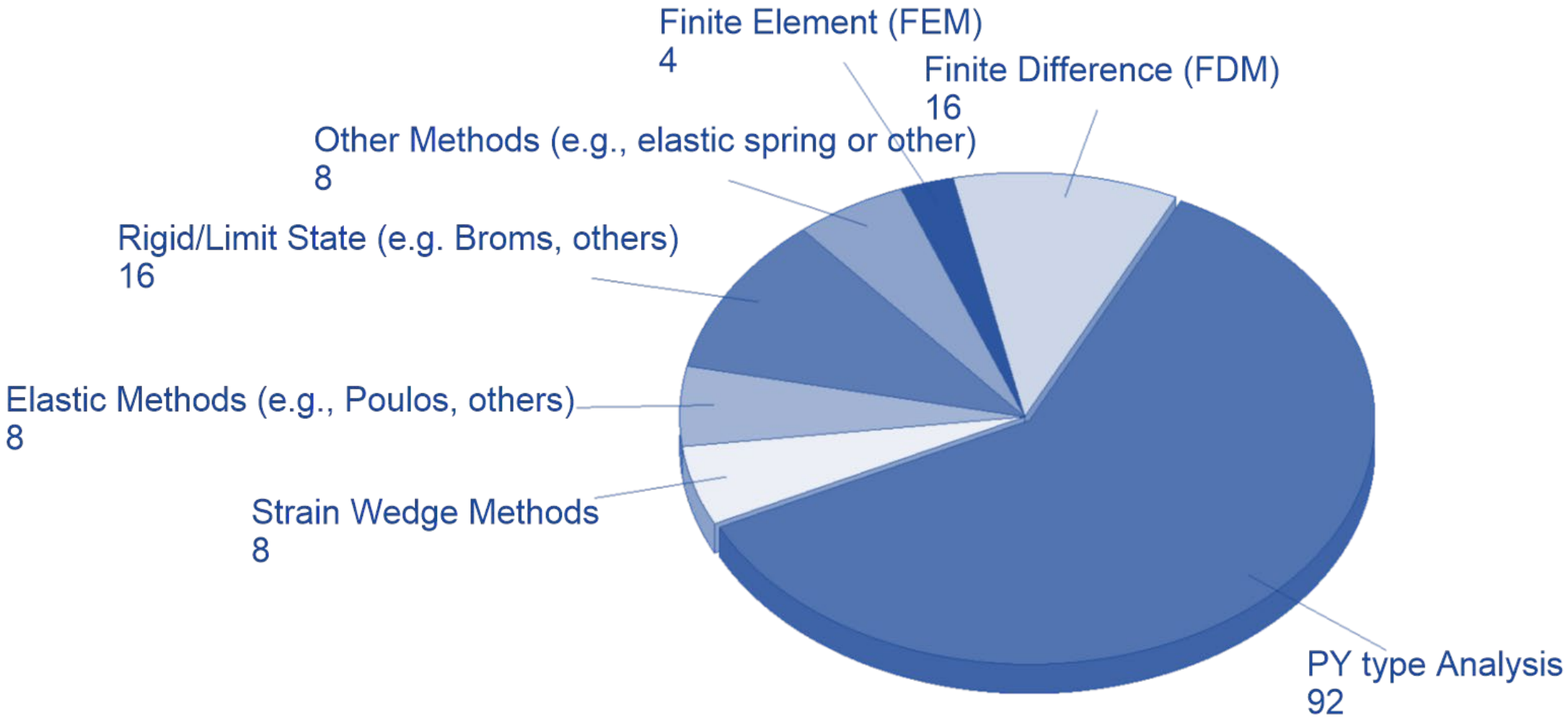
Example *p-y* Curves Along Pile



$$E_{\mathrm{p}} I_{\mathrm{p}} \frac{\mathrm{d}^4 y}{\mathrm{d}x^4} + P_x \frac{\mathrm{d}^2 y}{\mathrm{d}x^2} - p + W = 0$$

Laterally loaded pile design popularity contest



Source: PEER Presentation, A. Lemnitzer, 2018, Berkeley, California

How did we develop *p-y* curves?

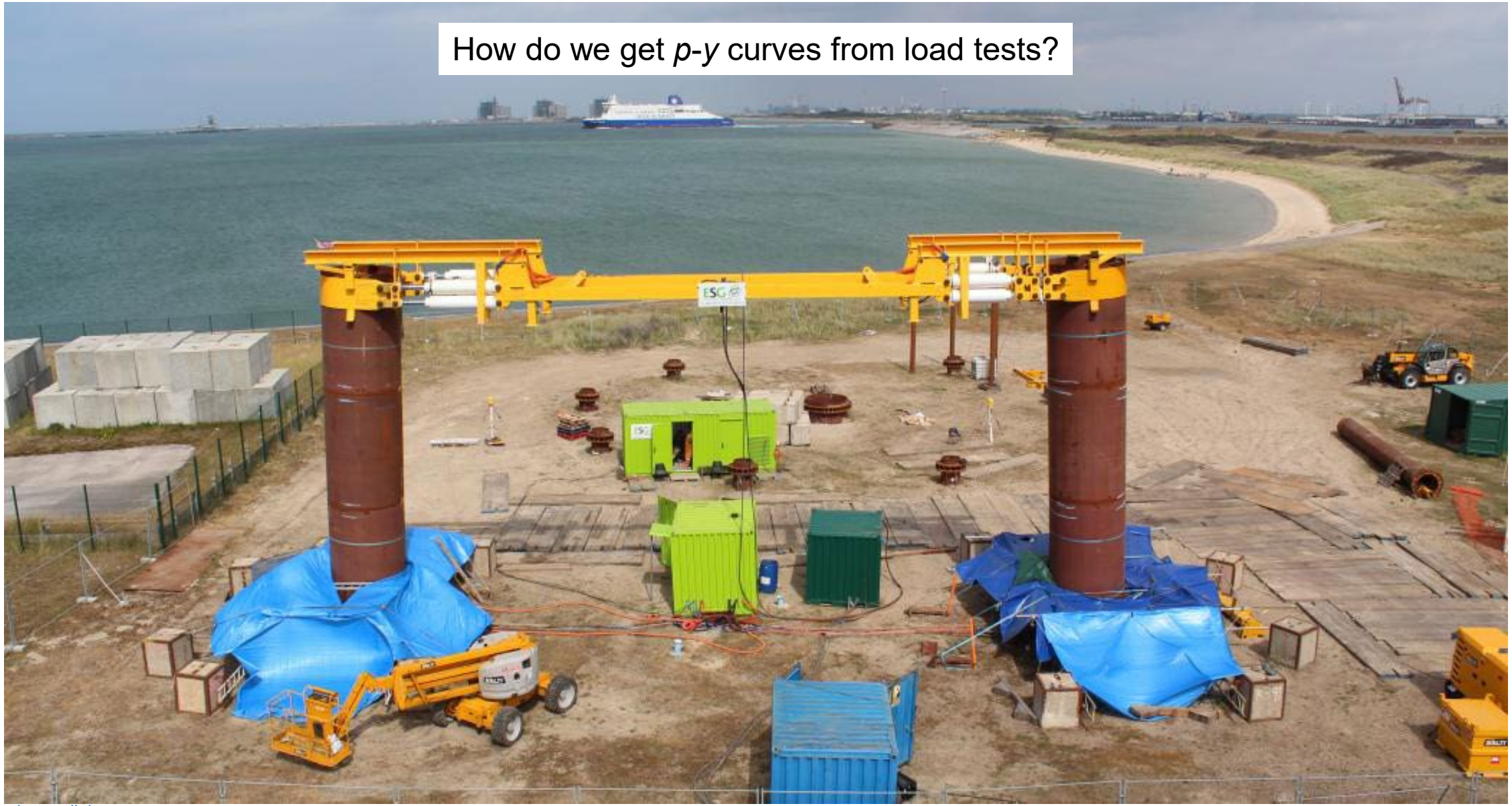
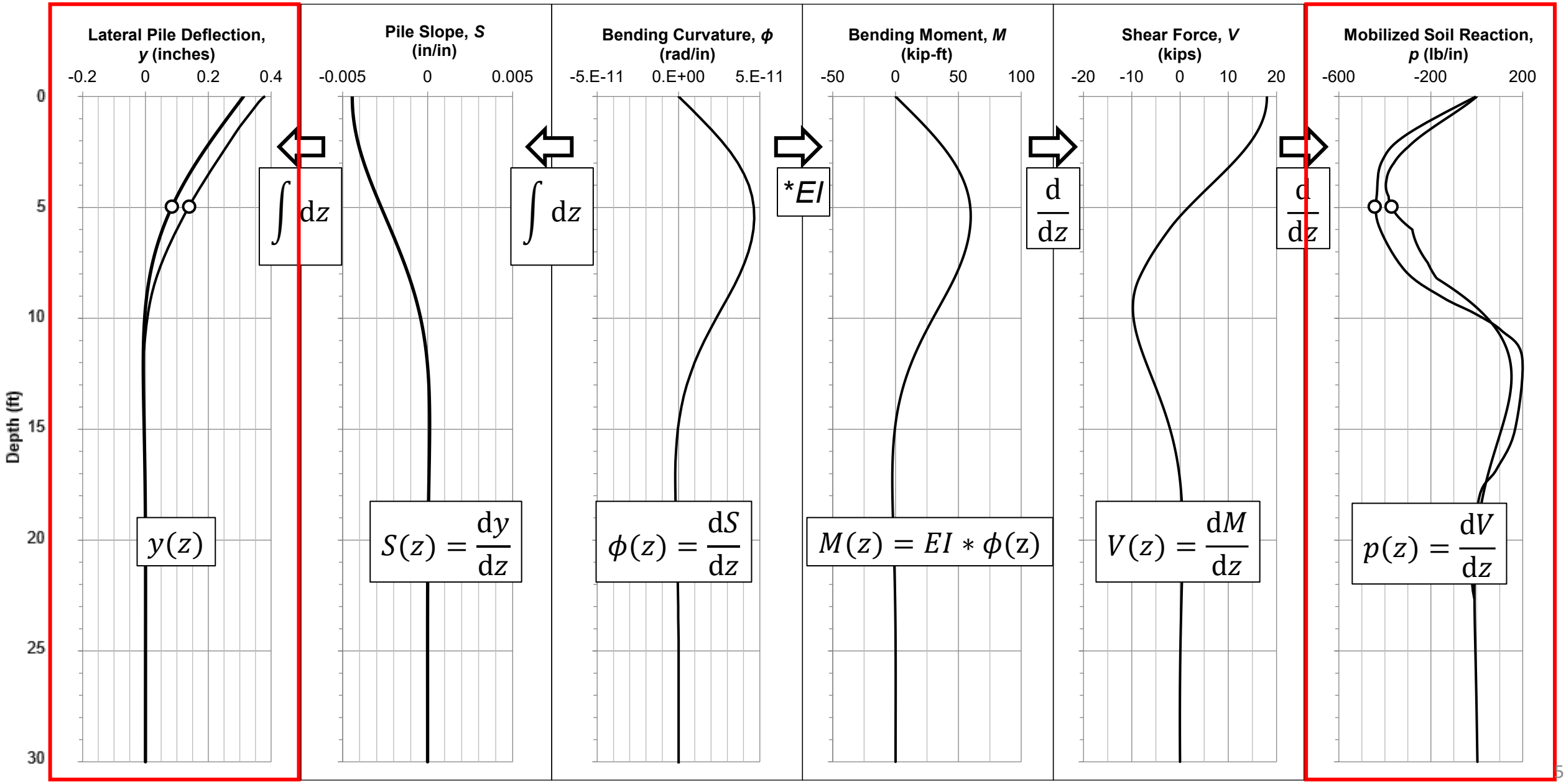
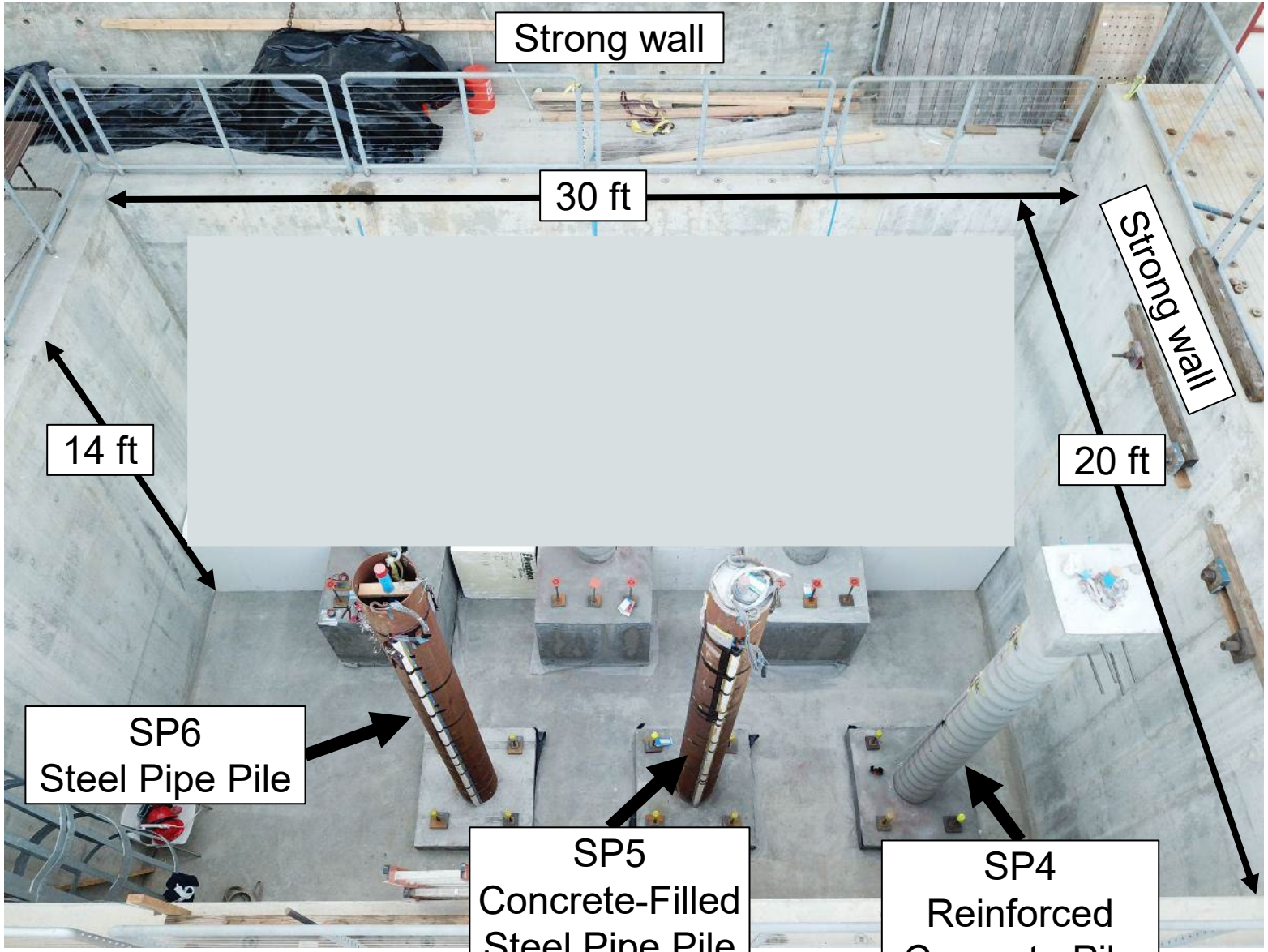


image link

Euler beam theory to derive *p-y* curves



UCI Soil Pit Load Tests



UCI Soil Pit Load Test Construction

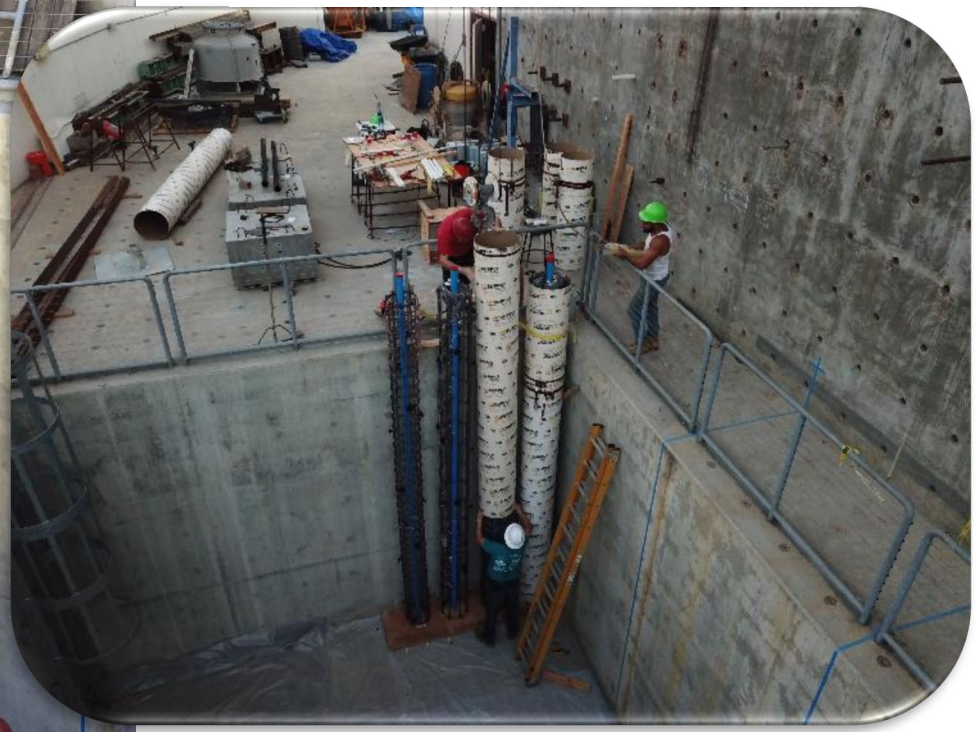
Assemble Rebar Cage and Attach Instrumentation



Assemble Rebar Cage and Attach Instrumentation

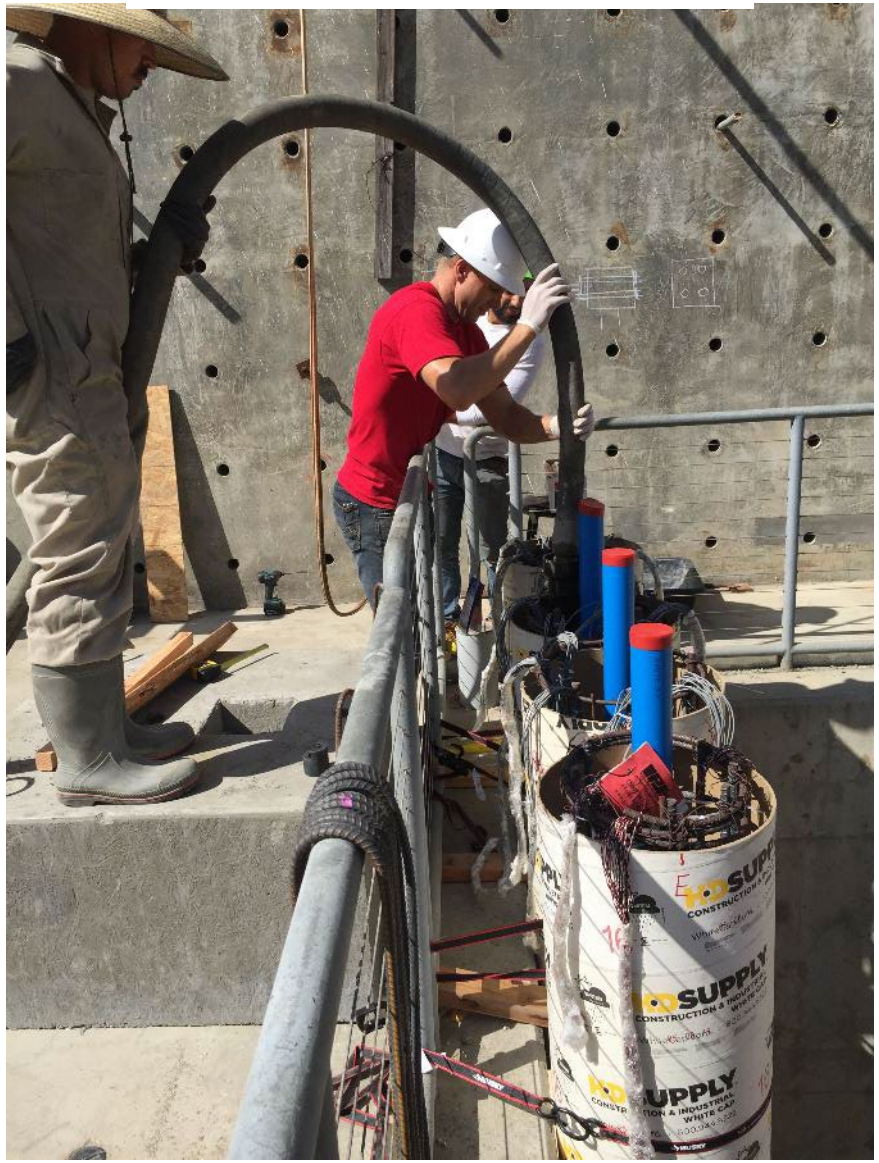


Place Sonotube Form Around Test Pile

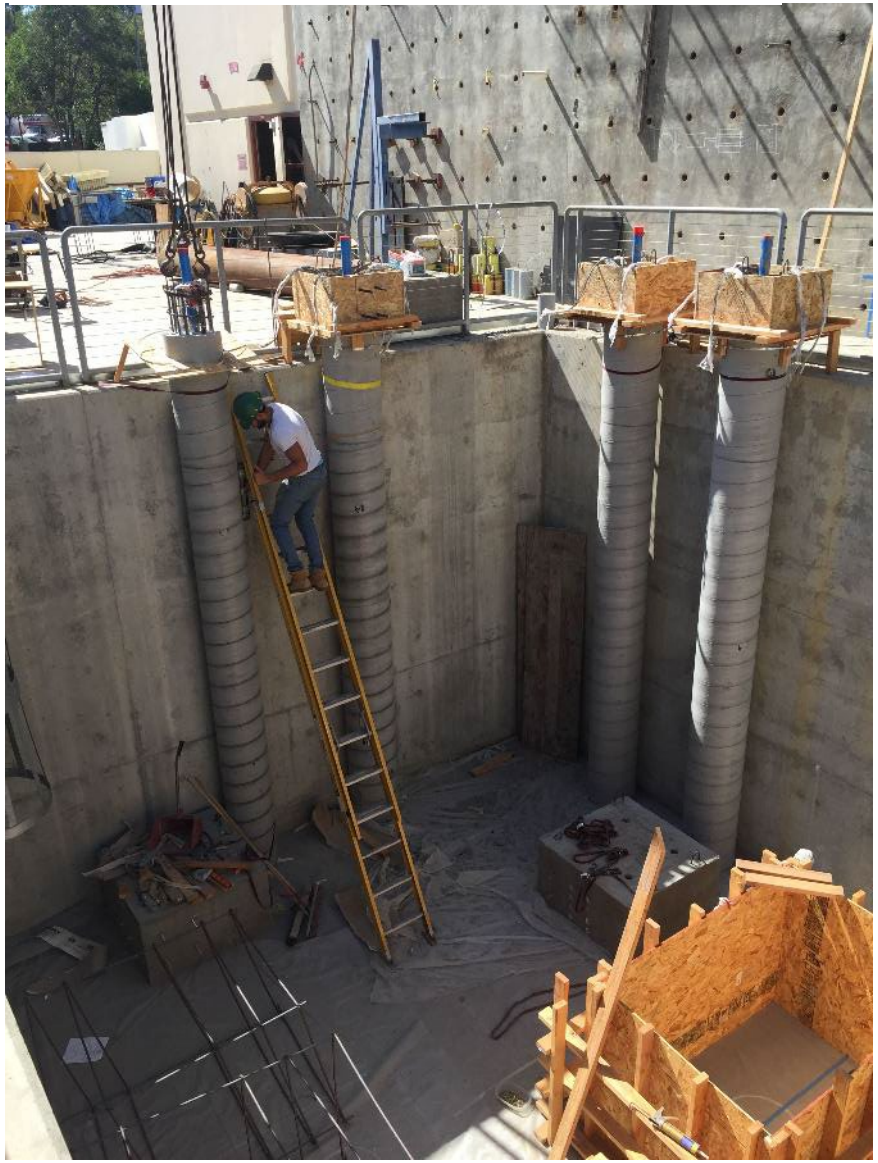


UCI Soil Pit Load Test Construction

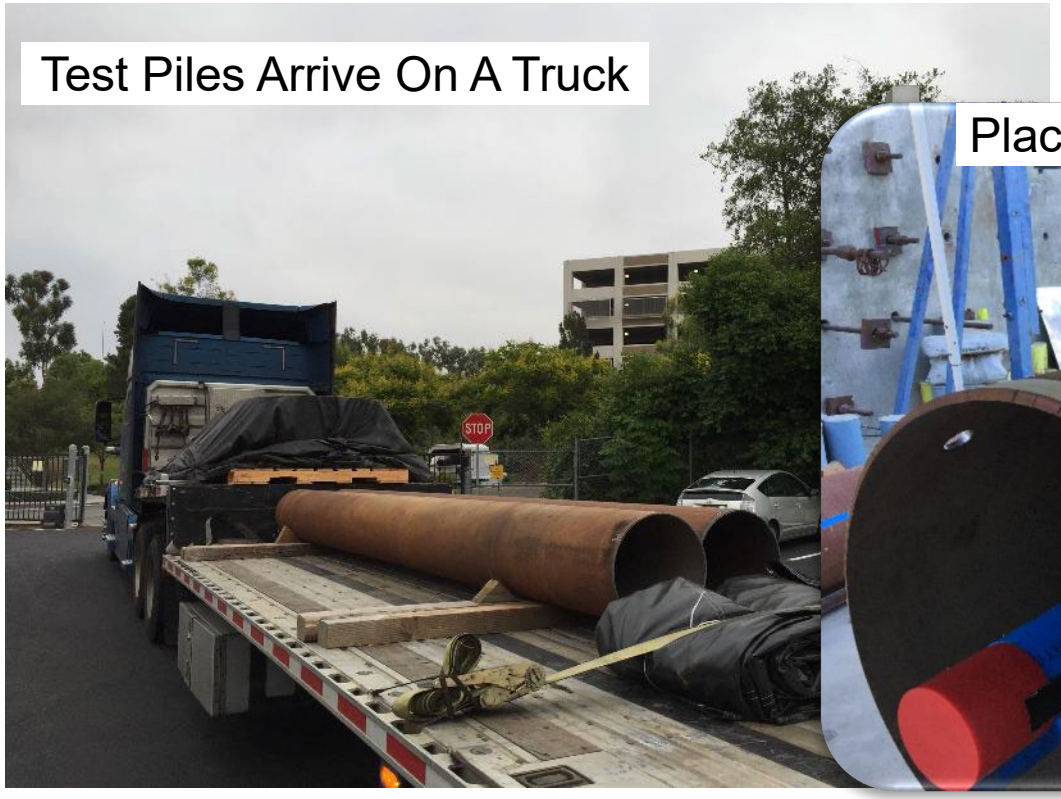
Pour Test Pile Concrete

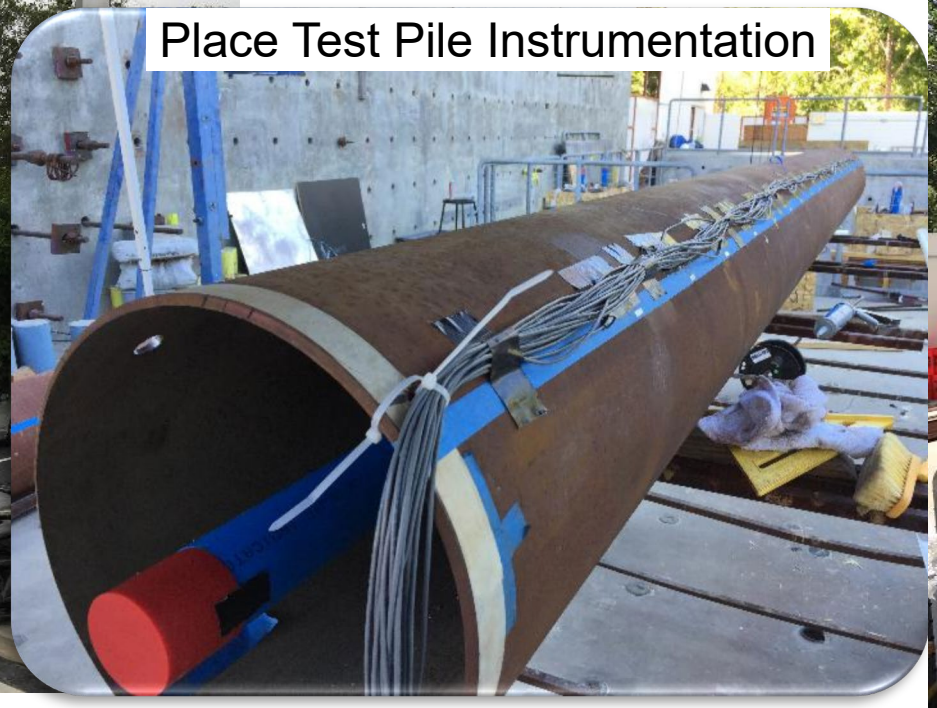


Place Concrete Surface Instrumentation and Cap Concrete



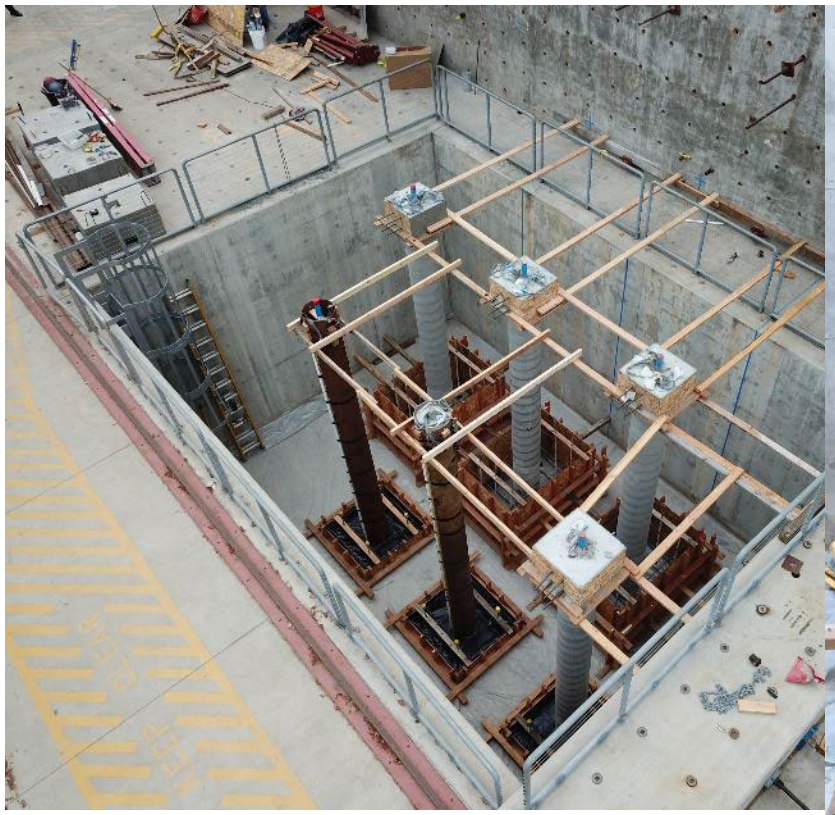
UCI Soil Pit Load Test Construction







UCI Soil Pit Load Test Construction







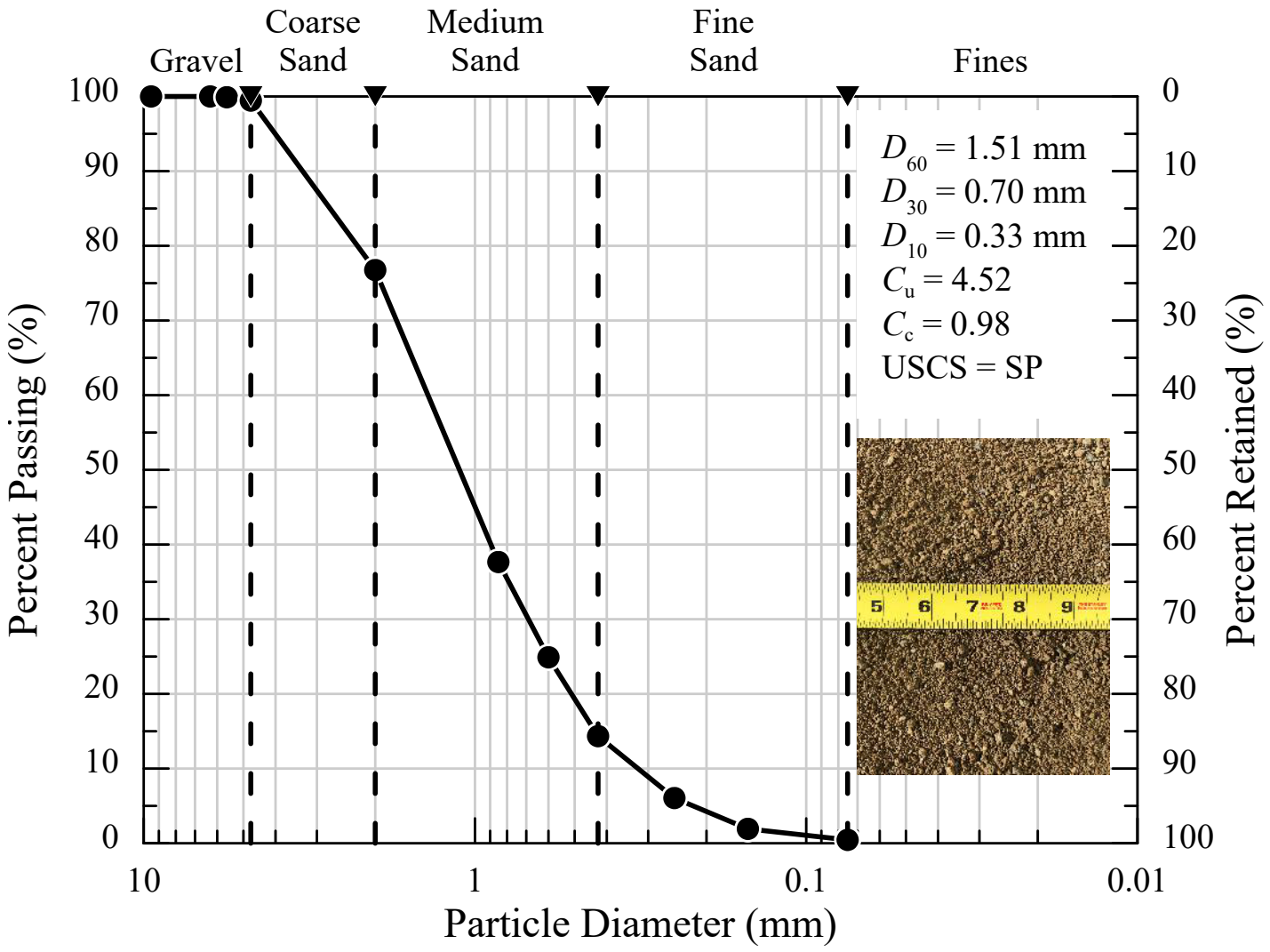
UCI Soil Pit Load Test Construction

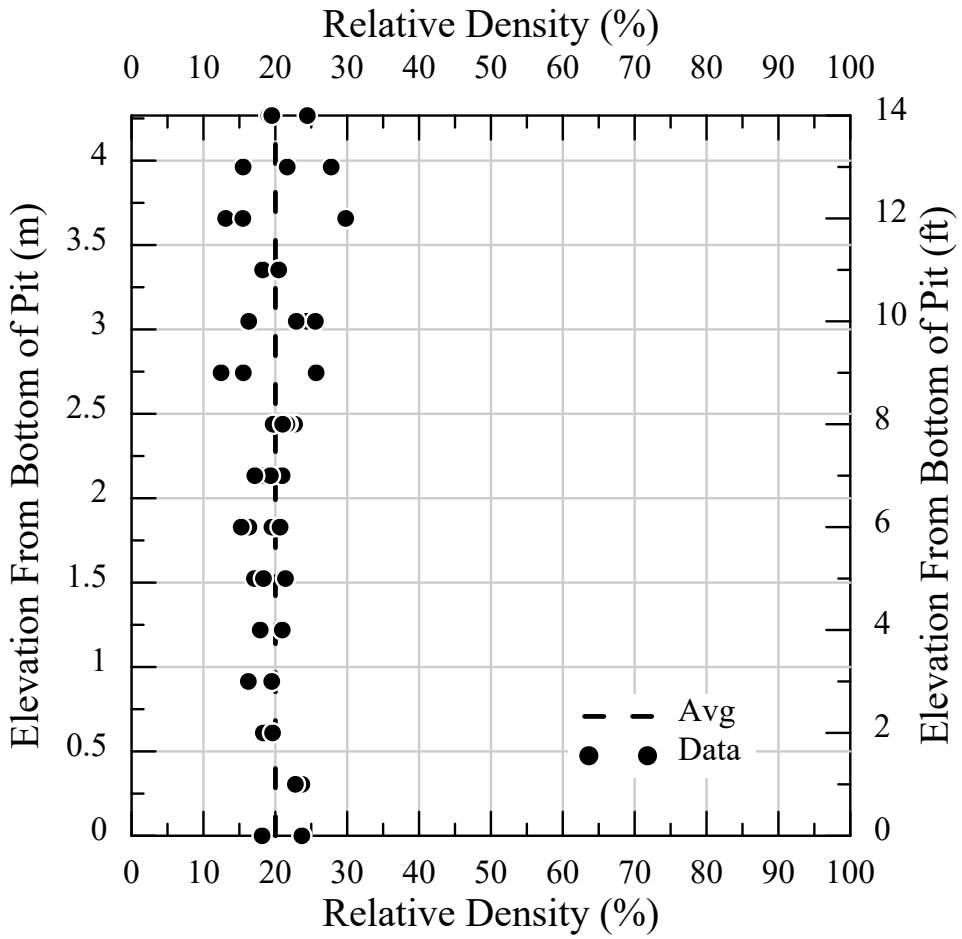




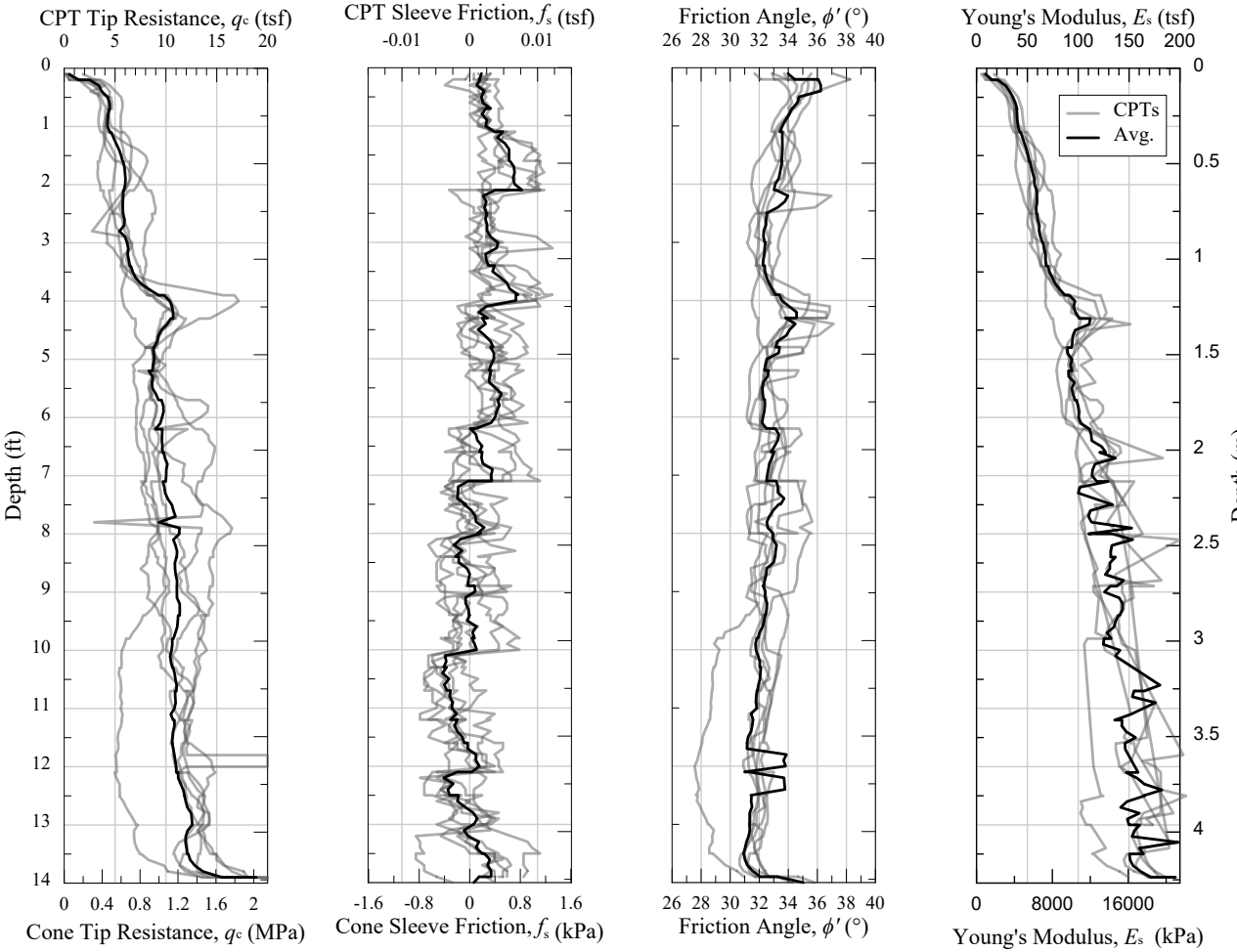


Soil Data

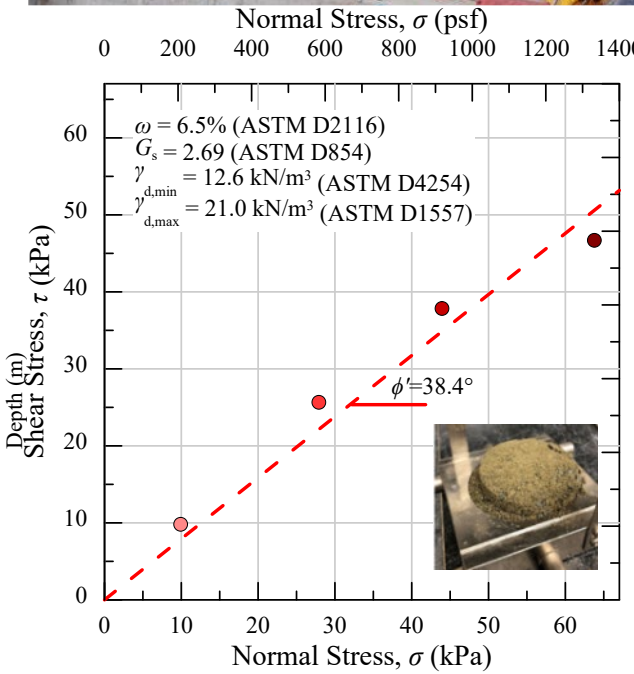

Gravel
Coarse Sand
Medium Sand
Fine Sand
Fines
D_{60} = 1.51 mm
D_{30} = 0.70 mm
D_{10} = 0.33 mm
C_u = 4.52
C_c = 0.98
USCS = SP
Percent Passing (%)
Percent Retained (%)
Particle Diameter (mm)
100
90
80
70
60
50
40
30
20
10
0
10
1
0.1
0.01

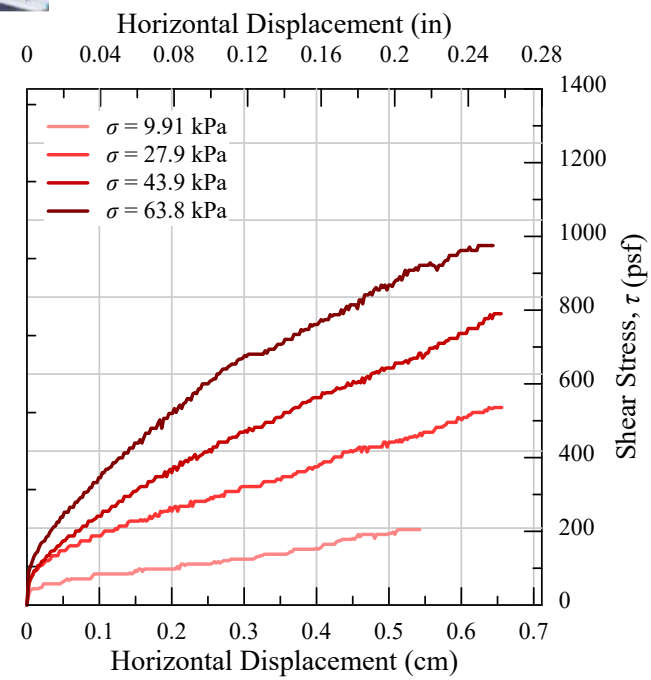

Relative Density (%)
Elevation From Bottom of Pit (m)
Elevation From Bottom of Pit (ft)
Avg
Data
0
10
20
30
40
50
60
70
80
90
100
0
0.5
1
1.5
2
2.5
3
3.5
4
0
2
4
6
8
10
12
14

Soil Data







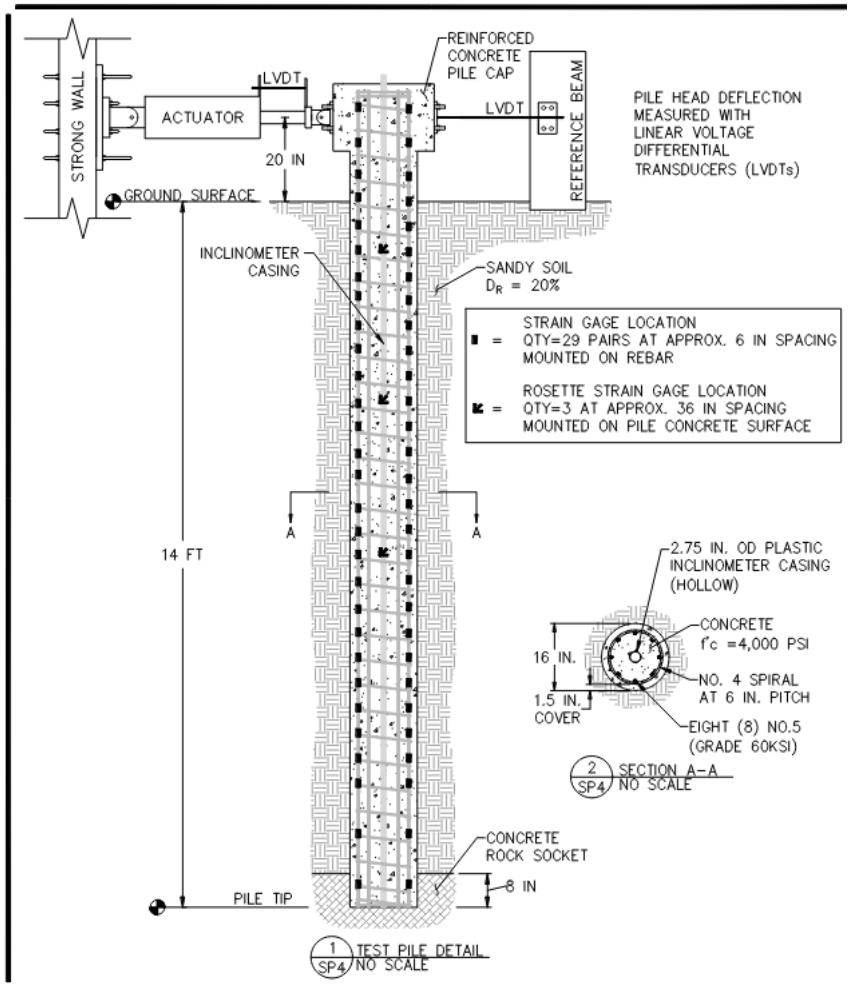


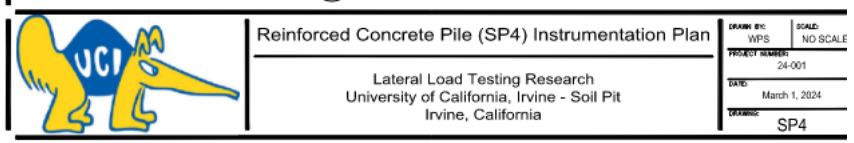


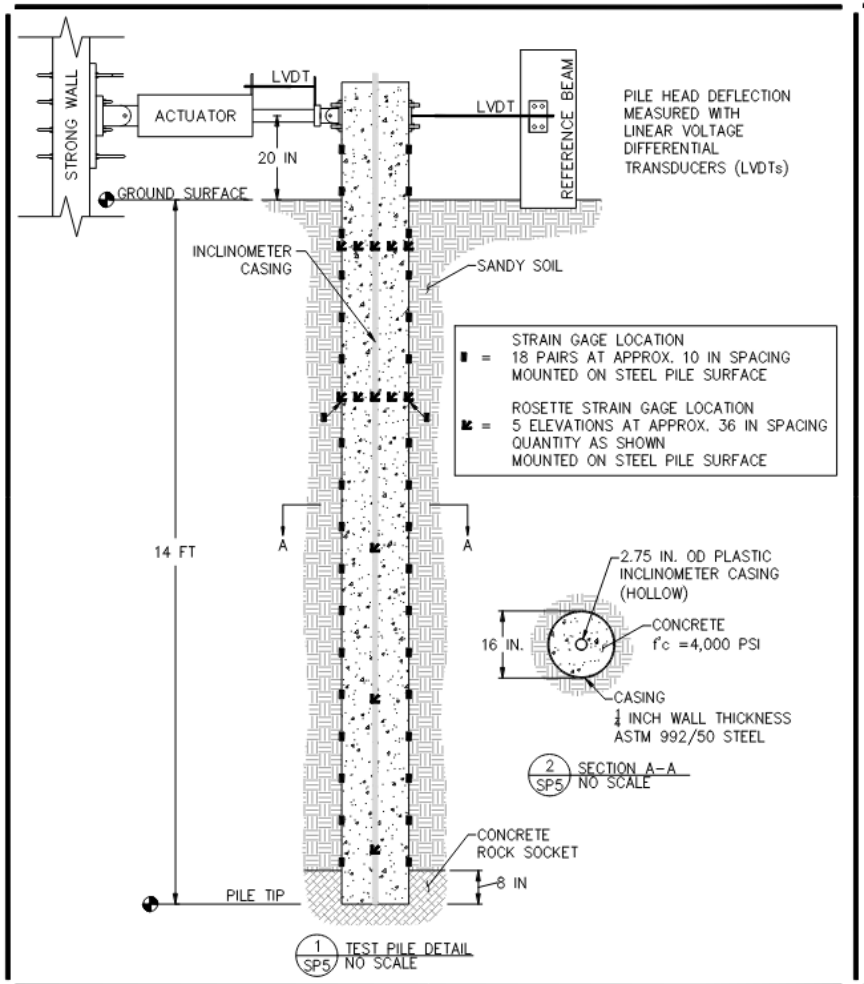
Equipment used: Ramset, with hydraulic pump
Standard CPT rod: 15 cm^2 with Shear Wave Velocity measurements

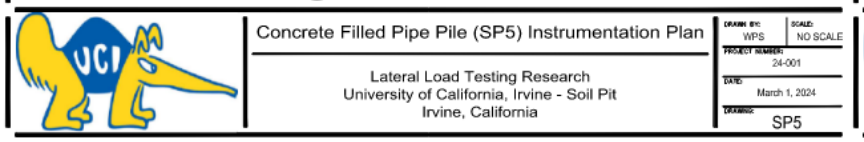
Pile Instrumentation

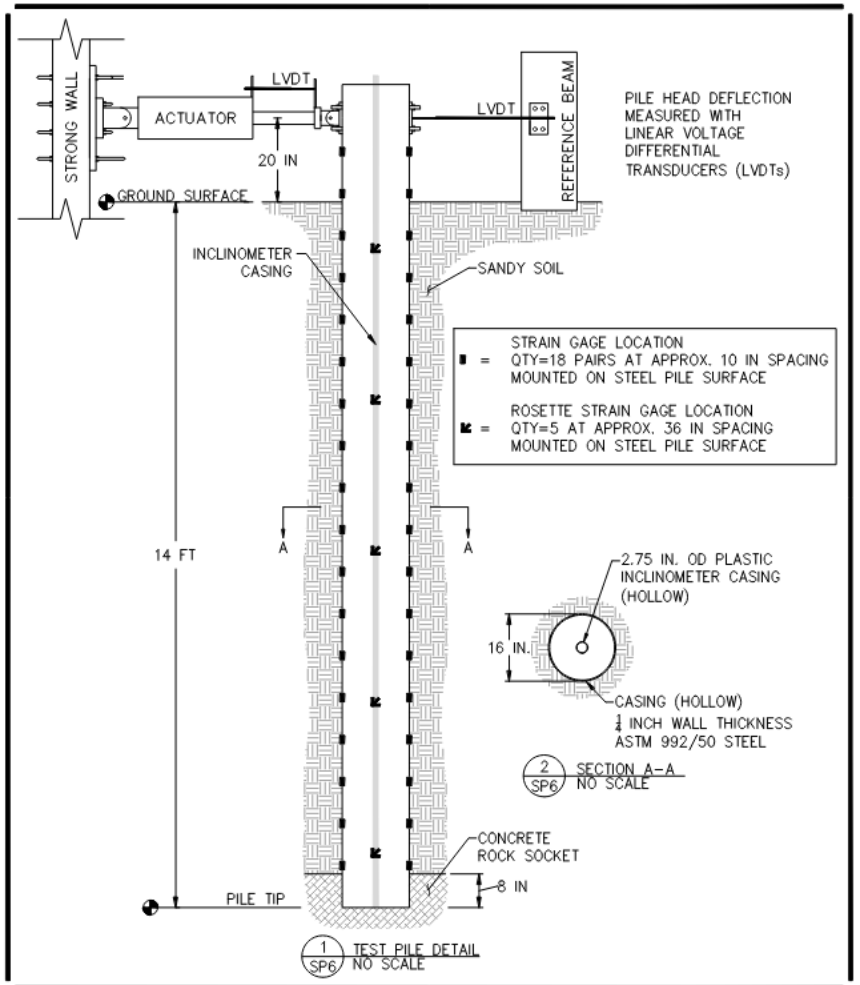
Analyze Previous Load Test from UCI Soil Pit

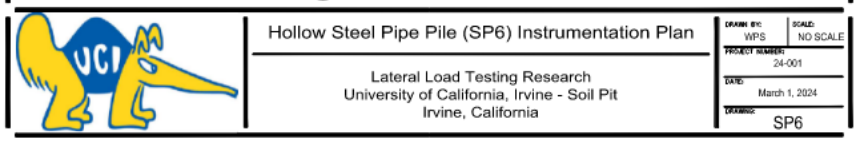




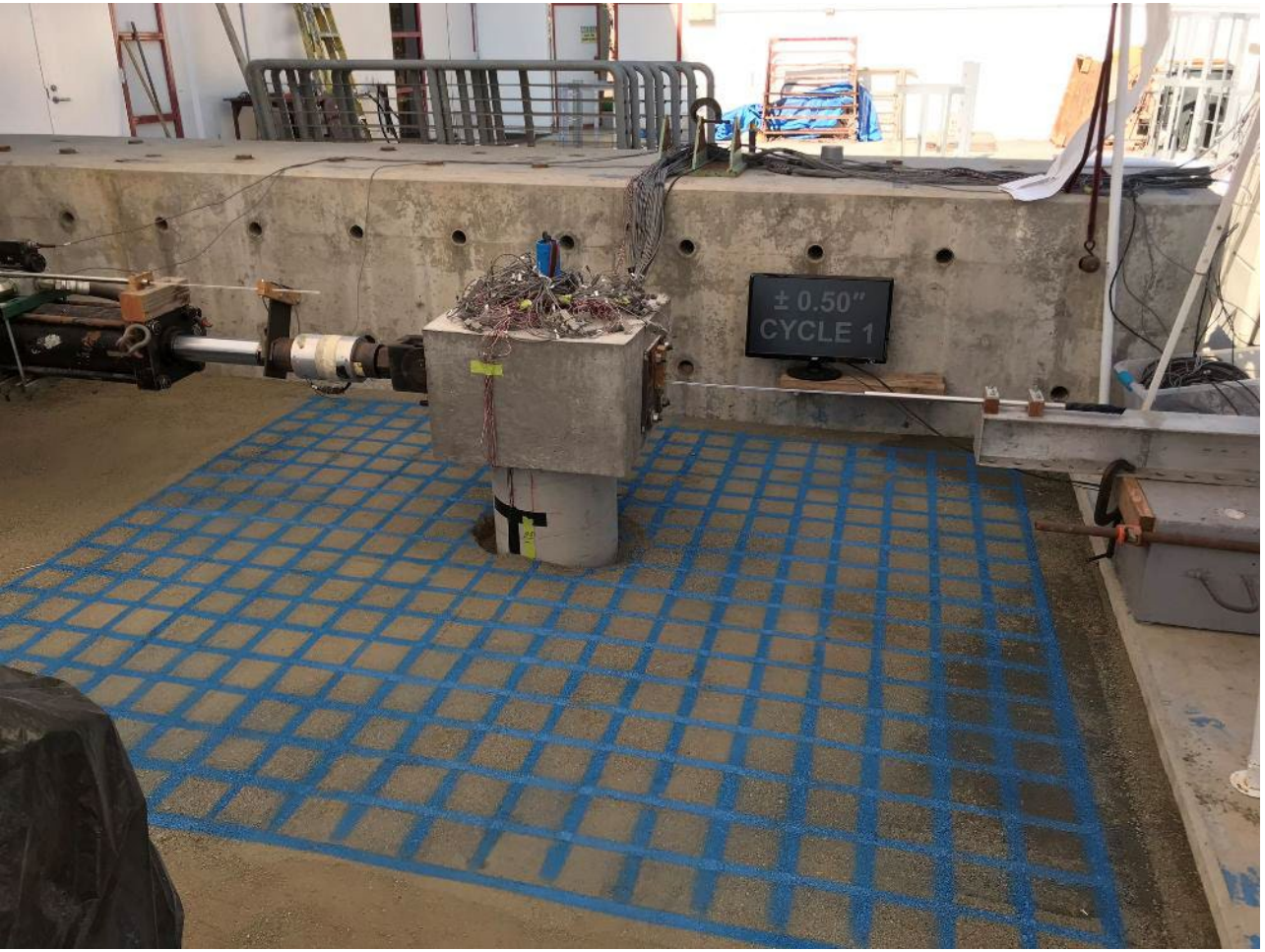


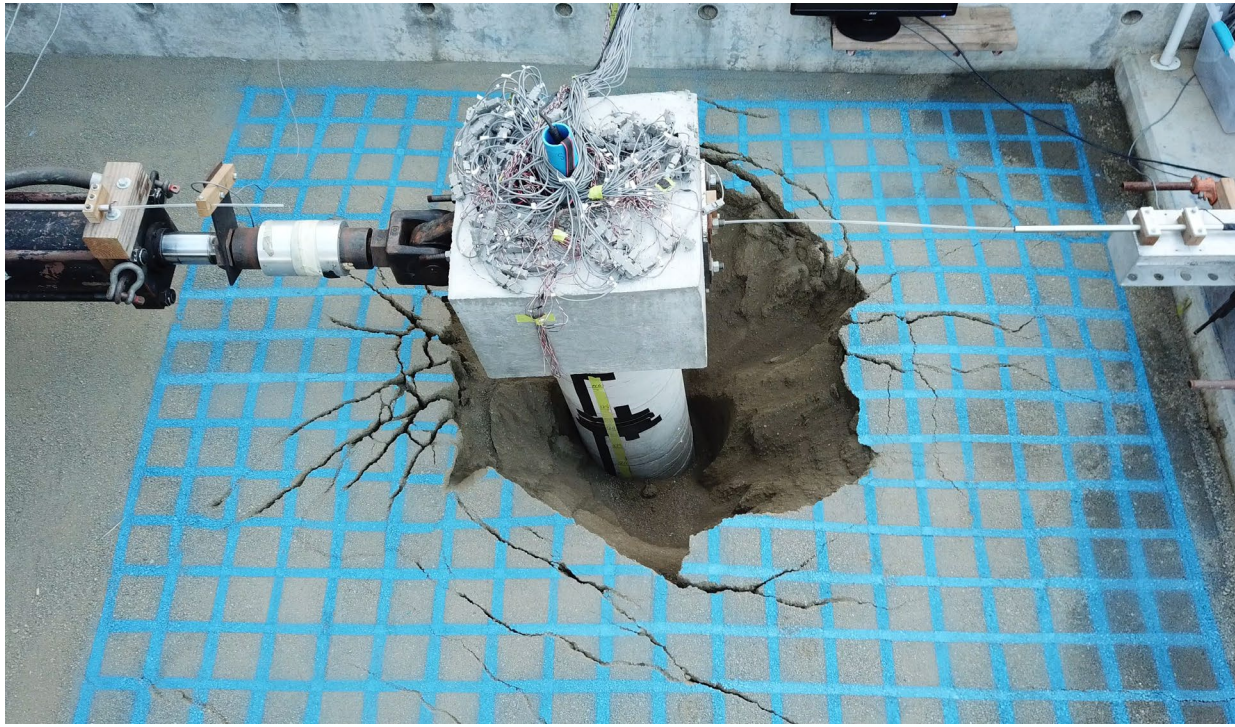




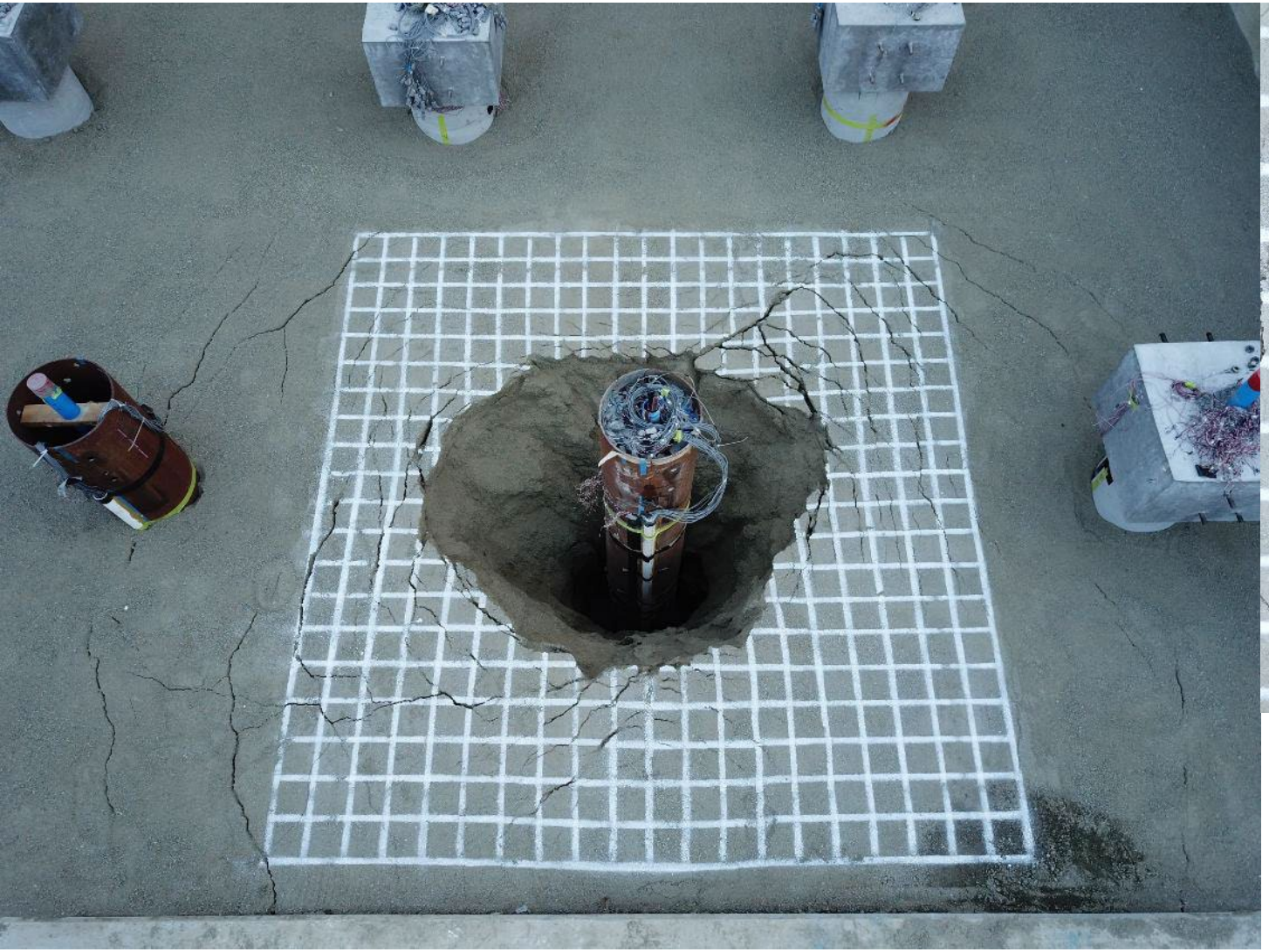


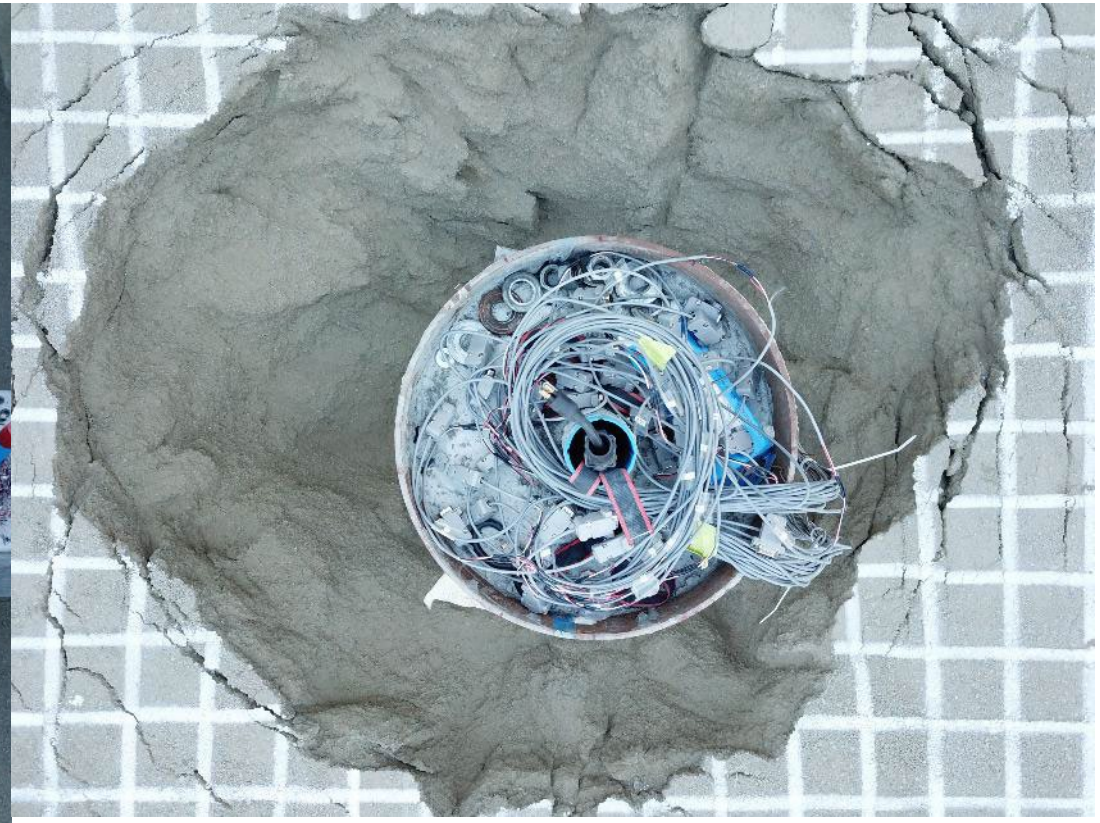
Pile Testing



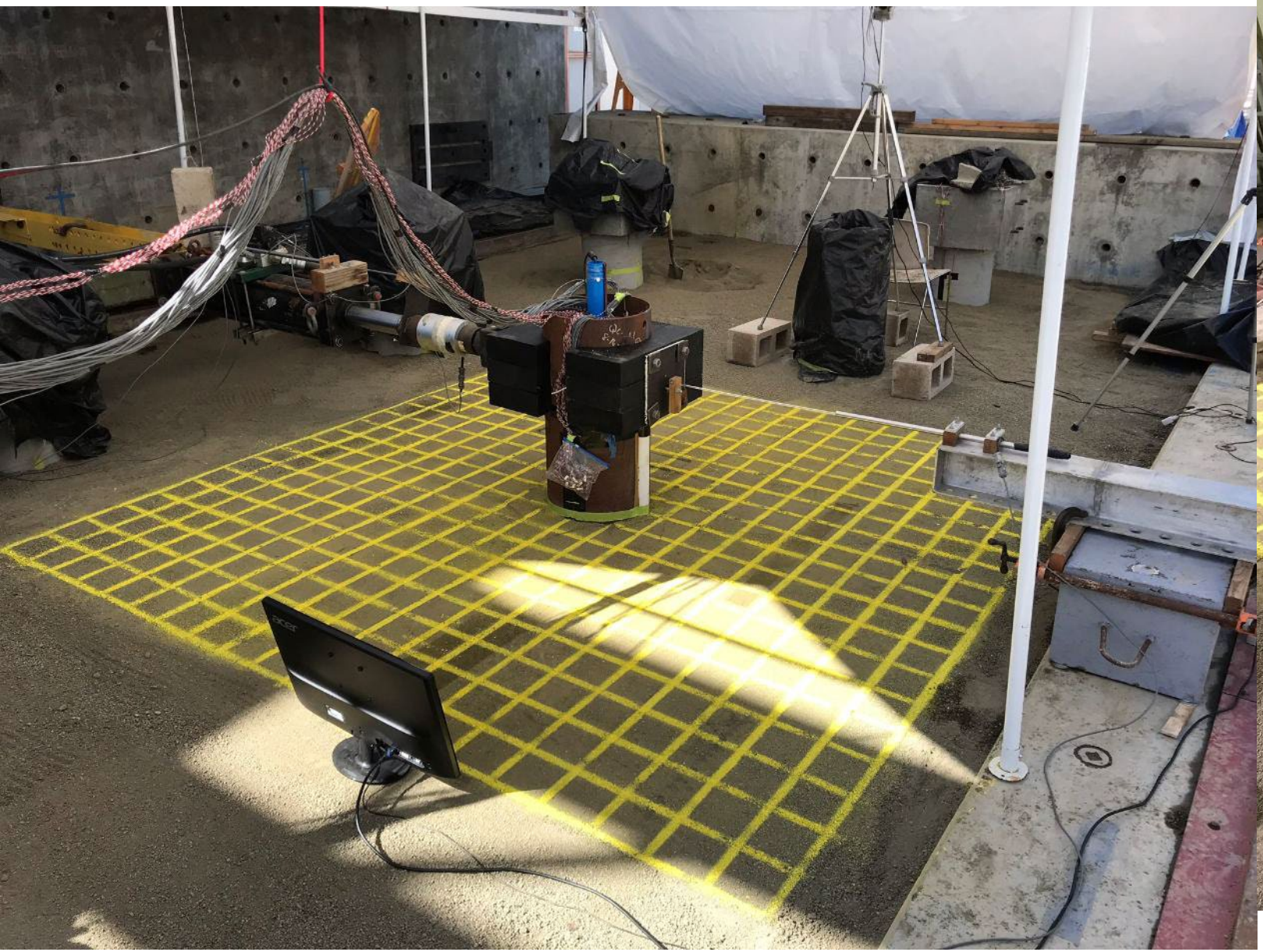


Pile Testing



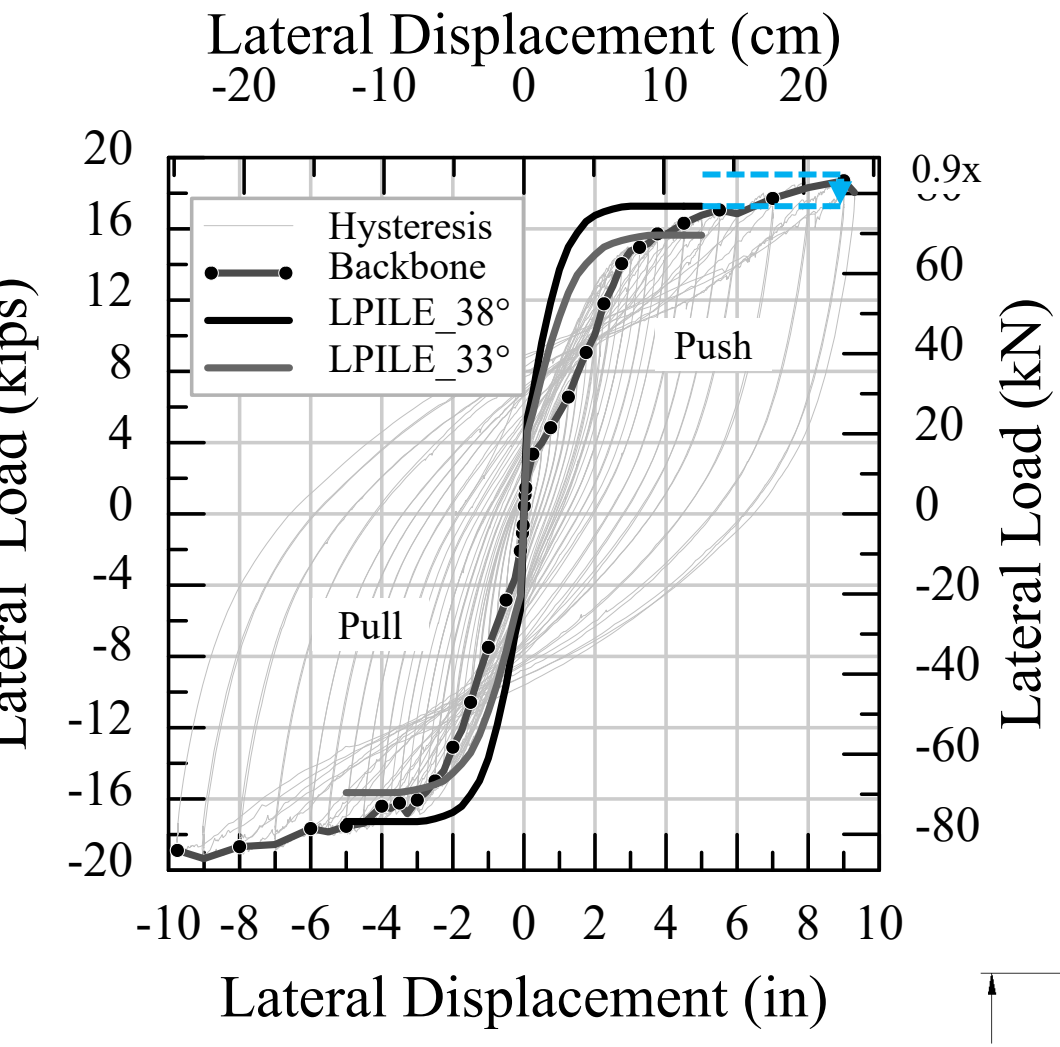


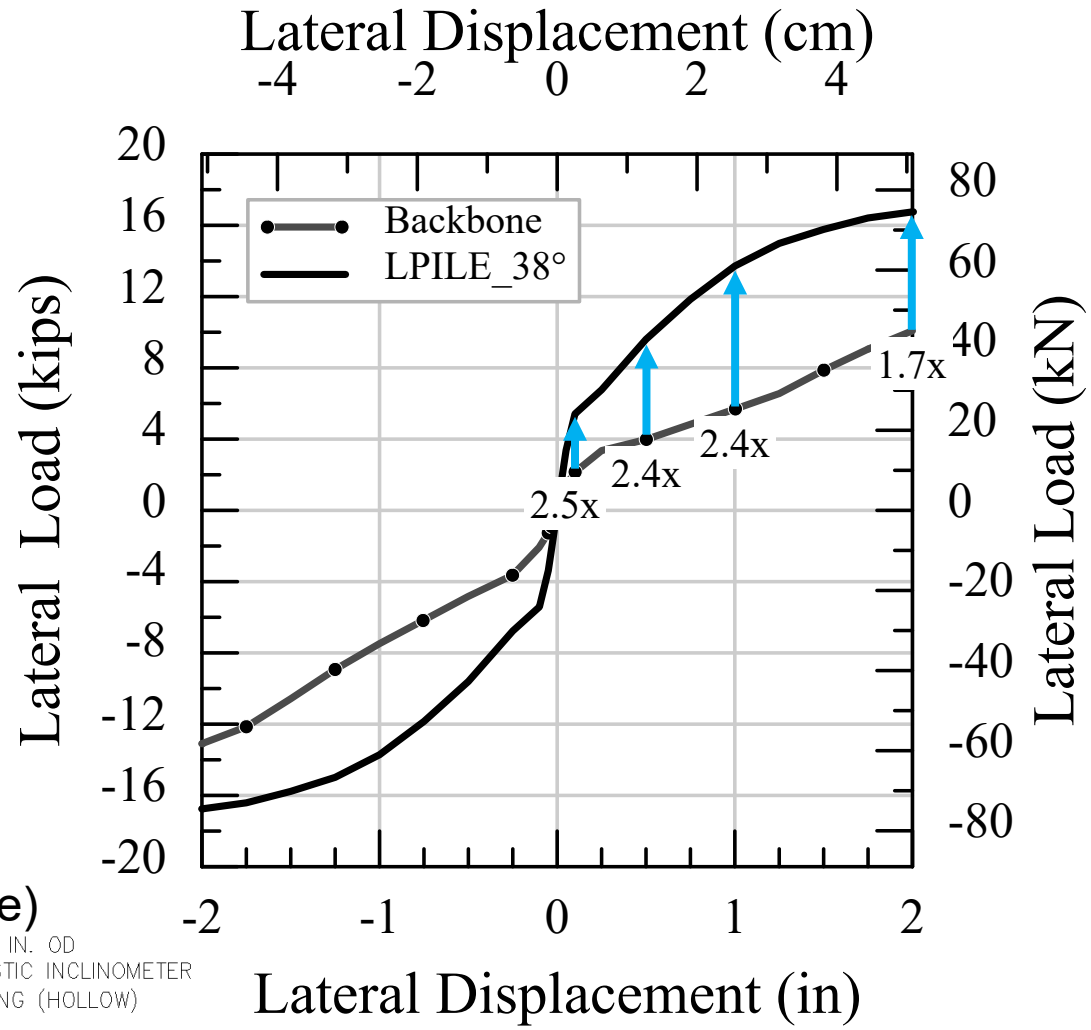
Pile Testing

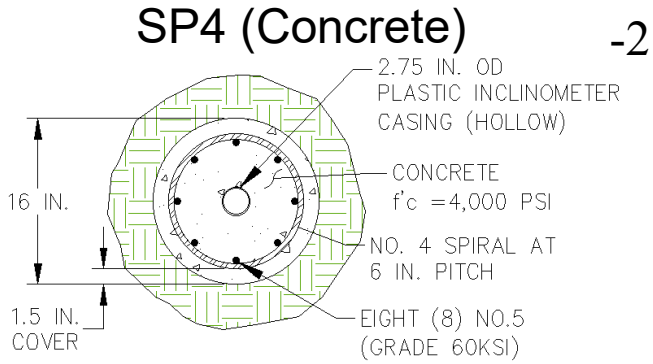




SP4 (Concrete) Backbone Curves

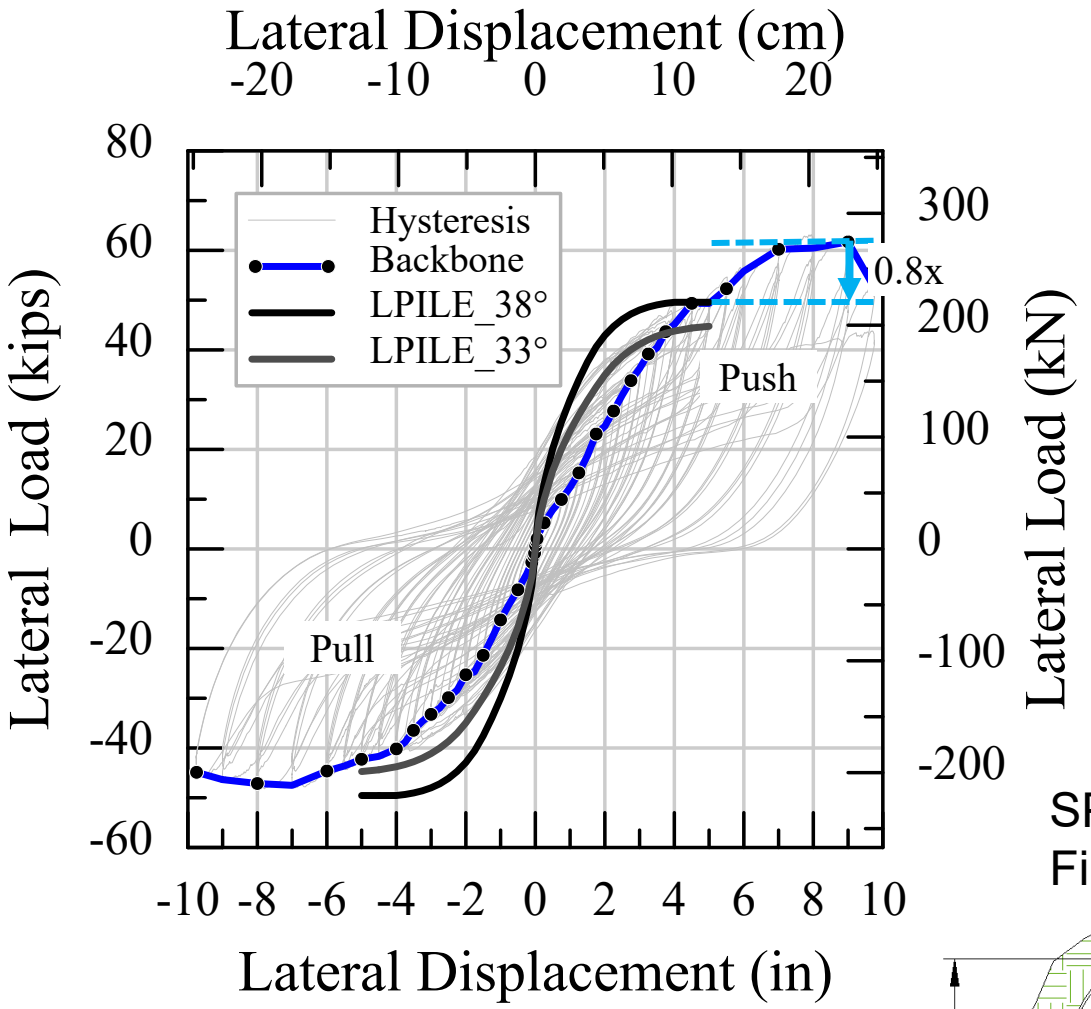


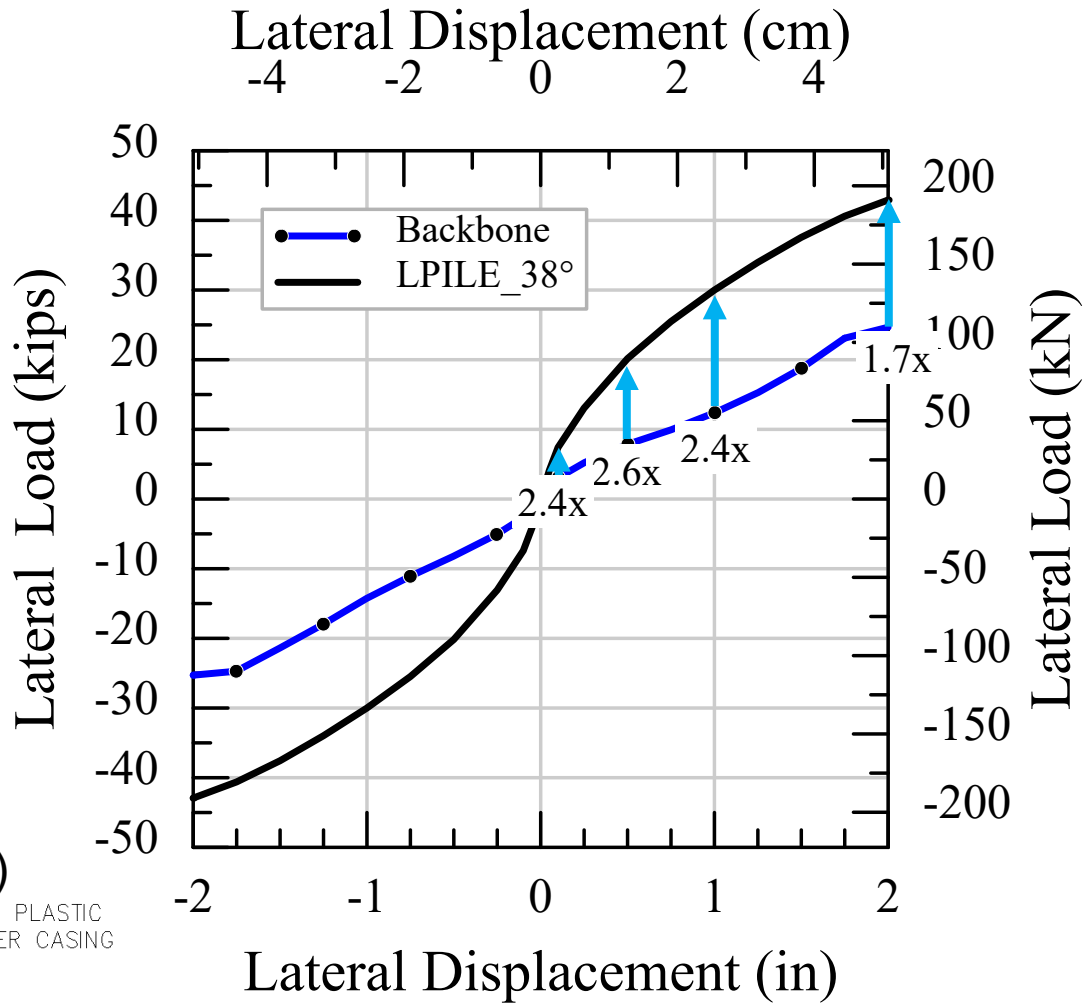




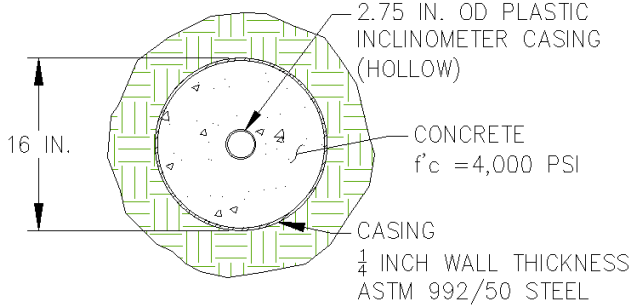
Test Date: February 8, 2019

SP5 (Concrete-Filled Steel Pipe) Backbone Curves



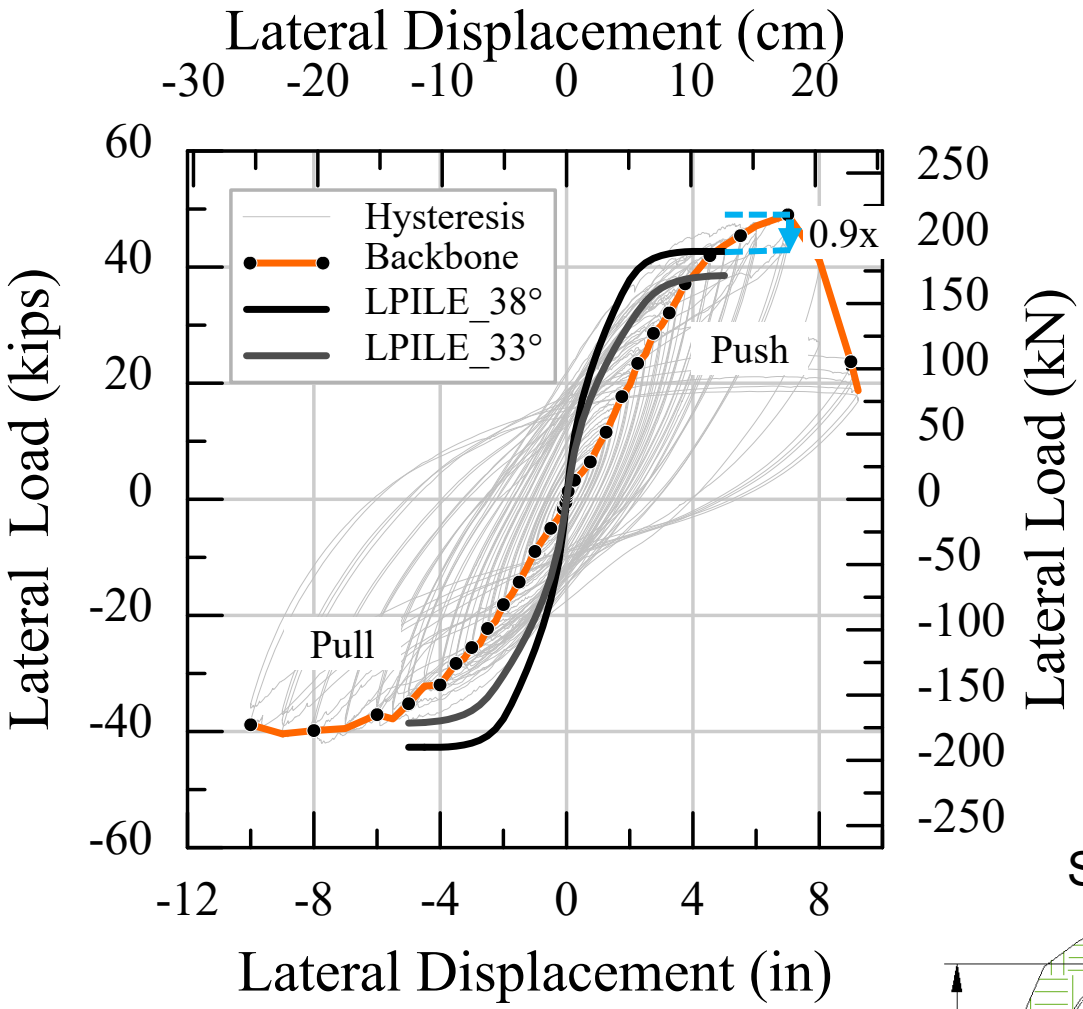


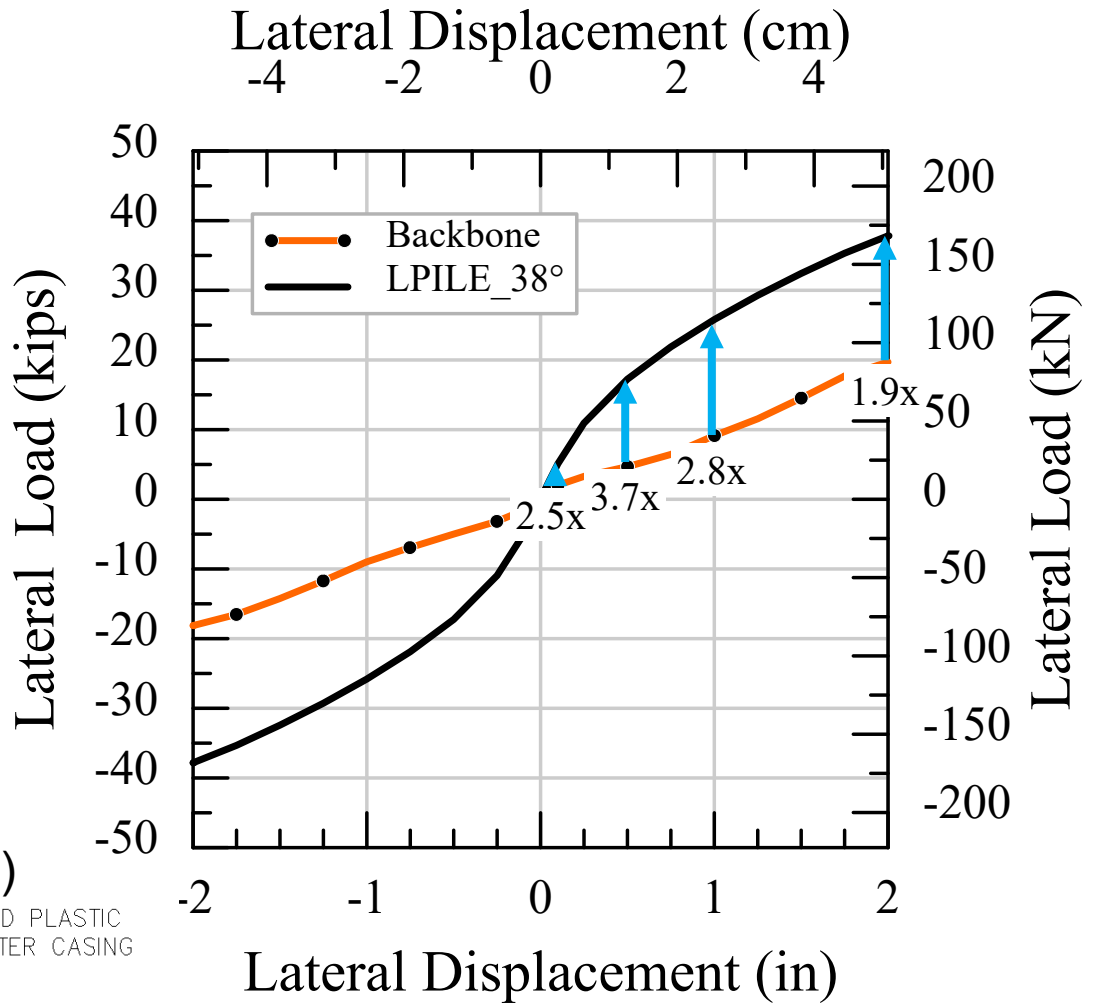
SP5 (Concrete-Filled Steel Pipe)

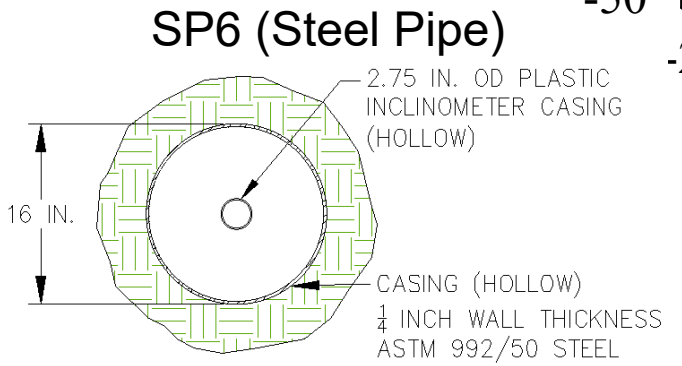


Test Date: February 22, 2019

SP6 (Steel Pipe) Backbone Curves

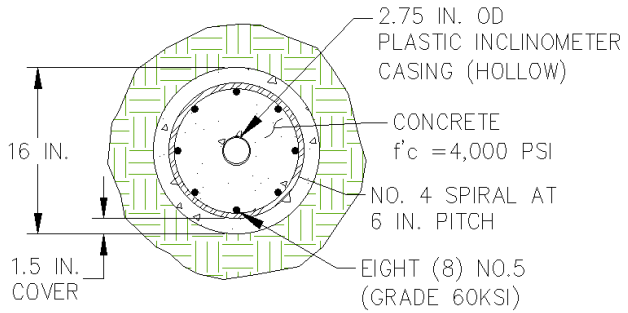




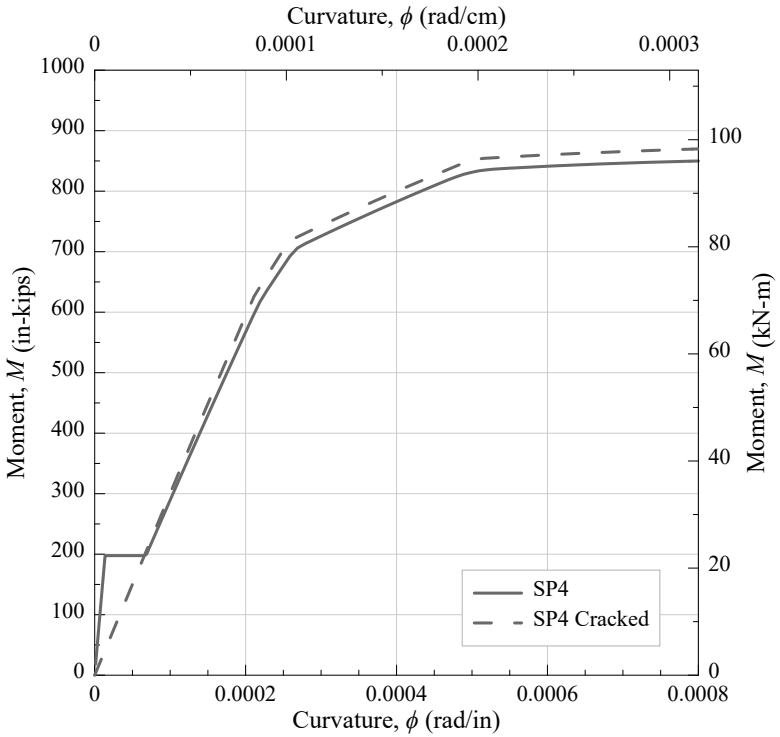


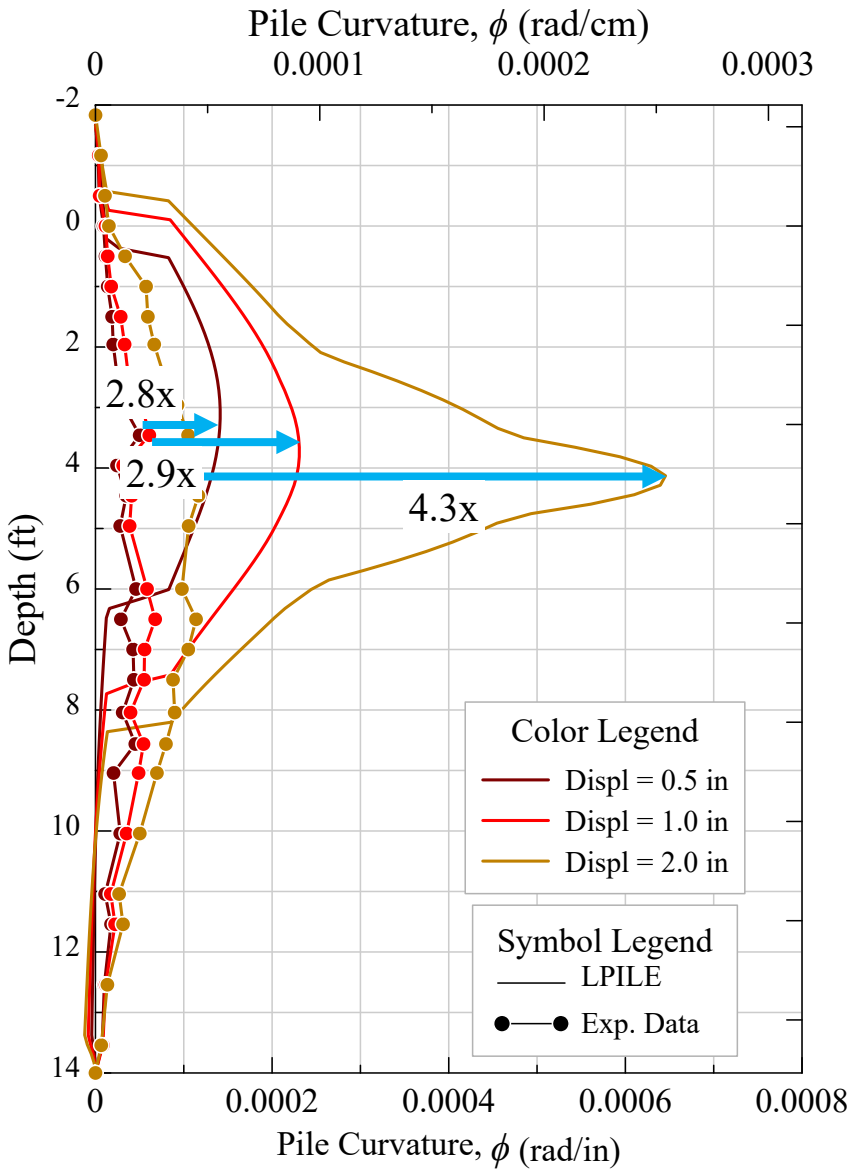
Test Date: February 16, 2019

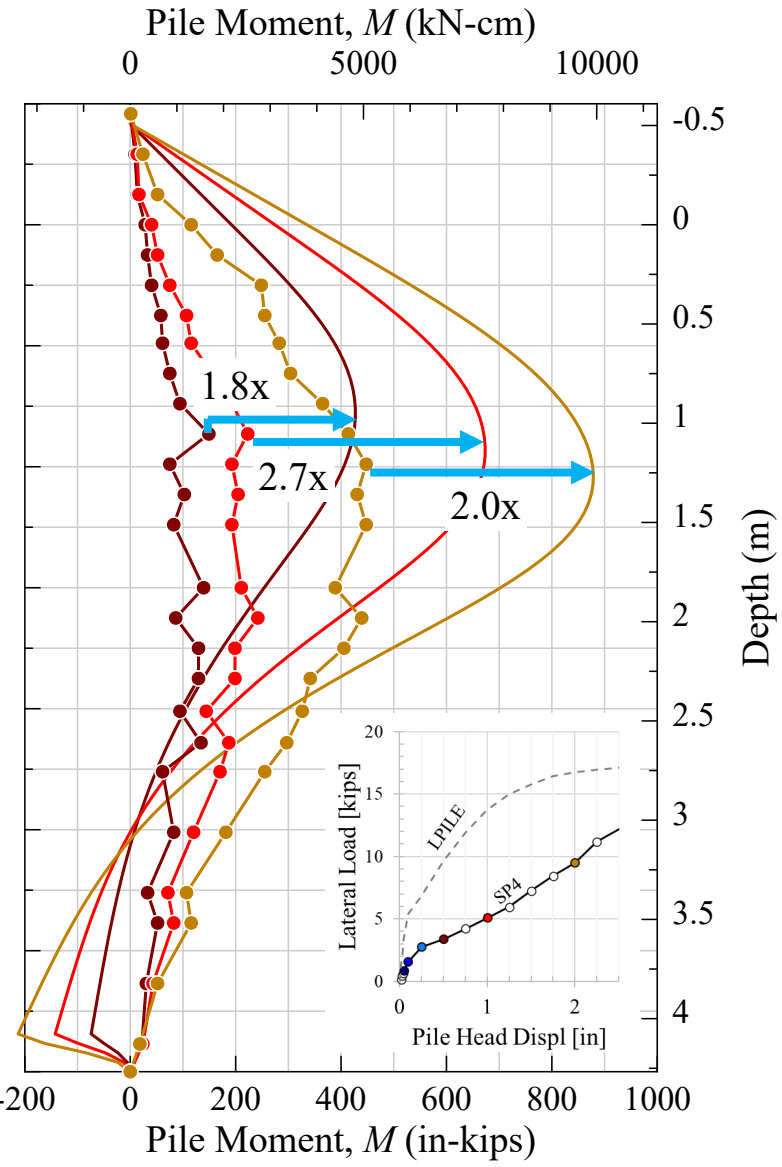
SP4 (Concrete) Pile curvature and moment profiles



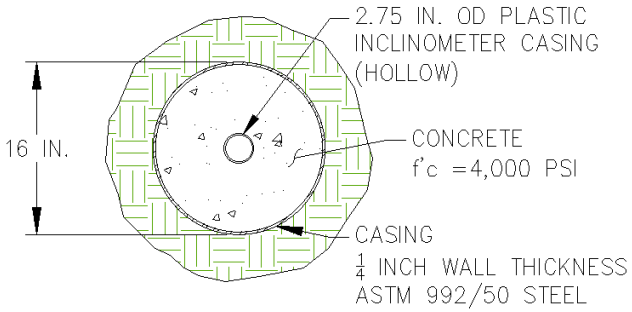
SP4 (Concrete)



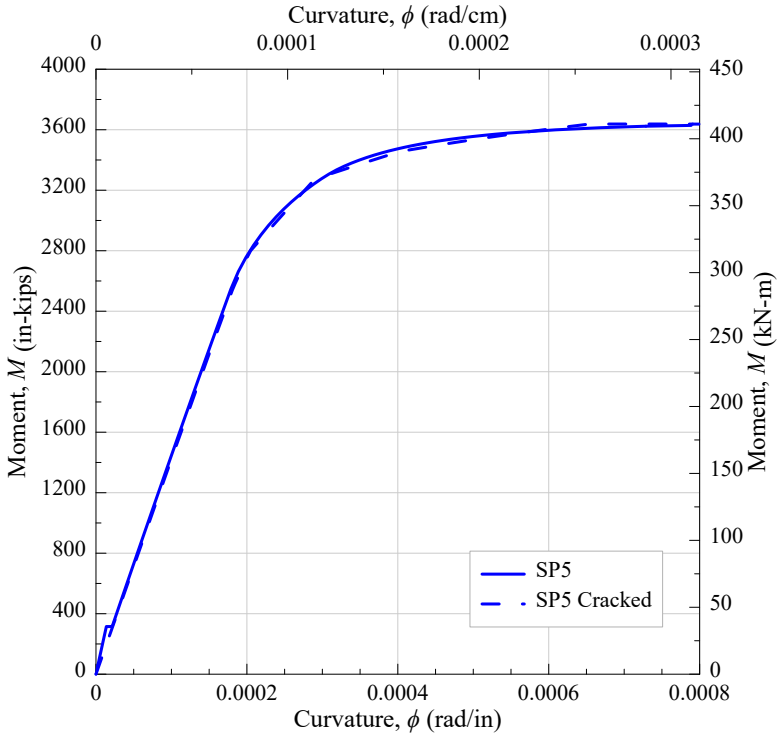


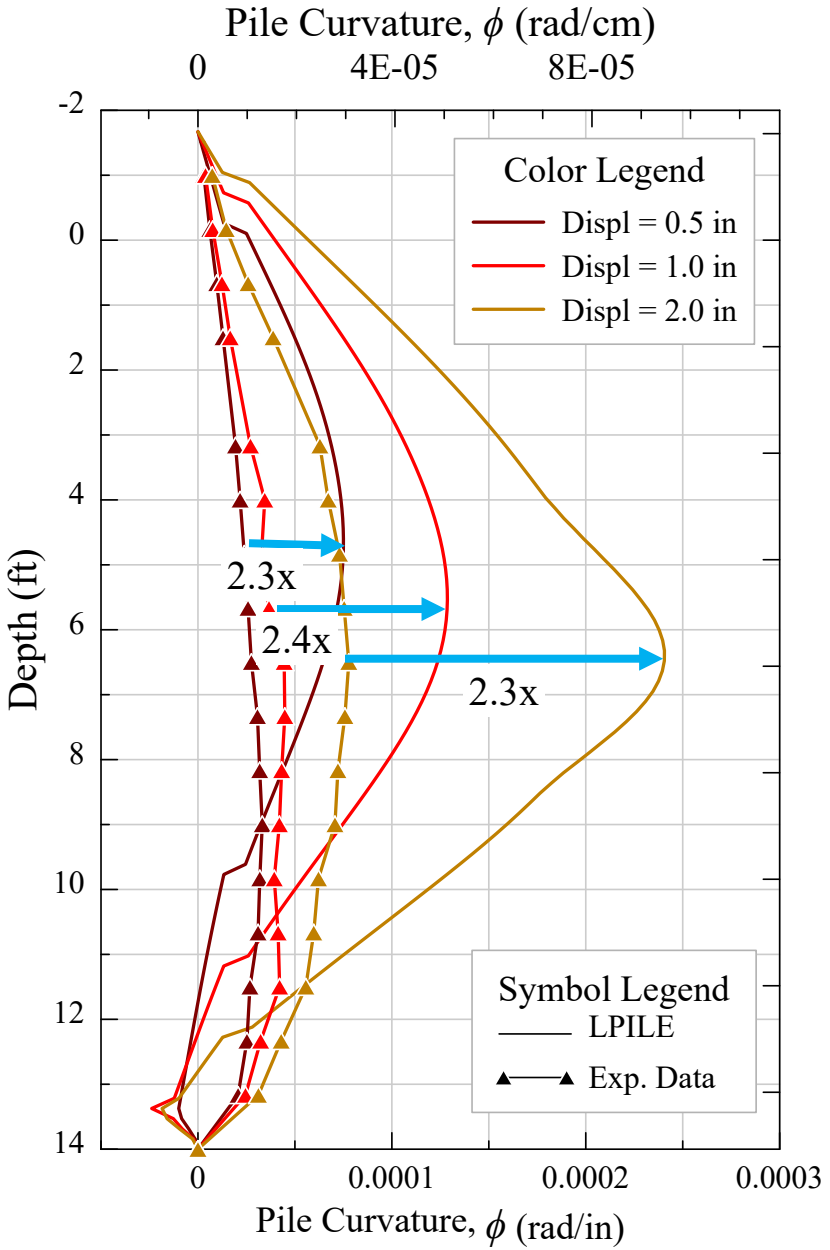


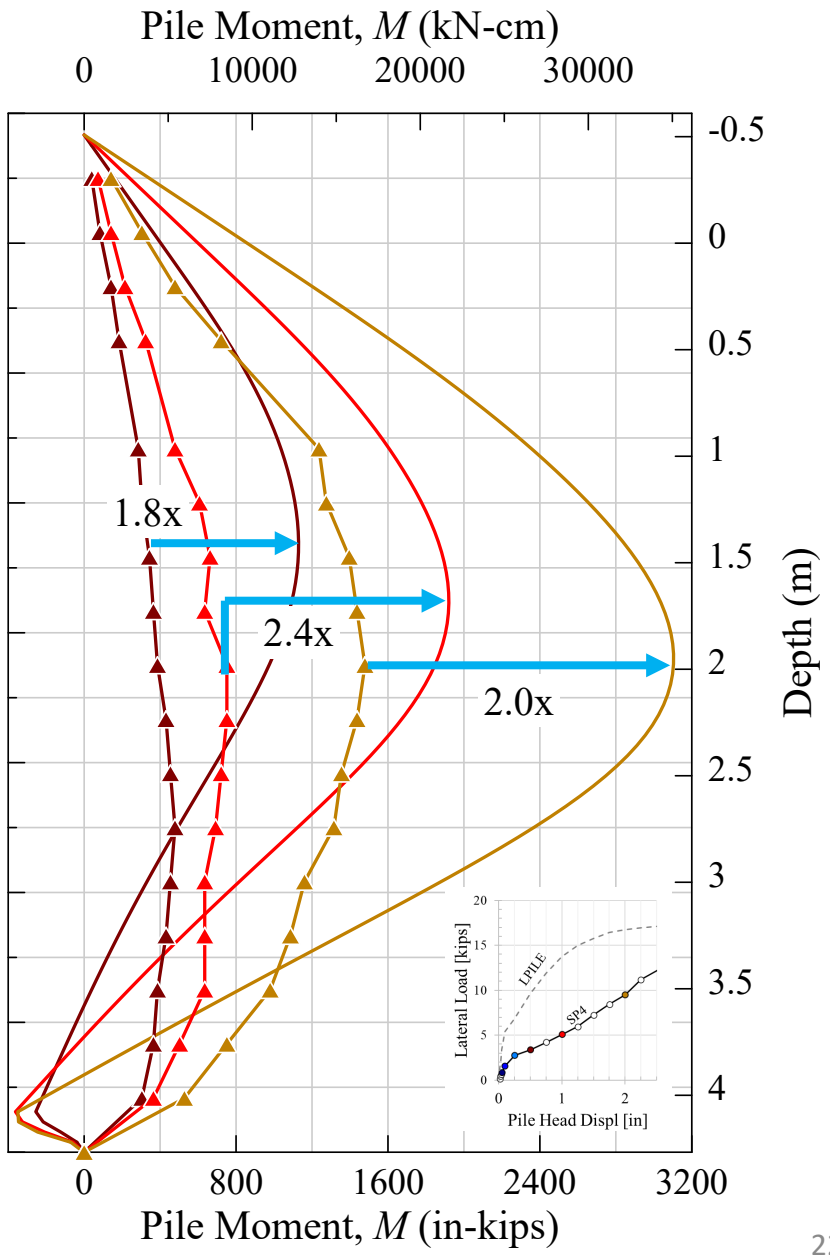
SP5 (Concrete-Filled Steel Pipe) Pile curvature and moment profiles



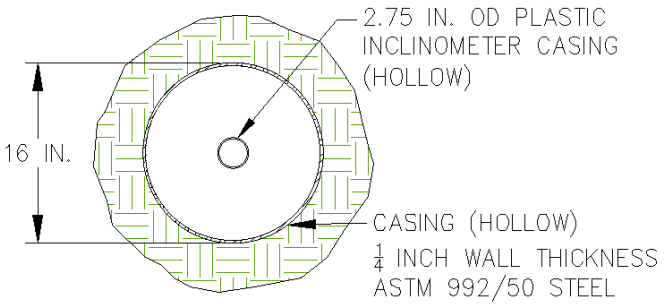
SP5 (Concrete-Filled Steel Pipe)



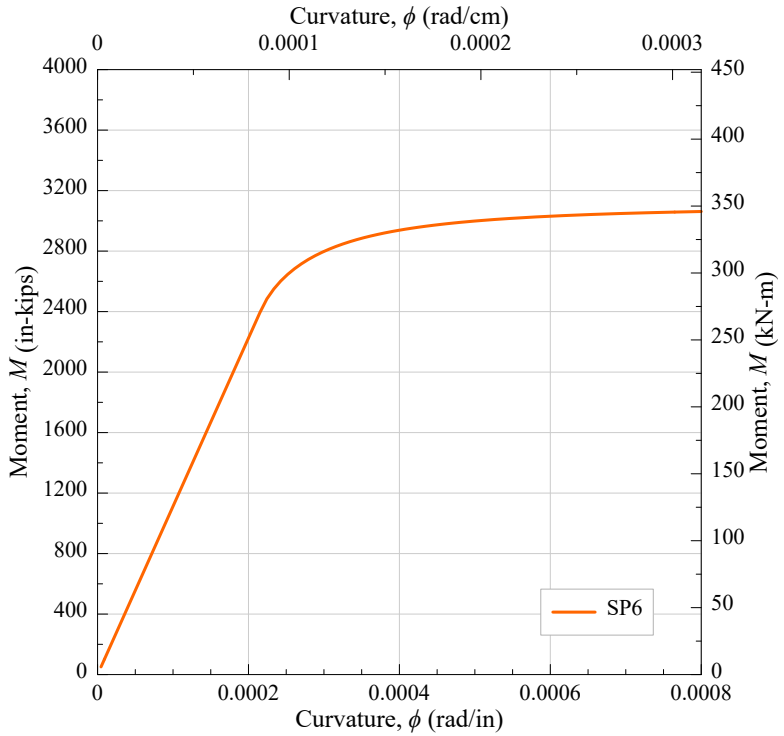


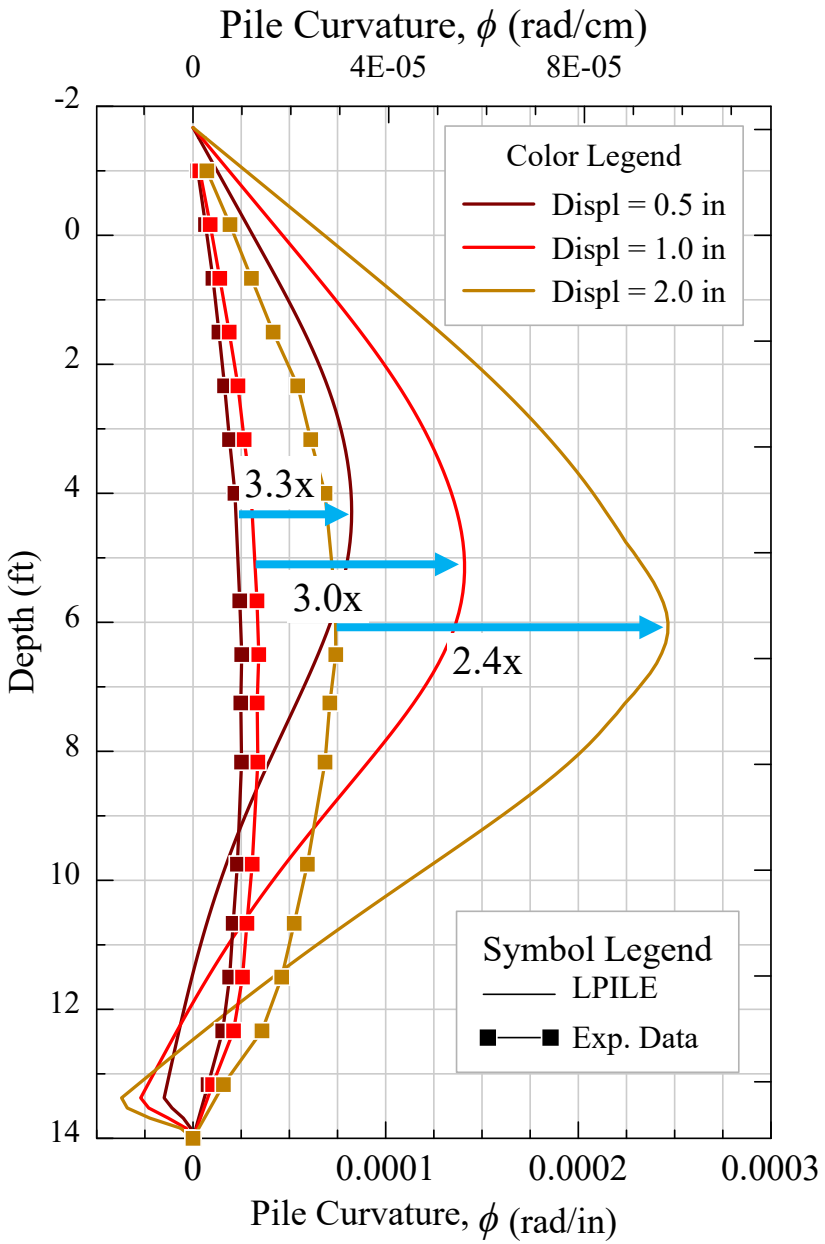


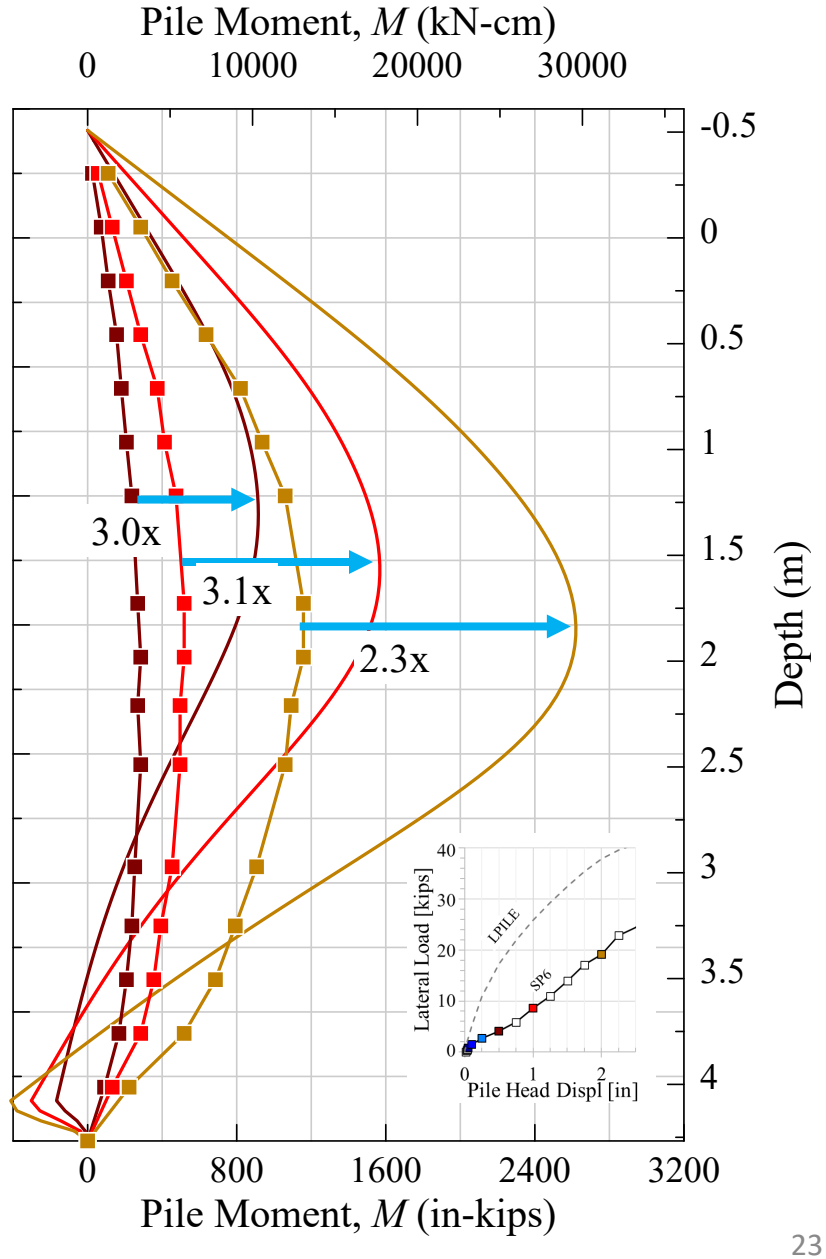
SP6 (Steel Pipe) Pile curvature and moment profiles



SP6 (Steel Pipe)







- *p-y* calculations
  - Under-estimated the maximum pile resistance to lateral load
  - Over-estimated the initial pile resistance to lateral load
  - Over-estimated the maximum pile internal moment
  - Accurately estimated the depth to maximum moment
  - Under-estimated pile internal moment deeper than the plastic hinge
- Cracked moment-curvature relationships are recommended for deriving pile profiles and *p-y* curves
- Design implications

Questions?

Amirkabir University of Technology
(Tehran Polytechnic)

50th Annual Conference on Deep Foundations

DEEP FOUNDATIONS INSTITUTE DFI

Conceptual Axial Stress-Strain Behavior of Drilled Displacement Piles Using Full-Scale Load Tests Database

(PID: DFI50-63)

By:

Prof. Abolfazl Eslami

Arshiya Mehrazma

Amirhossein Ebrahimipour

Nashville, Tennessee, 20-23 October 2025

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Outline

1	Background to Load-Displacement
2	Drilled Displacement (DD) Piles Database
3	Ultimate Pressure Analysis
4	Dominant Factors Appraisal
5	Stress – Strain Implementation
6	Conclusions and Remarks

Scope

Stress-Strain Behavior of DD Piles

Objectives

Load-Displacement Significancy

Stress – Strain Framework

Data-Centric Approach

Energy-Based Evaluation

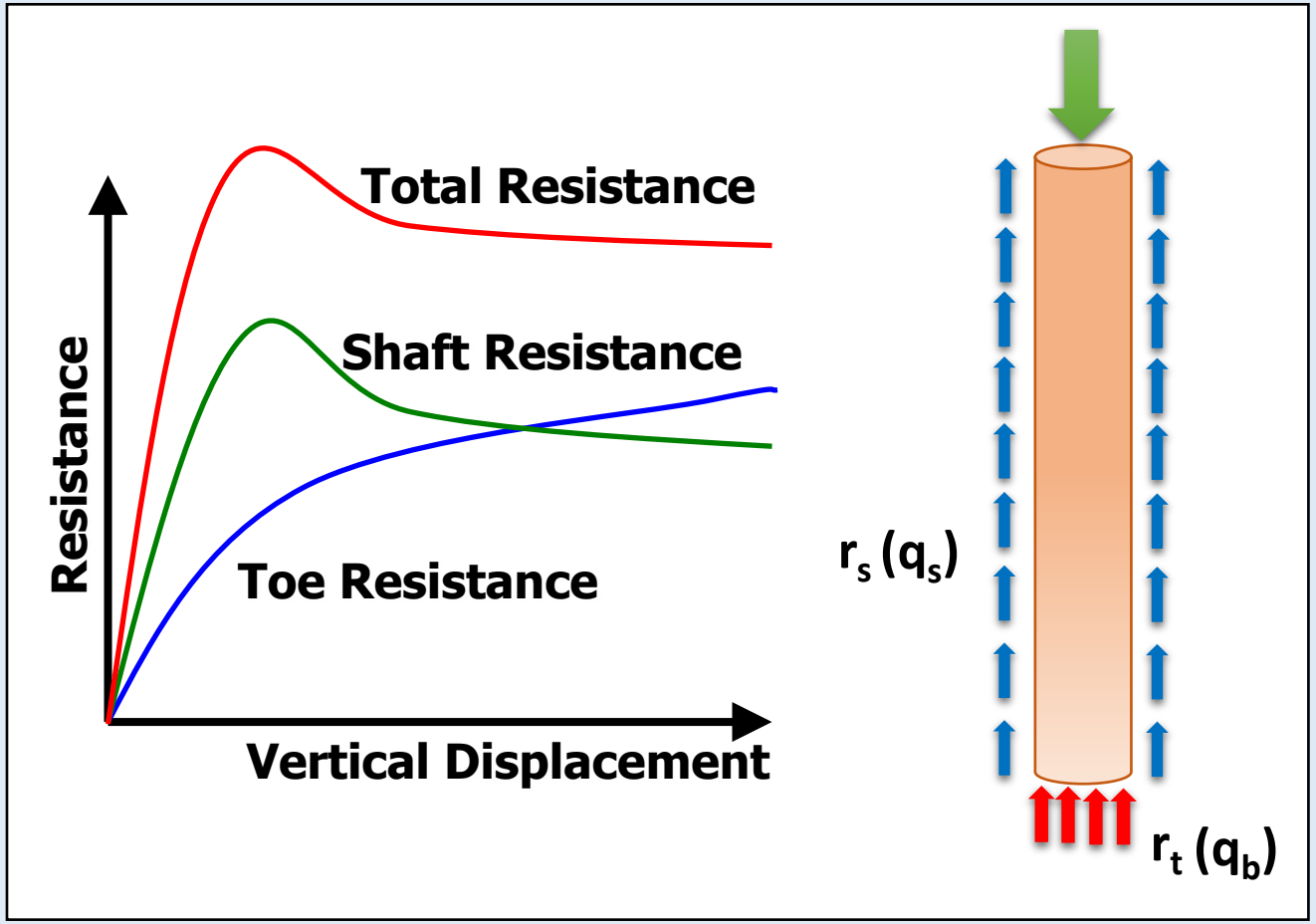
Foundation Alternatives

Aim

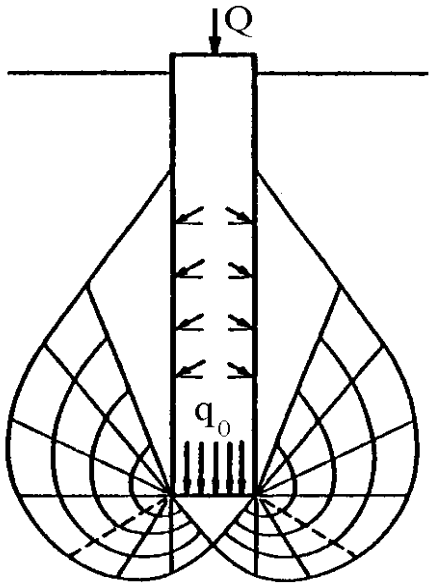
Resilient Foundations

Common Approaches for Bearing Capacity Appraisal

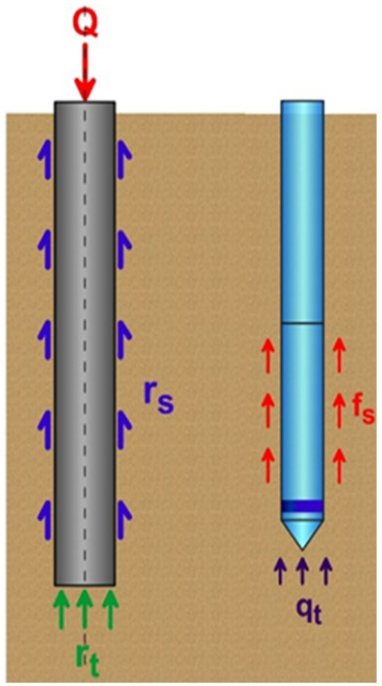
1. Static Analysis
2. In-Situ Tests Application
3. Static Load Test
4. Dynamic Analysis & Load Test
5. Numerical & Laboratory Modeling
6. Soft Computing: AI & ML



Common Approaches for Bearing Capacity Appraisal



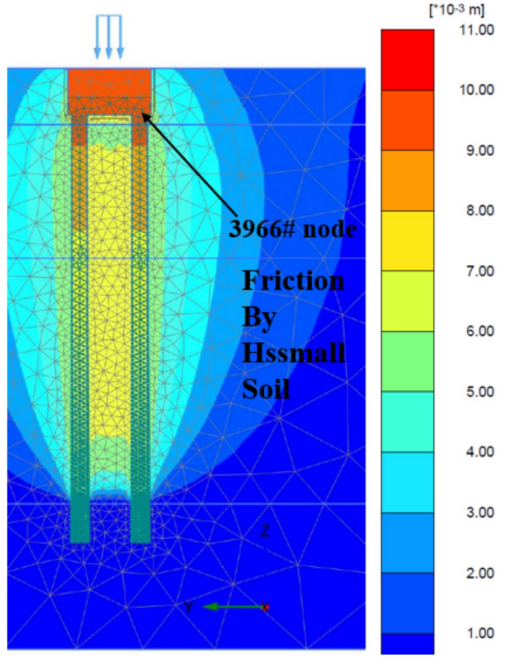
Pile failure zone
(Meyerhof, 1951)



CPT & pile resemblance
(Eslami et al., 2020)



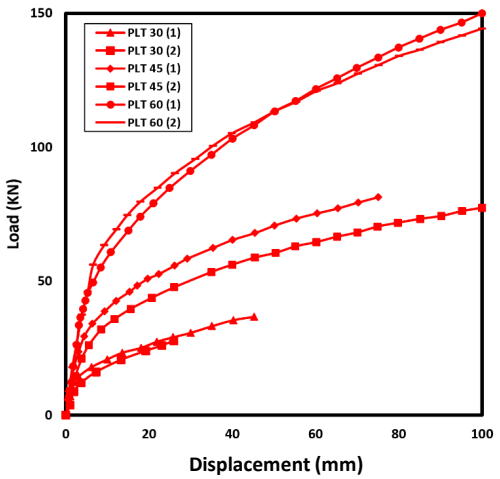
Field testing set-up
(Van Impe et al., 2013)



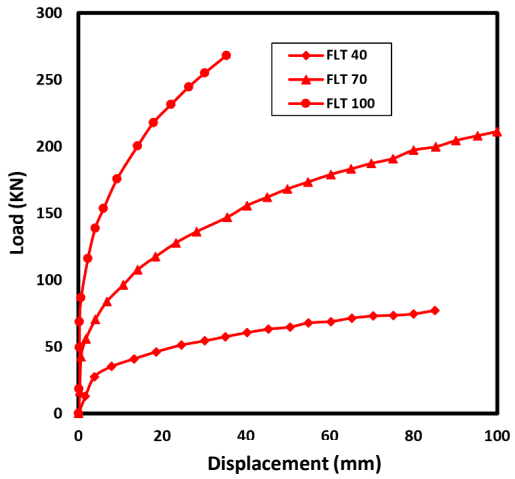
Numerical modeling of
piled raft (Xie, 2022)

Significancy of Load - Displacement

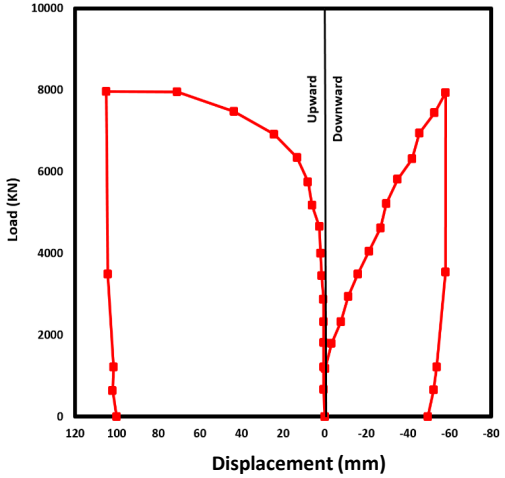
- Directly Reflects Critical Design Parameters
- Enables Accurate Bearing Capacity Determination
- Facilitates Settlement and Serviceability Checks
- Provides Essential Stiffness Parameters
- Proceeds Optimized Foundation Design



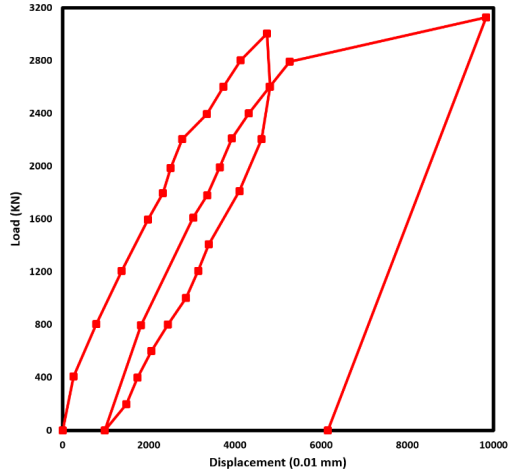
Circular PLTs
(Consoli et al., 1998)



Square Concrete Footings
(Consoli et al., 1998)



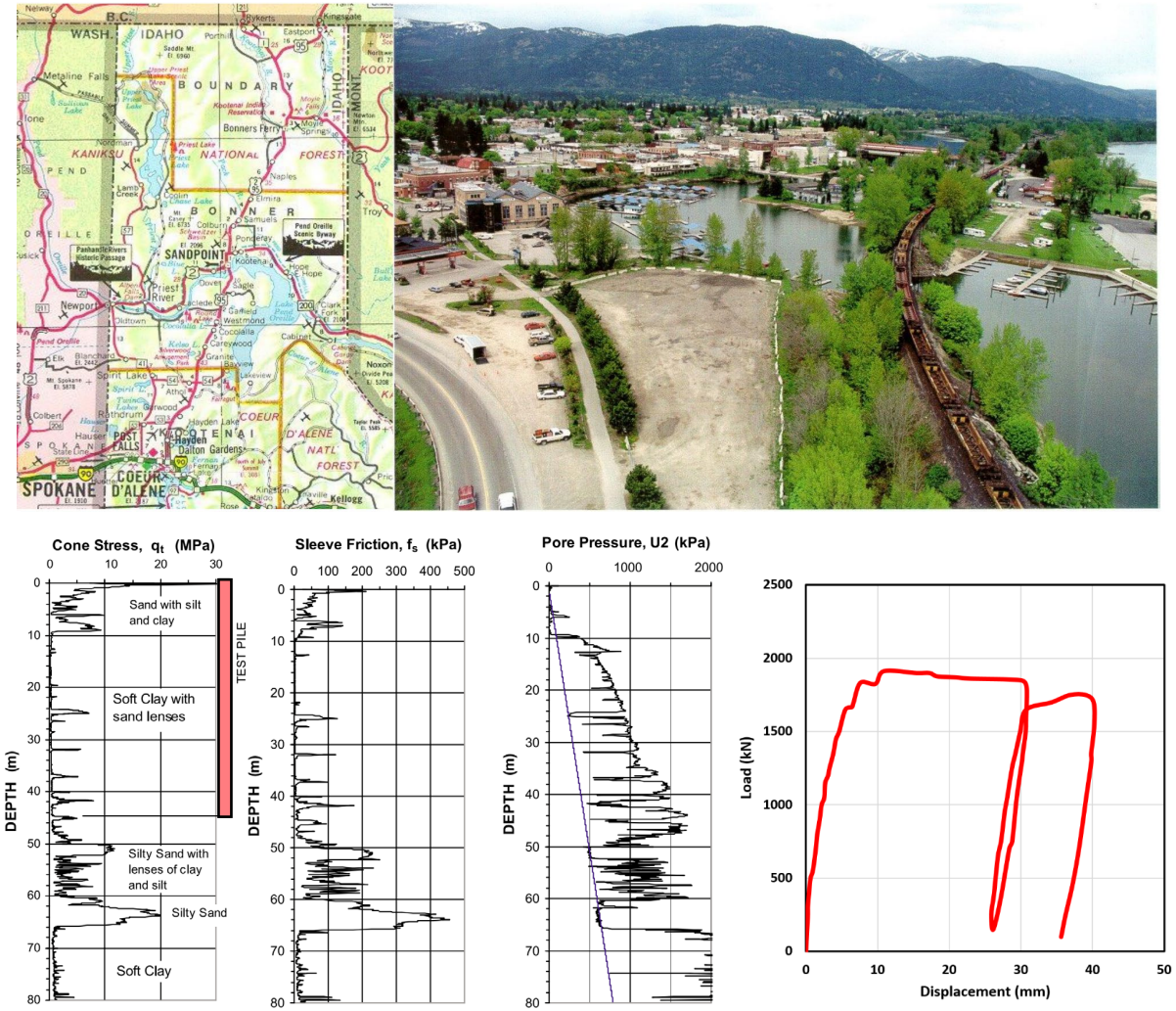
Bidirectional test on a bored pile
(Fellenius, 2023)



Driven pile, pull out test
(Eslami et al., 2011)

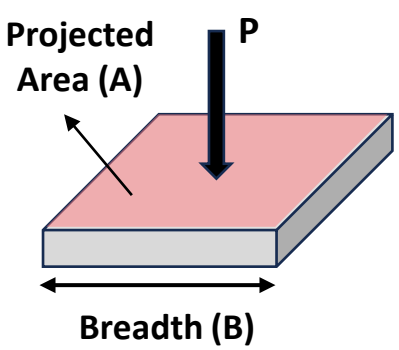
Load – Displacement; Dominant factors

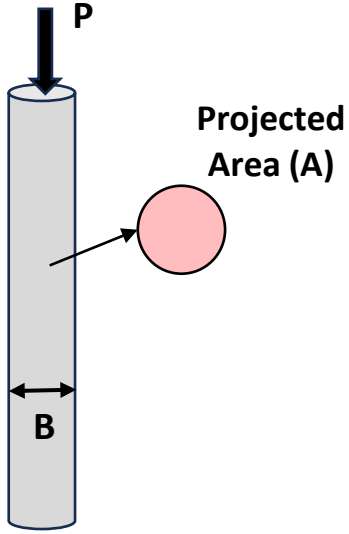
- **Foundation**
  - Geometry & Embedment
  - Installation Effects
  - Interface Properties
  - Structural Material
- **Geomaterial**
  - Strength & Stiffness
  - Drainage Condition
  - Time History & Aging
  - Sensitivity & Thixotropy
- **Loading**
  - Pattern: Slow or Rapid
  - Type: Compressive, Tensile, Cyclic



Static Loading Test on a 45 m Long Pipe Pile in Sandpoint, Idaho (Fellenius et al., 2004)

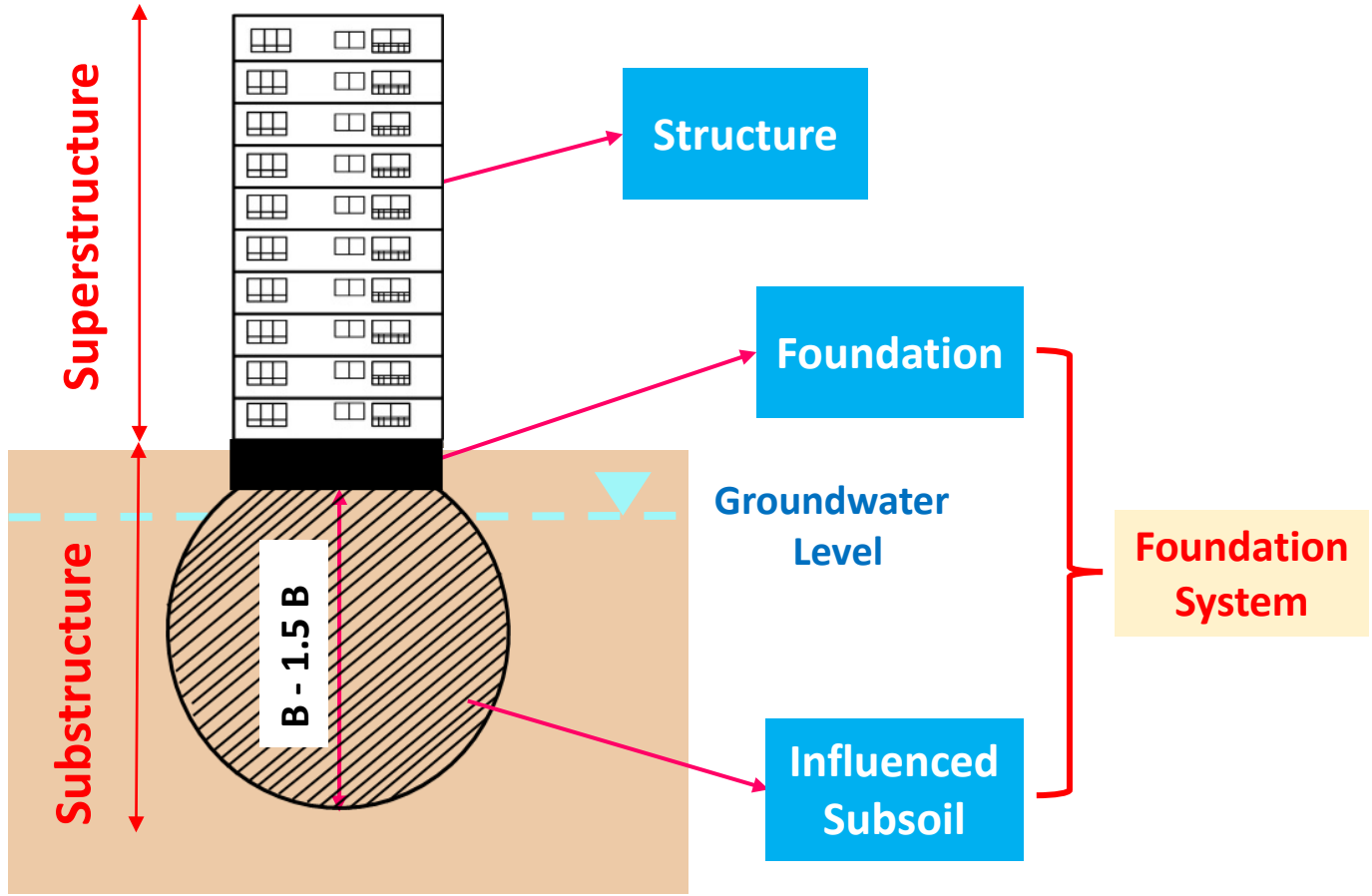
From Load-Displacement to Stress – Strain





$$\text{Stress } (\sigma) = \frac{\text{Axial Load (P)}}{\text{Projected Area (A)}}$$

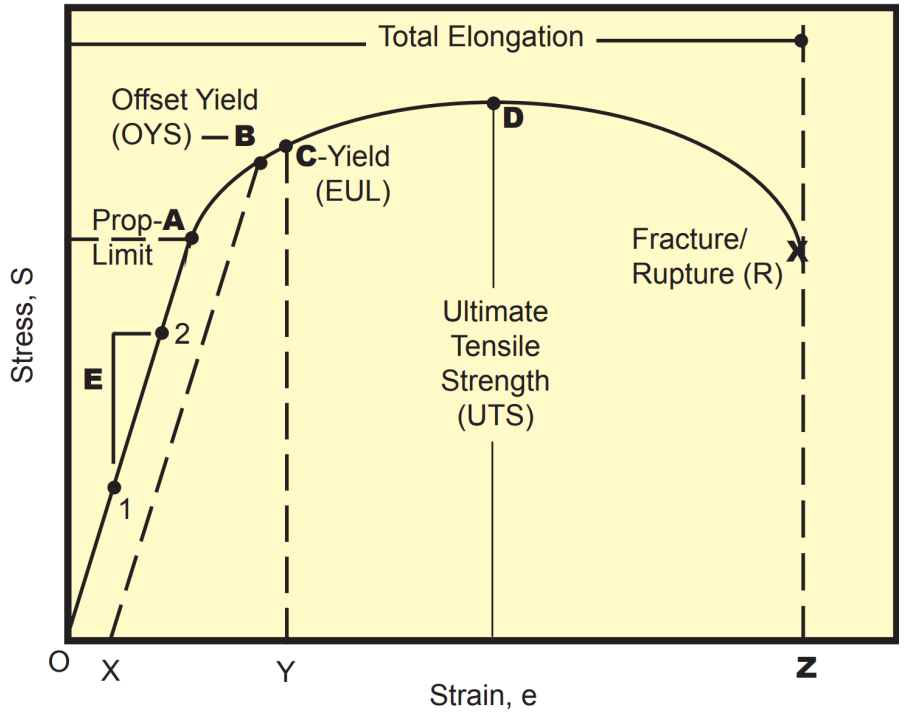
$$\text{Strain } (\varepsilon) = \frac{\text{Displacement}}{\text{Breadth (B)}}$$



Stress – Strain Behavior; Solid Mechanics

Engineering Parameters from Stress-Strain Curve

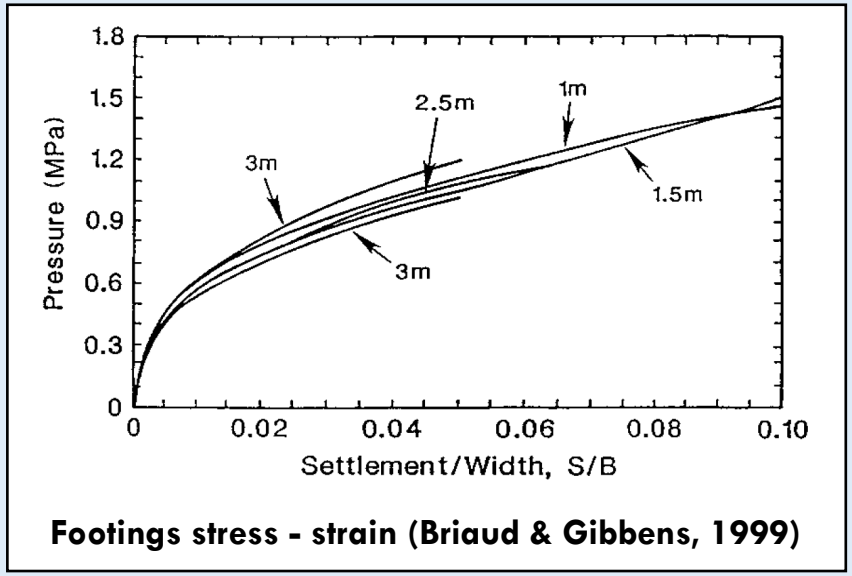
- Strength
- Toughness
- Elasticity
- Yield Point
- Strain Energy
- Resilience
- Elongation during Load



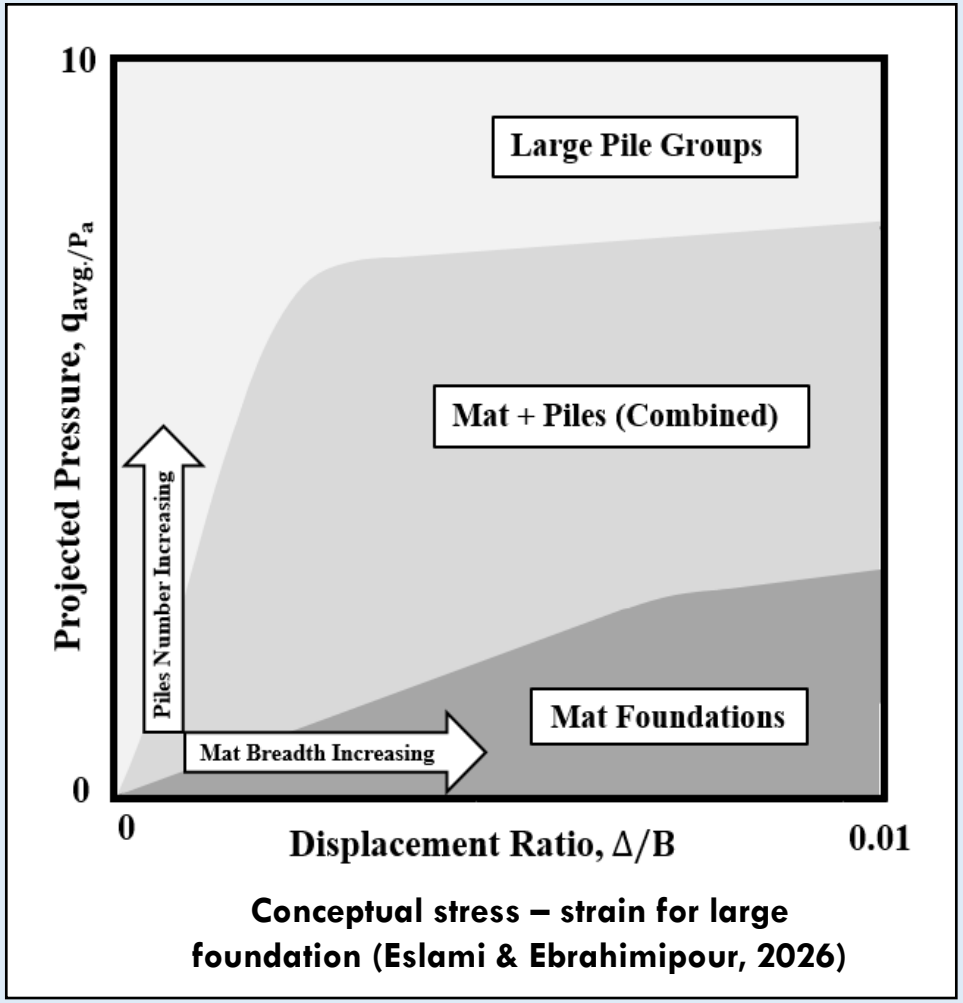
Typical stress-strain diagram (ADMET, 2013)

Stress – Strain in Foundation Engineering

- Providing a Unitless Framework
- Facilitating Broader Practical Application
- Enhancing Foundation Selection
- Aligning with Established Engineering Principles



Footings stress - strain (Briaud & Gibbens, 1999)



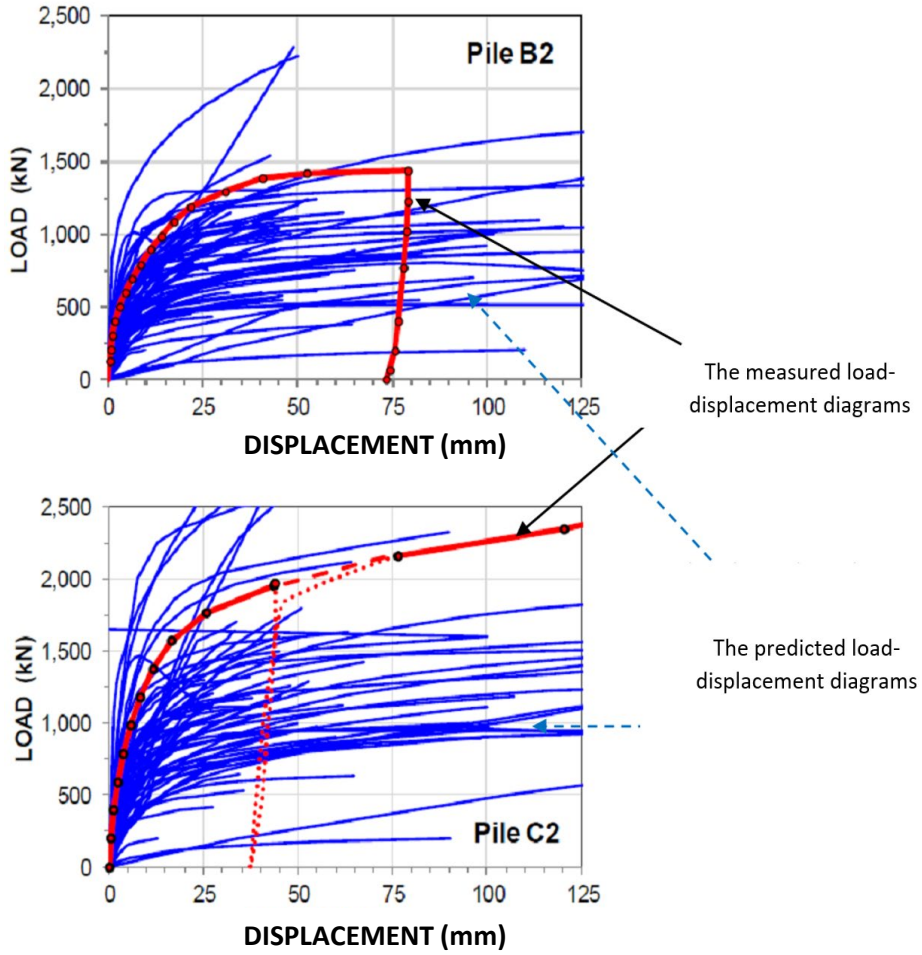
Conceptual stress – strain for large foundation (Eslami & Ebrahimipour, 2026)

Why Databases?

Databases: Collections of data, organized to facilitate access and retrieving data

❖ **Some Topics of Databases in Geotechnics:**

1. Geohazards
2. Earthquake Induced Liquefaction
3. Pile Loading Test
4. Geostructures Performance
5. In-situ Tests for Foundation Design



Predicted load-displacement diagrams vs. measured for Bolivian Contest (Fellenius et al., 2017)

Why Databases?

❖ Advantages and Applications:

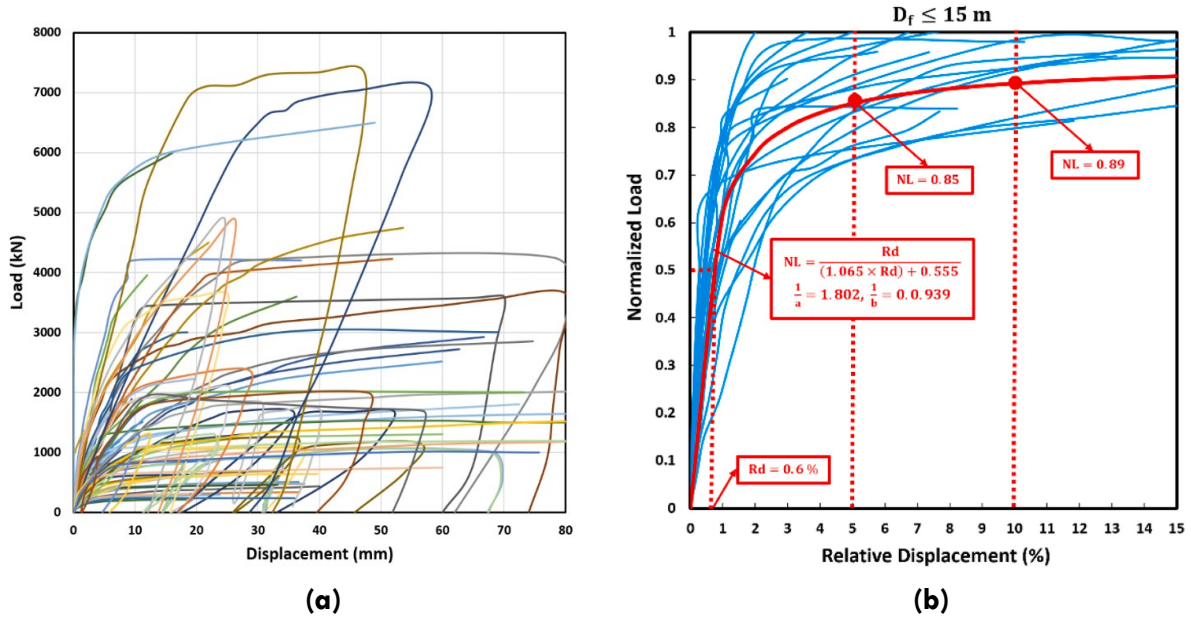
I. Cost saving and project execution time

II. Optimization of design methods

III. Evaluation of design methods

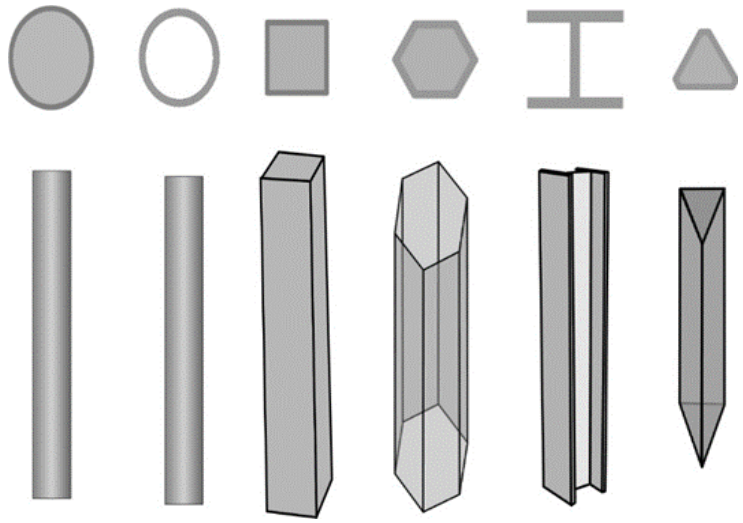
IV. Development of new methods

V. Improvement of geotechnical studies



Driven piles database: (a) Bunch of load-displacement records, (b) Analysis for normalized load vs. relative displacement (Eslami & Ebrahimipour, 2024)

Types of Deep Foundations: Conventional Piles

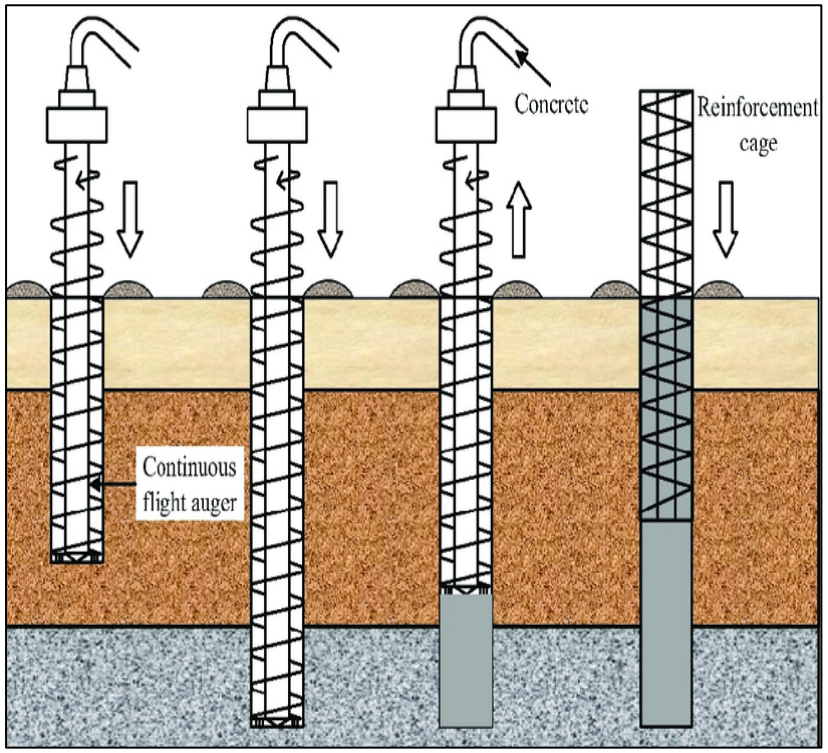


Driven piles variety

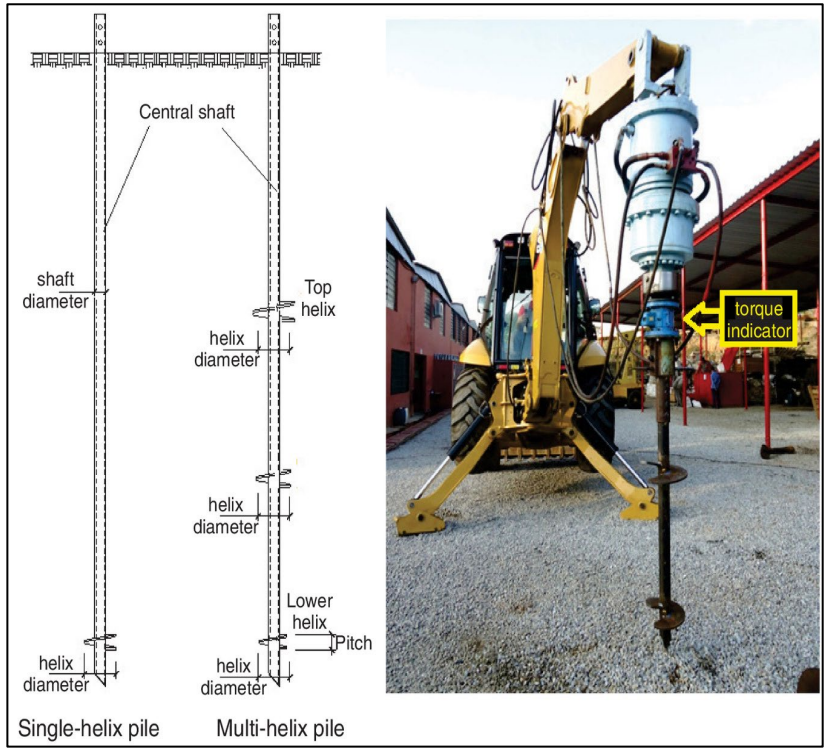


Drilled shafts implementation
(FHWA, 2018)

Types of Deep Foundations: Special Piles



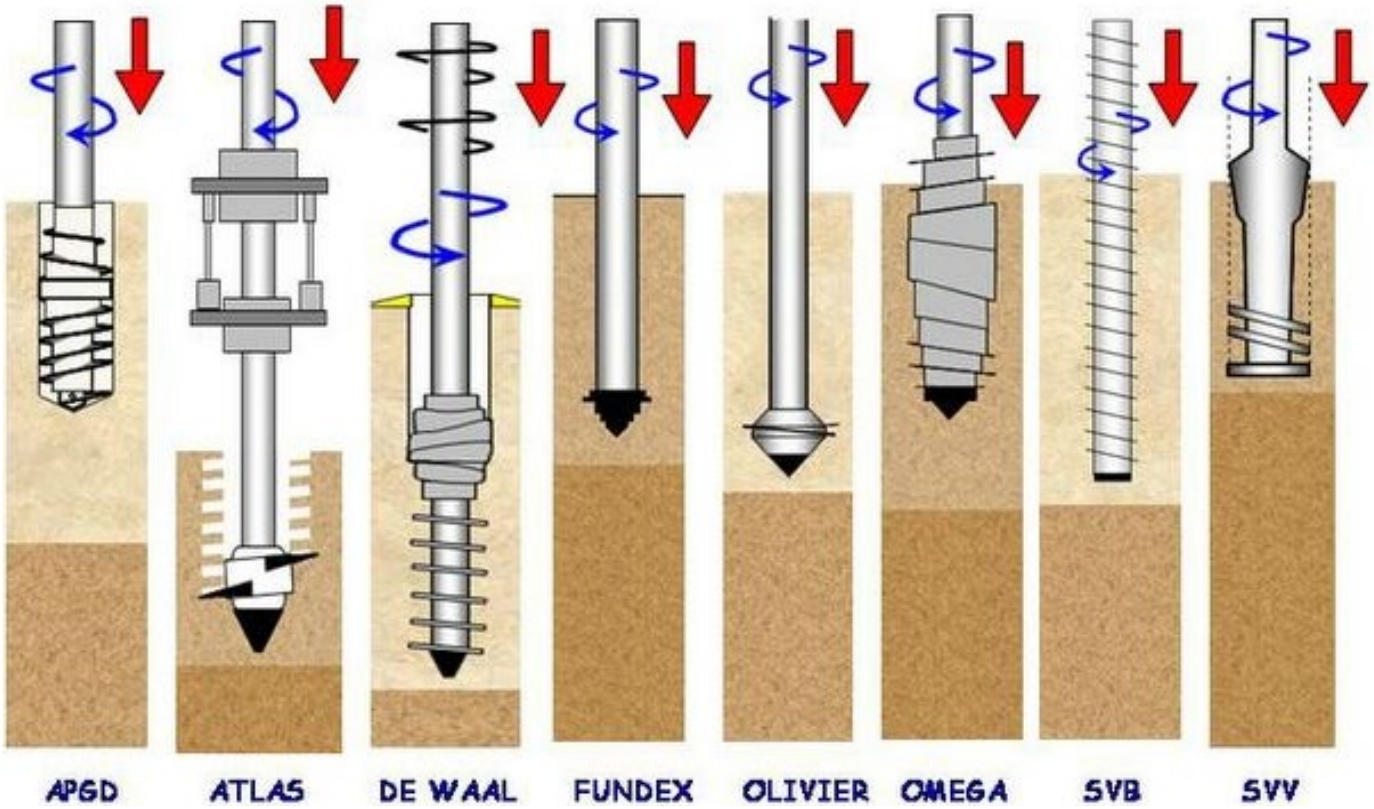
Schematic of CFA pile installation
(Josa et al., 2021)



Helical piles installation
(Mosquera et al., 2015)

DD Piles Advantages

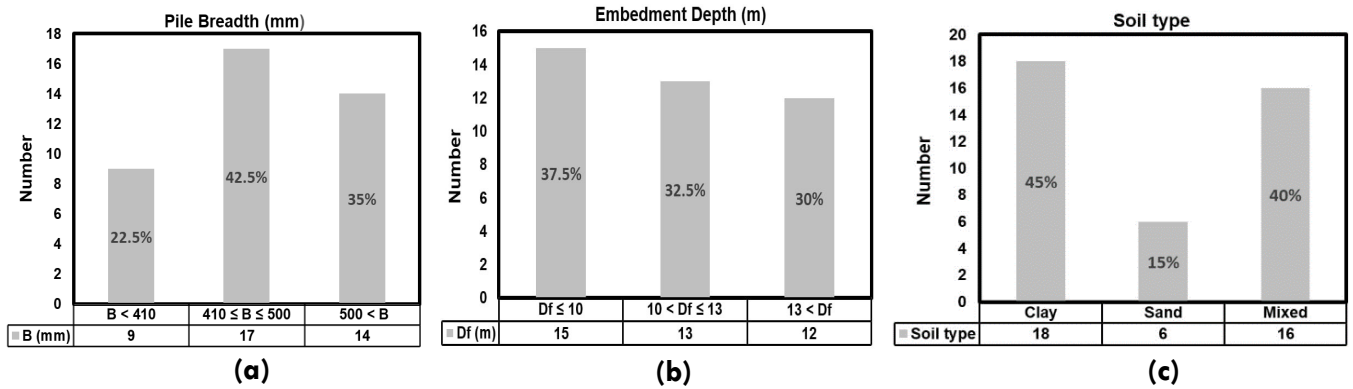
- Enhanced Soil Strength
- Minimal Boring Spoil Generation
- Reduced Environmental Impact
- Proven Reliability
- Economic and Feasibility Advantages



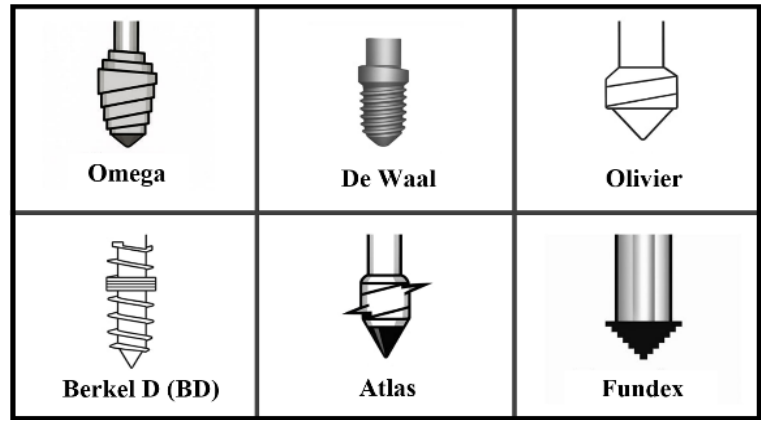
Installation procedure for common DD piles (Basu & Prezzi, 2005) ©Purdue University

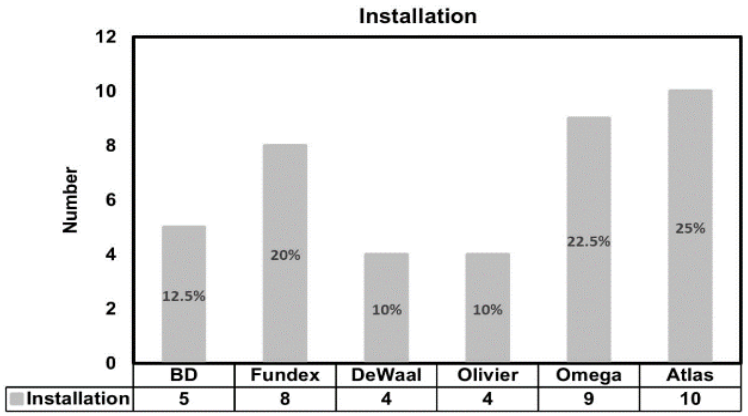
Compiled Database Characteristics

- 40 full-scale static load tests from multiple countries
- Type: Fundex, Atlas, Omega, and Berkel D
- Diameter: from 356 mm to 650 mm
- Embedment Depth: from 7 m to 19.5 m
- Soil Conditions: Clay, sand, and mixed deposits



Graphical distribution of compiled records characteristics: a) breadth, b) embedment depth, c) surrounding soil type





Included DD piles configuration & number

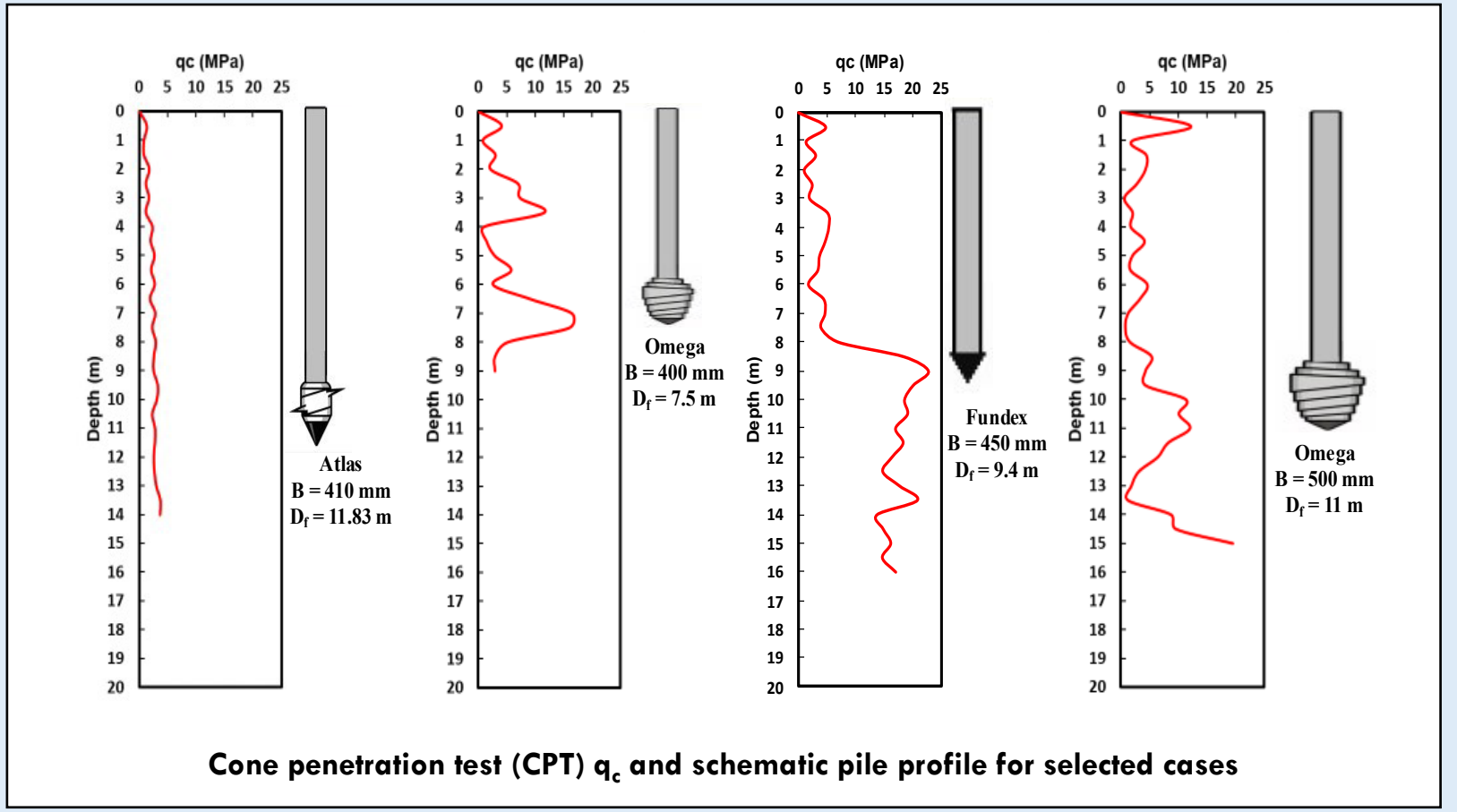
Table of Cases

Case No.	Reference	Location	Pile Data				Deposit	Maximum Measurement	
			Installation	D_{SN} (mm)	D_{BN} (mm)	D_f (m)		Load (kN)	Disp. (mm)
01	Holeyman et al. (2000a, b)	Belgium	Fundex	380	400	11.5	Clay	2000	15
02			De Waal	410	410	11.73	Clay	1800	18
03			Olivier	510	510	11.68	Clay	1900	20
04			Atlas	510	510	11.76	Clay	2100	22
05	De Cock (2001)	Belgium	Atlas	360	460	13.1	Clay	1765	35.5
06		Belgium	Atlas	360	460	13.1	Sand	2763	46.6
07		Belgium	Atlas	460	460	13.5	Sand	3000	66
08		Germany	Atlas	410	510	9.5	Mixed	2300	25.8
09		Germany	Atlas	410	510	7	Mixed	1710	29.8
10		Belgium	Atlas	360	500	13	Clay	1950	24.1
11		Belgium	Atlas	510	650	13.2	Clay	2487	64.9
12		Belgium	Atlas	360	500	13	Clay	2055	26
13		Netherlands	Fundex	450	560	11	Sand	3000	35.8
14		Germany	Fundex	420	560	14	Mixed	3000	31.4
15		Germany	Fundex	440	560	8.5	Sand	1752	28.2
16		Belgium	Fundex	380	450	13	Mixed	2400	45.6
17		Belgium	Fundex	380	450	13	Mixed	2063	42.1
18		Belgium	Omega	410	410	8.2	Sand	1900	55.9
19		Belgium	Omega	360	360	14.5	Clay	1200	46.2
20		Belgium	Omega	360	360	14.5	Clay	1350	46.6
21		Belgium	Omega	360	600	11	Clay	1500	42.9
22		Belgium	Omega	360	600	11	Clay	1800	42.5

Table of Cases; Continued

Case No.	Reference	Location	Pile Data				Deposit	Maximum Measurement	
			Installation	D_{SN} (mm)	D_{BN} (mm)	D_f (m)		Load (kN)	Disp. (mm)
23	Holeyman & Charue (2003)	Belgium	Fundex	390	450	9.5	Clay	3012	69
24			Olivier	550	550	9.2	Clay	3368	31.5
25			Omega	410	410	9.5	Clay	2826	54
26			DeWaal	410	410	9.5	Clay	2441	66
27			Olivier	550	550	9.1	Clay	2910	30
28			DeWaal	410	410	9.5	Clay	2267	86.5
29	NeSmith & NeSmith (2007)	USA	BD	406.4	406.4	18.3	Sand	3430	38.1
30	Park et al. (2012)	USA	BD	356	356	15.3	Mixed	3350	46
31			BD	356	356	19.5	Mixed	3000	30.5
32			BD	406	406	17.4	Mixed	3400	31
33			BD	356	356	15.2	Mixed	3200	32
34	Moshfeghi & Eslami (2018); Charue (2002)	Belgium	Atlas	510	510	9.6	Mixed	3603	65.5
35			DeWaal	410	410	9.5	Mixed	2653	63
36			Fundex	390	450	9.6	Mixed	3129	69.5
37			Omega	410	410	9.5	Mixed	3206	100
38			Olivier	550	550	9.5	Mixed	3062	31.5
39	Krasiński (2021)	Poland	Omega	400	400	12.6	Mixed	890	44.5
40	Krasiński (2023)	Poland	Omega	300	500	11	Mixed	993	40

Subsoil Condition; CPT Logs



Cone penetration test (CPT) q_c and schematic pile profile for selected cases

Case Study: Oil Tanks - Belgium (Van Impe et al., 2013 & 2015)

❖ Tanks

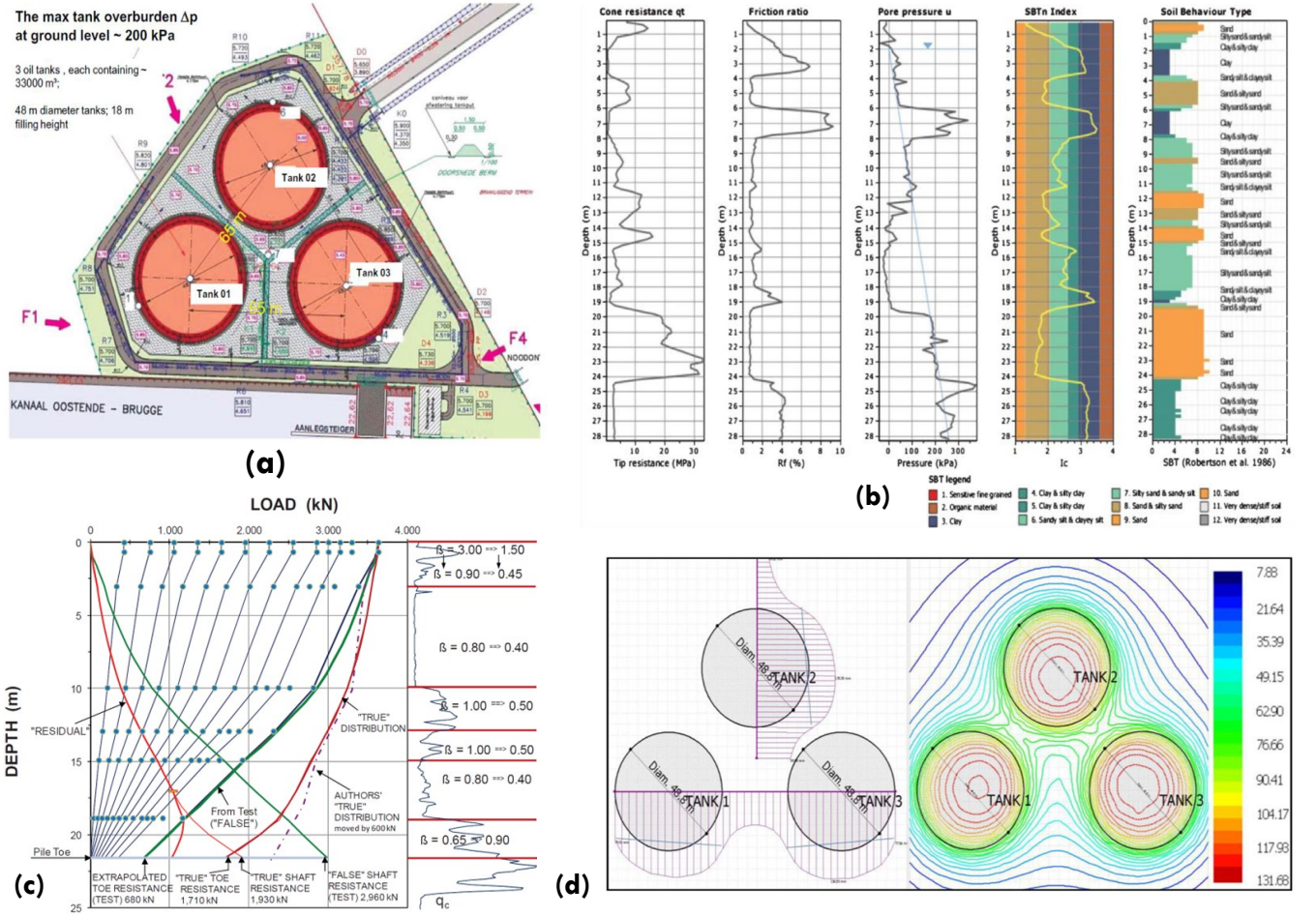
- 48 m diameter and 19 m height,
- Triangle configuration - 65 m center to center

❖ Soil Profile

- 0–12 to 15 m depth: Old fill (GWT at 2 m)
- 12-15 to 18 m depth: Sand
- 18 to 100-120 m depth: OC Clay

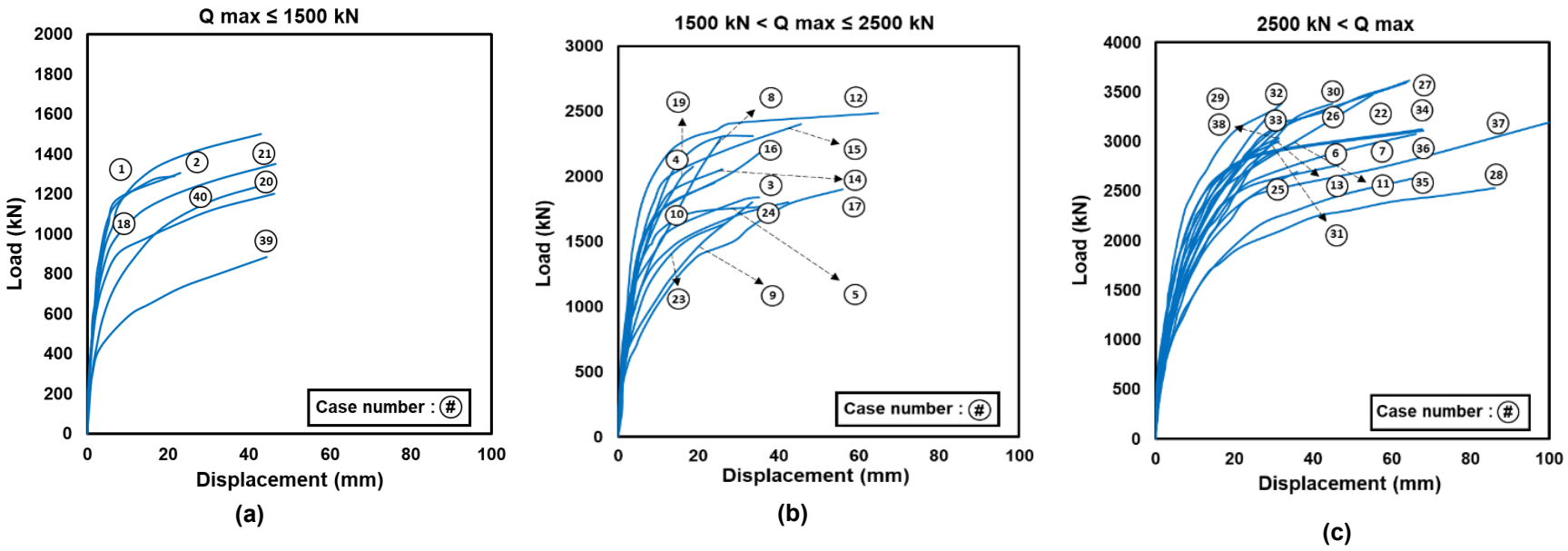
❖ Foundation System

- Pile Cap: 0.6 m thick and 49 m diameter
- 422 Omega Piles as pile group
- Piles: 460 mm in diameter and 22 m in depth



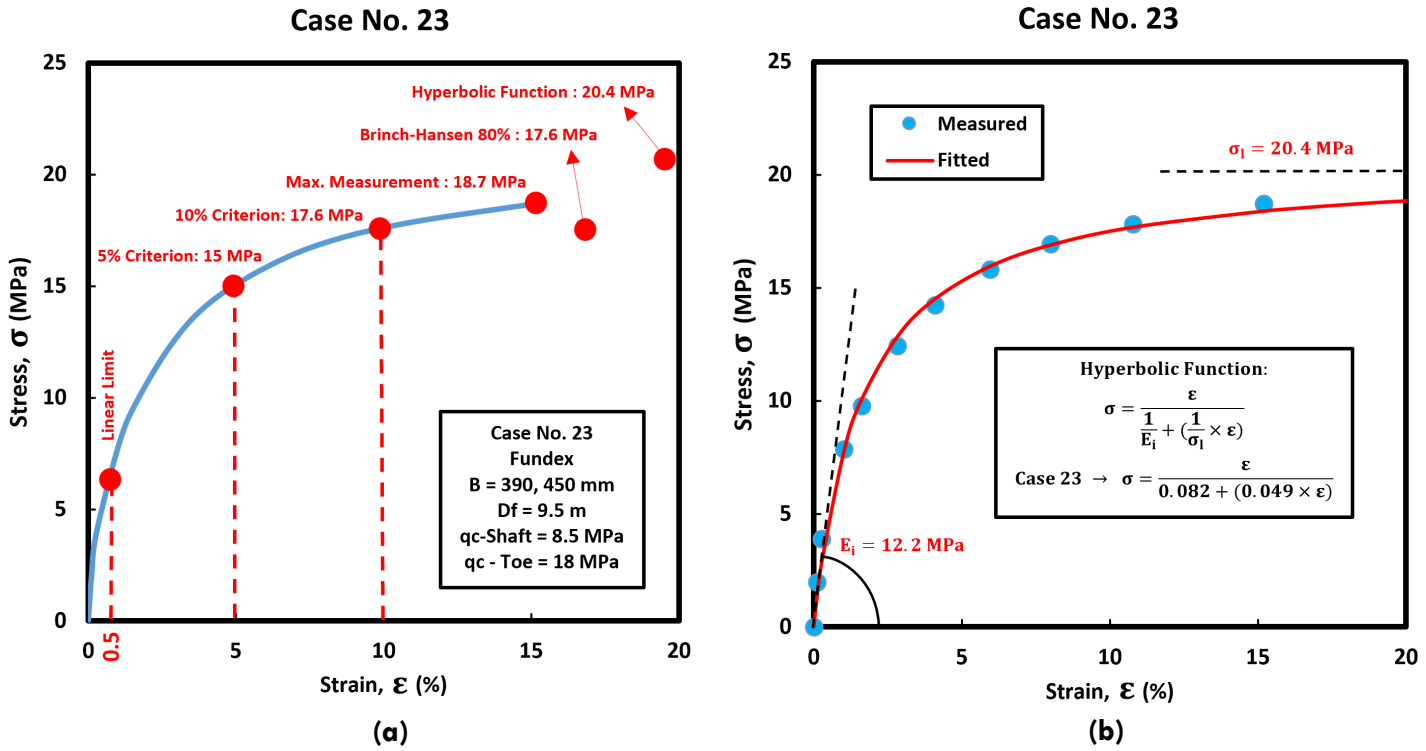
Three oil tanks case study: a) General view tanks, b) CPTu log (Van Impe et al., 2013), c) Load and resistance distribution (Fellenius, 2014), d) Settlement analysis under operational load (Van Impe et al., 2015)

Compiled Load-Displacement Records



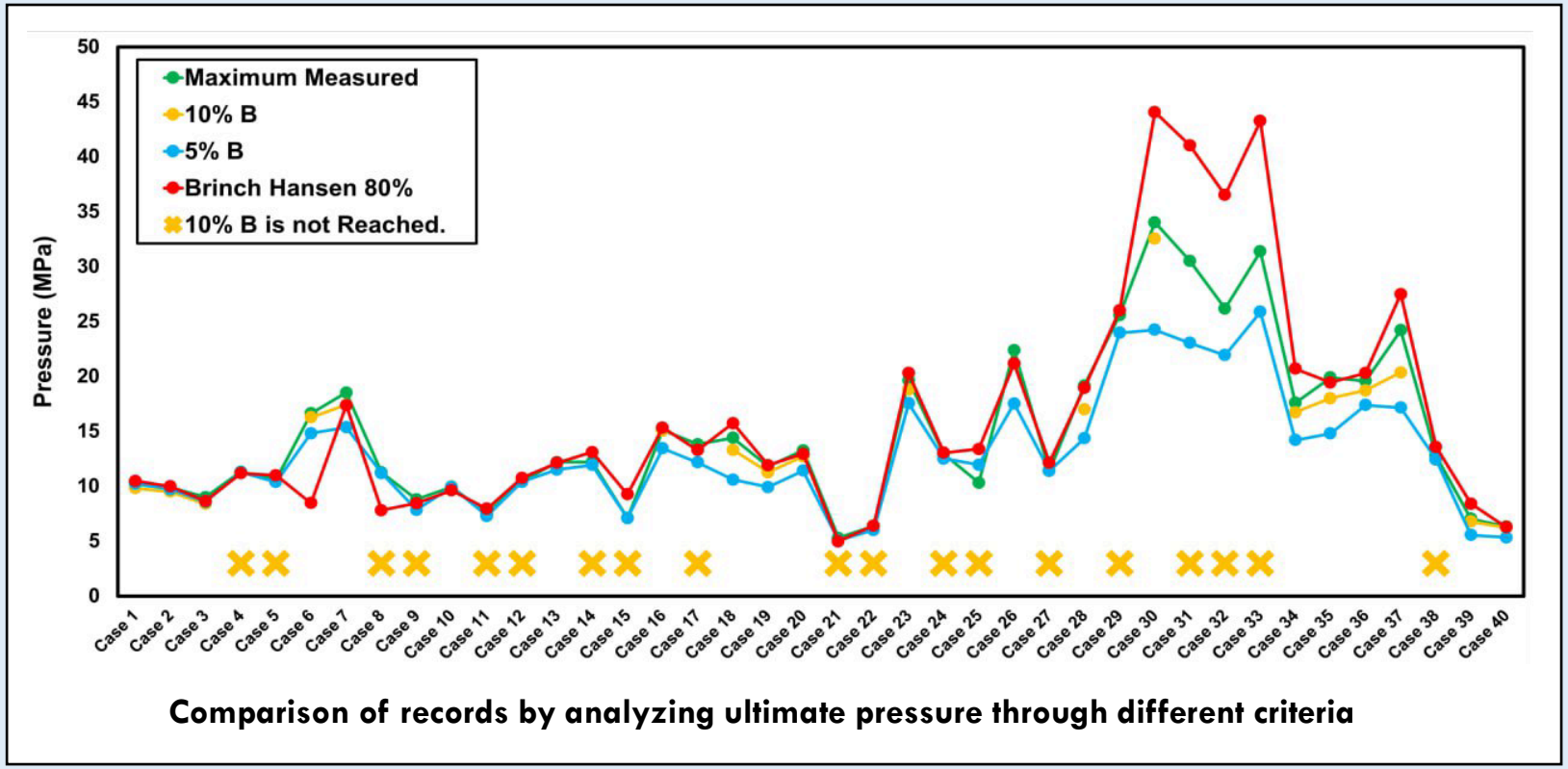
Load-displacement diagrams for all compiled cases: a) $Q_{max} \leq 1500kN$, b) $1500kN < Q_{max} \leq 2500kN$, c) $2500kN < Q_{max}$

Criteria Variety & Hyperbolic Fitting



Typical stress-strain and criteria for Case No. 23: a) Stress resultant from various criteria, b) Hyperbolic fitting and concepts

Ultimate Pressure Through Different Criteria

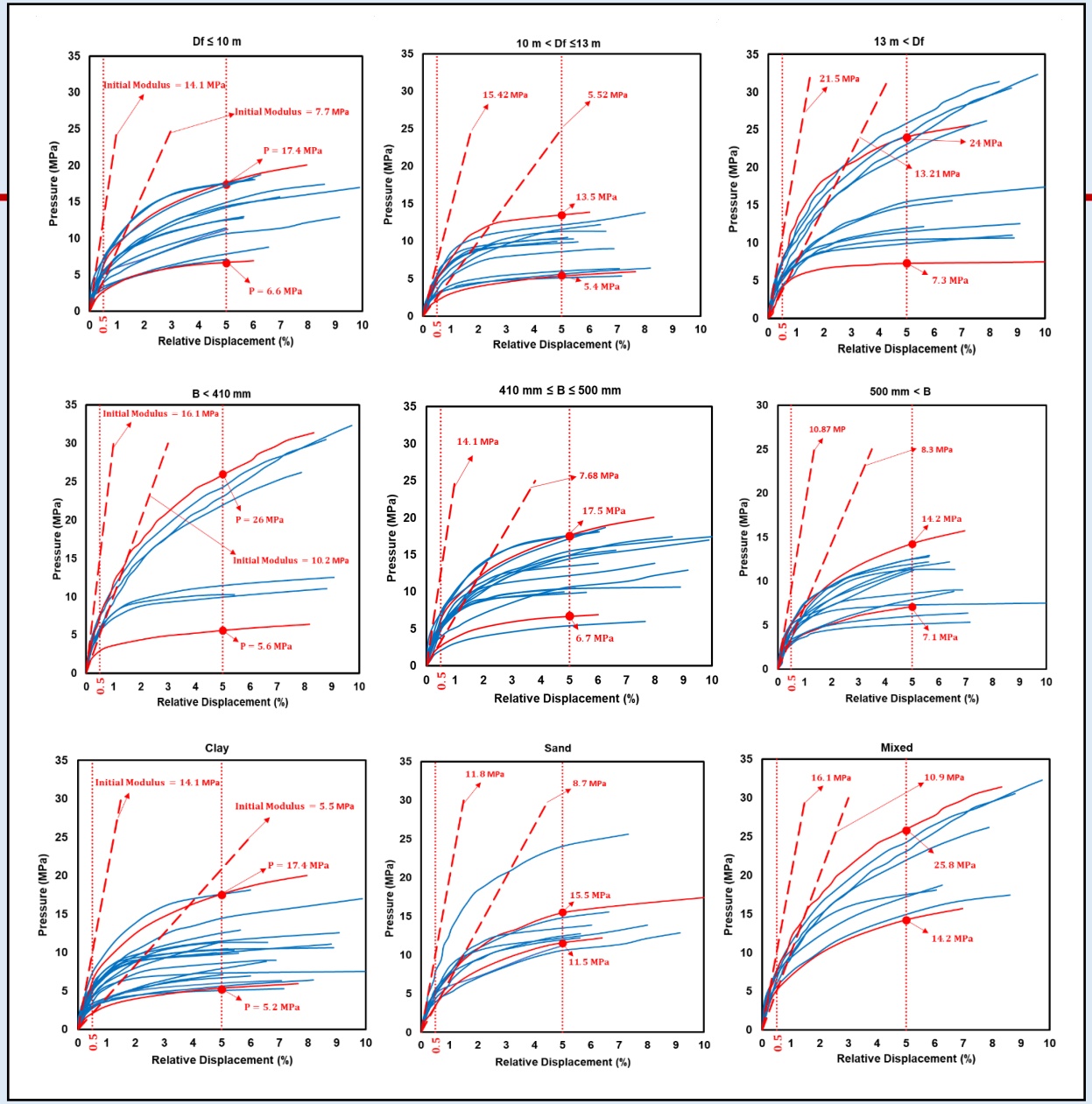


Comparison of records by analyzing ultimate pressure through different criteria

***5% B** and **Brinch-Hansen 80%** criteria opted for further analysis due to:*
***Simplicity, Efficiency** and **Adoptable** for All Cases*

Dominant Factors Assessment

- Embedment Depth
- Pile Breadth
- Surrounding Deposit: (Shaft and Toe)



Energy-Based Evaluation: Toughness and Resilience

$$\frac{Energy}{Volume} = \int_0^{\varepsilon} \sigma \, d\varepsilon$$

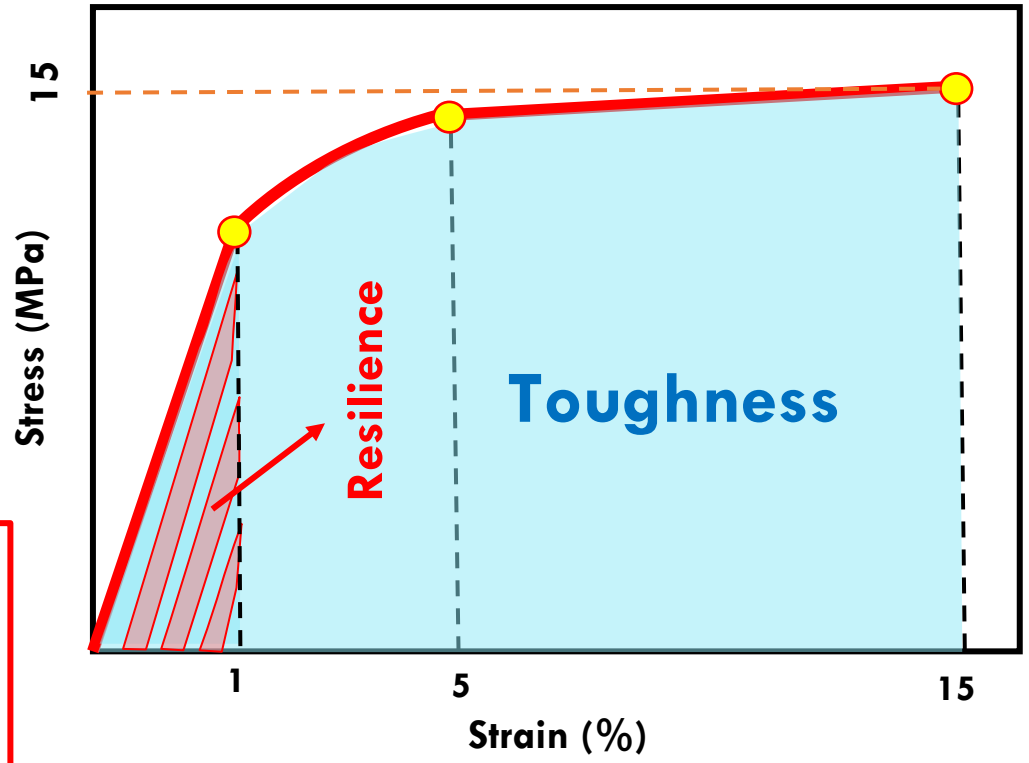
ε = Strain,

σ = Stress

Energy Absorption;

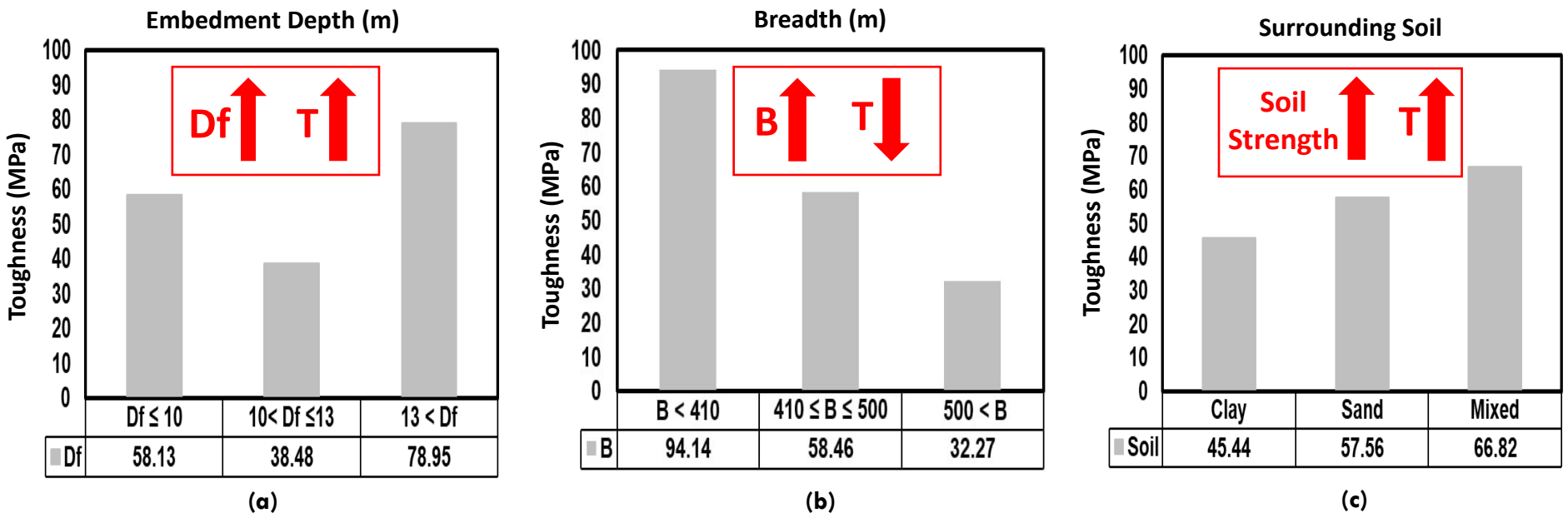
Resilience: Elastic Region

Toughness: Fracture Threshold



Resilience and toughness Concept

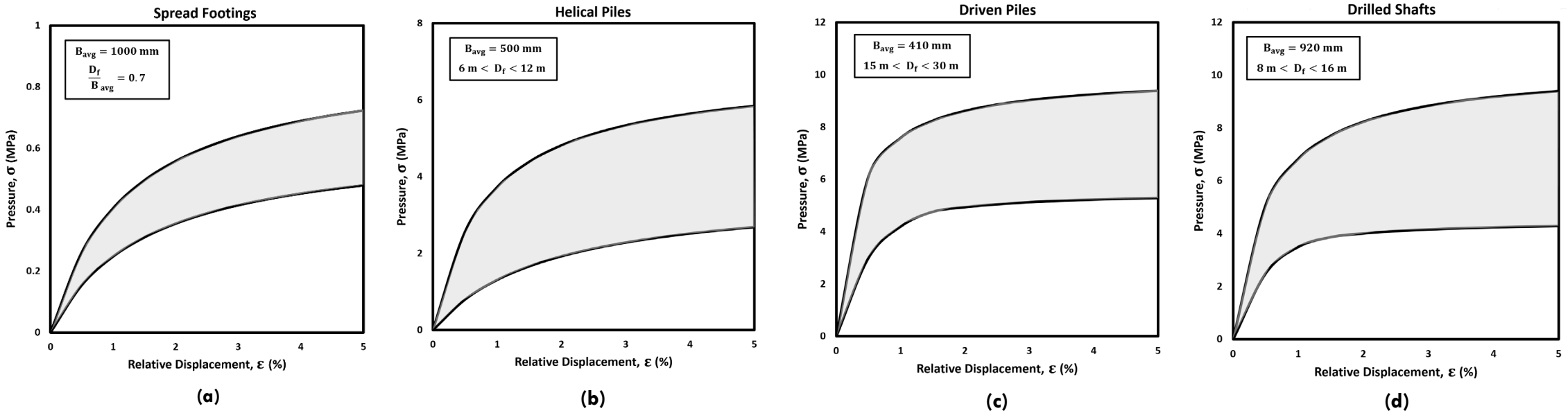
Dominant Factors on Toughness (T) Value



Average toughness values up the displacement of 1 inch: a) Embedment depth, b) Breadth, c) Surrounding soil type

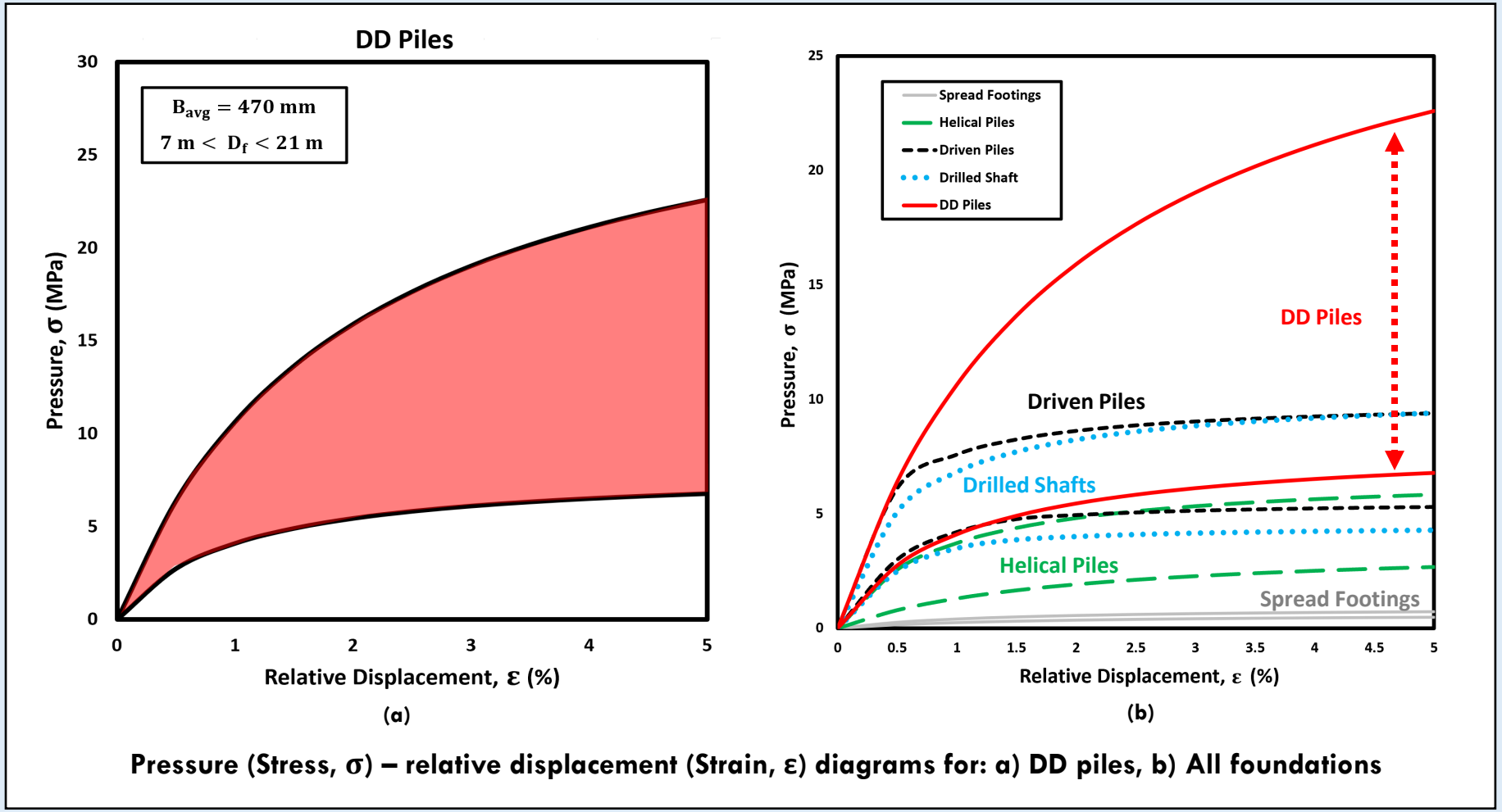
Longer, Narrower piles in Mixed soil condition lead to Higher toughness value

σ – ε Performance for Various Foundations



Pressure (Stress, σ) – relative displacement (Strain, ε) diagrams for various foundations: a) Spread Footings (Heidari et al., 2025), b) Helical Piles (Rahimi et al., 2024), c) Driven piles (Eslami & Ebrahimipour, 2024), d) Drilled shafts

σ – ε Performance for Various Foundations



Pressure (Stress, σ) – relative displacement (Strain, ε) diagrams for: a) DD piles, b) All foundations

Remarks & Conclusions

Drilled Displacement (DD) Piles; Efficient and Sustainable Solution

Load-Displacement Records; Embracing Major Design Criteria

Stress-Strain Framework; Normalized Uniform Appraisal/Comparison

Data-Centric Approach; Tackling Uncertainty/Enhancing Prediction

Energy-Based Paradigm; Towards Resilient Foundations

Major References

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THANKS FOR

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PARALLEL SESSION B5

Modeling is Better Than Pyrite

Investigation of Limit Equilibrium and Finite Element Methods for Ground-Anchored Slopes

Farnyuh Menq, Daqing Xu, Luis Vasquez, and Shin-Tower Wang

Ensoft Inc.



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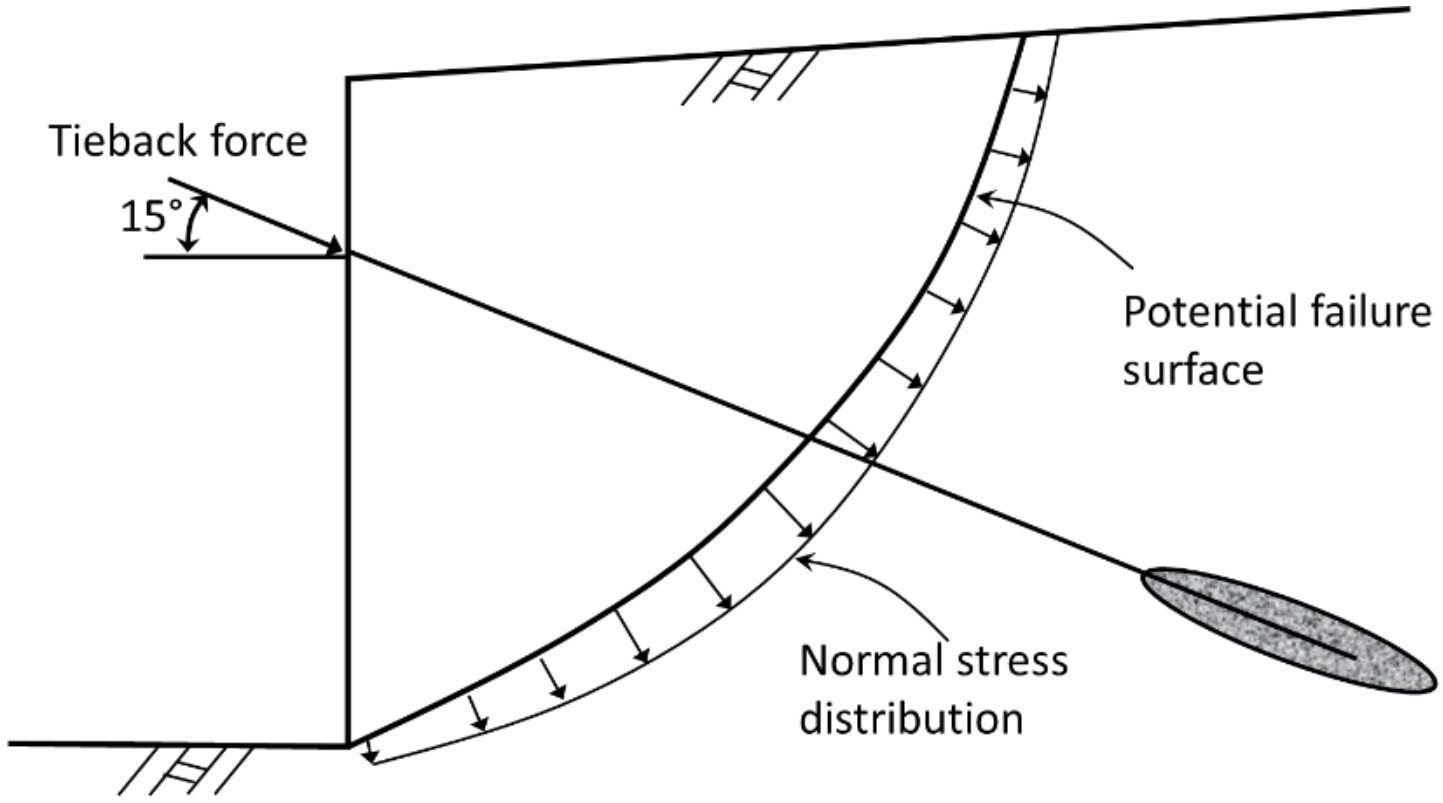
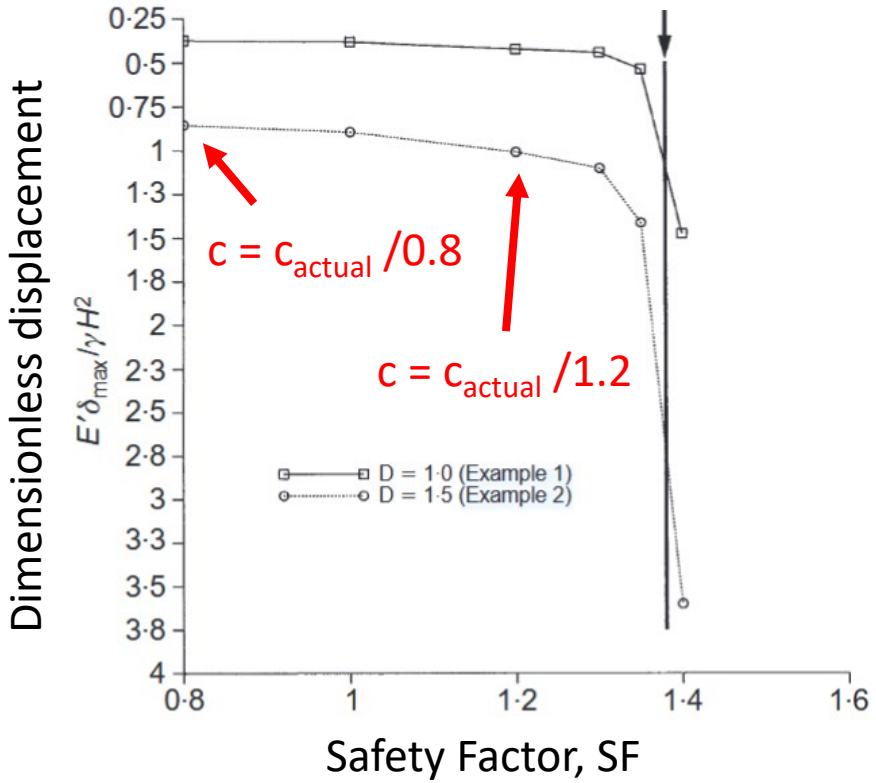
Load Distribution Method (Carpenter, 1986)

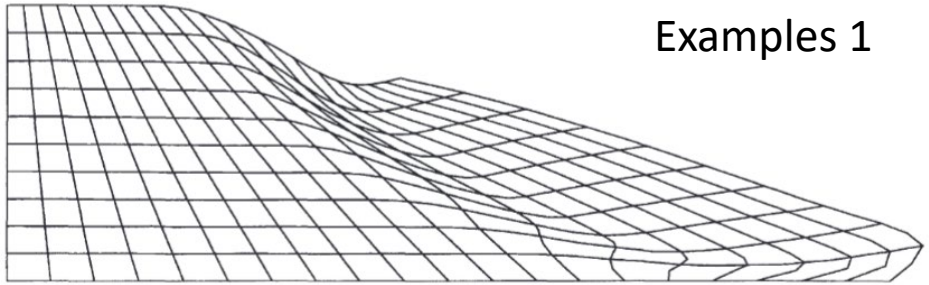


Based on Flamant's (1886) stress distribution model: $\sigma_r = \frac{2\,P\,\cos(\theta)}{\pi\,R}$

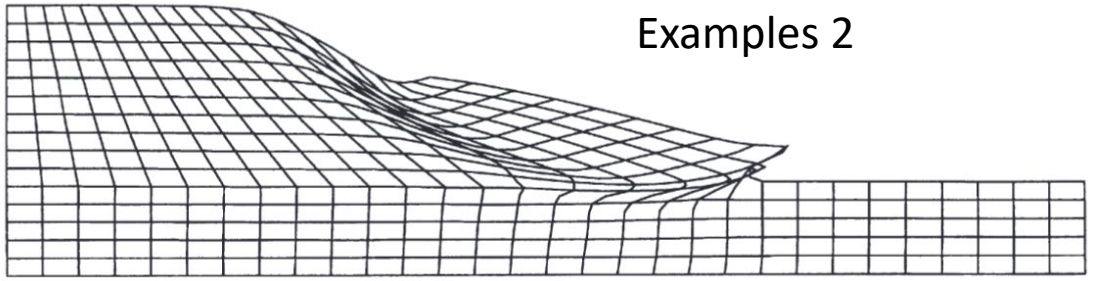
Strength Reduction Finite Element Method (SRFEM)

Griffiths & Lane (1999) SF = 1.4
Bishop & Morgensten (1960) SF = 1.380





Examples 1



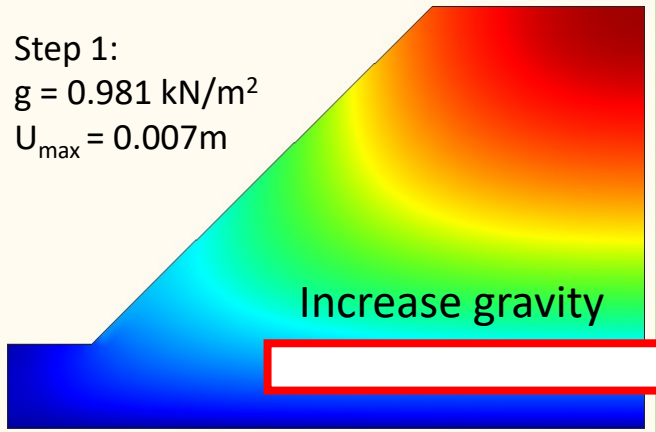
Examples 2

Griffiths, D., Lane, P., 1999. Slope Stability Analysis by Finite Elements. Géotechnique 51, pp. 653–654.

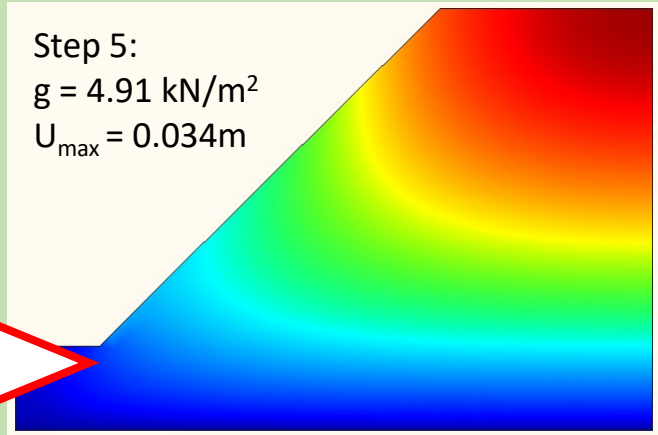
SRFEM implemented in this study using ENFEM

Stage 1 Apply gravity: SF = 0.5, $c_{FS=0.5} = c' /0.5$, $\tan(\phi_{FS=0.5}) = \tan(\phi') /0.5$, ($c' = 4.79$ kPa, $\phi' = 28°$)

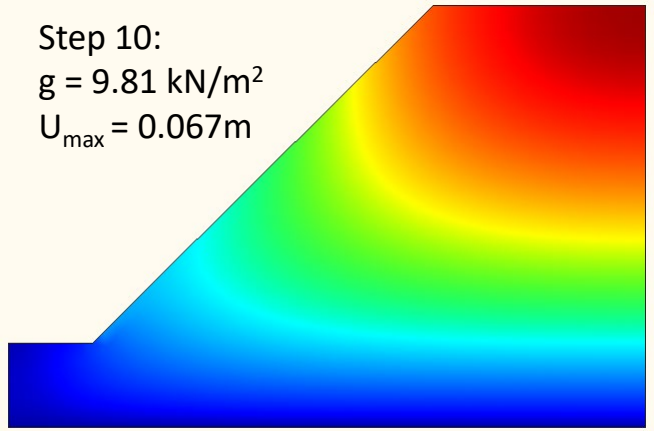
Step 1:
$g = 0.981$ kN/m^2
$U_{max} = 0.007$m



Step 5:
$g = 4.91$ kN/m^2
$U_{max} = 0.034$m

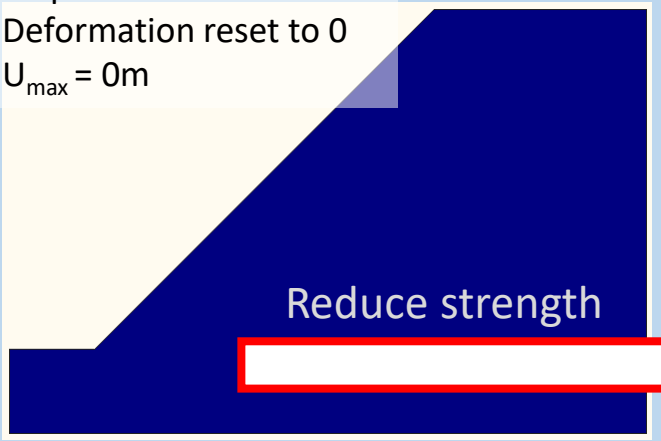


Step 10:
$g = 9.81$ kN/m^2
$U_{max} = 0.067$m

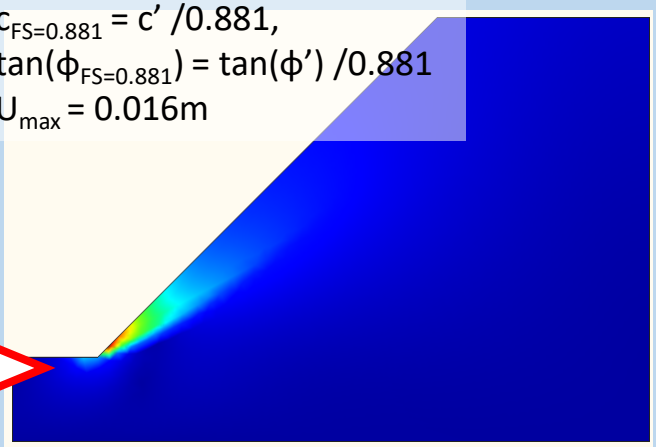


Stage2: Strength reduction: $g = 9.81$ kN/m^2

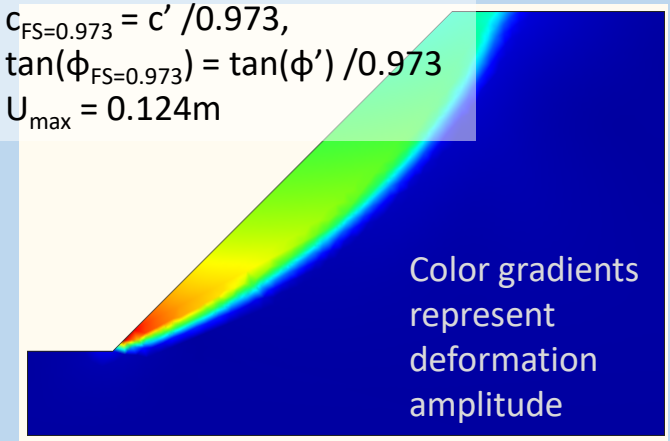
Step 0: SF = 0.5
Deformation reset to 0
$U_{max} = 0$m



Step 5: SF = 0.881,
$c_{FS=0.881} = c' /0.881$,
$\tan(\phi_{FS=0.881}) = \tan(\phi') /0.881$
$U_{max} = 0.016$m



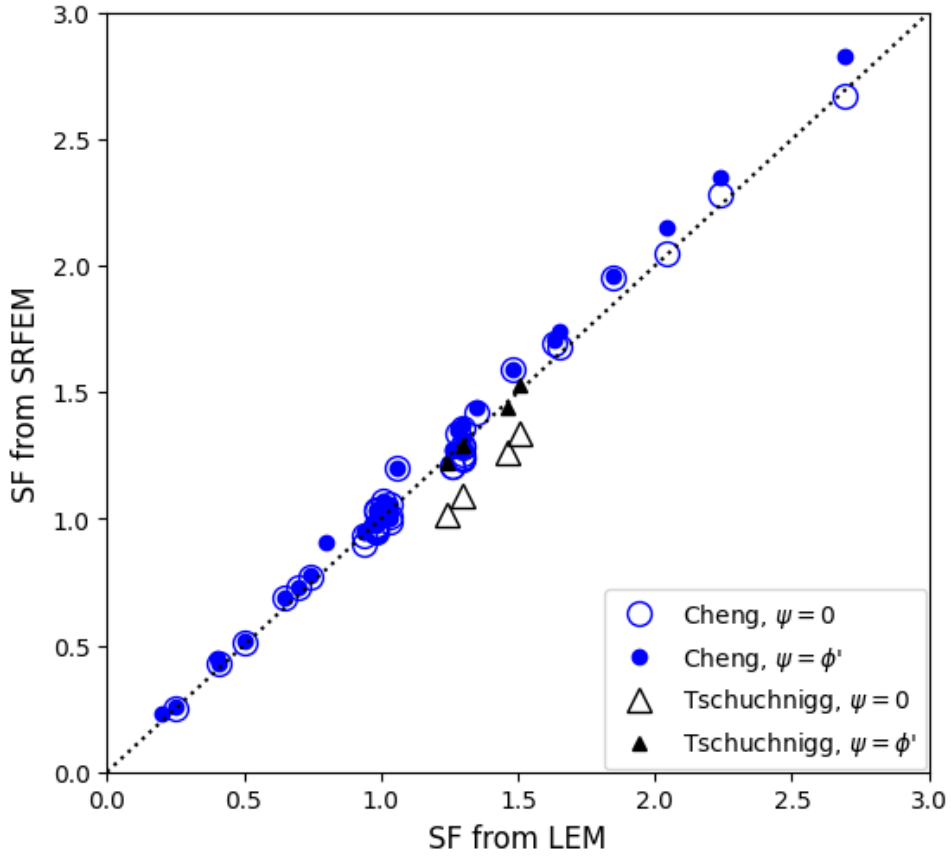
Step 17: SF = 0.973
$c_{FS=0.973} = c' /0.973$,
$\tan(\phi_{FS=0.973}) = \tan(\phi') /0.973$
$U_{max} = 0.124$m

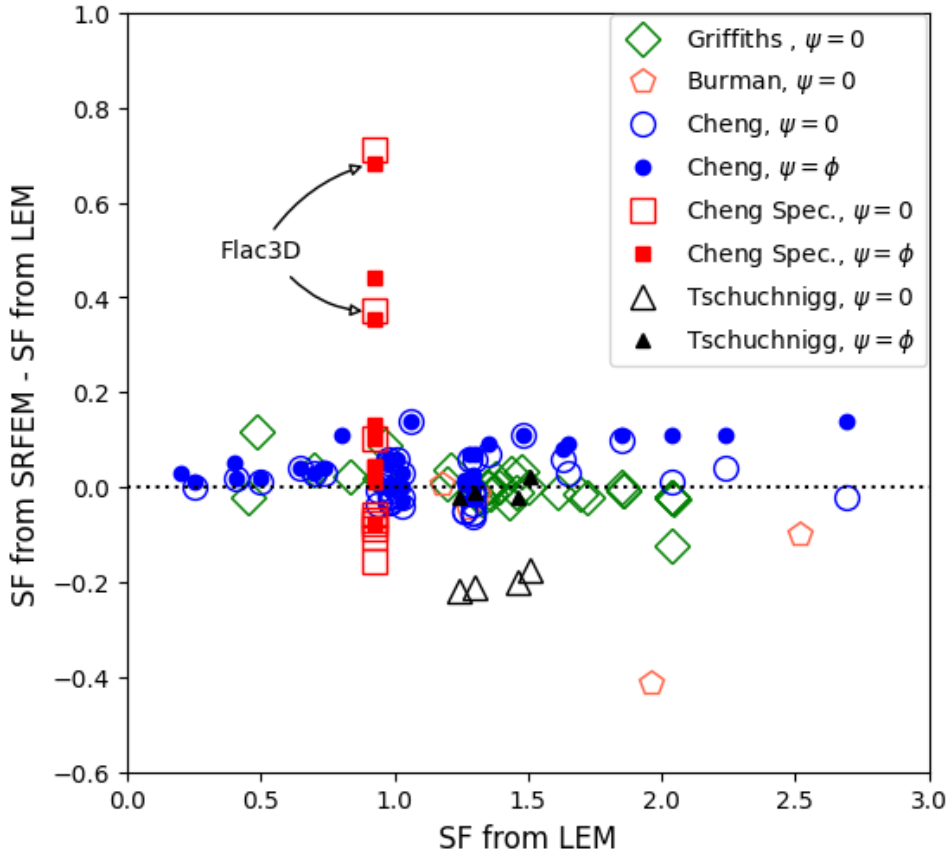


Input parameters

- The Mohr-Coulomb model in Limit Equilibrium Method (LEM) requires 3 parameters: c’, ϕ', ϒ
- The Mohr-Coulomb model in SRFEM requires 6 parameters: c’, ϕ', ϒ + Young's modulus (E), Poisson's ratio (μ), dilation angle (ψ)
- In finite element analyses,
  - Young’s modulus and Poisson's ratio define the deformations in the linear range – limited effect on safety factor (Cheng et al., 2007).
    - Griffiths & Lane (1999) suggested $E = 10^5$ kPa, μ = 0.3.
  - The dilation angle defines the flow rate of the soil element after yielding – some effects on safety factor.
    - Associated flow rule: ψ = ϕ’, large volume change
    - Non-associated flow rule: ψ = 0, no volume change but can have difficulties to converge.

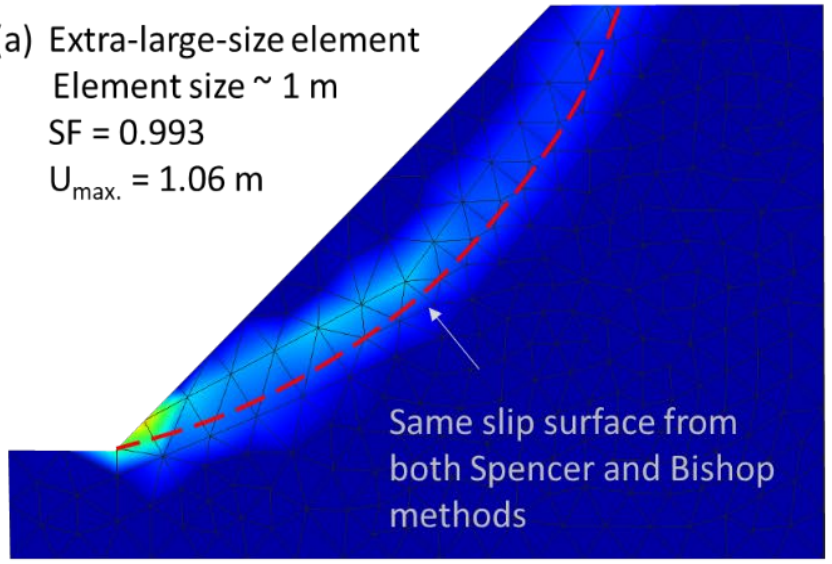
Safety factors from SRFEM vs. from LEM in the literature



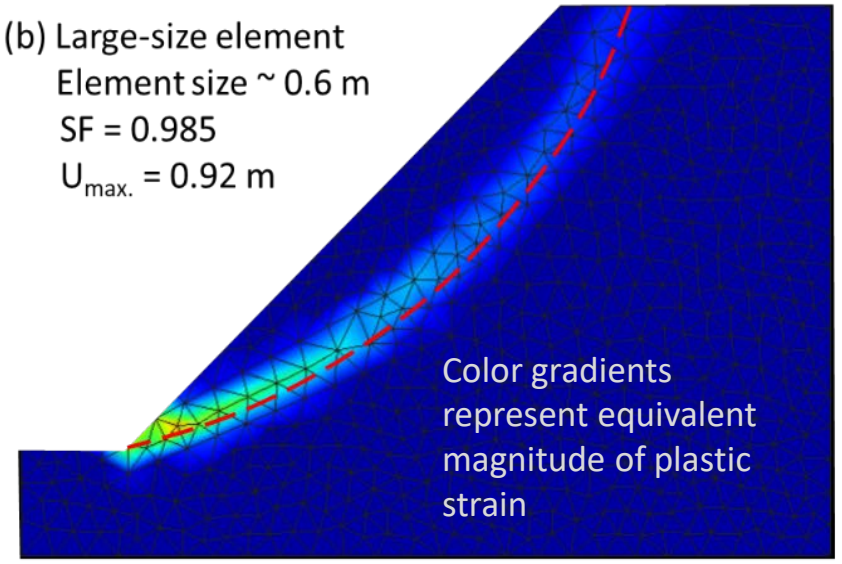


Results from different element sizes

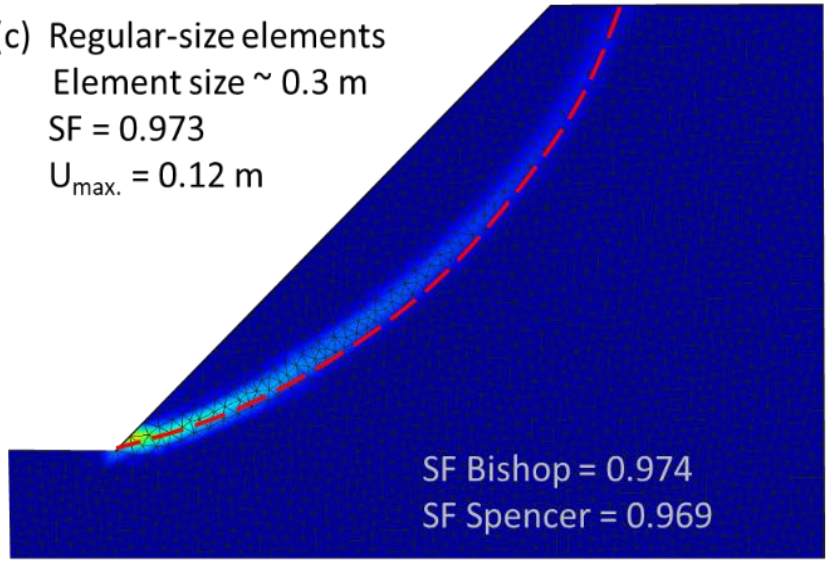
(a) Extra-large-size element
Element size ~ 1 m
SF = 0.993
$U_{max.}$ = 1.06 m



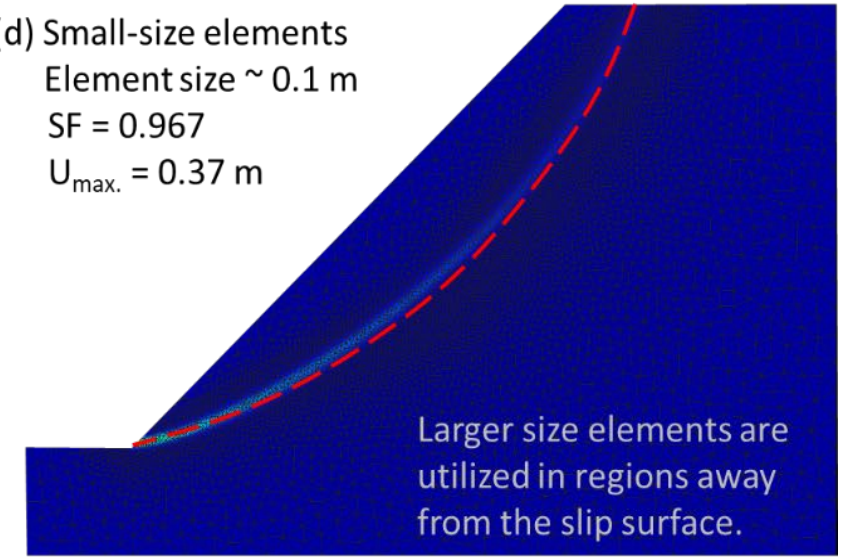
(b) Large-size element
Element size ~ 0.6 m
SF = 0.985
$U_{max.}$ = 0.92 m



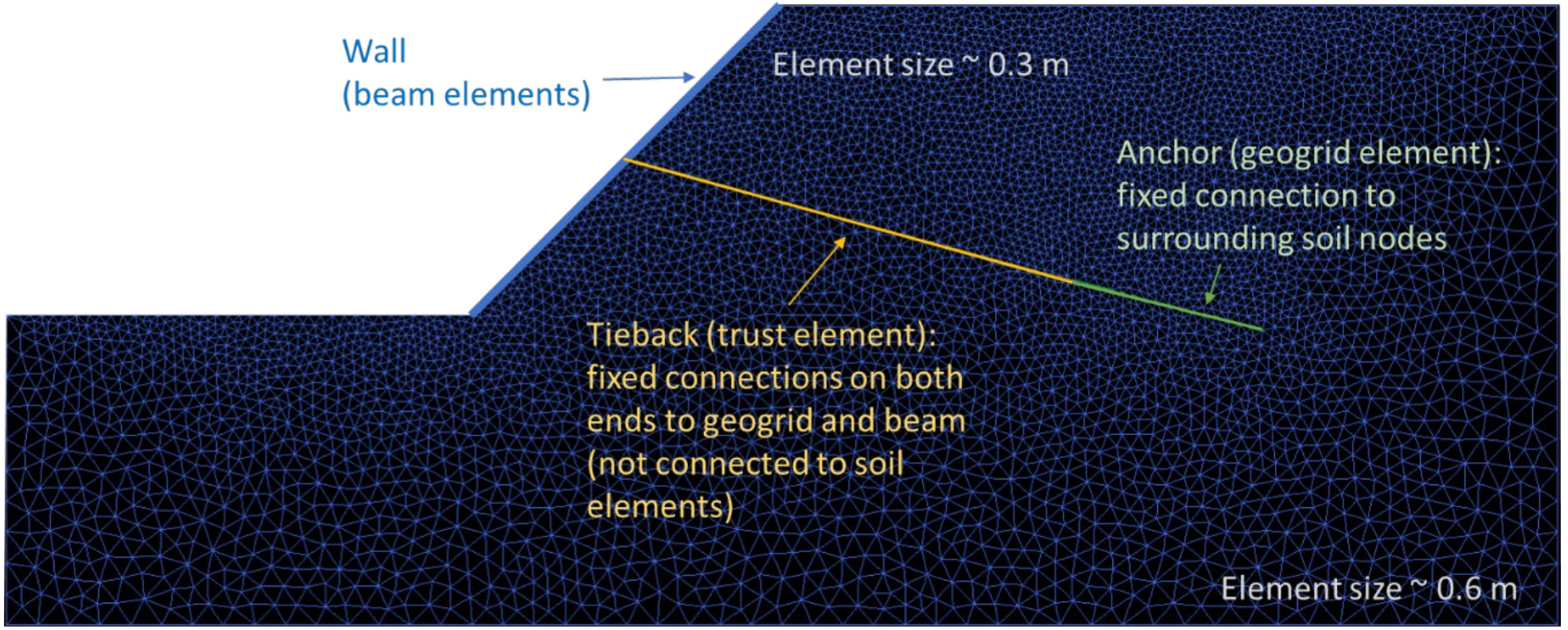
(c) Regular-size elements
Element size ~ 0.3 m
SF = 0.973
$U_{max.}$ = 0.12 m



(d) Small-size elements
Element size ~ 0.1 m
SF = 0.967
$U_{max.}$ = 0.37 m



Mesh plot of the slope with a single row of tiebacks



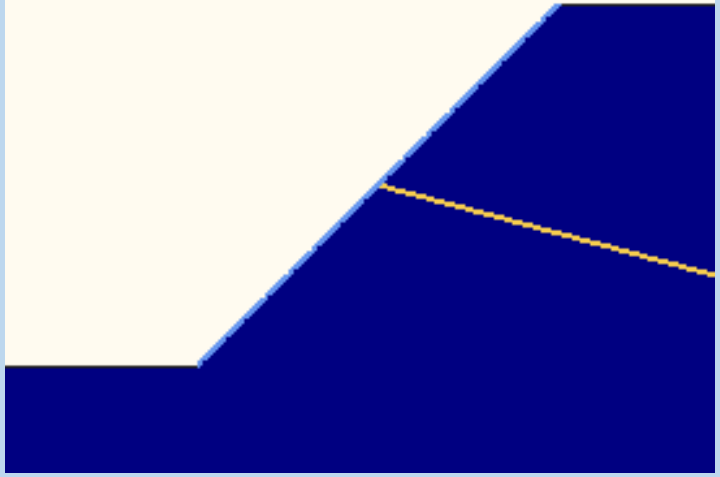
Soil and tieback parameters utilized in SRFEM

	Tieback		Soil					
Model	Diameter (inch/cm)	Prestress (kN)	c' (kN)	ϕ' (°)	ϒ (kN/m³)	E (kPa)	u	Ψ (°)
A	**None**	N/A	4.79	28	19.63	1.0E+05	0.3	28
B	**1.0/2.54**	0	4.79	28	19.63	1.0E+05	0.3	28
C	1.0/2.54	0	4.79	28	19.63	1.0E+05	0.3	**0**
D	1.0/2.54	**450**	4.79	28	19.63	1.0E+05	0.3	28
E	1.0/2.54	0	4.79	28	19.63	**1.0E+04**	0.3	28
F	**2.5/6.35**	0	4.79	28	19.63	1.0E+05	0.3	28

Model B - strength reduction analyses

Stage2: Strength reduction

Step 0 - Deformation reset to 0



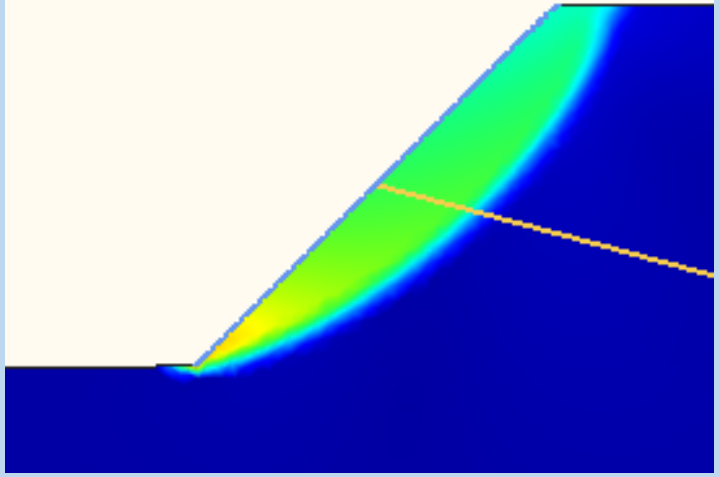
FS = 1.0

$c_{FS=1.0} = c' /1.0$

$\tan(\phi_{FS=1.0}) = \tan(\phi') /1.0$

Tieback force = 37 kN

Step 1



FS = 1.01

$c_{FS=1.01} = c' /1.01$

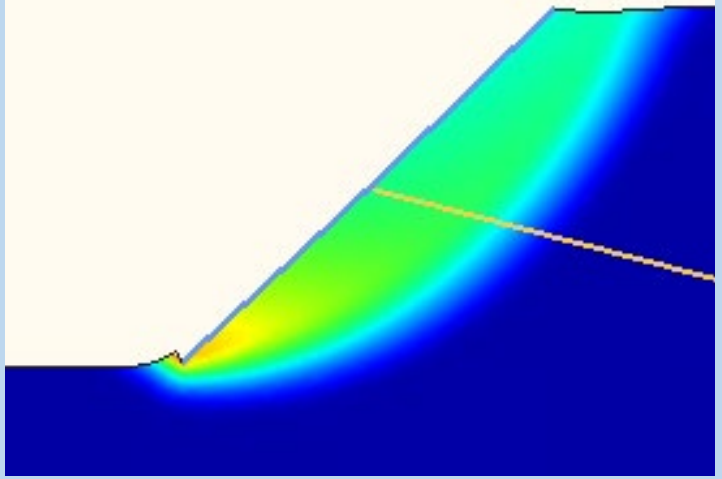
$\tan(\phi_{FS=1.01}) = \tan(\phi') /1.01$

Tieback force = 47 kN

...

...

Step 102 - Last converged step



FS = 1.606

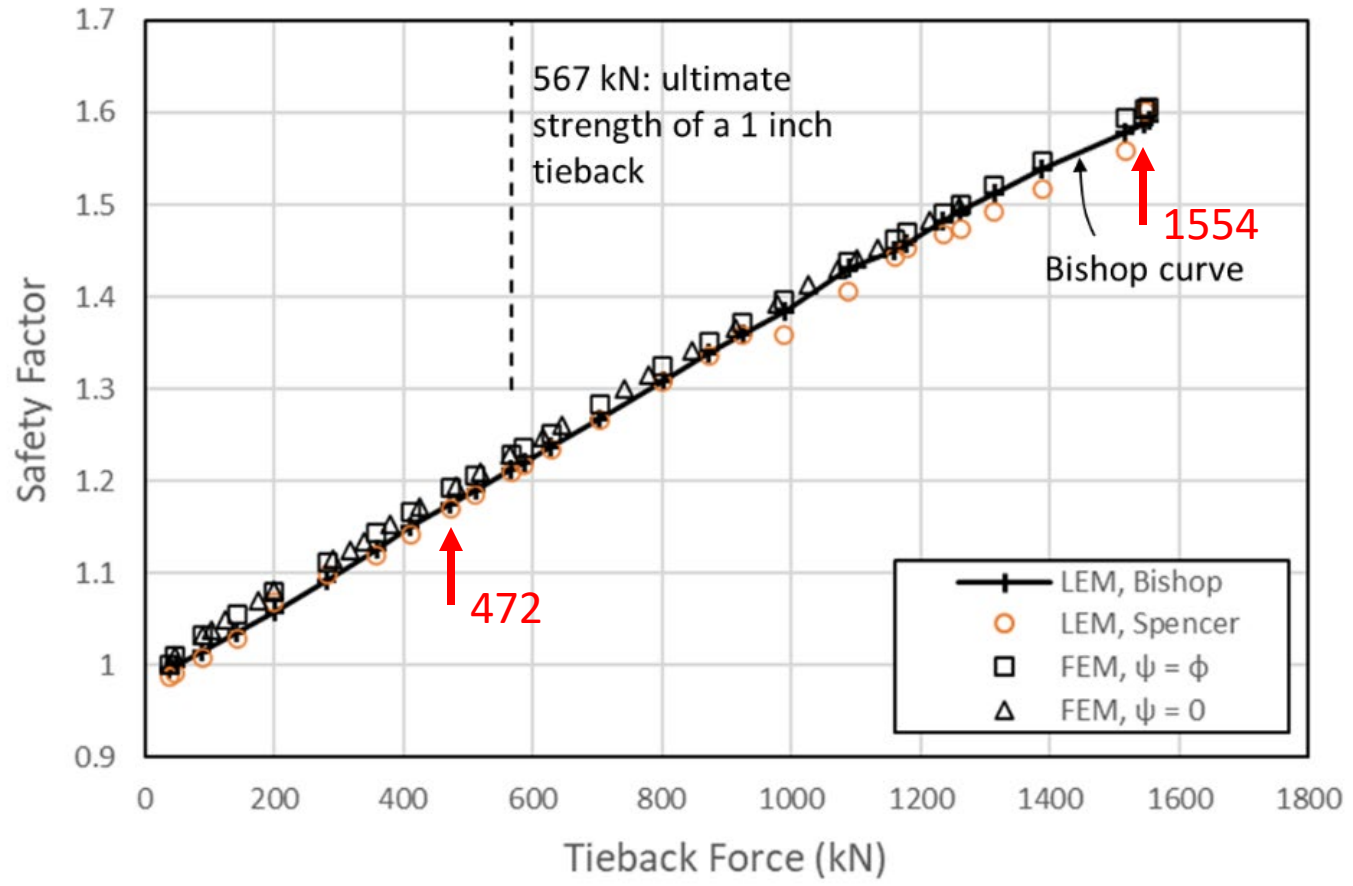
$c_{FS=1.606} = c' /1.606$

$\tan(\phi_{FS=1.606}) = \tan(\phi') /1.606$

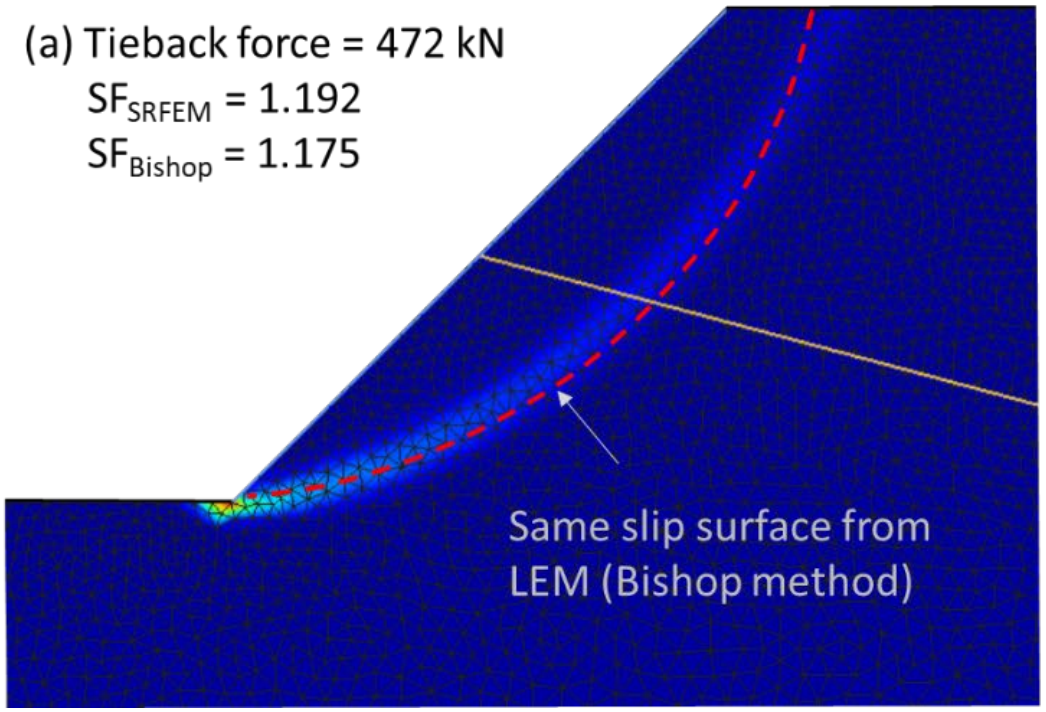
Tieback force = 1554 kN

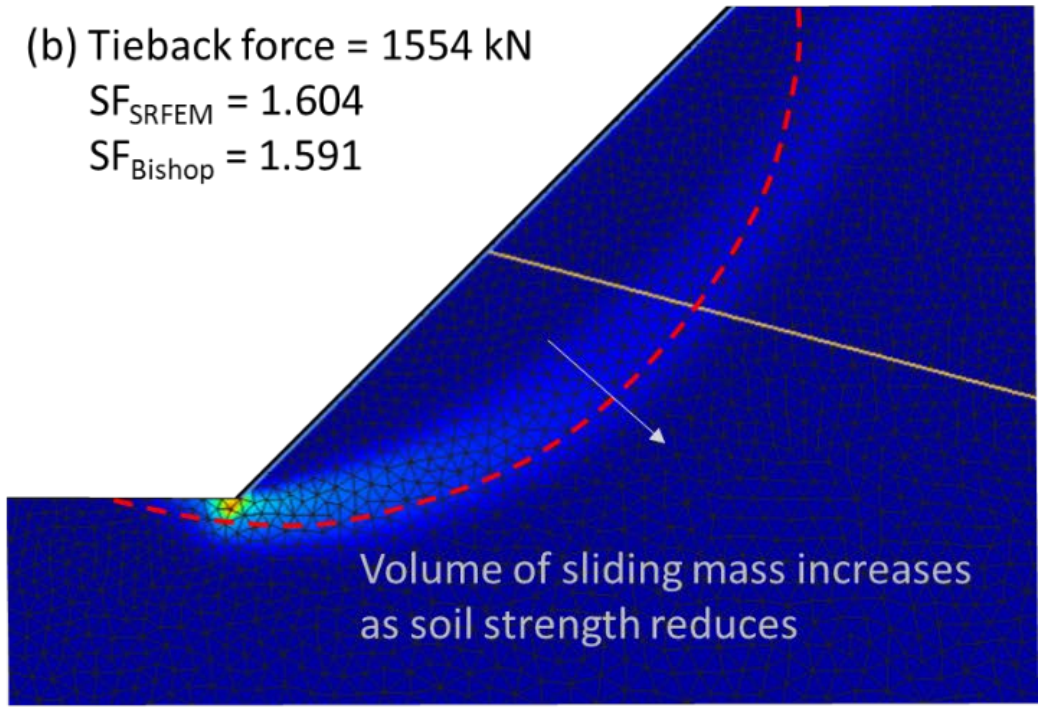
Strength reduces as the safety factor increases

Safety factors versus resulting tension from Model B

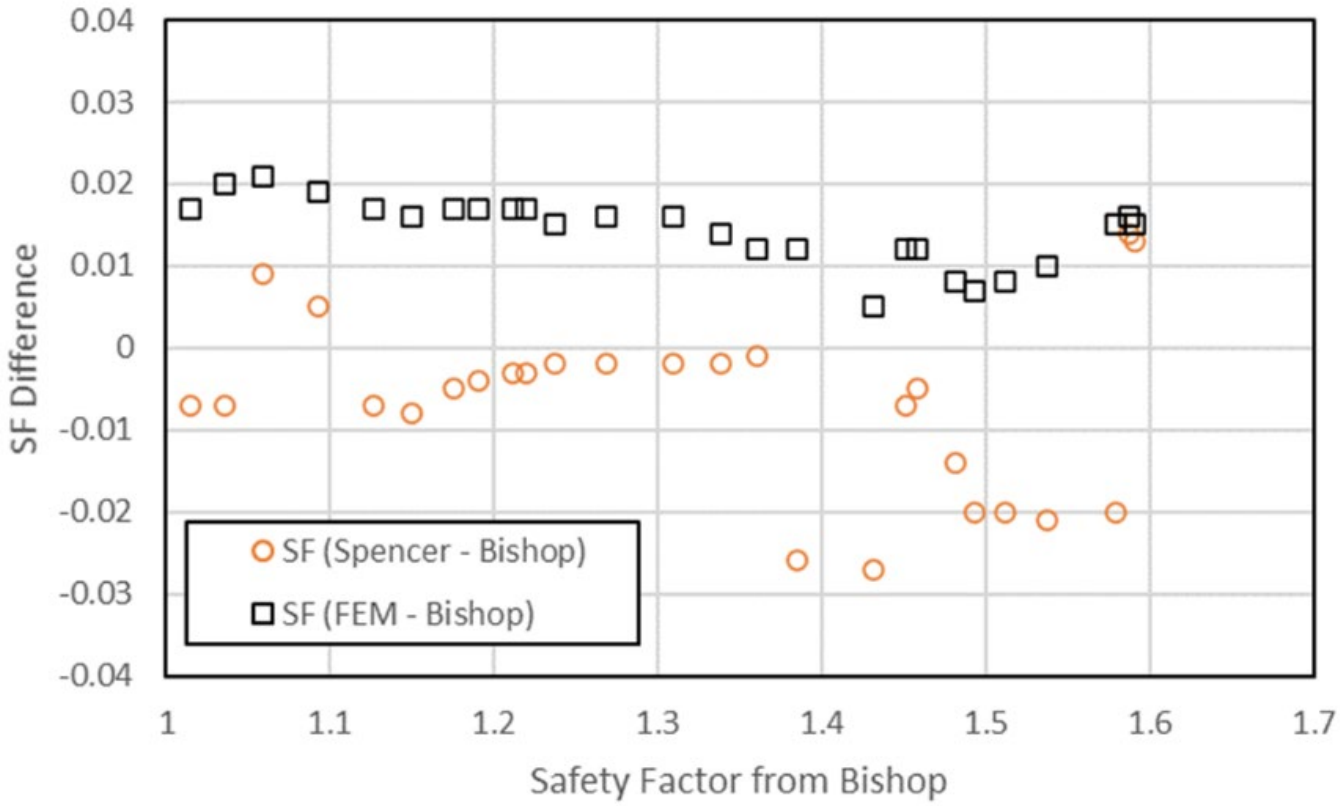


Tieback force and SF from two different steps in Stage 2 SRFEM analyses (Model B)

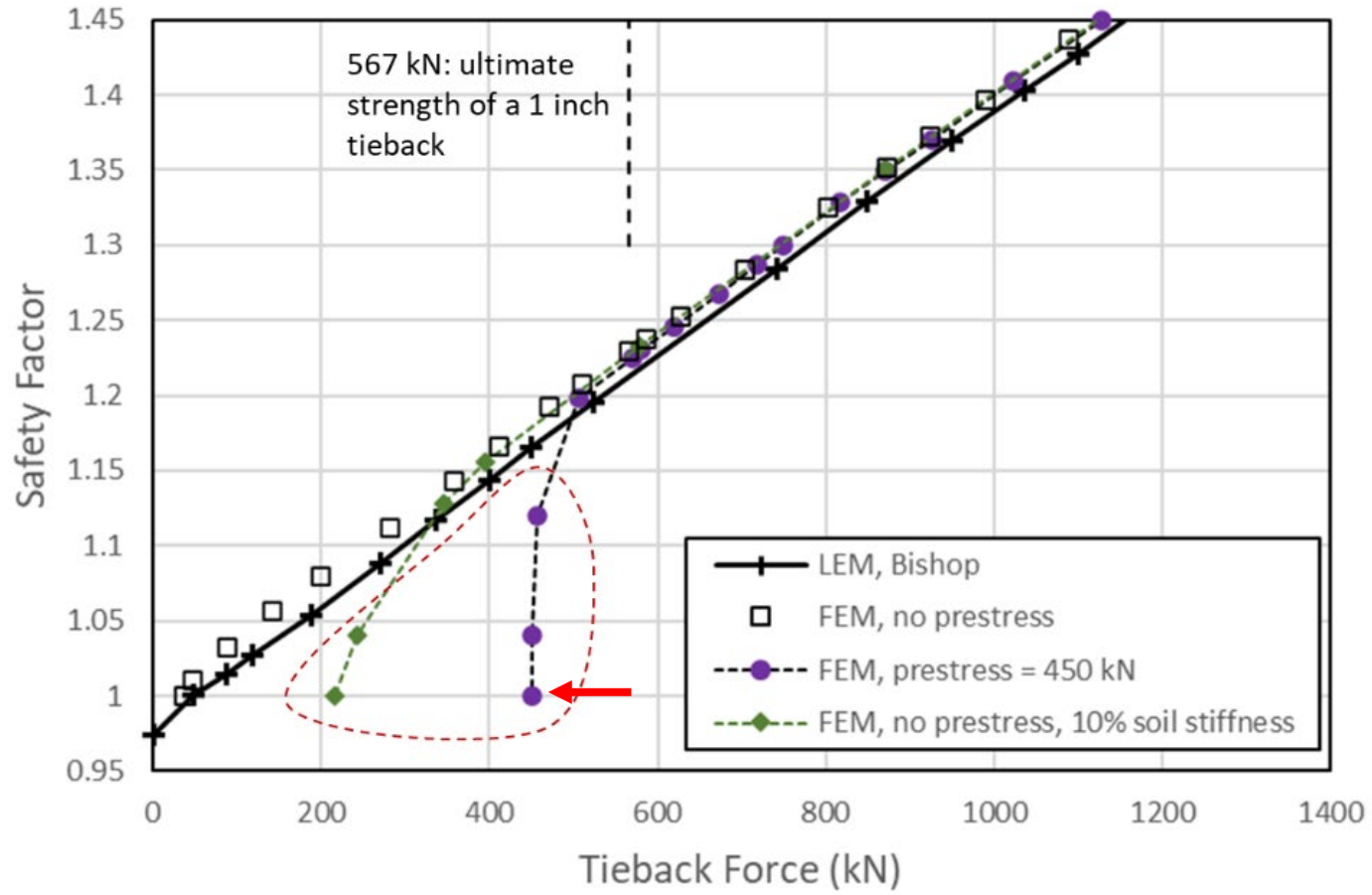




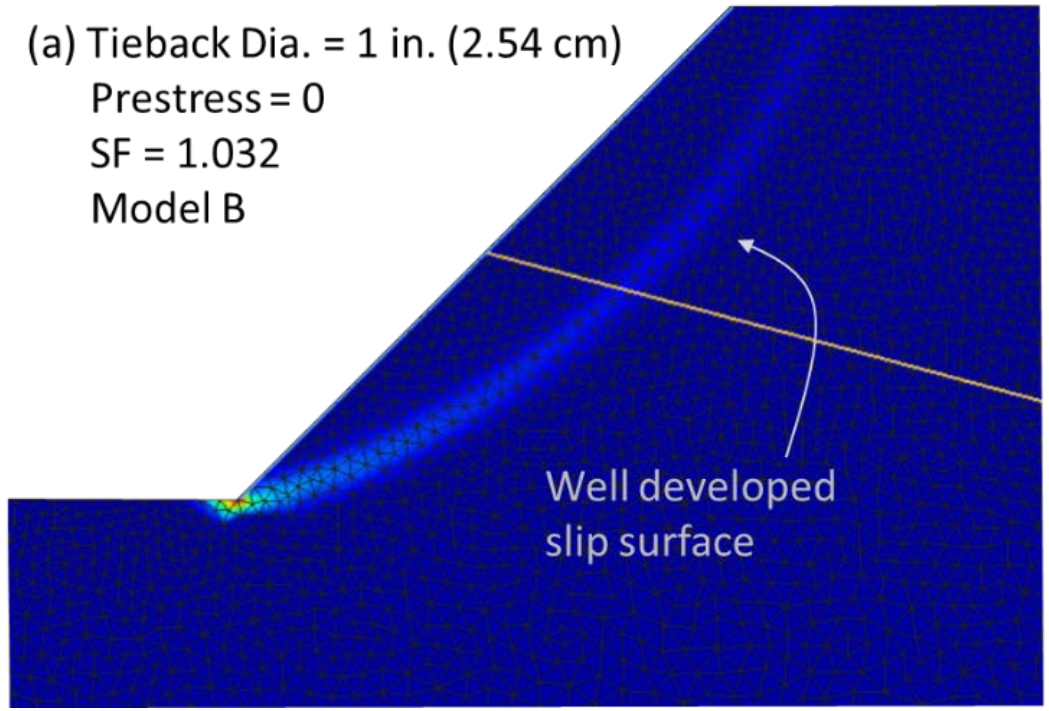
Safety factor difference between LEM and SRFEM from Model B

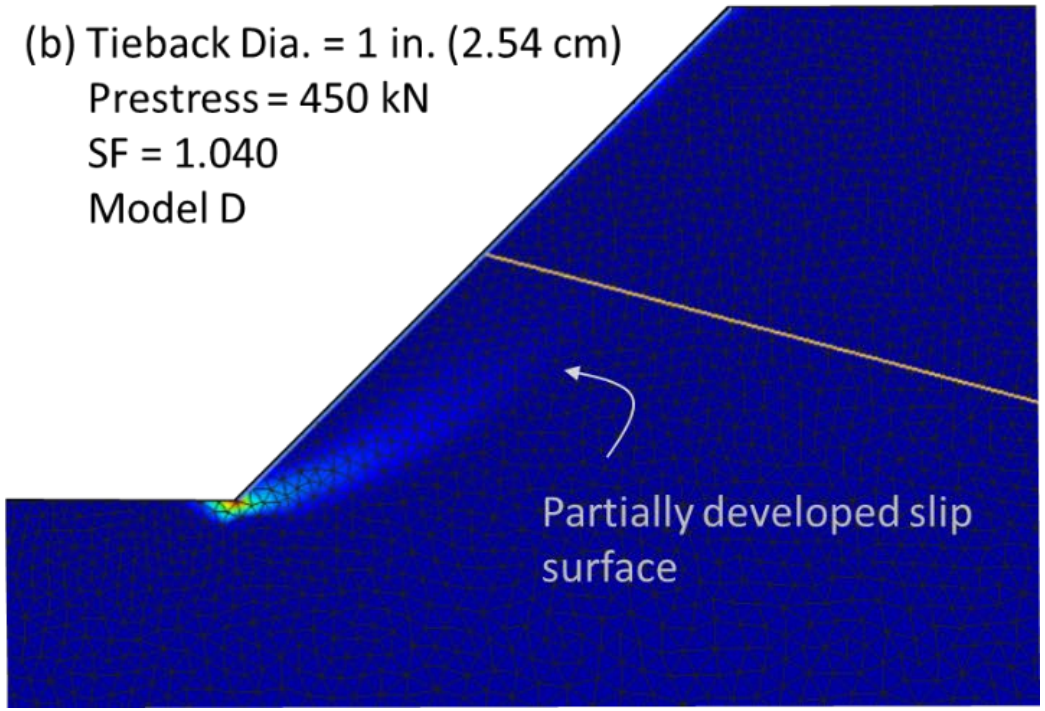


Effects of prestress and soil stiffness

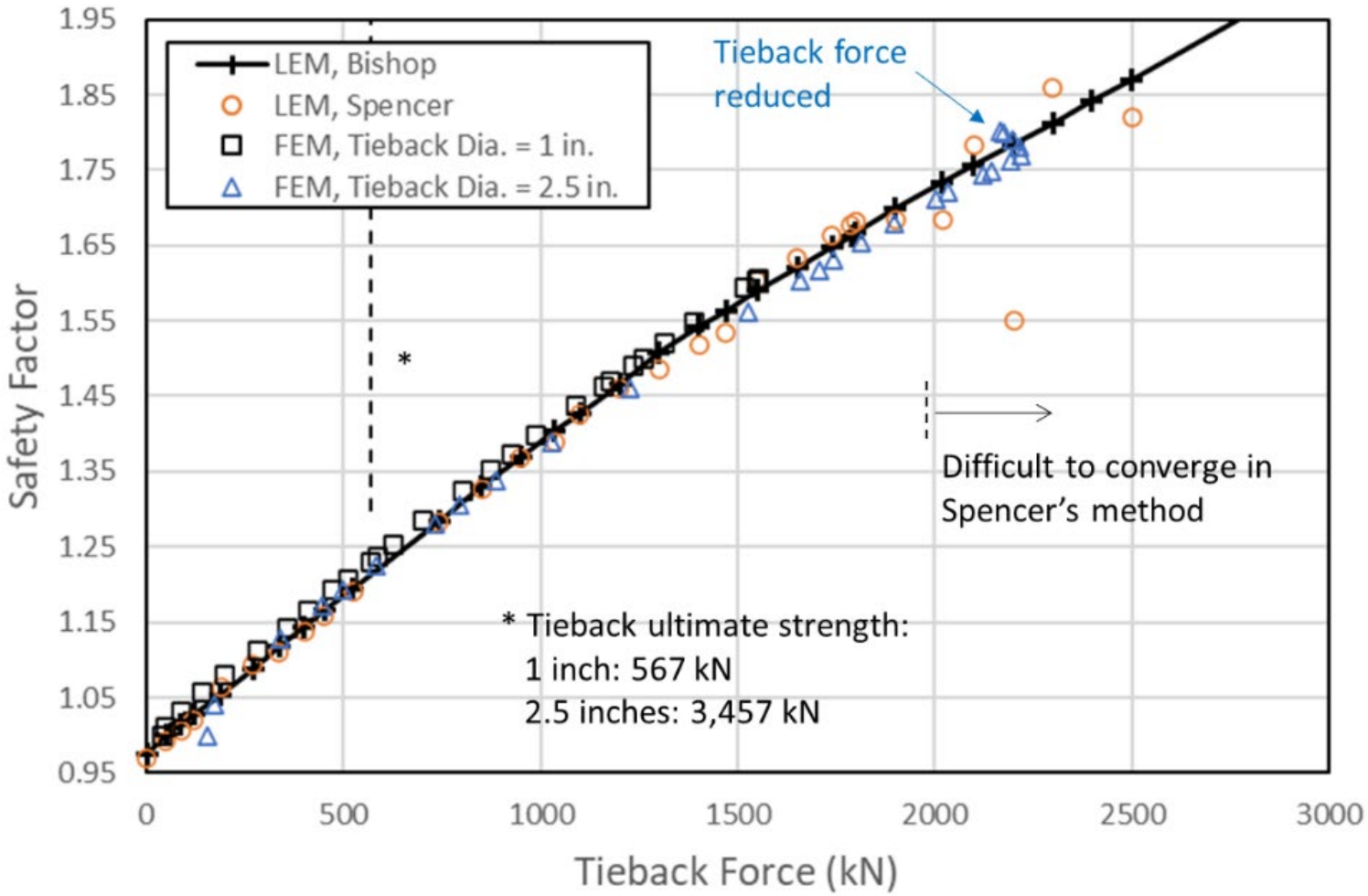


Equivalent magnitude of plastic strain from the 2nd step of stage 2 from Models B and D

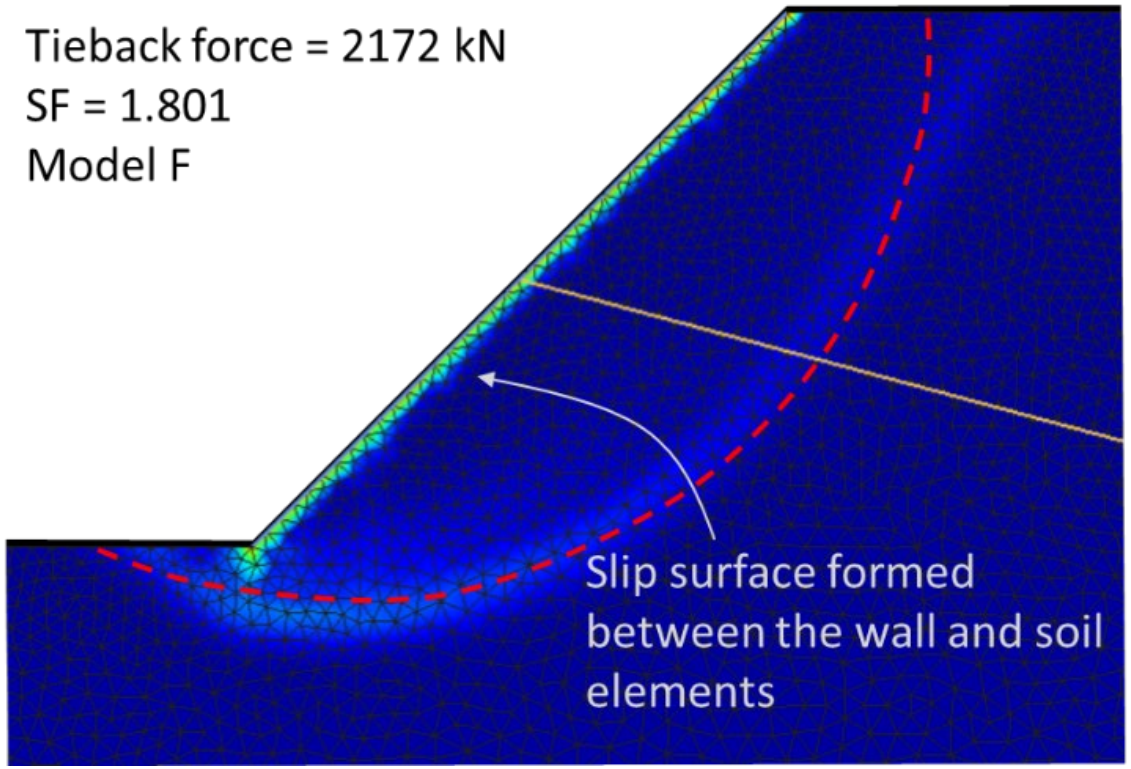




Effects of the size of the tieback diameter

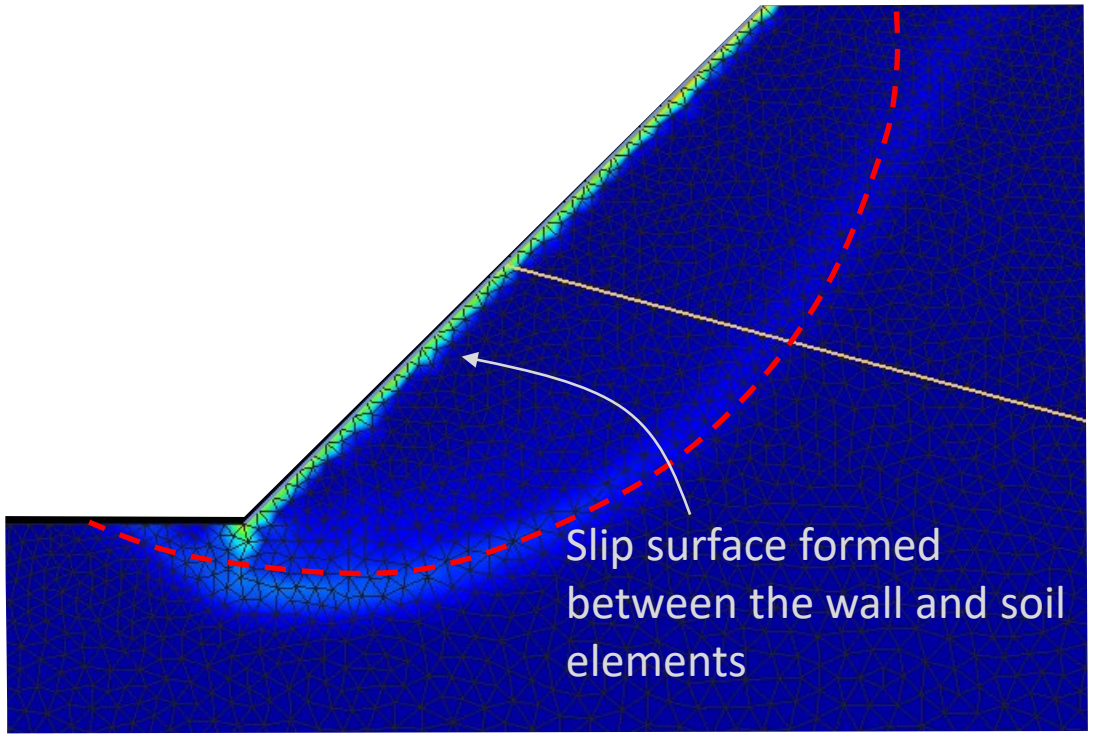


Failure mechanism at the last converged step of the SRFEM

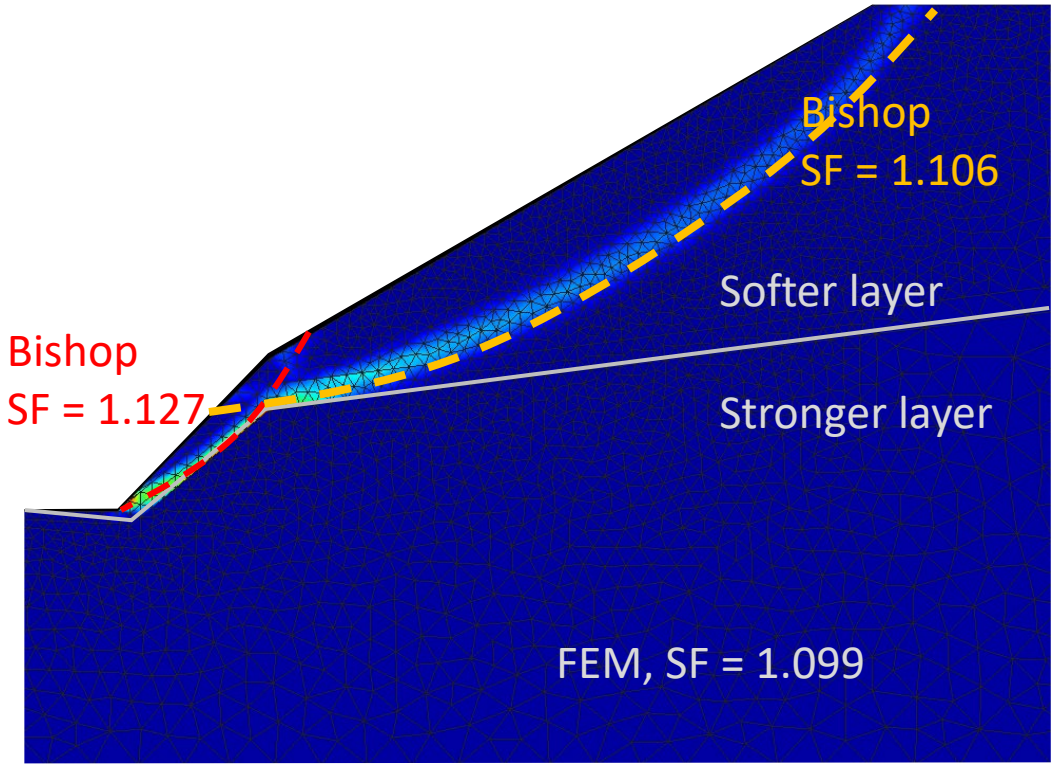


When to use FEM for slope stability analyses?

(1) When deformation is important



(2) Find the critical slip surface



Summary

- Safety factors obtained from SRFEM are close to those obtained from LEM using the Load Distribution Method when a slip surface is well developed in the soil mass.
- SRFEM shows there is a limit to applying the tieback force. For a slope with a 2.5-inch (6.35 cm) diameter steel bar, a slip surface between the wall and the soil mass developed at a tieback force of around 2,200 kN. The safety factor calculated from LEM with a tieback force higher than 2,200 kN may not be valid.
- Increasing the size of the tieback can have an influence on the resulting safety factor as more loads are carried by the tieback.

Thank you!



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THE UNIVERSITY OF QUEENSLAND
AUSTRALIA

Improving the Reliability of Pile Drivability Studies by Explicitly Simulating Ground Disturbance

Edward Smith
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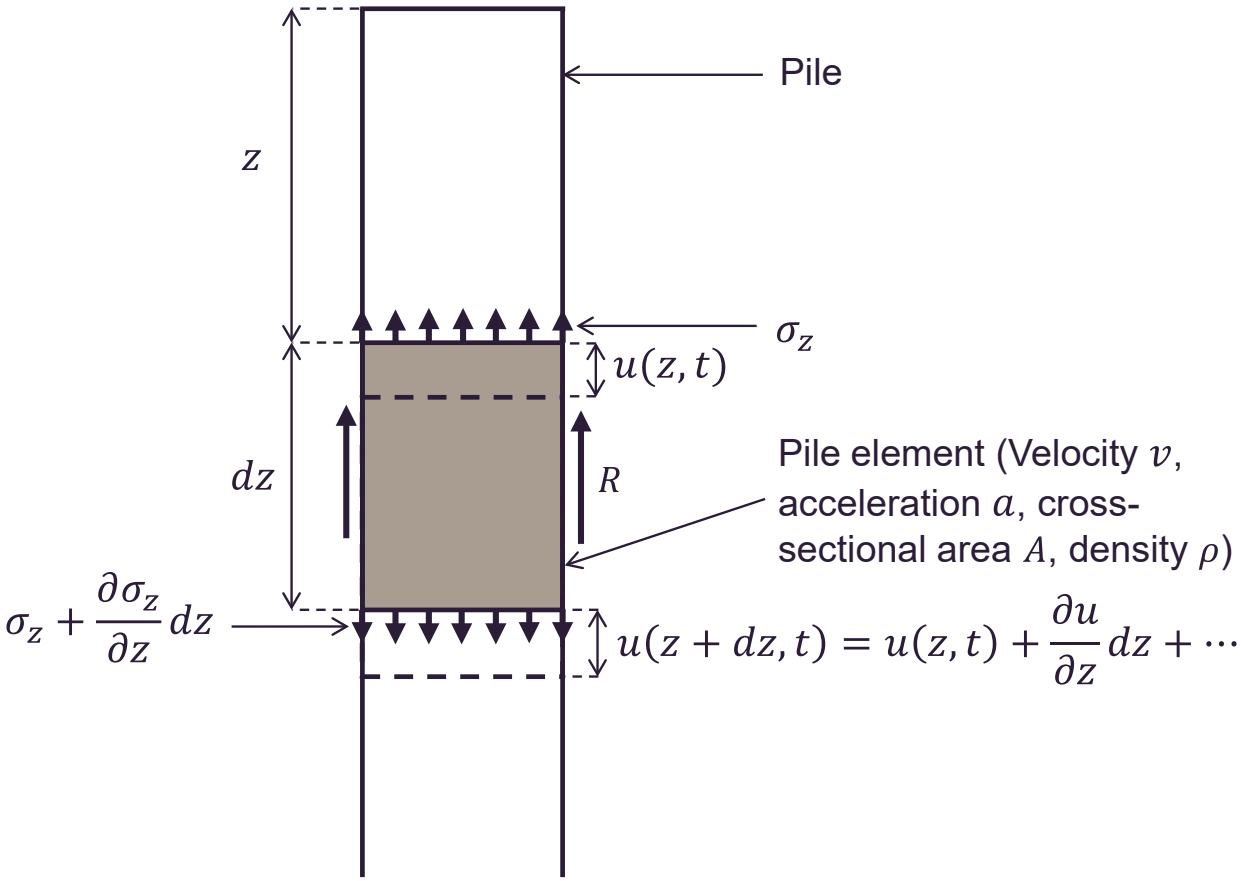
DFI50 – Deep Foundation's Institute 50th Anniversary Conference
Nashville, 20-23 October, 2025

Contents

01	Simulation of pile driving
02	Why is estimating ground resistance difficult?
03	What is current practice to capture this complexity?
04	What is the overall modelling approach being proposed?
05	How is ground stiffness and resistance simulated?
06	How is ground disturbance simulated?
07	Application of the modelling approach.
08	Opportunities for further development.

Simulating pile driving

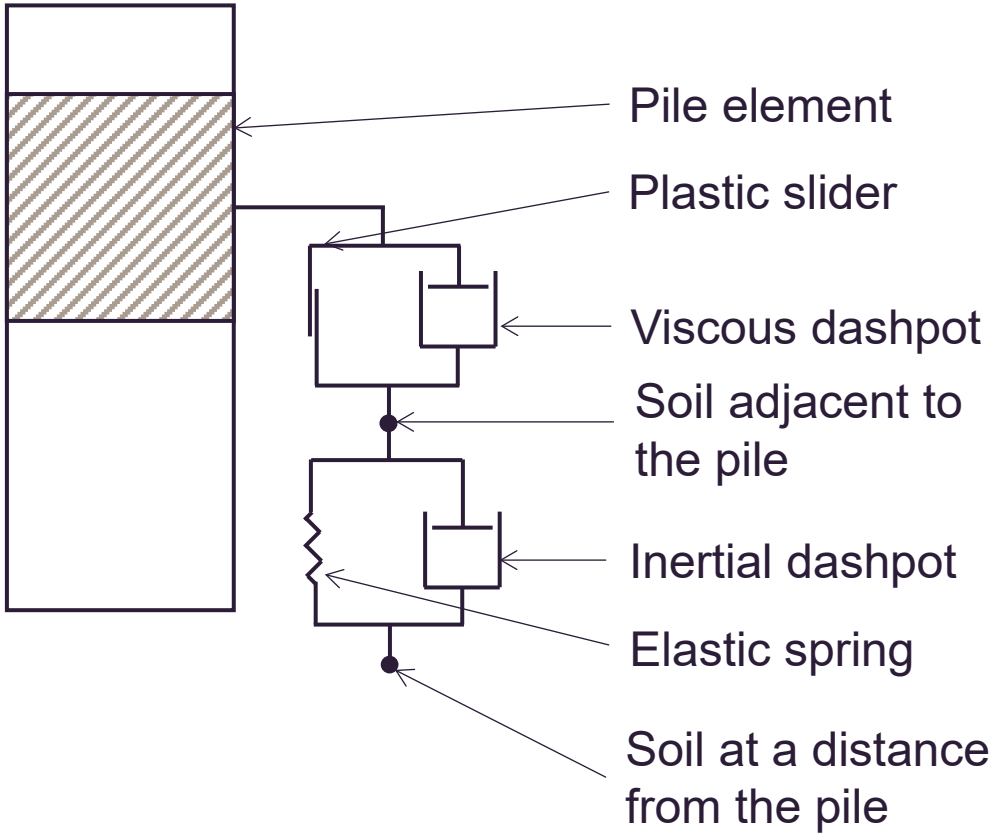


- One dimensional wave equation

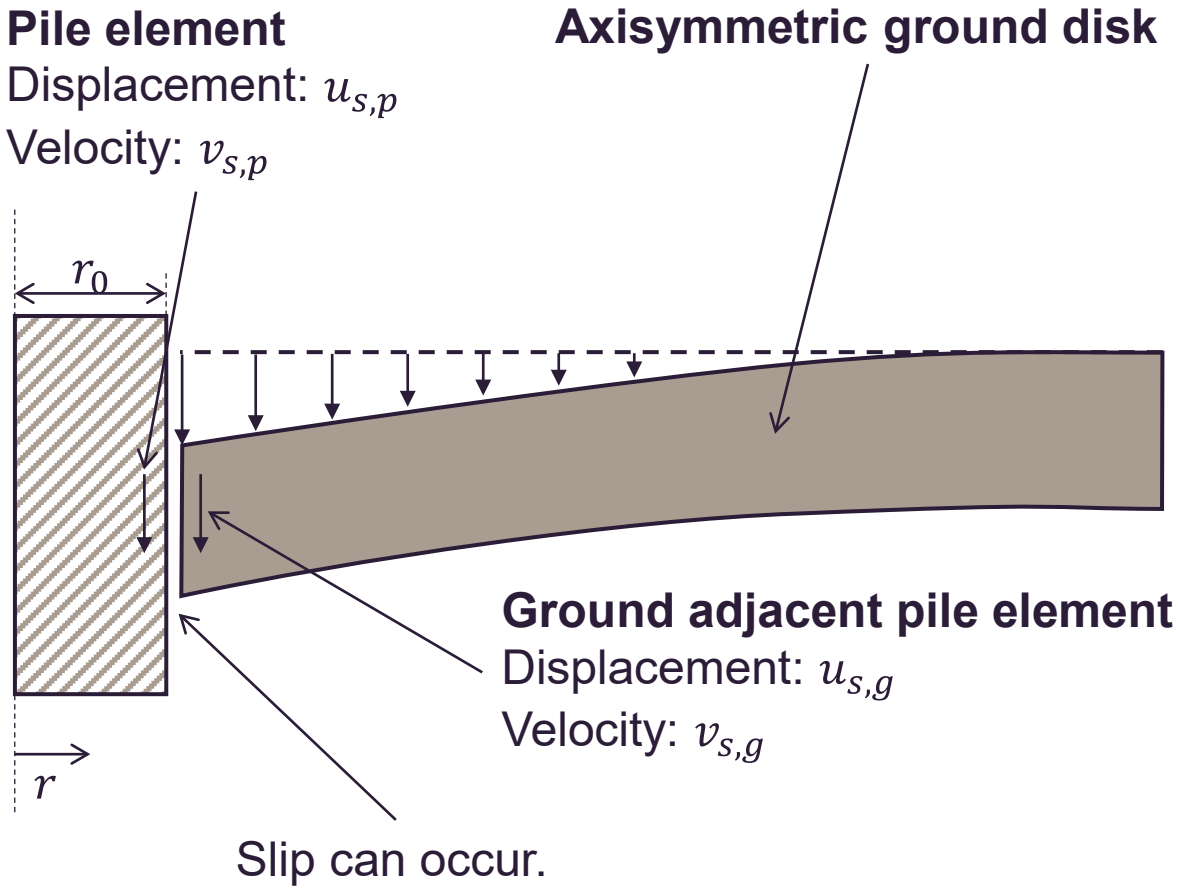
$$c^2 \frac{\partial^2 u}{\partial z^2} - \frac{R}{A_p \rho_p} = \frac{\partial^2 u}{\partial t^2}$$

- Initially applied to pile driving problems by Smith (1960) with the ground resistance simulated using elastic-plastic springs and viscous dashpots
- Subsequent advancements have reduced some of the empirisism in the way that the ground resistance is simulated

Simulating pile driving



Continuum model – Simons and Randolph (1984)



Difficulties in estimating ground resistance

Cohesive Soils

- Excess porewater pressures
- Soil structure alteration
- Shaft failure mode and location

Foundation system and installation

- Pile material, size, end condition
- Installation method – Driving, jacking, vibrating
- Pile whipping

Granular Soils

- Excess porewater pressures
- Relaxation of arching stresses
- Soil structure alteration
- Enhanced constrained dilation

Other considerations

- Axial cyclic loading
- Environmental variability
- Groundwater level

Current practice to estimate installation and time-dependent effects

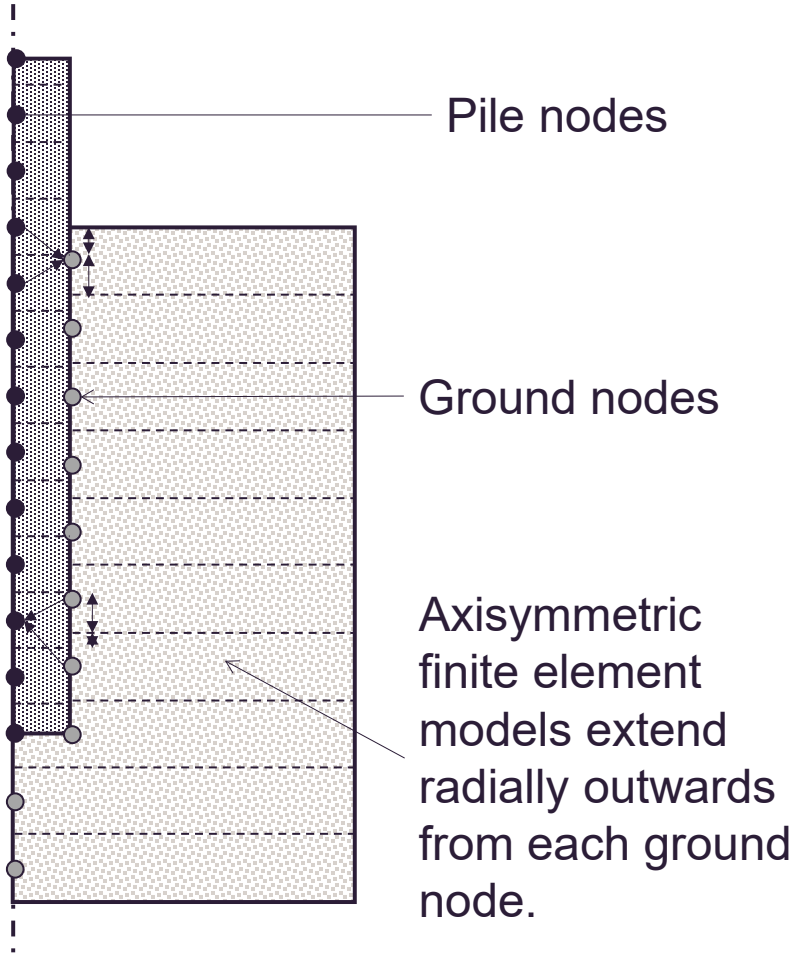
- **Alm and Hamre (2001)**

$$f_s(p) = f_{s,res} + \left(f_{s,i} - f_{s,res}\right)e^{\boldsymbol{k}(d-p)}$$

- **Skov and Denver (1988)**

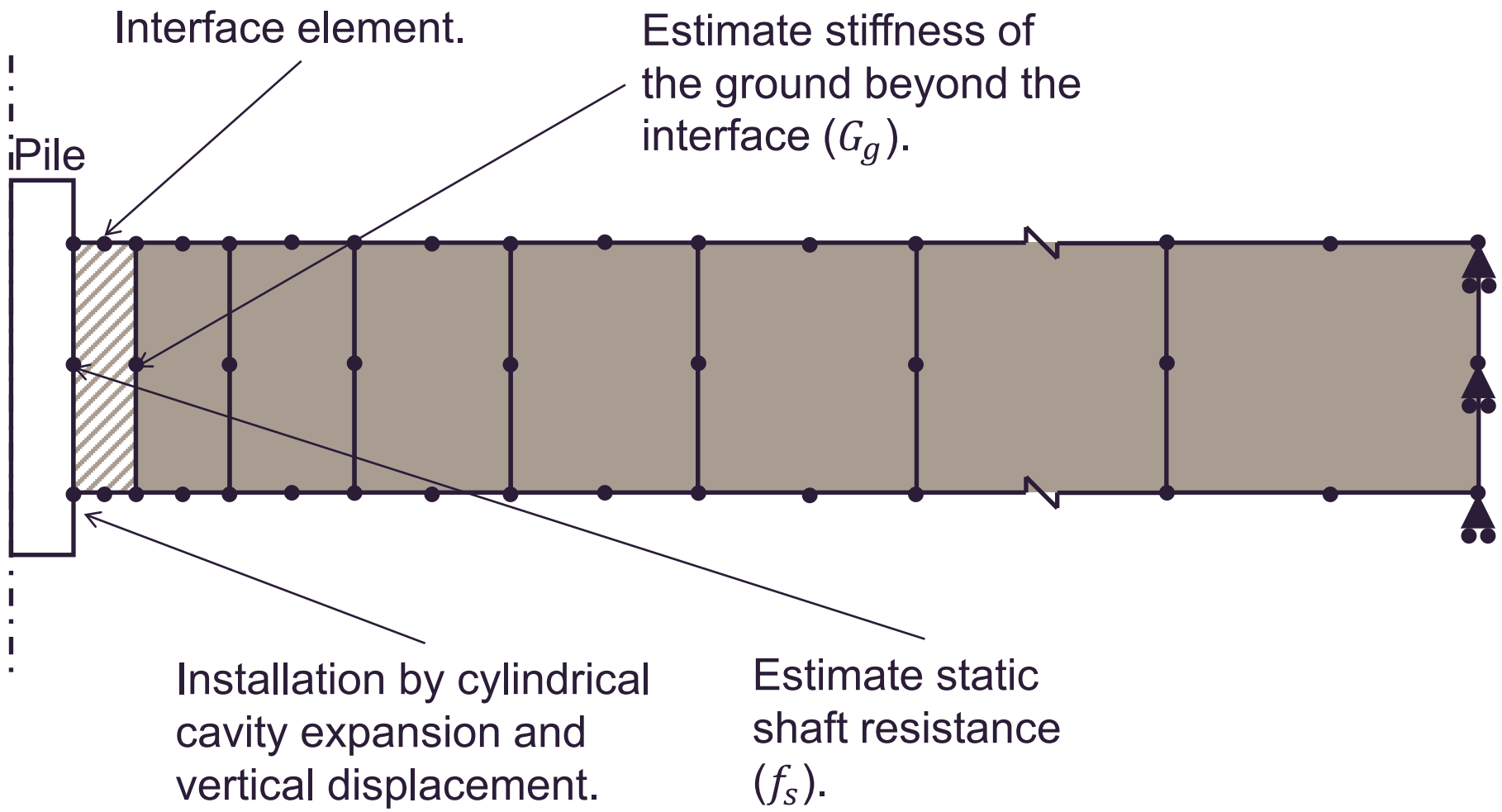
$$f_s(t) = f_{s,t_0}\left(\boldsymbol{A}\log_{10}\left(\frac{t}{\boldsymbol{t_0}}\right) + 1\right)$$

Proposed modelling approach

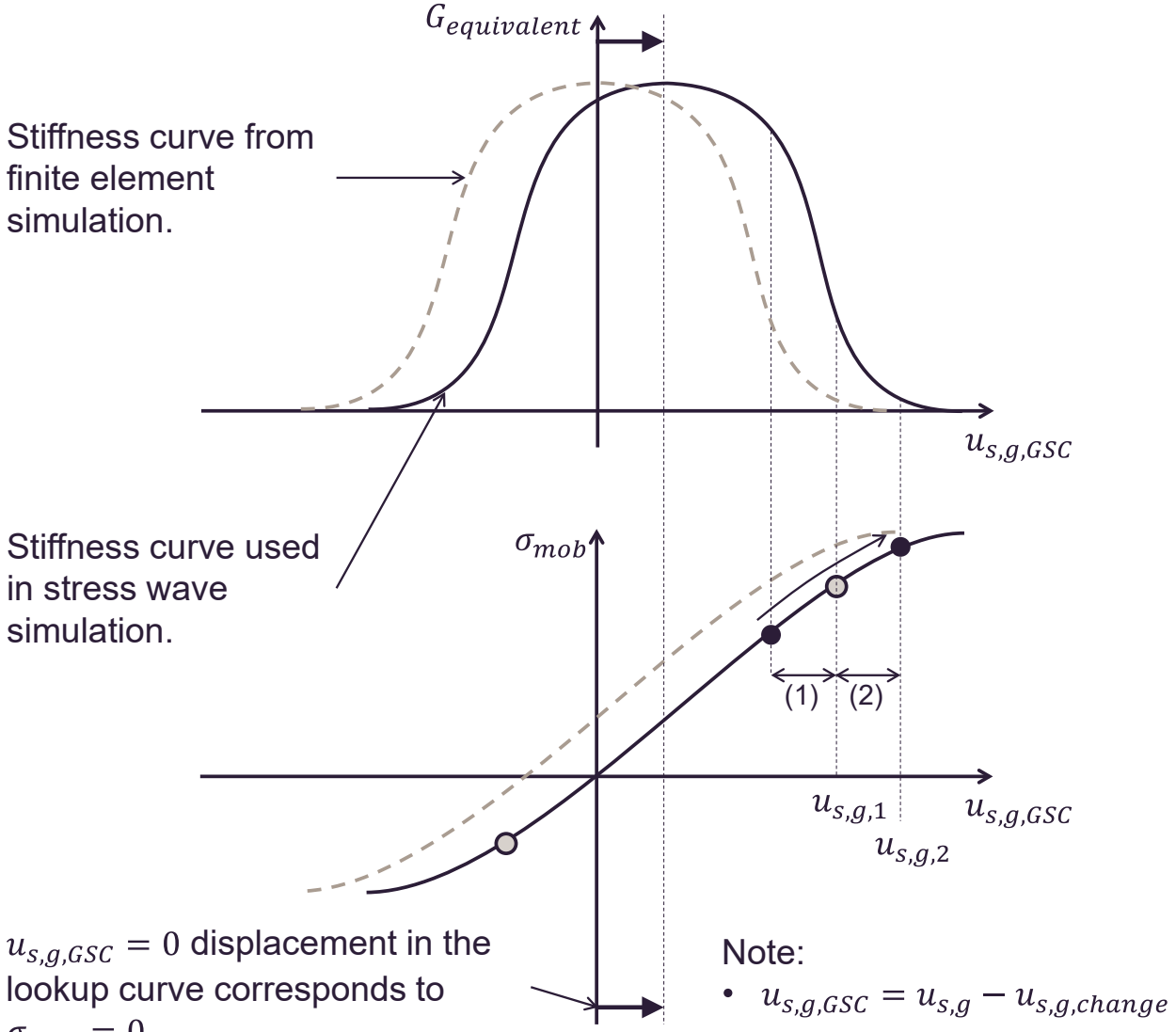


- **Stress wave theory:** Interaction between input energy, pile, and ground.
- **Ground nodes:** Allow ground disturbance to be tracked throughout the entire driving sequence.
- **Independent axisymmetric finite element models**: Estimate ground stiffness/resistance, simulate ground disturbance.
- **During stress wave simulation:** Information transfer from pile nodes to ground nodes, and vice versa to estimate ground stiffness/resistance.
- **After stress wave simulation:** Apply disturbance to ground node models.
- **Installation/Temporal effects:** By using appropriate constitutive model in the ground node finite element model.

Estimating ground stiffness and resistance



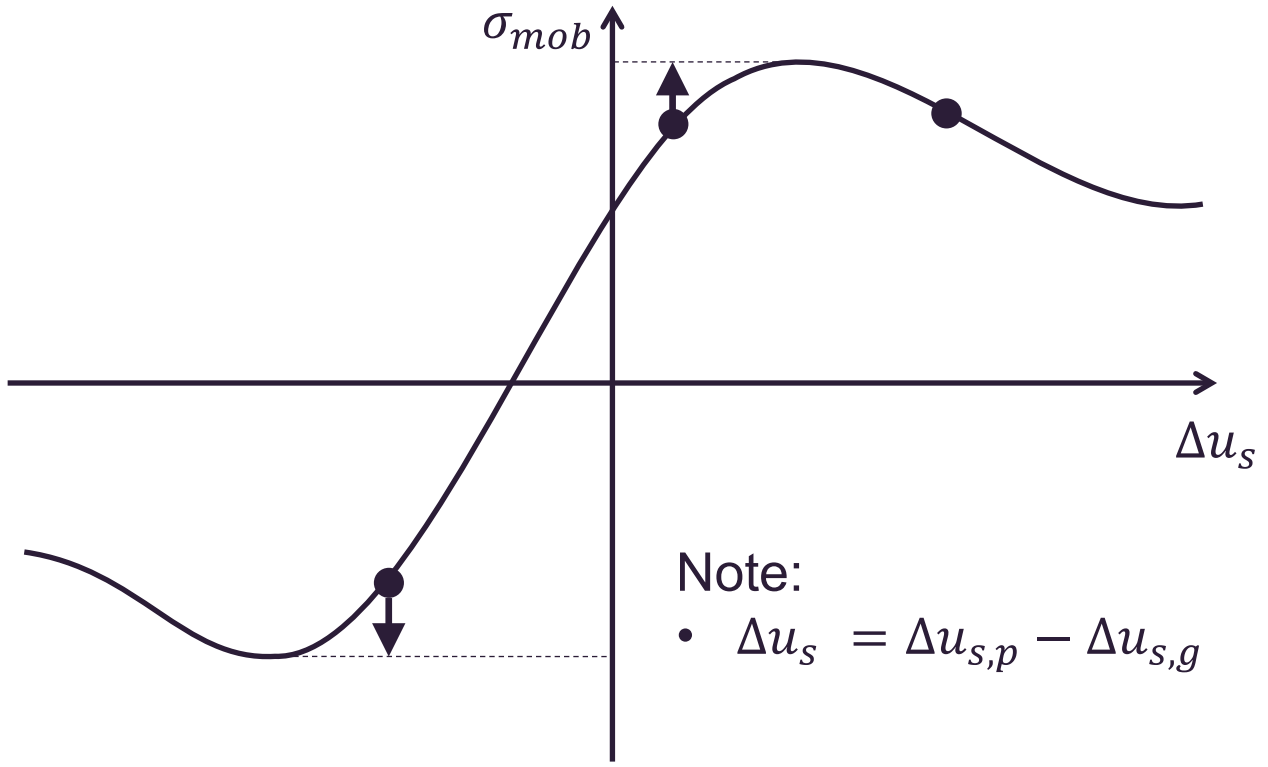
Estimating ground stiffness



Note:

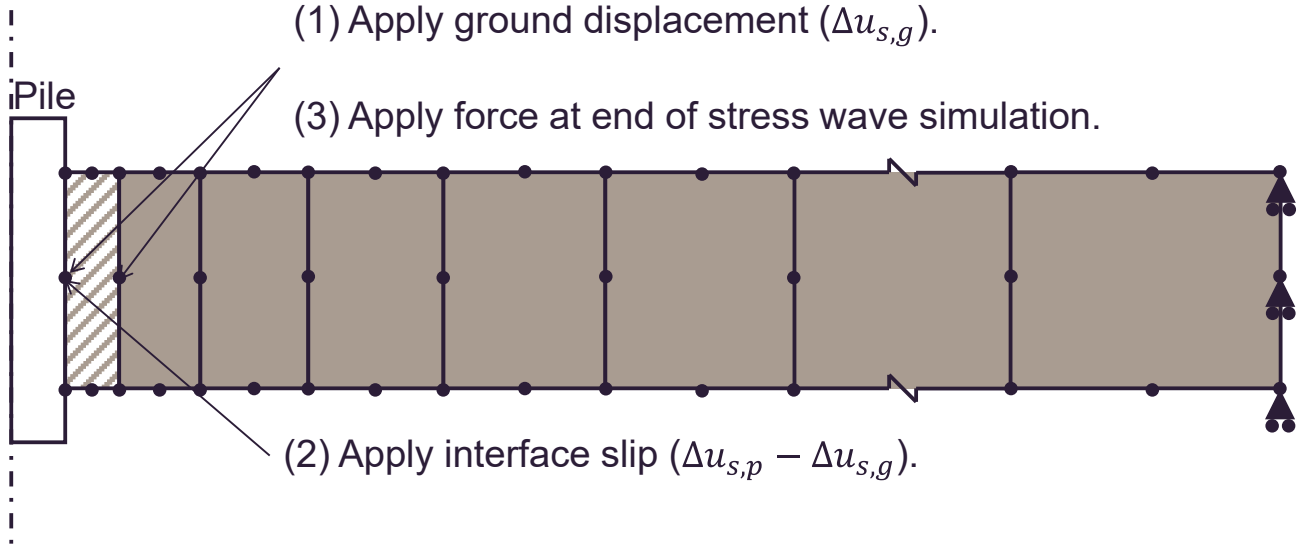
- $u_{s,g,GSC} = u_{s,g} - u_{s,g,change}$

Estimating ground resistance



Simulating ground disturbance

- Apply the **maximum displacement from the outside of the interface** (i.e., away from pile, $\Delta u_{s,g}$) to both sides of the interface (i.e., no slip).
- **Apply the maximum interface slip** (i.e., $\Delta u_{s,p} - \Delta u_{s,g}$) to the pile side of the interface with vertical movement of the other side (i.e., away from pile) of the interface fixed.
- Ensure that the **load on the finite element model matches that at the end of the stress wave simulation**.
- Simulate porewater pressure dissipation?

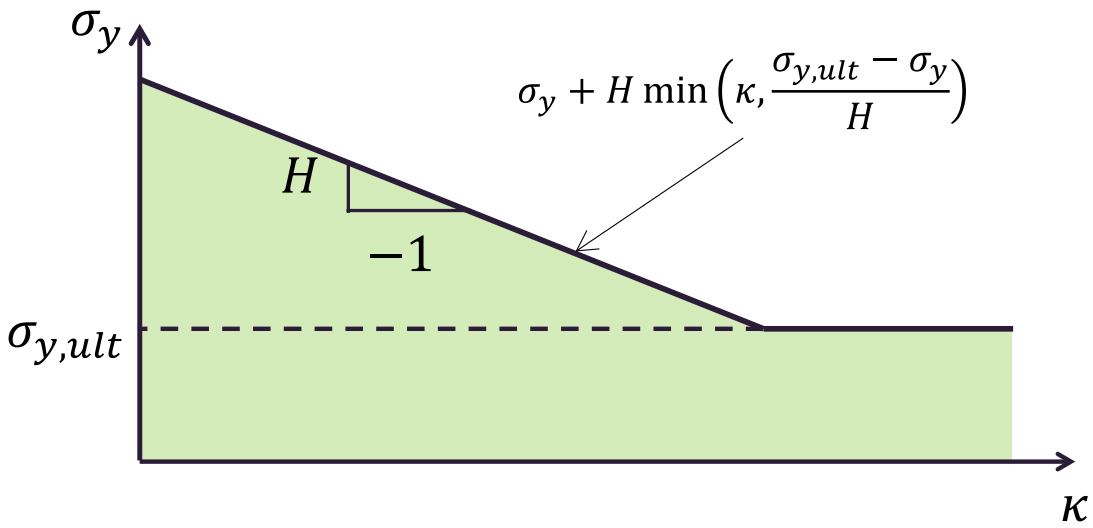


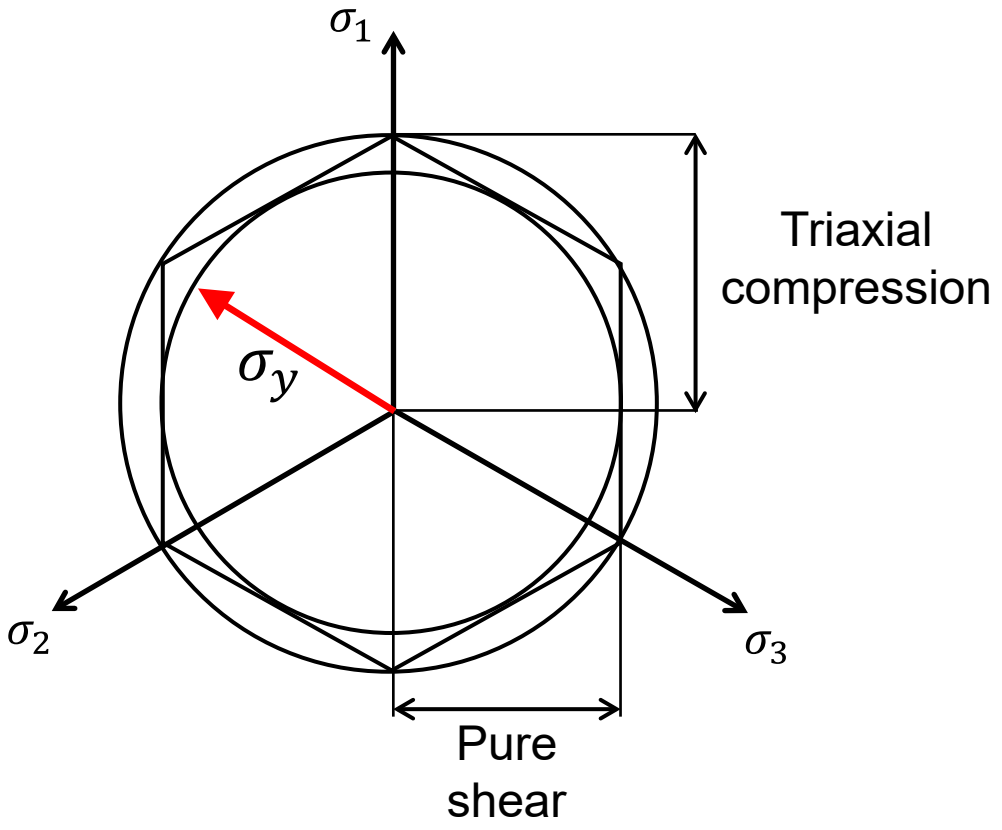
Application of the modelling approach – Conventional continuum model

$$f = \sqrt{3J_2} - \left(\sigma_y + H \min\left(\kappa, \frac{\sigma_{y,ult} - \sigma_y}{H}\right)\right)$$

$$\kappa = \int d\kappa = \int \sqrt{\frac{2}{3} d\epsilon^p : d\epsilon^p}$$

$$\sigma_y = \sqrt{3} f_s$$





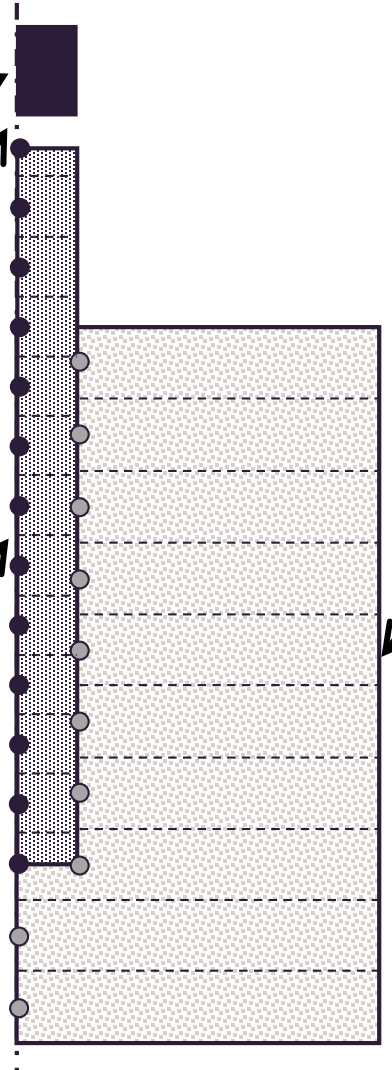
Failure surface in deviatoric plane

Application of the modelling approach – Conventional continuum model

Hammer
Drop height: 1 m
Mass: 3000 kg
Efficiency: 100%
E: 210 GPa

Cushion
Thickness: 0.1 m
E: 0.5 GPa

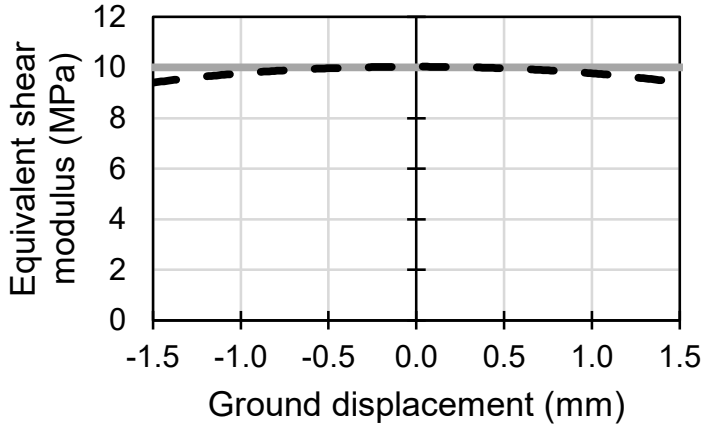
Pile
Length: 10 m
Embedment: 9 m
Diameter: 0.3 m
E: 40 GPa
No. elements: 40

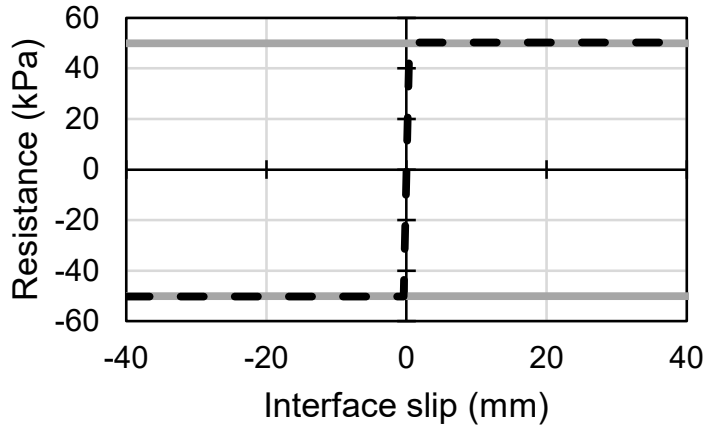


Ground
E: 30 MPa
Shaft resistance: 50 kPa
End bearing resistance: 900 kPa
No. ground nodes: 40

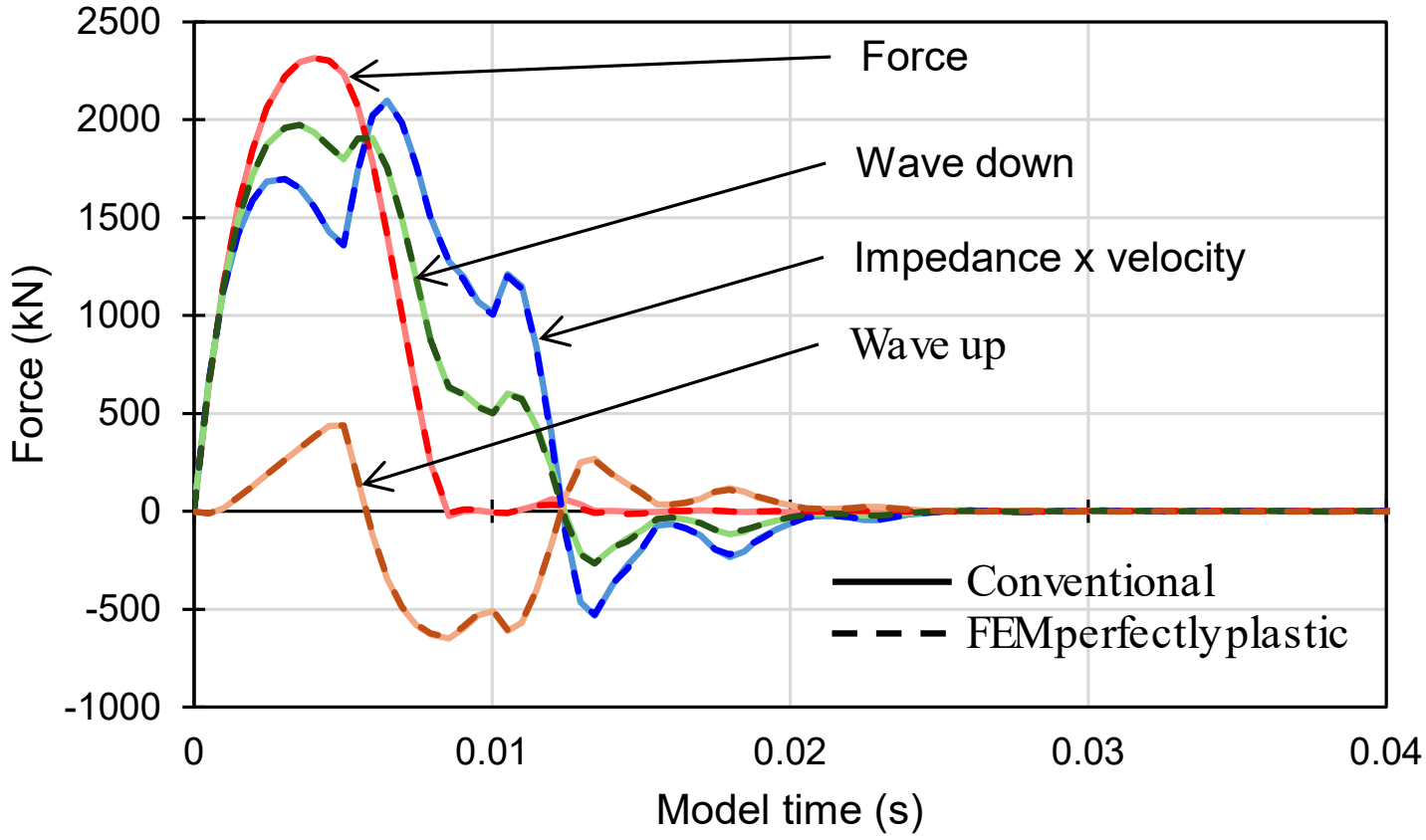
Axisymmetric finite element models
Width: 4 m
No. elements: 15

Application of the modelling approach – Conventional continuum model

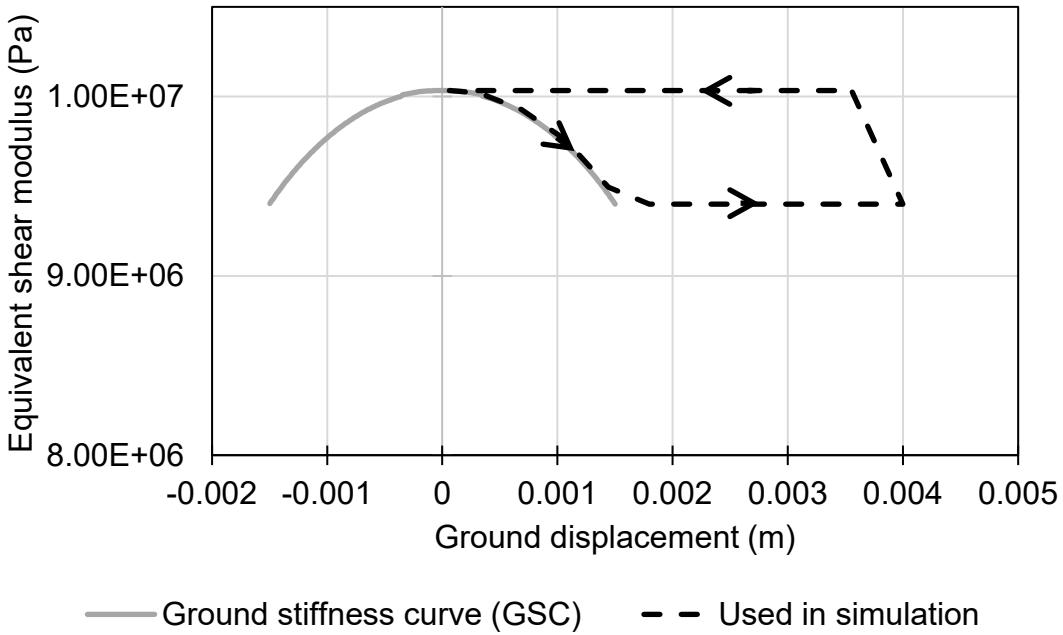


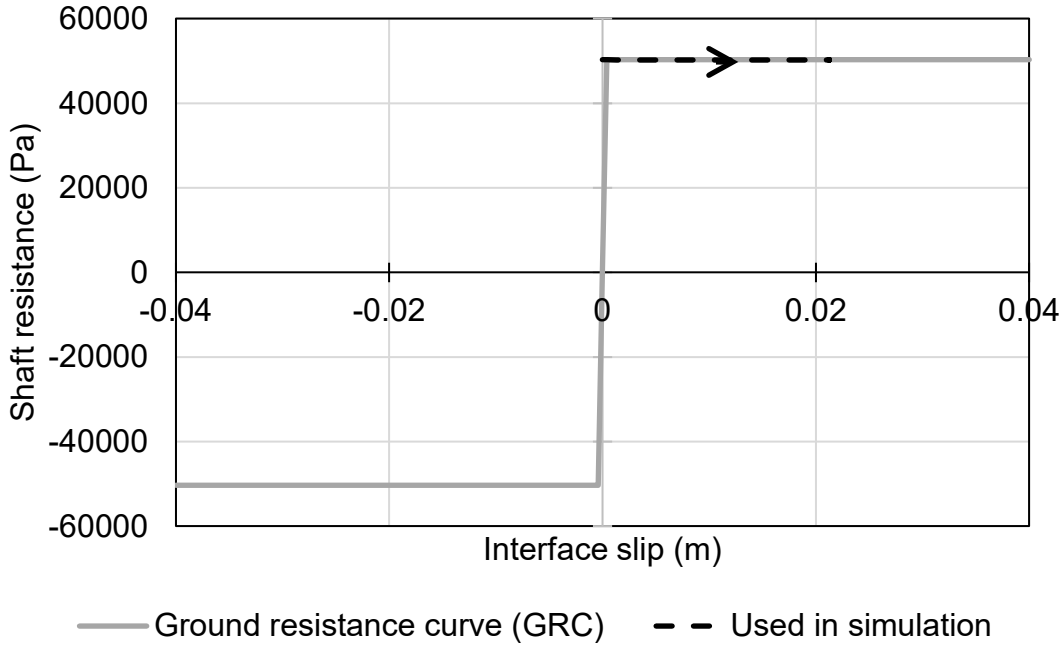


Conventional FEM perfectly plastic



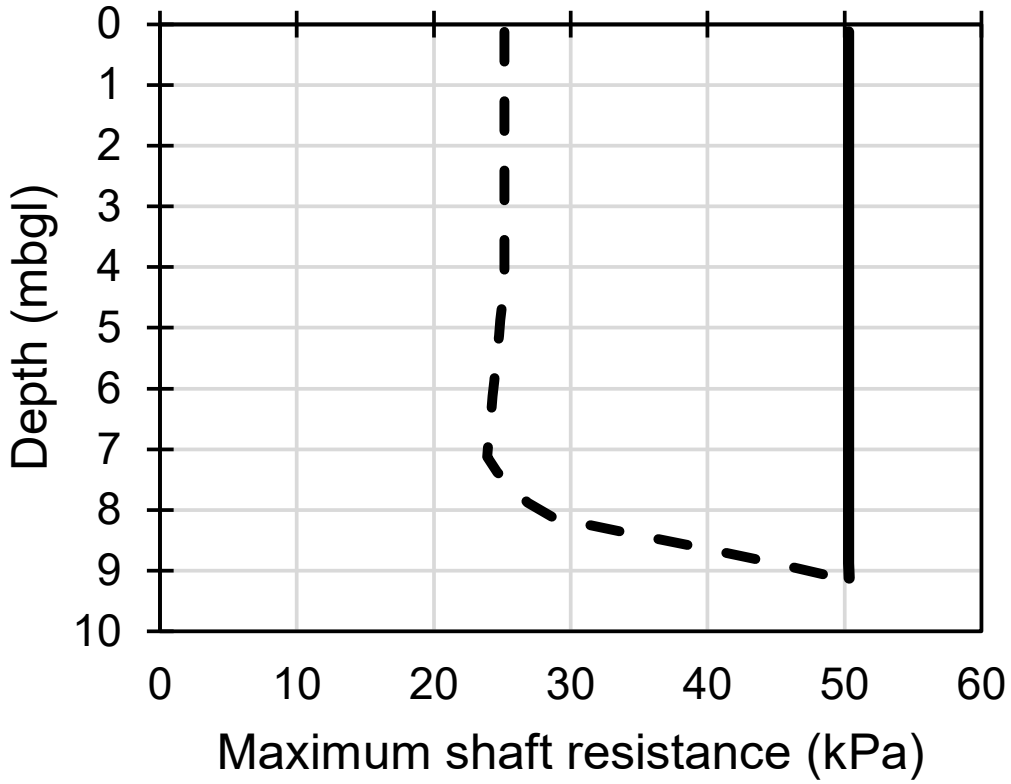
Application of the modelling approach – Conventional continuum model

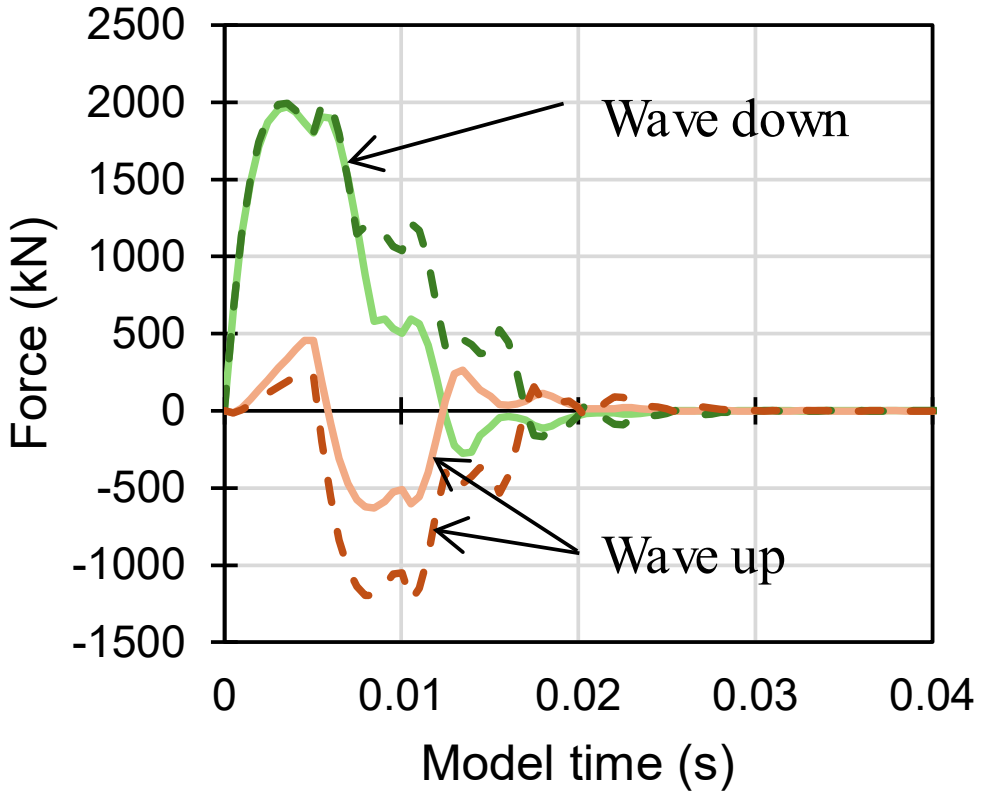




Application of the modelling approach – Simulation of installation effects

Softening: H = -0.5 kPa, $f_{s,ult}$ = 25 kPa





FEM perfectly plastic ~220 hammer blows

FEM softening ~130 hammer blows

Further development opportunities

Opportunities

- More **advanced constitutive model** able to simulate the mechanisms often associated with installation and temporal effects.
- Use of the **strain path method** (Baligh, 1985) account for the two-dimensional nature of pile installation.
- **Interface coefficient** which reduces the shaft resistance that can be mobilized at the pile-soil interface.
- Simulation of **waiting periods** between pile installation and testing to allow temporal effects such as excess pore pressure dissipation and creep.

To establish

- Confirm that **static simulations** capture disturbance from dynamic loading.
- Ground node finite element models can continue predictions of **ground response at large deformations**.
- How to determine **ground node finite element dimensions**.

Conclusions

- Pile installation and temporal effects
  - Caused by the complex interaction of several mechanisms.
  - Highly variable and difficult to predict.
- Current approach - establish the values of empirical coefficients
- Proposed modeling approach
  - Creates the opportunity to **explicitly simulate the effect of pile installation on the ground.**
  - Parameter estimation transformed into an exercise based on **laboratory testing and more clearly defined soil mechanics principles**.

Acknowledgements

Prof. David Williams, Dr Chenming Zhang
Geotechnical Engineering Centre, The University of Queensland

FSG Geotechnics and Foundations

Australian Government – Department of Education
RTP Stipend, National Industry PhD Program

Questions?

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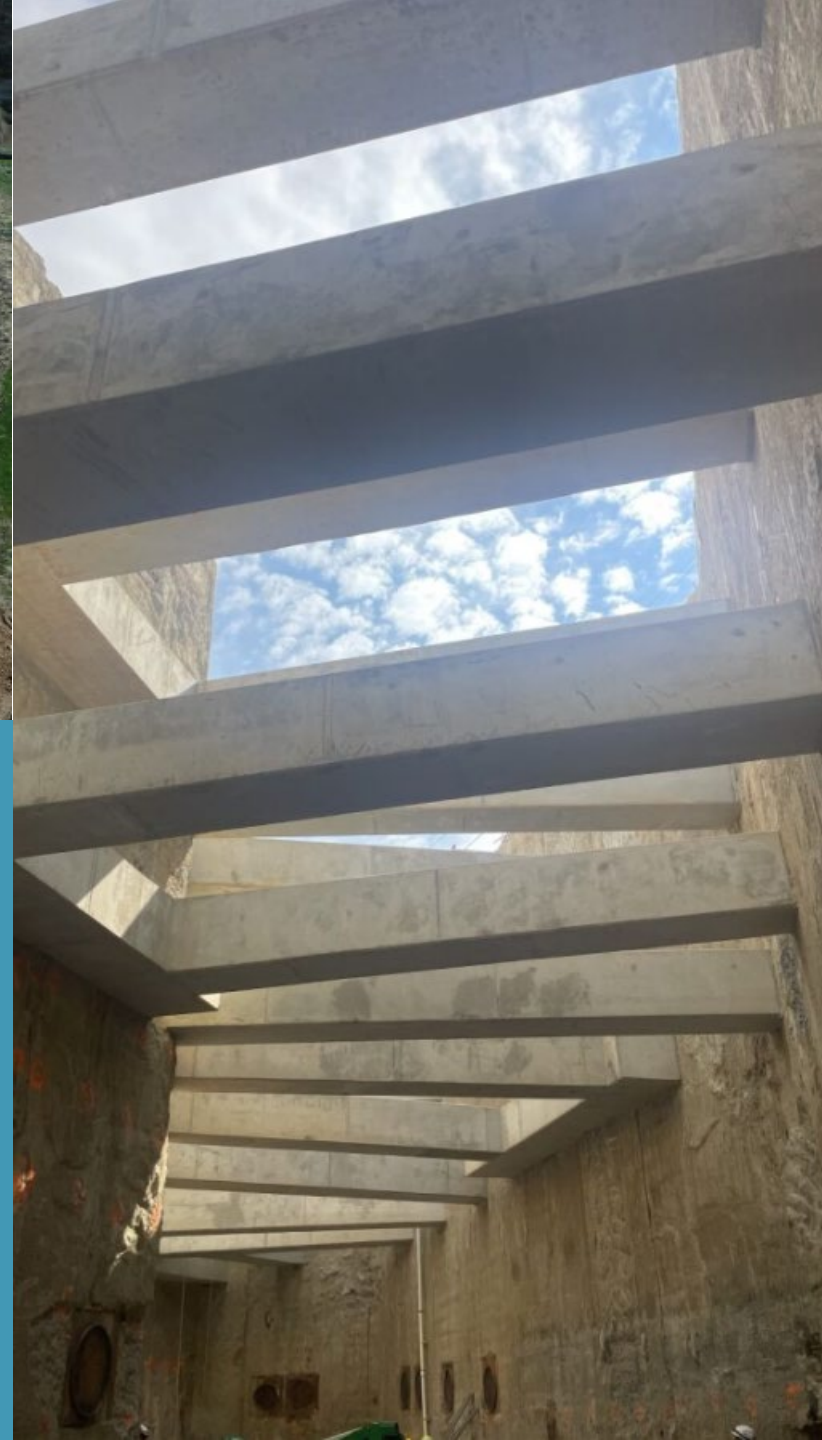

Modern View of In-Situ Wall Prediction and Movement

Enrique Farfan, PE, PhD

Adam Hacker, PE

Tom Pace, PE, PhD

October 22, 2025

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Agenda

1. Introduction
2. Design Solution
3. Idealized Soil Profile
4. The Pump Station Structure
5. From 3D to 2D and Back
6. Influent Channel Analysis
7. Monitoring Movements
8. Reality vs Design

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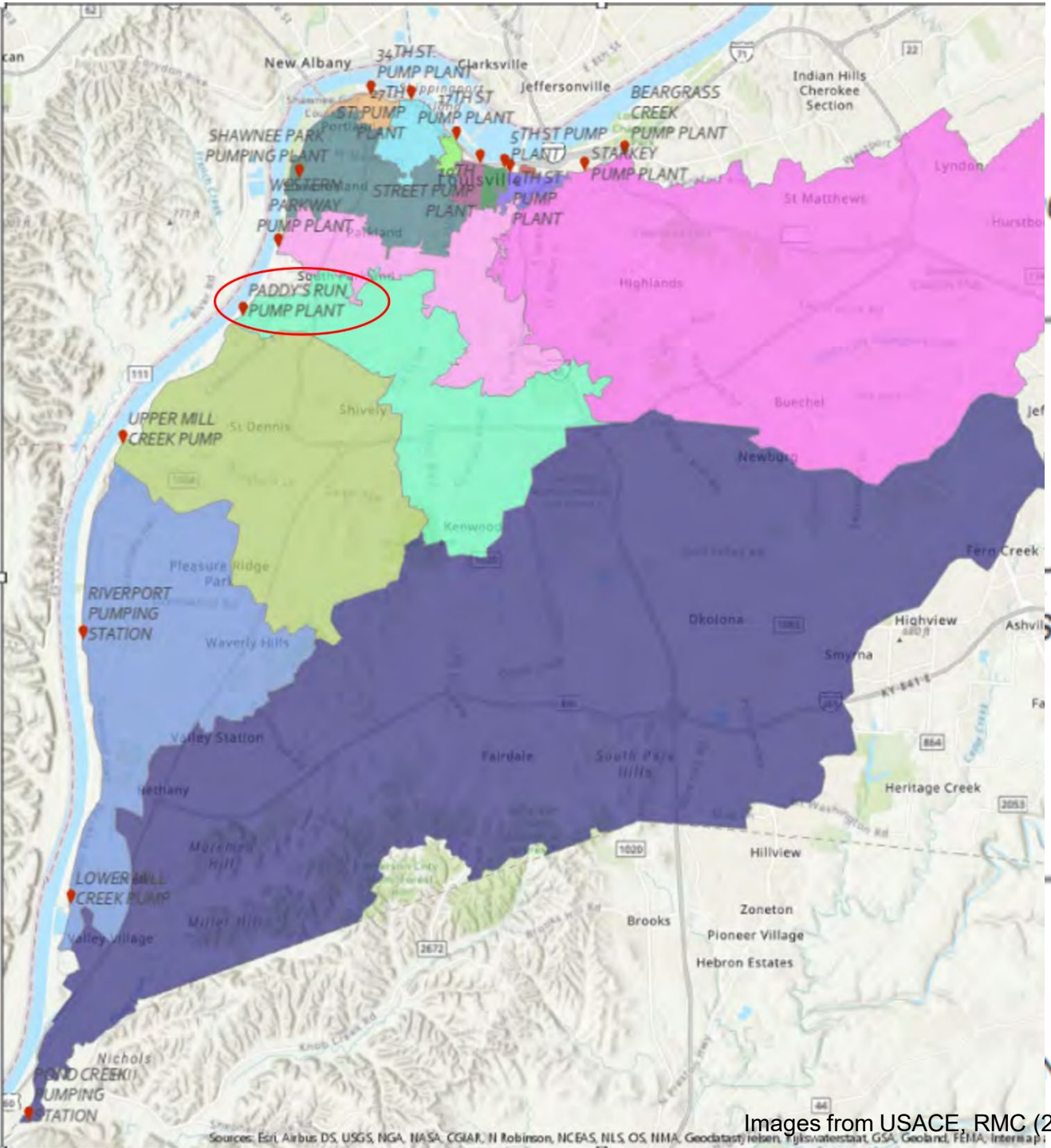
01 INTRODUCTION





Images from USACE, RMC (2020), Semi-Quantitative Risk Assessment for the Louisville Metro Levee System, Louisville, KY



Images from USACE, RMC (2020), Semi-Quantitative Risk Assessment for the Louisville Metro Levee System, Louisville, KY

- Flood Control Act – June 1938
- Sponsor – Louisville and Jefferson County Metropolitan Sewer District (MSD)
- Built by USACE
- Levee System Total Length 25.9 miles
  - 19.7 miles of earth levee
  - 3.92 miles of concrete wall
  - 16 pump stations

Image from Weather.gov (2024)



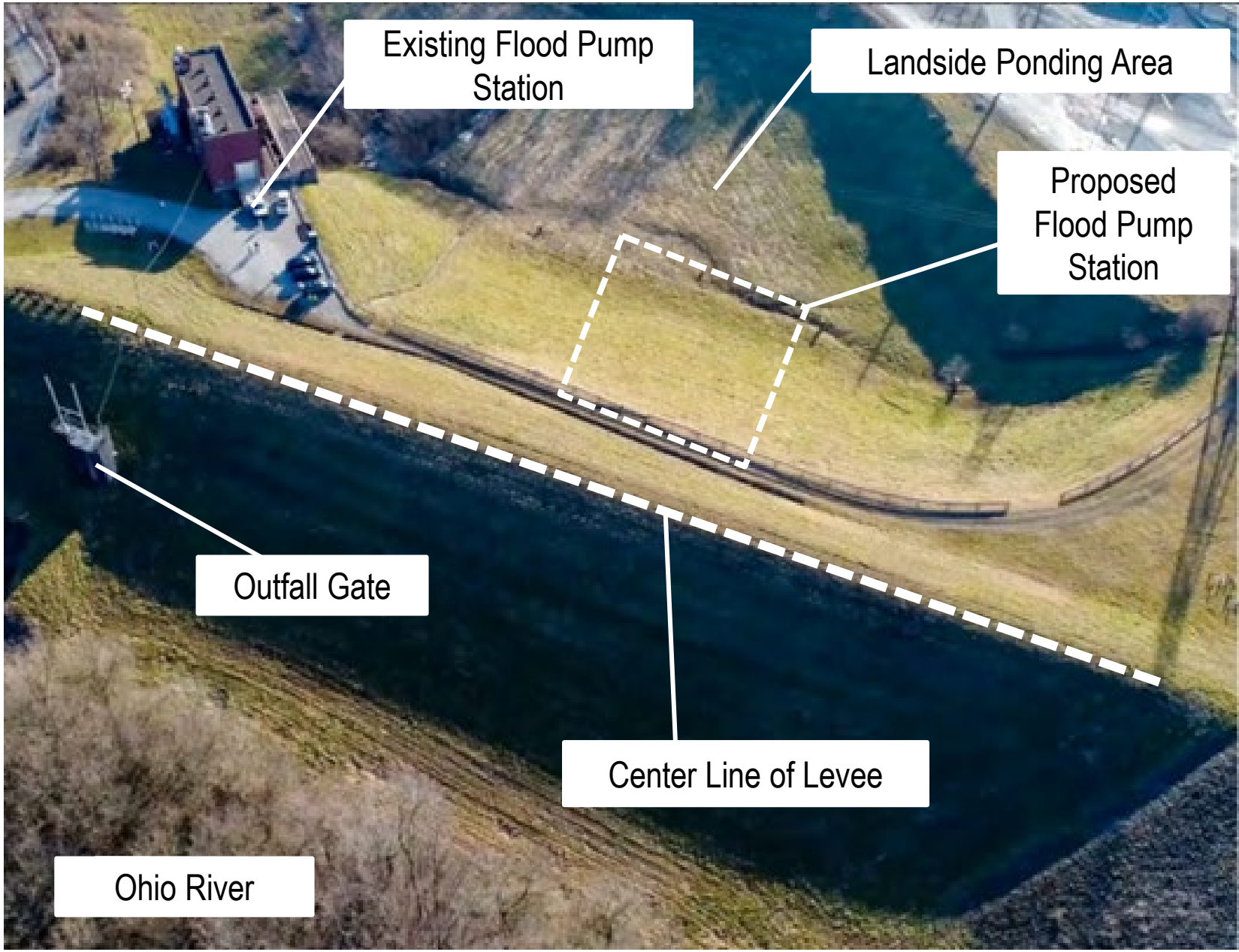


Existing Flood Pump Station

- Commissioned in 1953
- Pumping Capacity = 875 MGD
- 6 Pumps

Proposed Flood Pump Station

- Pumping Capacity = 1,900 MGD
- 8 Pumps
- Storm Event: Project Year 2035, 10-year, 24-hour storm event (5.0 inches in 24 hours)





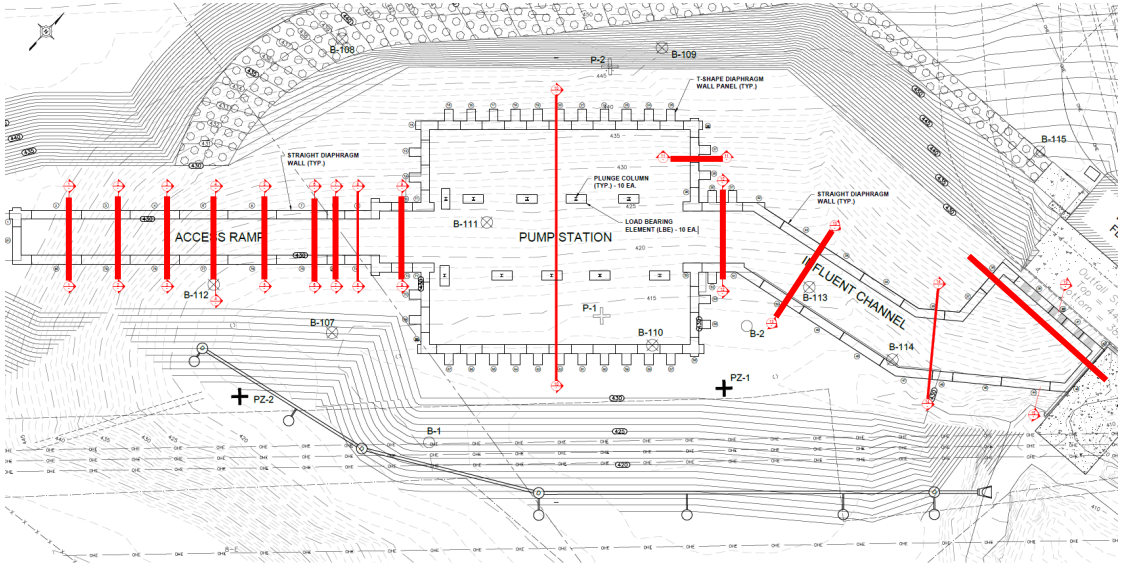
02 DESIGN SOLUTION

Challenges: Deep sands, gravels, high groundwater.

Objectives: Control groundwater, limit seepage, ensure levee stability.

Solution: Diaphragm wall to bedrock – no dewatering, seepage cutoff, dual use (temporary + permanent).

Design Tool: Finite Element Analysis (FEA).

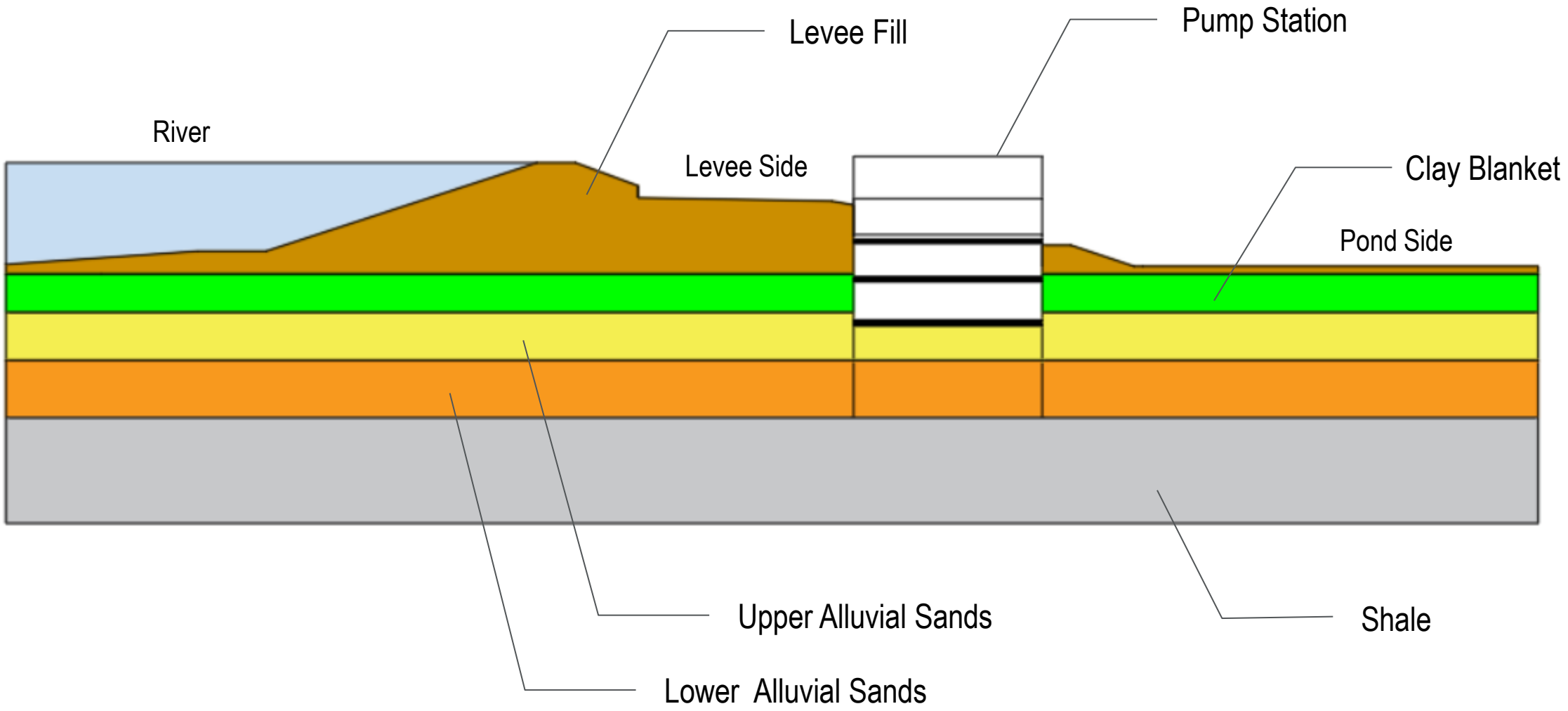


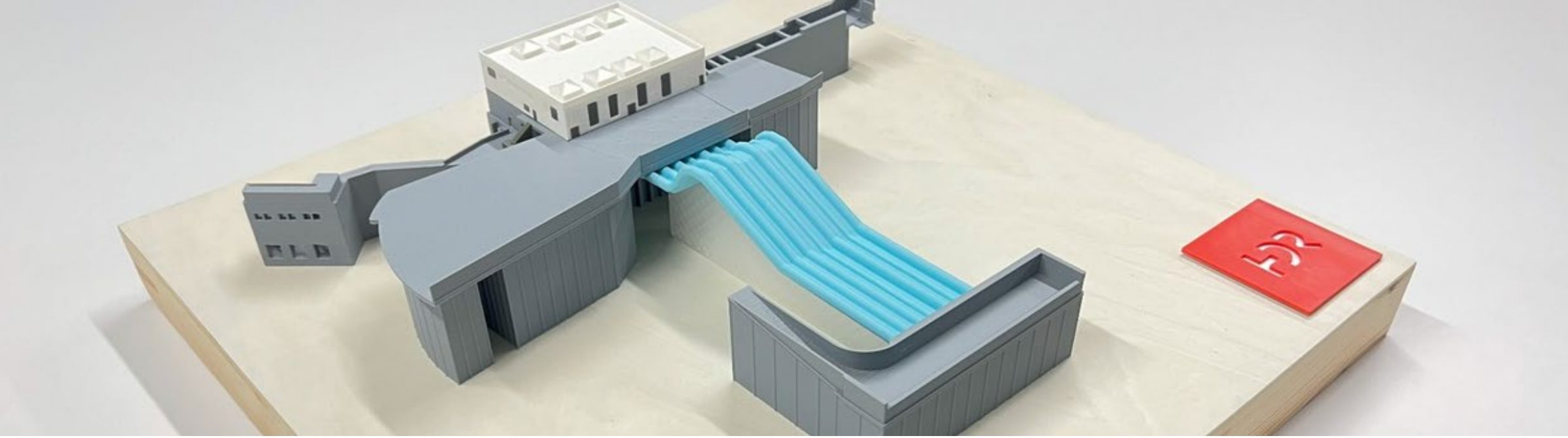




03 IDEALIZED SOIL PROFILE

Idealized Soil Profile

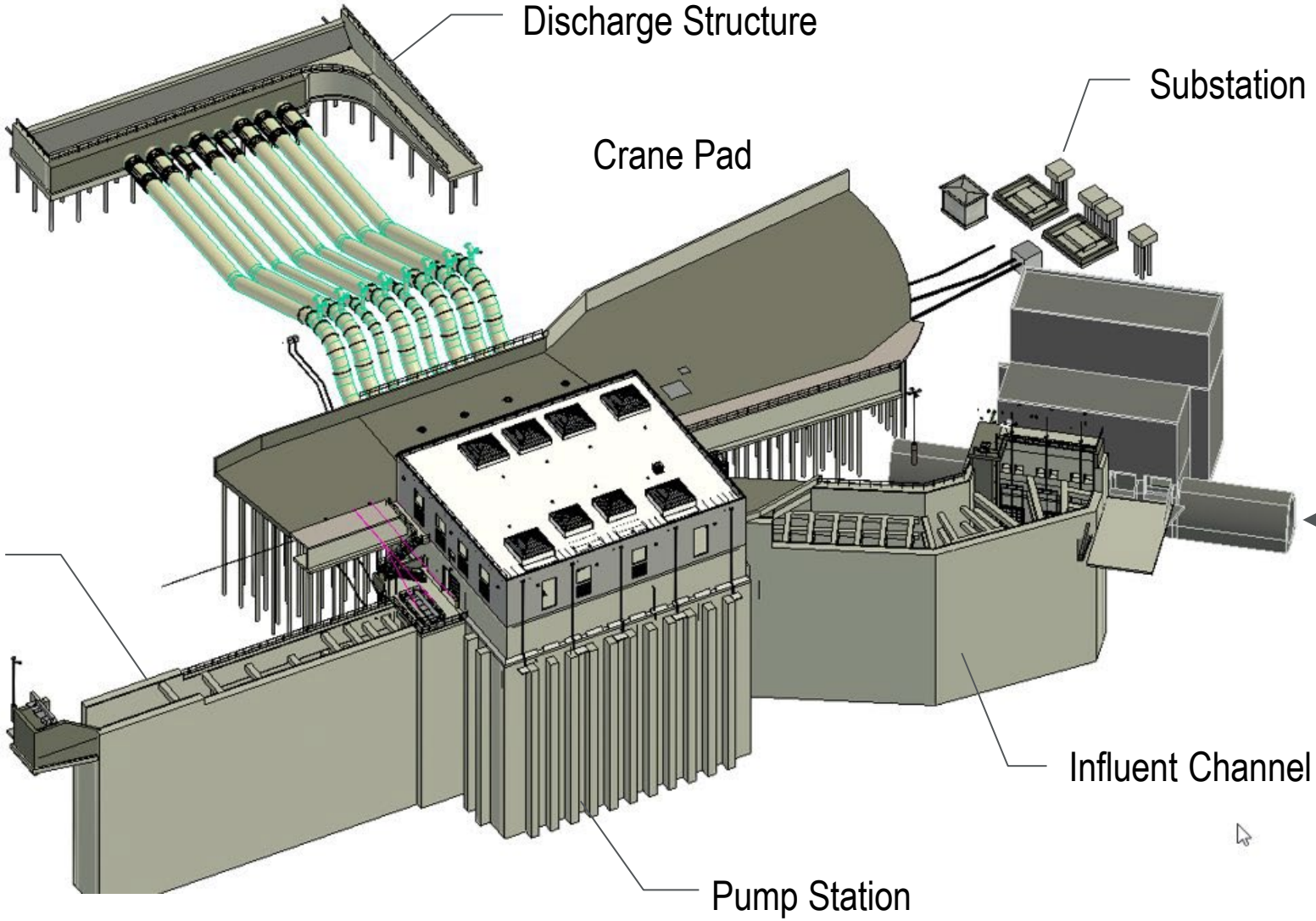


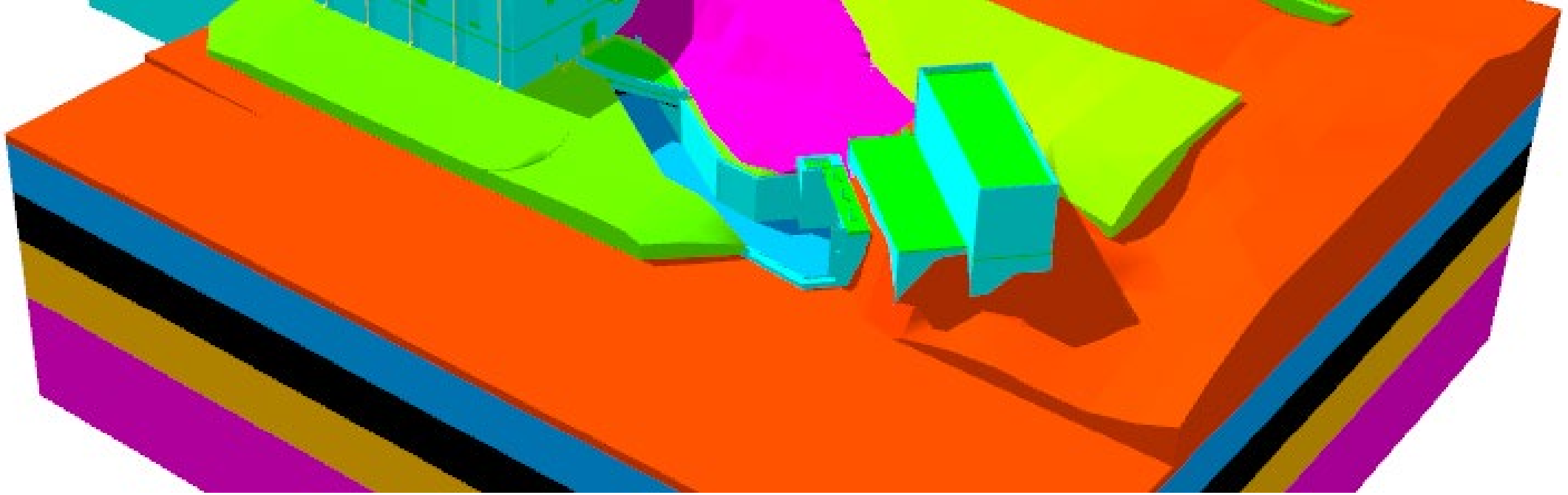




04 THE PUMP STATION STRUCTURE

Structural System



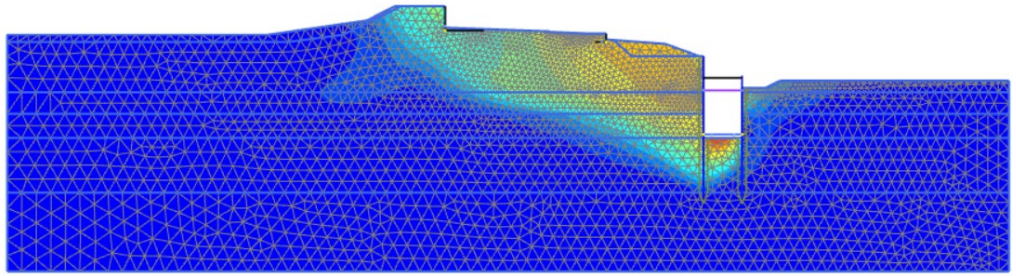




05 FROM 3D to 2D and BACK

PLAXIS®

PLAXIS 3D



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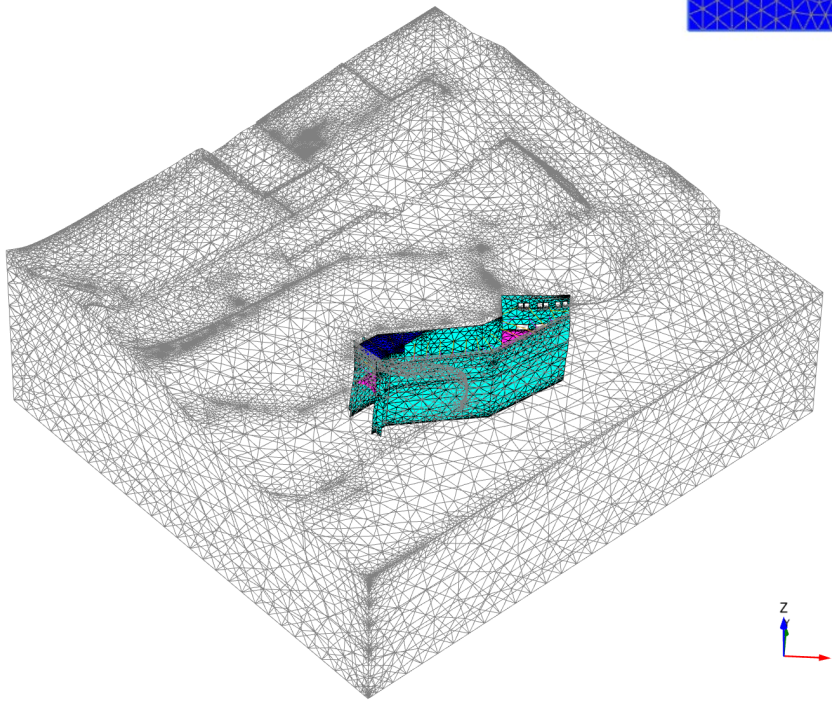

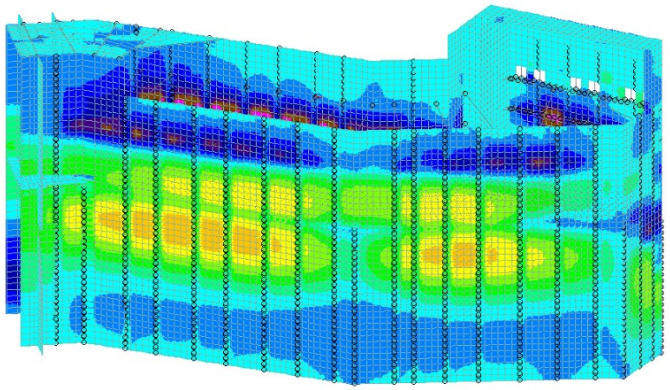



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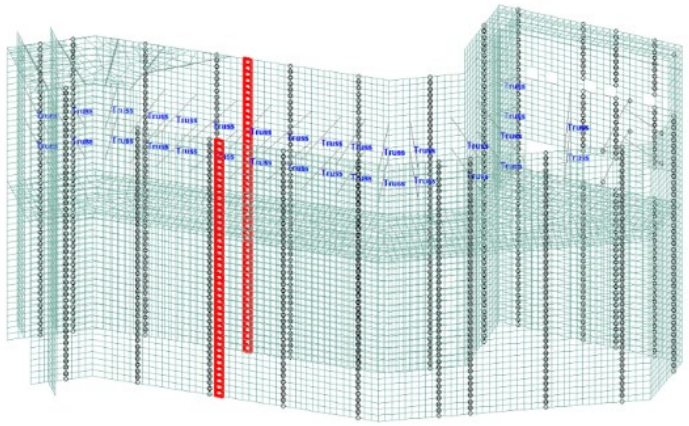
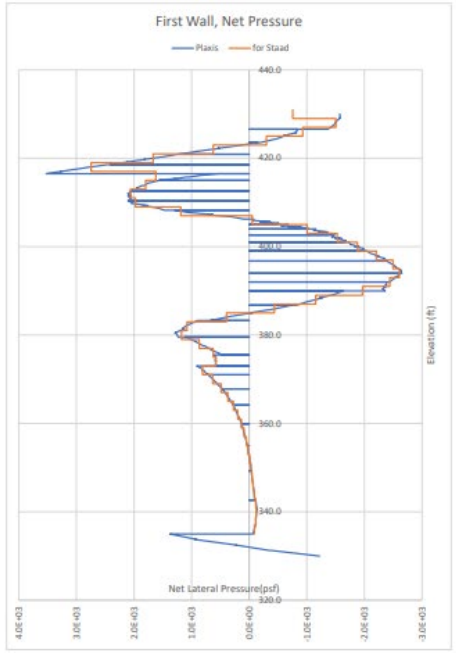
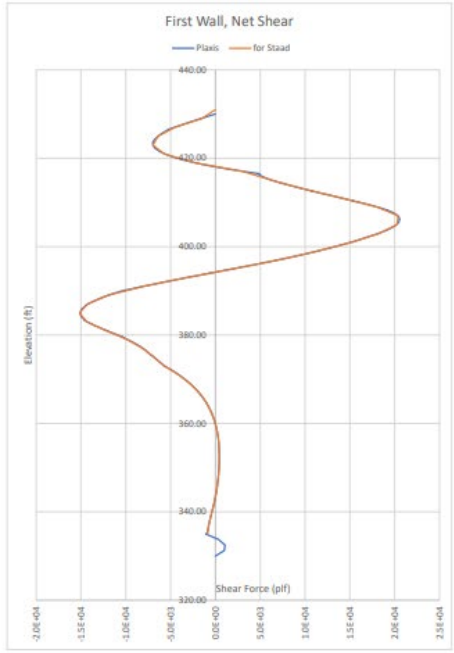
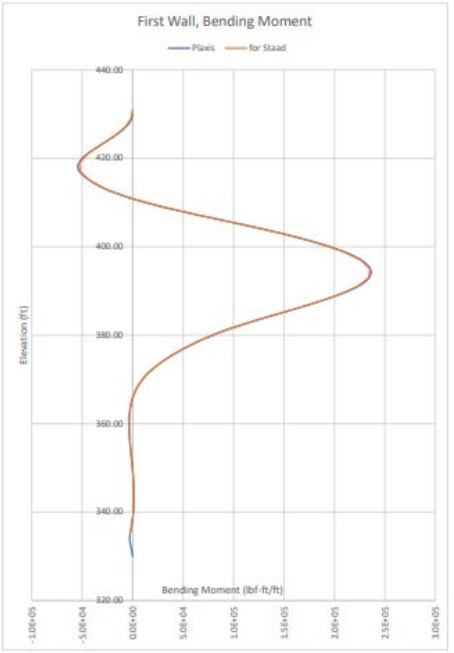
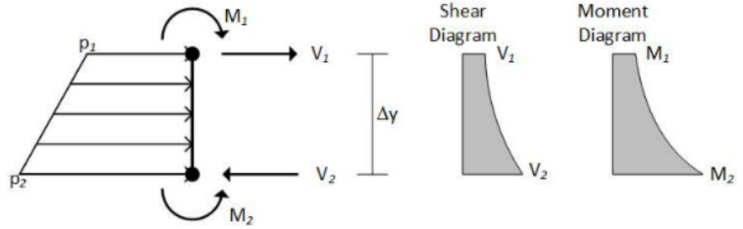


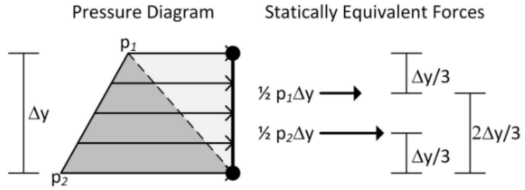
Excav 1 [Phase_3]

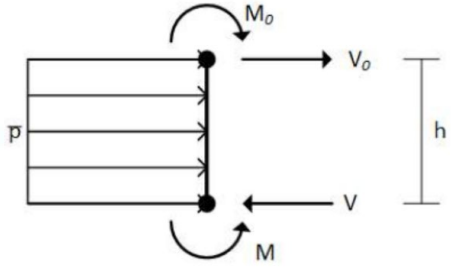
















06 INFLUENT CHANNEL ANALYSIS

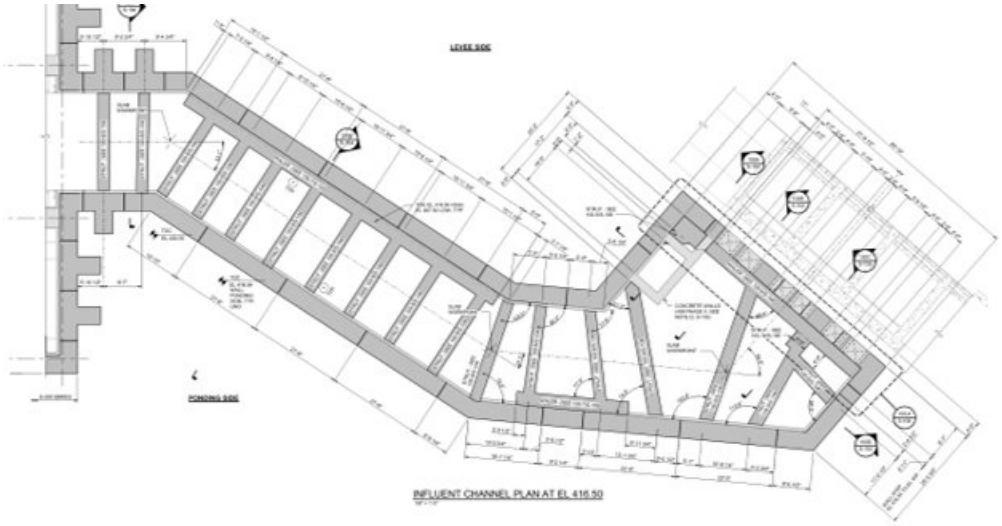
Influent Channel

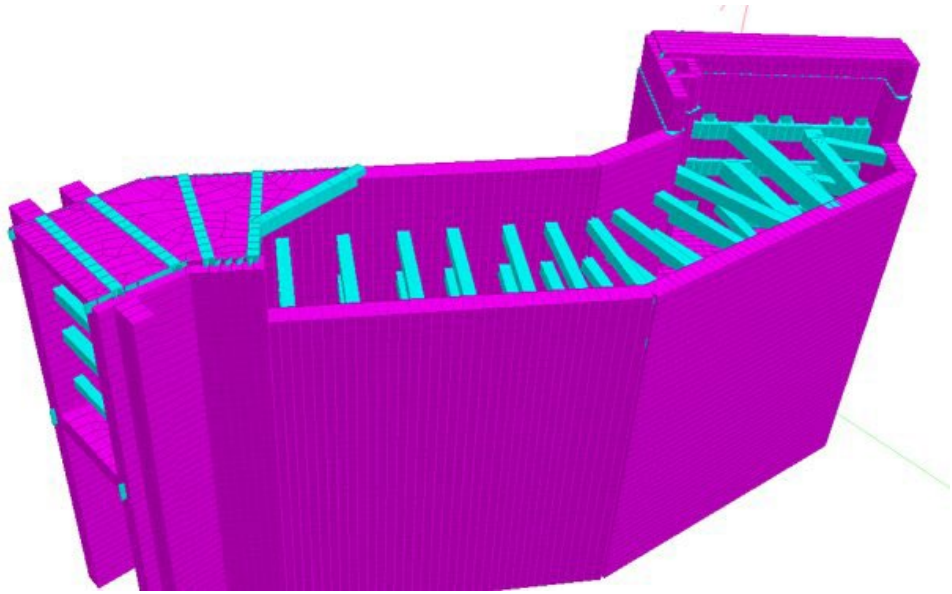
Abuts Existing pump station structure

Complex Geometry

Approximately 50 feet in depth

Multiple levels of struts and walers





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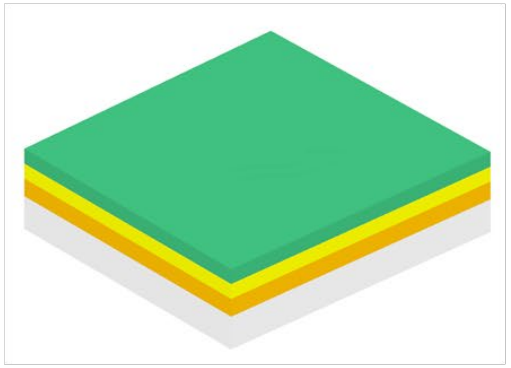

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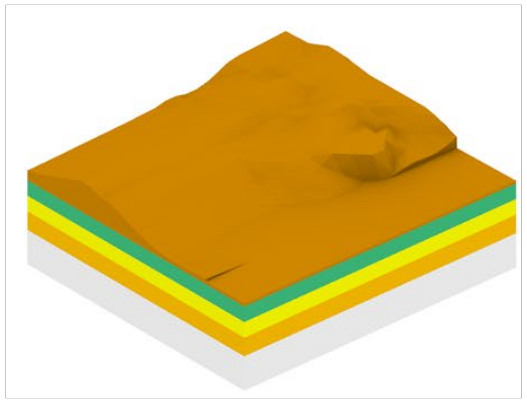

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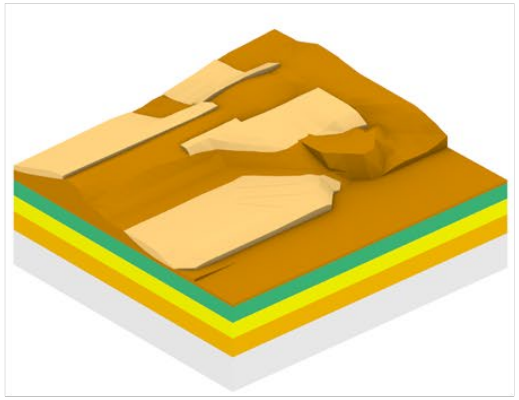

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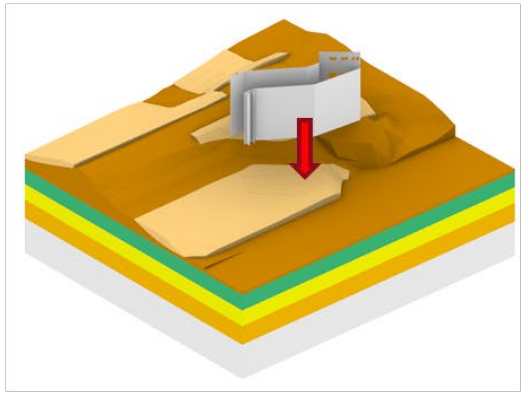

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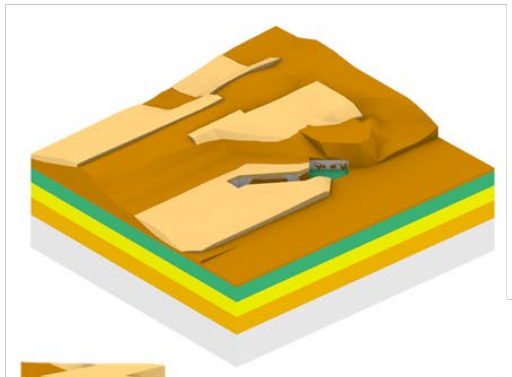

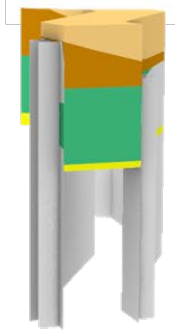

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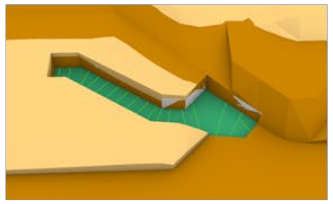



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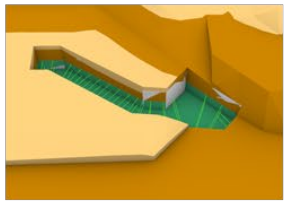
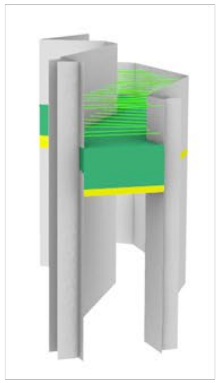
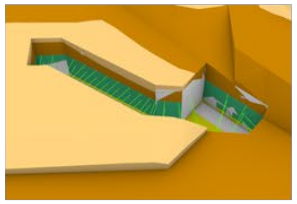
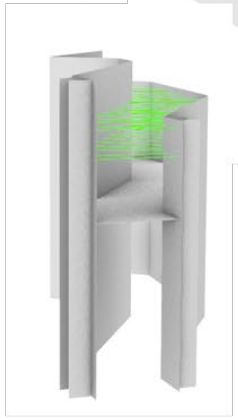




Stage 7





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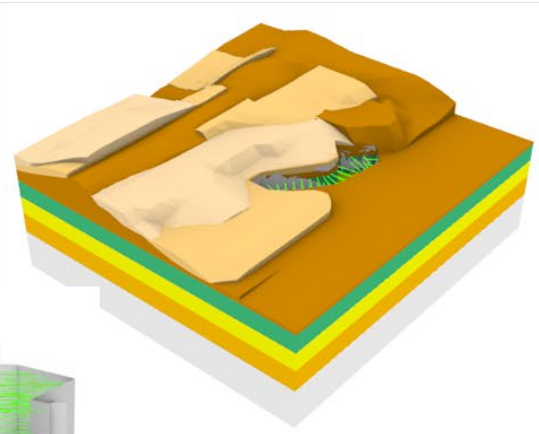
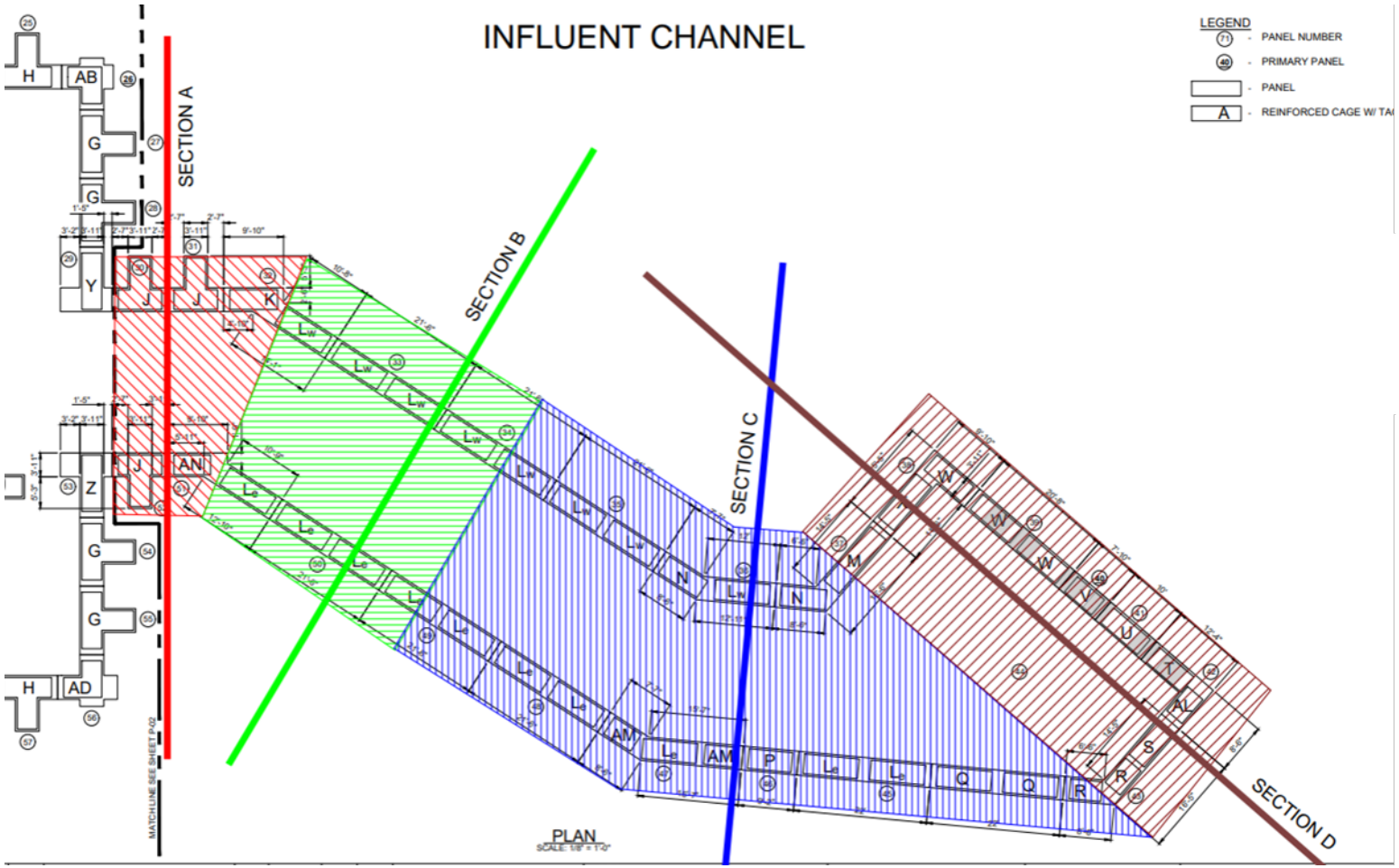
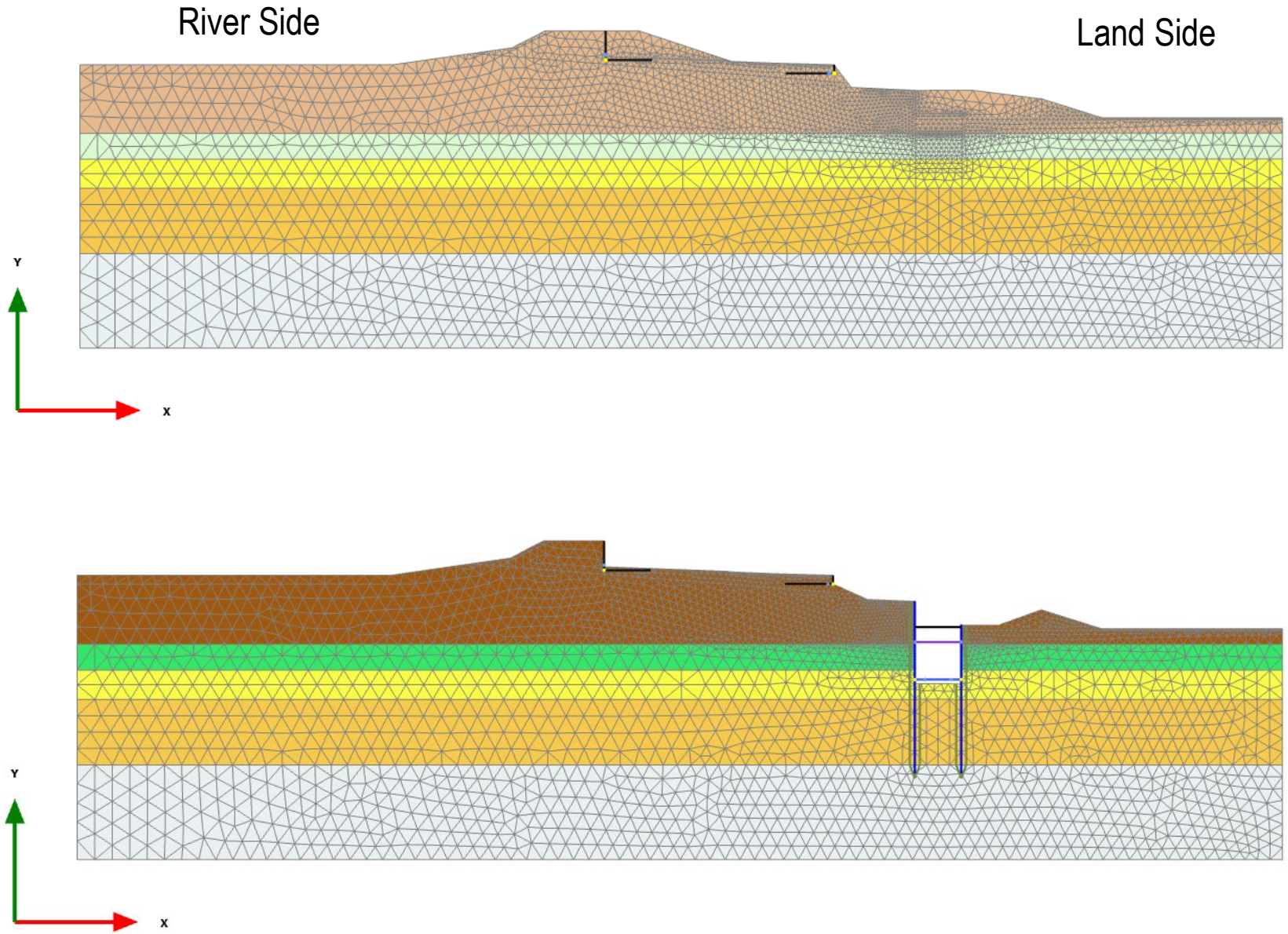


Evaluated Sections (PLAXIS 2D)



Phases

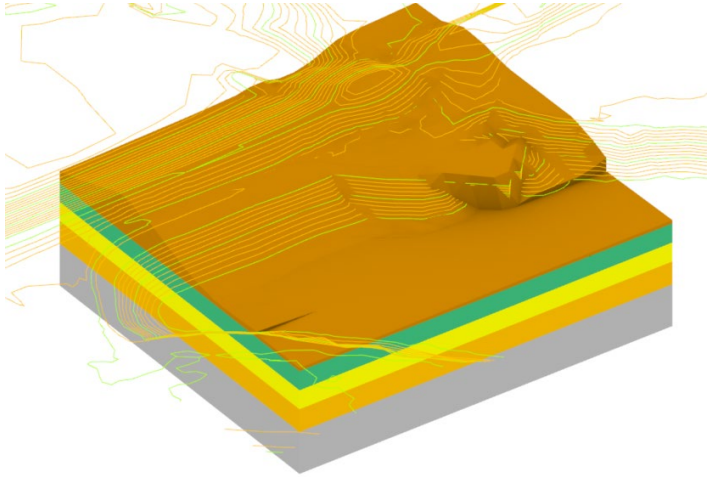
1.Excavation 1
2.Top Strut
3.Excavation 2
4.Middle Strut
5.Excavation 3
6.Temp Brace
7.Excavation BOE
8.Base Slab
9.Remove Temp Brace
10.Final Grade
11.Pressure Slab
12.Long Term



FE Model Limitations

3D Model

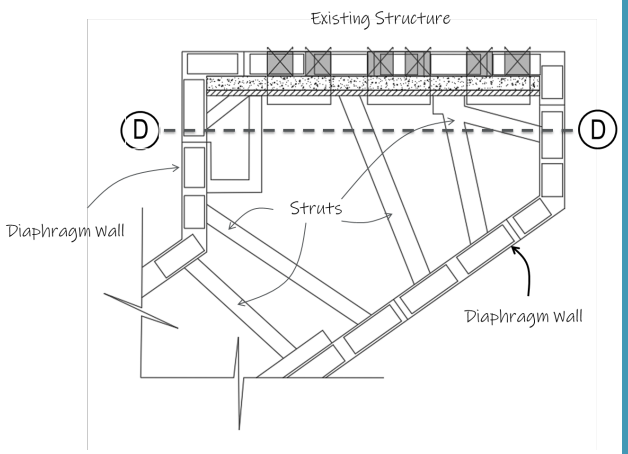
- Meshing – Complex geometry
- Model from Undrain to Drain conditions
- Converging problems



2D Model

- Capture 3D effects
- Limited to critical sections





Real-world examples

Study of 60 deep excavations with rigid diaphragm walls (Boonyarak et al, 2024):

Top-down: δmax/He = 0.10%–0.27%
Bottom-up: δmax/He = 0.20%–0.50%

Bottom-up + 1.0 m wall + longer excavation → up to 0.60%He

Shanghai soft soils:
Top-down ≈ 0.27%He
Bottom-up ≈ 0.40%He

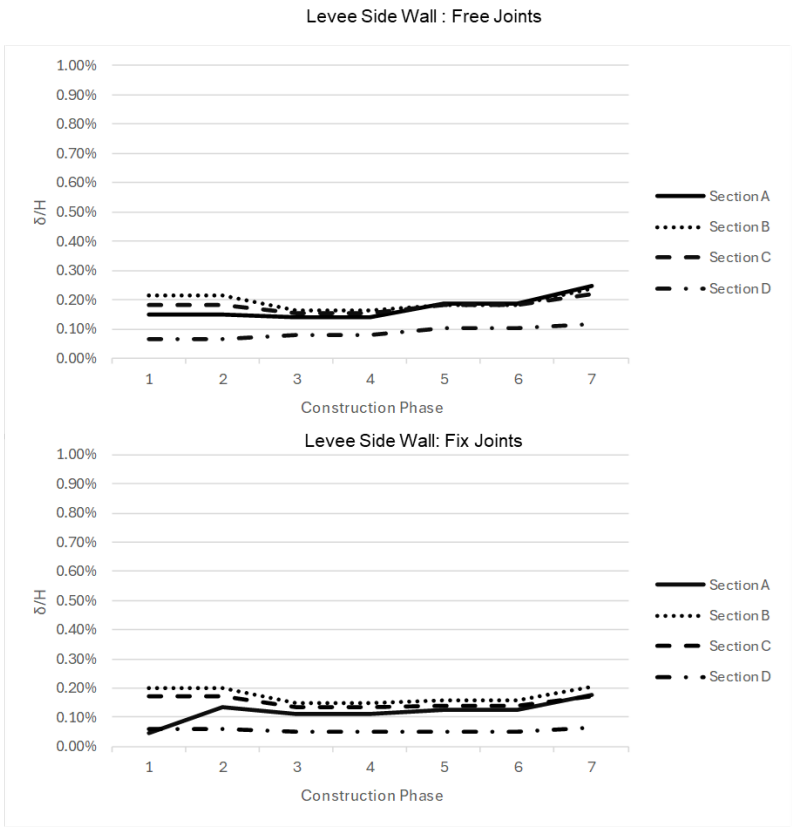
Case Histories (52 projects, 10–42 m depth)
(Masuda et al. ,1994)

δmax/He = 0.05%–0.5%



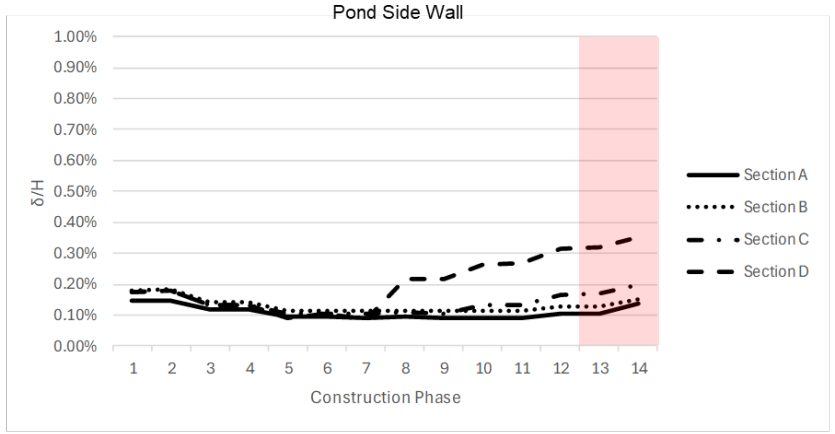
3D Model

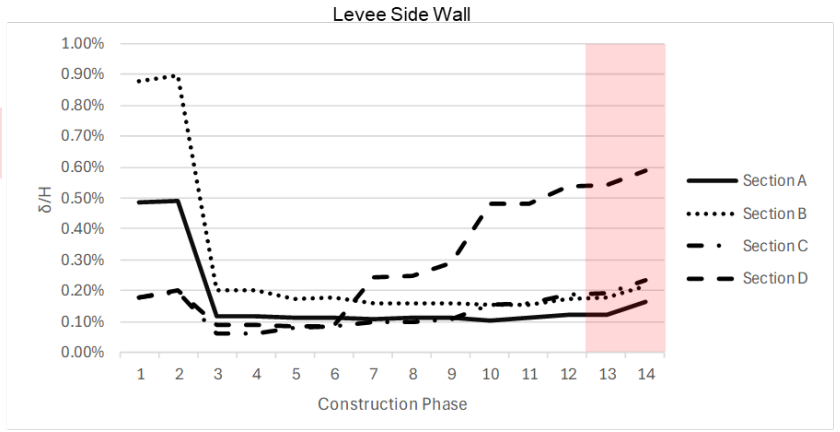
Phase	
1	Excavation 1
2	Top Strut
3	Excavation 2
4	Middle Strut
5	Excavation BOE
6	Base Slab
7	Final grade



2D Model

Phase	
1	Excavation 1
2	Top Strut
3	Excavation 2
4	Middle Strut
5	Excavation 3
6	Temp Brace
7	Excavation BOE
8	Base Slab
9	Remove Temp Brace
10	Final Grade
11	Pressure Slab
12	Long Term
13	Flood
14	Post Liquefaction







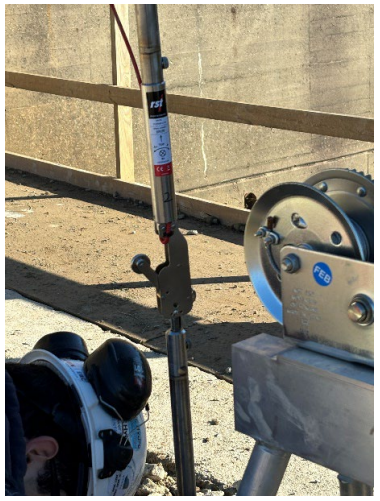


07 MONITORING MOVEMENTS

INSTRUMENTATION



Inclinometer casing

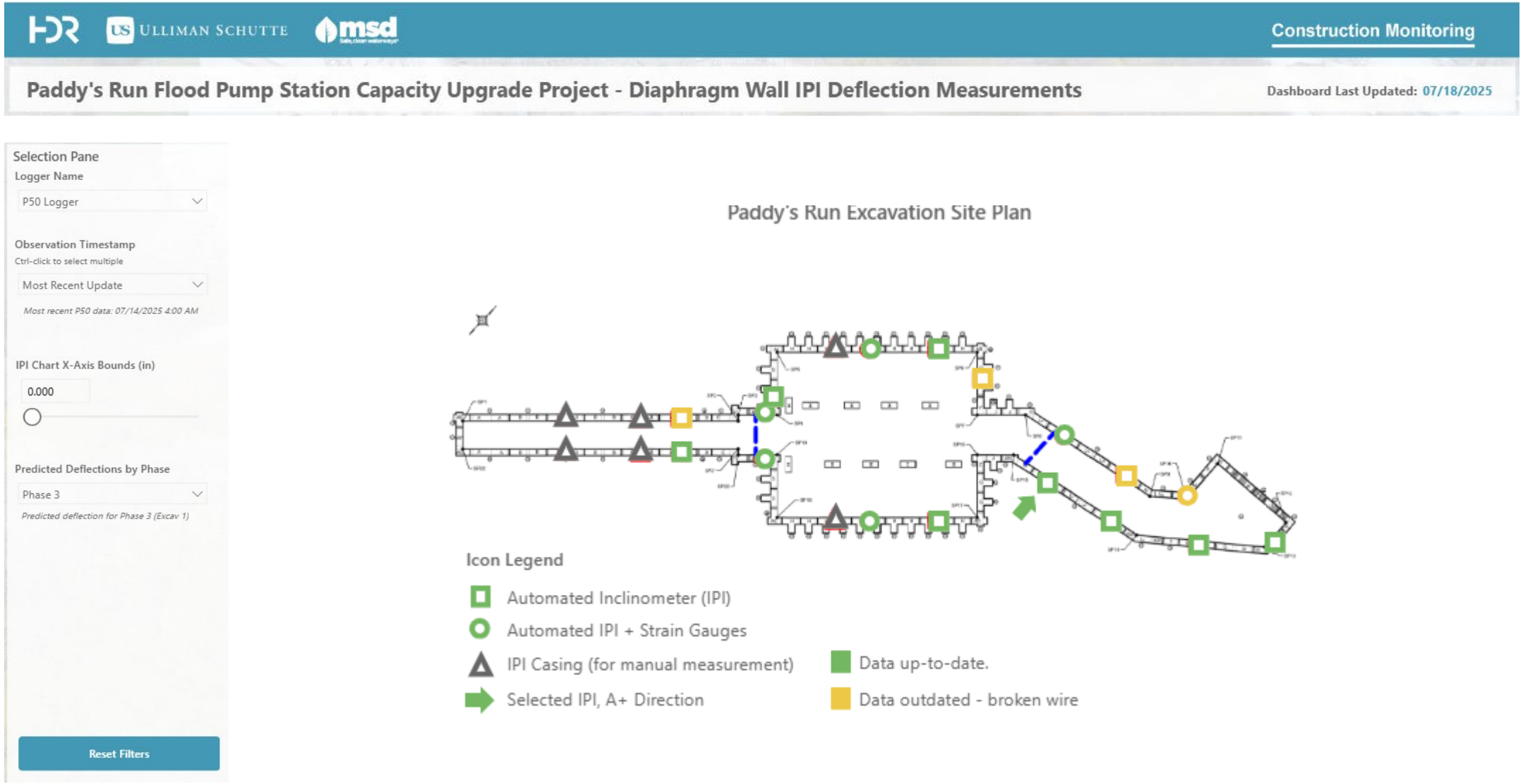


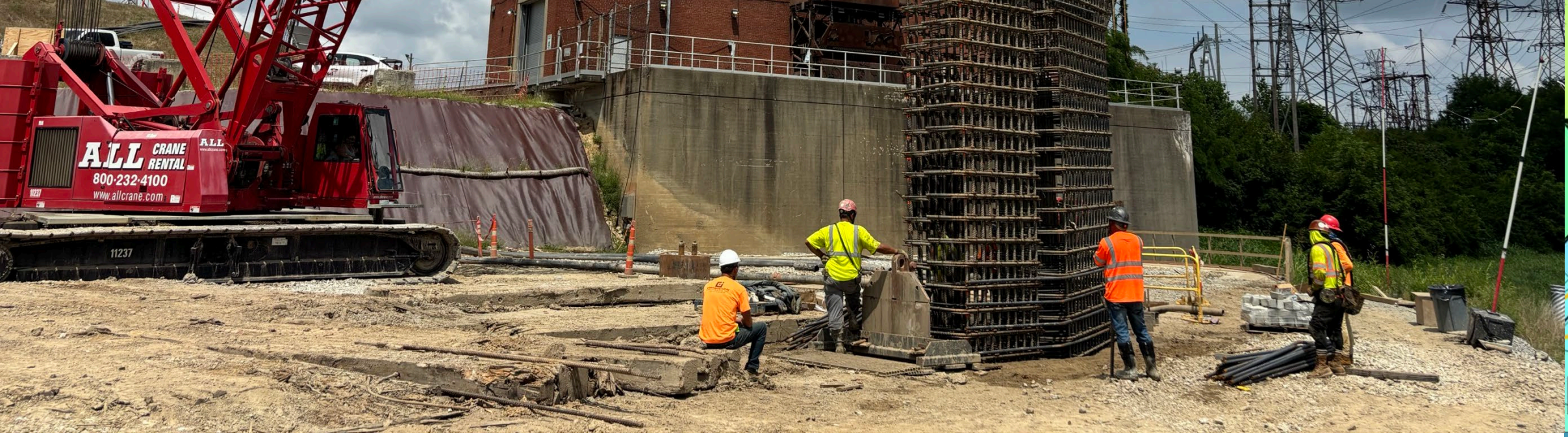
In-Place Inclinometer sensor



Strain gage installation

Monitoring System

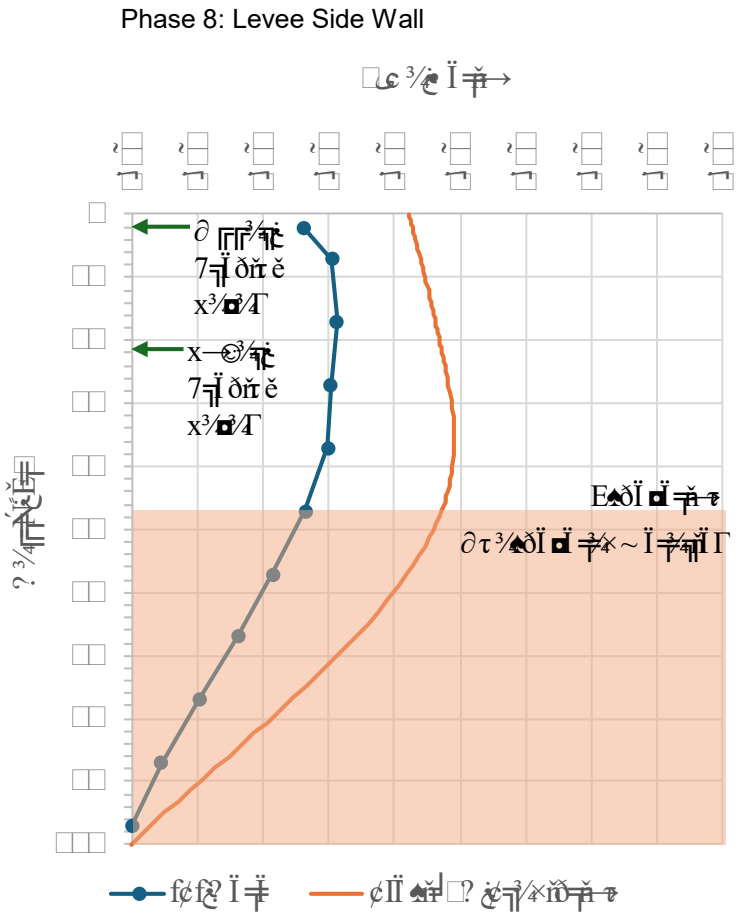






08 REALITY VS DESIGN

Comparison



Phase	
1	Excavation 1
2	Top Strut
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4	Middle Strut
5	Excavation 3
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7	Excavation BOE
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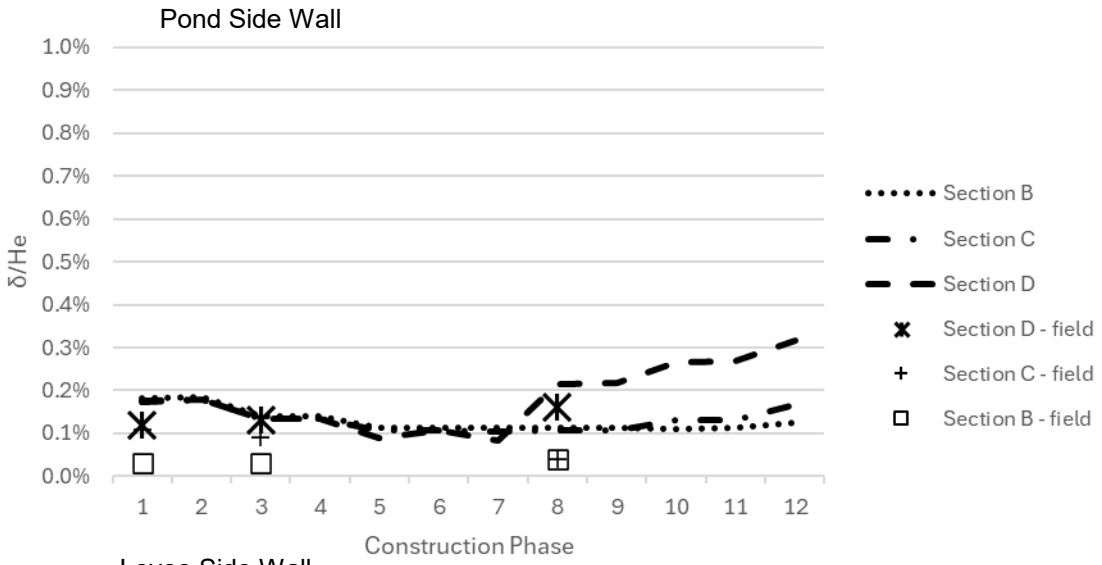
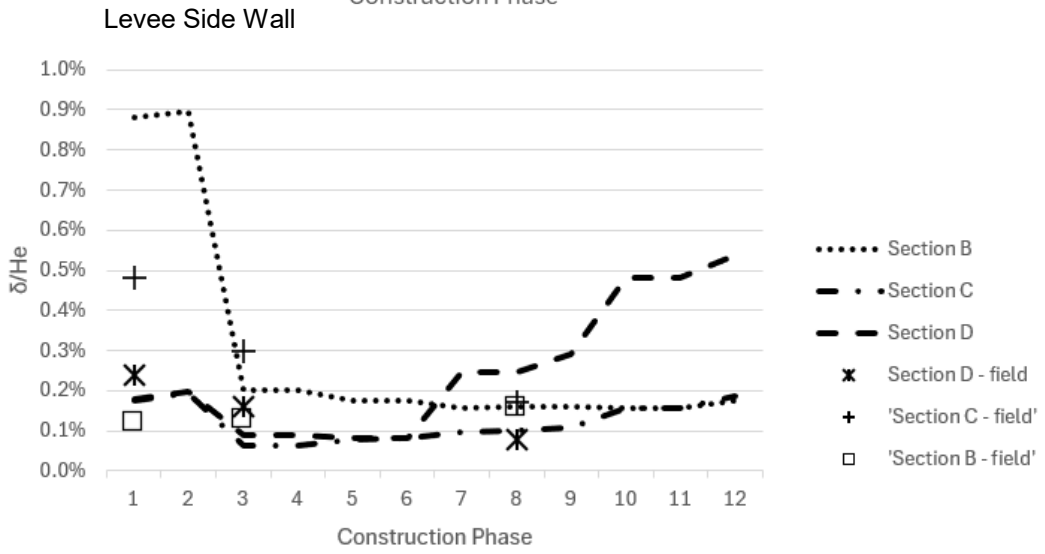
12	Long Term





SPECIAL THANKS

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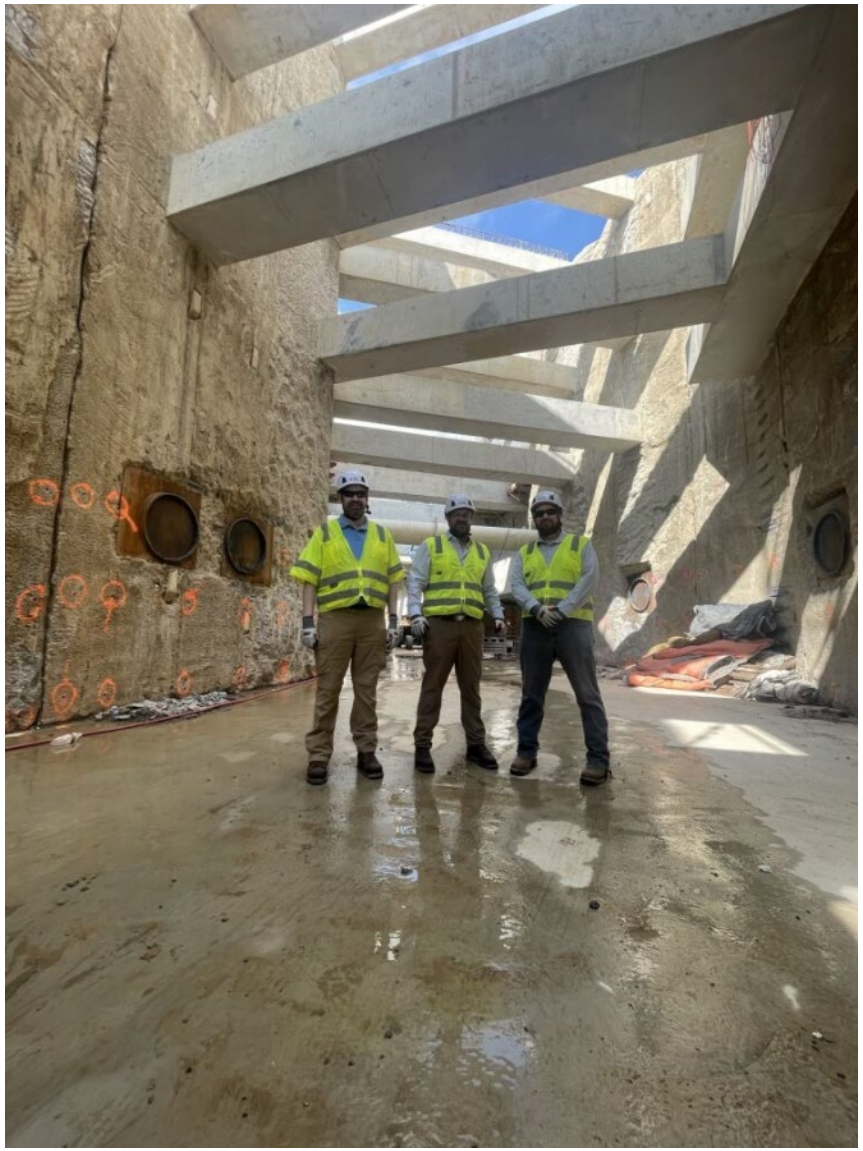



msd
Safe, clean waterways
US ULLIMAN SCHUTTE
HDR
NICHOLSON
Questions



Case History: Harbor Point Baltimore Evolution of Deep Foundations over 40 Years

Presented by: Adam M. Dyer, PE

Senior Associate, Mueser Rutledge Consulting Engineers, PLLC

Disclaimer:

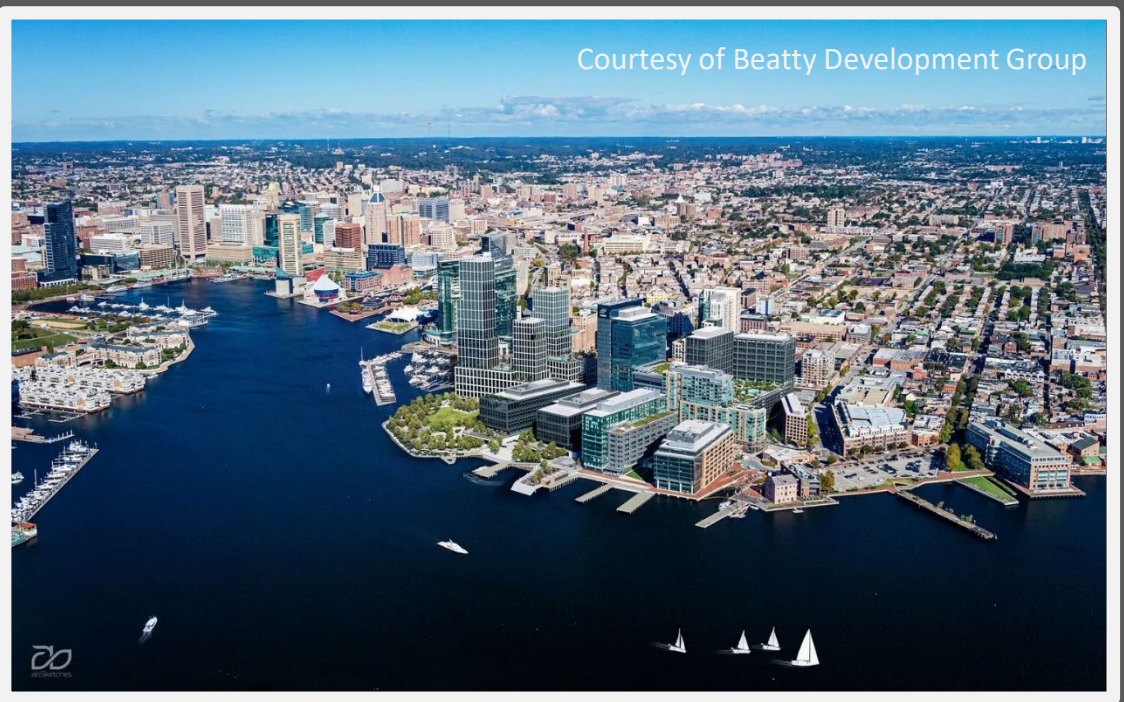
"The opinions expressed in this presentation and on the following slides are solely those of the presenter and not necessarily those of DFI. DFI does not guarantee the accuracy or reliability of the information nor endorse the recommendations offered by the presenter."

DFI 50 – Nashville, TN : October 22, 2025

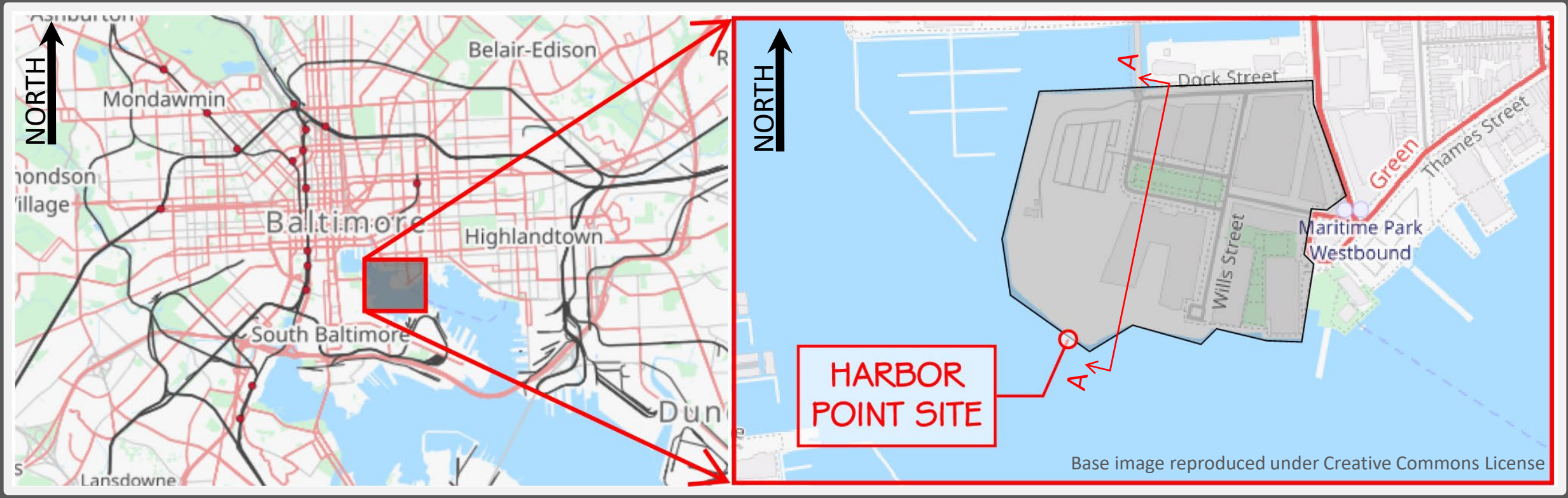
Introduction

- Location
- Site Geology
- Site History
- Environmental Remedy
- Redevelopment
- Future
- Questions



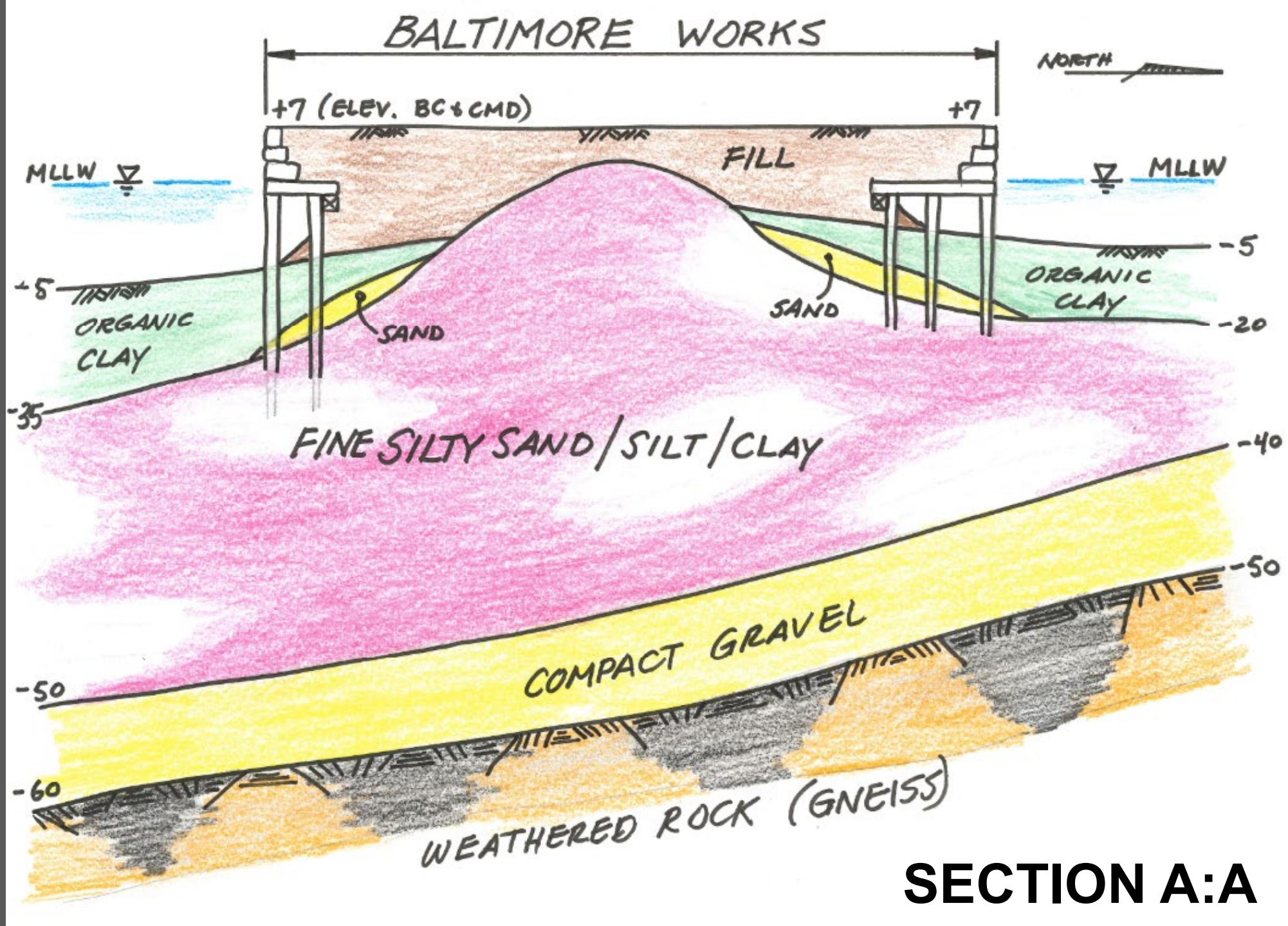


Site Location



Site Geology

- Peninsula with river bottom deposits along the waterside perimeter.
- Very compact / Hard silty sand / sandy silt creates peninsula.
- Continuous very compact gravel deposit.
- Weathered rock profile.
- Groundwater flows from northwest to southeast

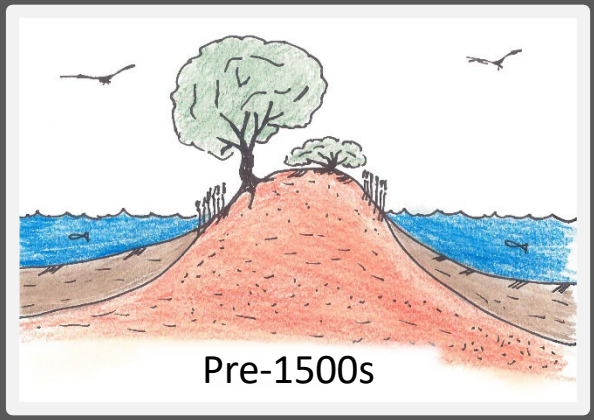


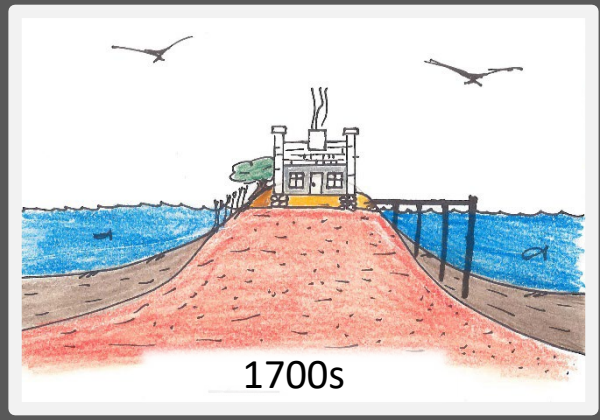
SECTION A:A

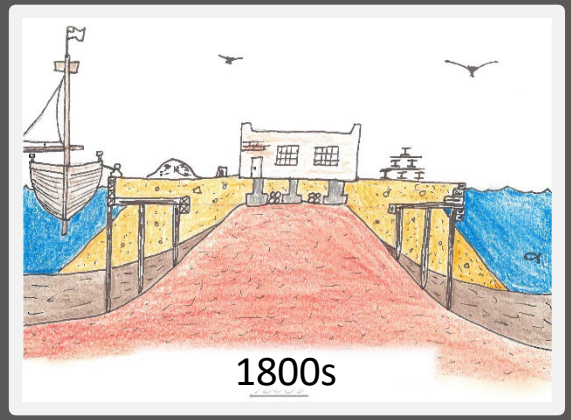
Site History

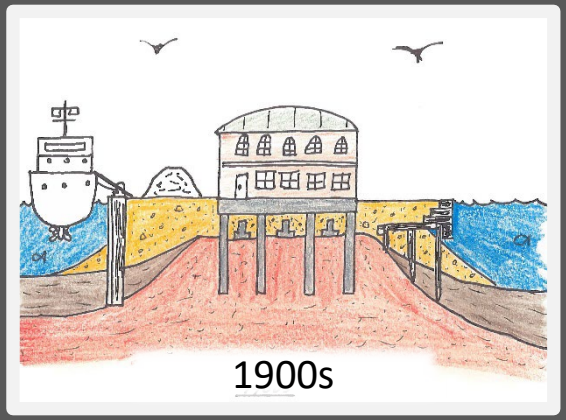
- 300+ Years of Human Activity
- 100+ Years of Industrial Activity
- Groundwater contamination:
  - Hexavalent Chromium (Cr^{6+})
- Environmental containment
- Redevelopment









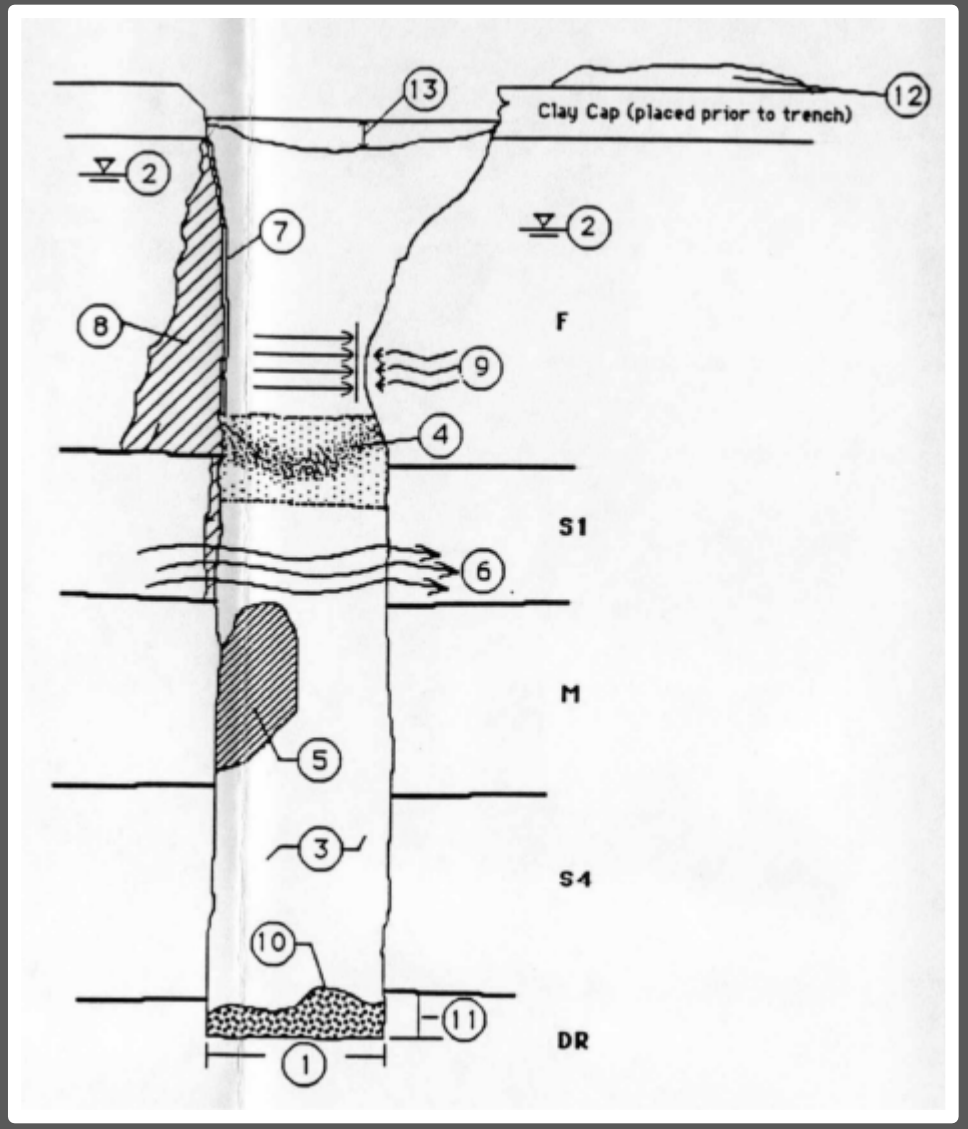


Remedy Design – I

- Industrial facility closed in mid-1980s
- Environmental investigation and remedy design: 1986 to 1991
- Selected remedy: Contain in place by low permeability soil-bentonite hydraulic barrier

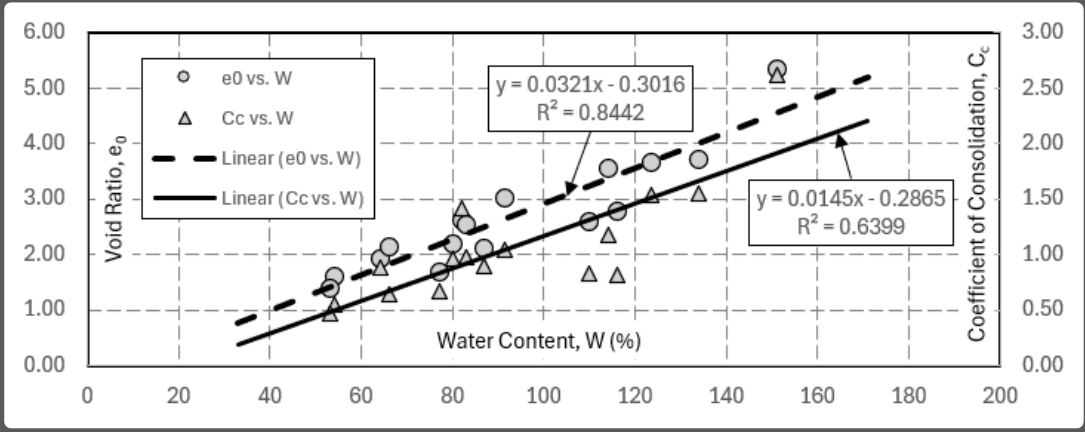
<u>Design Issues for Barrier:</u>

1. Thickness of Barrier
2. Differential Head
3. Backfill Material
4. Arching/Hydrofracture
5. Barrier Defects
6. Seepage Through
7. Filter Cake
8. Slurry Penetration
9. Trench Stability
10. Trench Sediment
11. Closure at Cutoff
12. Amount of Spoils
13. Surface Settlement



Remedy Design – II

- Geotechnical Investigations:
  - Numerous phases with over 100 historic borings and over 100 new borings in 6 phases from 1988 to 1993
  - Strong correlations identified for organic clays





Remedy Design – III

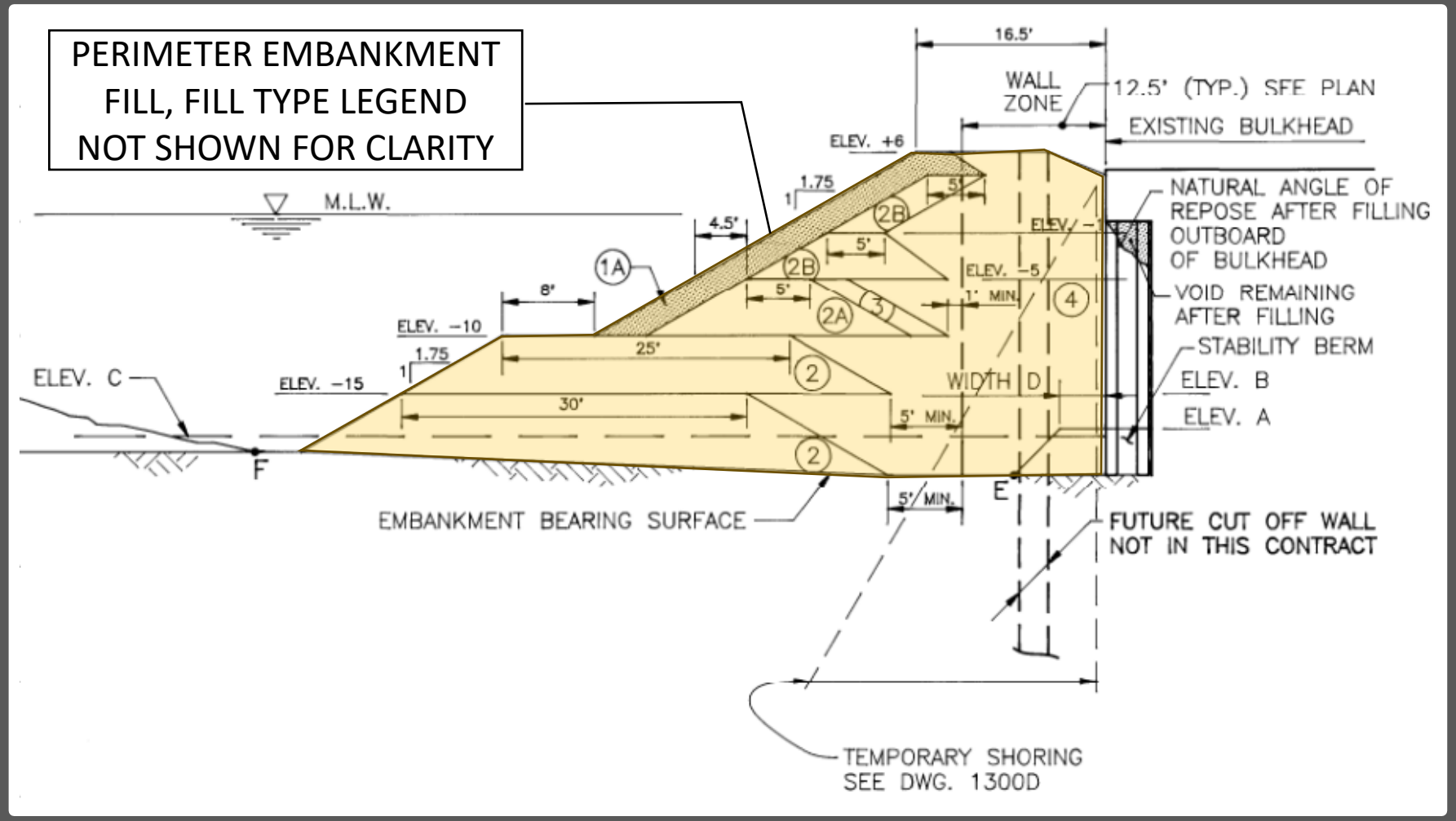
- Perimeter Embankment

Design Issues:

1. Stability of existing bulkheads
2. Stability of Embankment during Barrier construction
3. Settlement of embankment crest

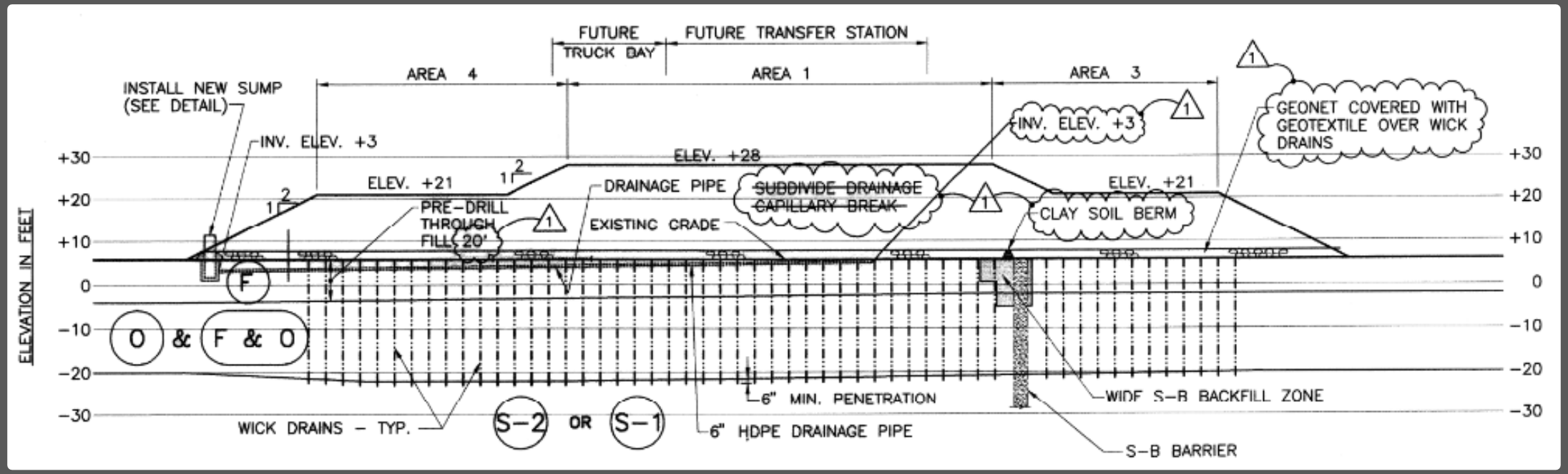
Analysis:

1. Analyzed using PC-Slope (early version of Slope/W).
2. Factor of Safety > 1.25 during dredging



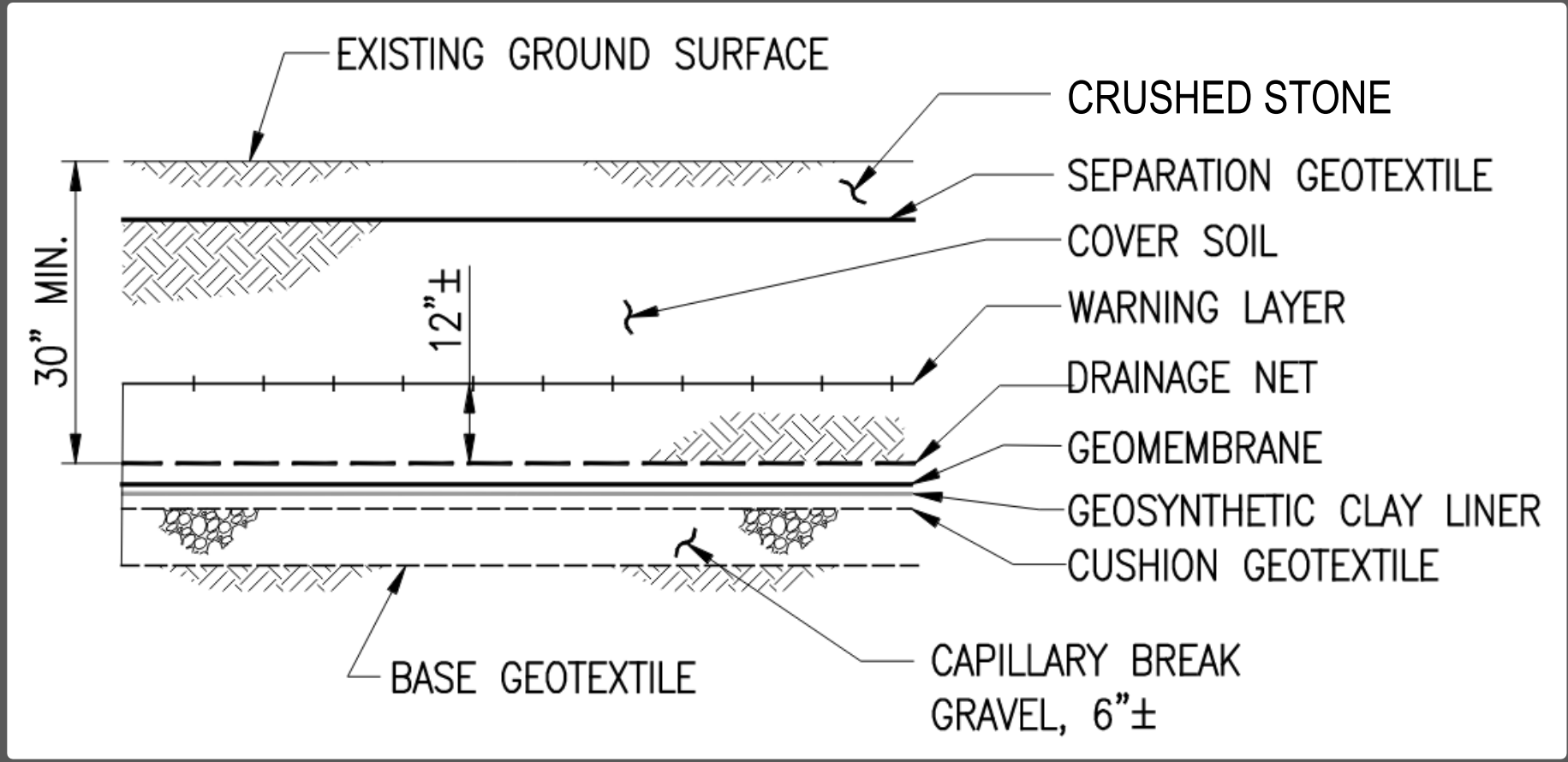
Remedy Design – IV

- Pre-compression of Organic Clay in advance of fill placement using surcharge and wick drains, controlled by settlement and hydrostatic pressure measurements



Remedy Design – V

- Multimedia Cap



Remedy Construction – I

- Perimeter Embankment
  - Some bulkheads stabilized with temporary bracing.
  - Dredged out organic clay in front of bulkheads.
  - Placed specified gradations to pre-determined elevations and plan extents.
  - Vibro-densified stone after placement.



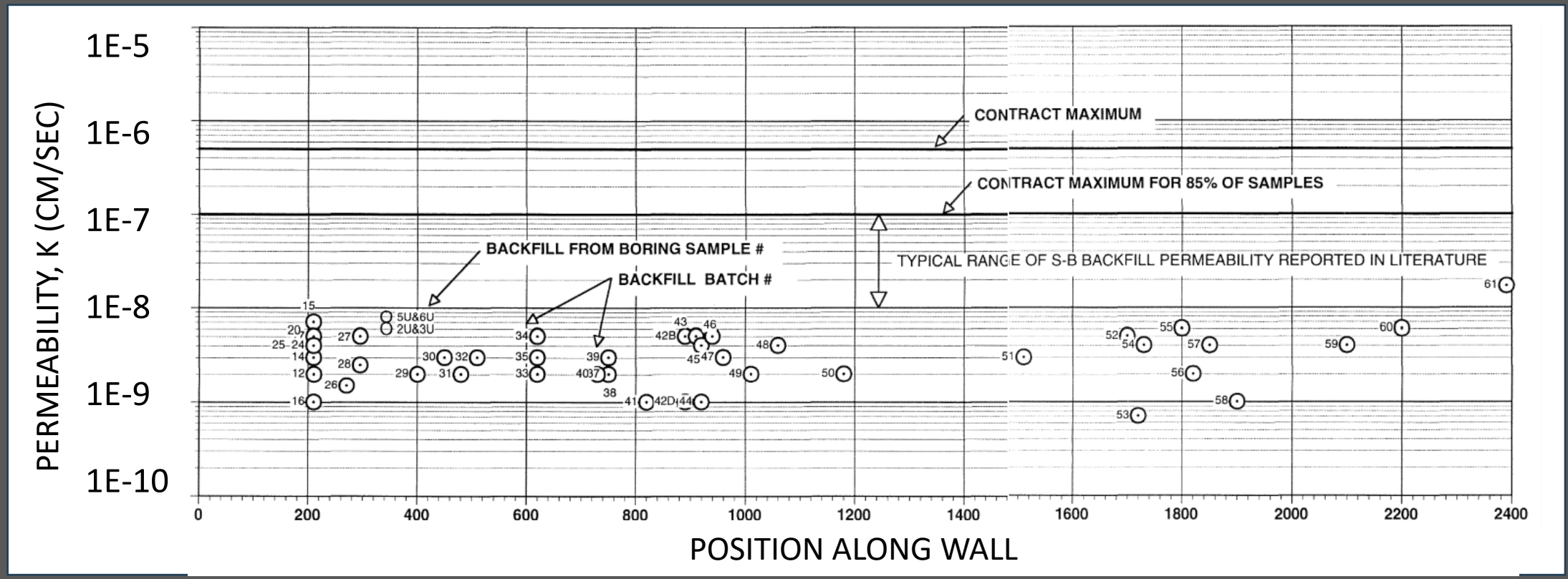
Remedy Construction – II

- Hydraulic Barrier
  - Difficulty with excavating through hard silts and clays
  - Trench Stability
  - Worker Safety
  - QA/QC Complications
  - QA/QC Results



Remedy Construction – III

- Hydraulic Barrier QA/QC Results



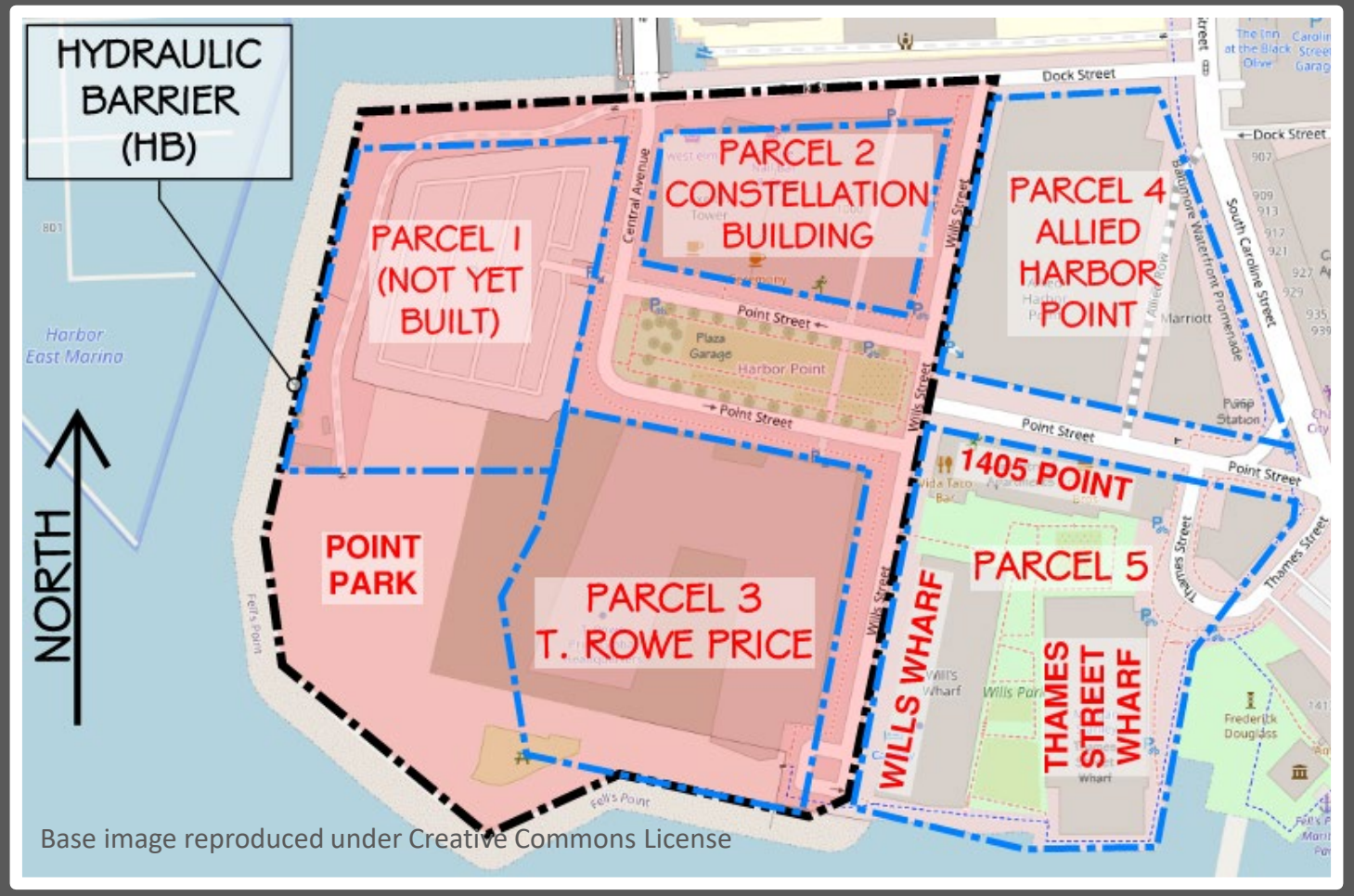
Remedy Construction – IV

- Multimedia Cap Construction
  - Connection to Hydraulic Barrier
  - Stability over Hydraulic Barrier
  - Spreading layers evenly



Re-Development – I

- Built in Phases from 2005 to Present to create the new neighborhood of Baltimore known as Harbor Point



Re-Development – II

- As is typical, foundation selection is driven by overall cost to the project. The presence of the active remedy system distorts which costs drove selection relative to typical:
  - High costs to collect, treat, and dispose of groundwater collected from below the Multimedia Cap (about 50 times higher per gallon than other less contaminated sites).
  - Foundation work occurring on a site which has tightly controlled contamination reduction zones to mitigate potential for cross contamination above multimedia cap.
  - Depth of uncontrolled fill requires deep foundations in most portions of the site.

Re-Development – III

- Deep Foundation Selection:
  - Cost estimates and studies performed with contractors indicated driven piles would control costs better than drilled piles.
  - Pile Load Test Program conducted during design. However, test pile program had to be performed adjacent to the site due to environmental restrictions. This limitation was mitigated by requiring a robust indicator and PDA pile program during construction.

Re-Development – IV
Offsite Load Tests - 2013



Re-Development – V
Pile Load Tests Performed:

- 2007 Load Tests:
  - 14 inch diameter pipe pile in two geologic conditions and two HP14 H-piles in soft profile
  - PDA Measurements during initial drive and re-struck after 48 hours (min.)
  - Maintained Test with 24 hour hold, maximum test load of 320 tons

- 2013 Load Tests:
  - 16 and 18 inch diameter conical tip pipe piles filled with concrete sealed at top with non-shrink grout in two geologic conditions and HP14 & HP18 H-piles in soft profile
  - PDA Measurements during initial drive and re-struck after 48 hours (min.)
  - 16 inch diameter: Maintained Test with 48 hour hold with maximum test load of 450 tons
  - 16 inch diameter: Excess Test with maximum load of 540 tons
  - 18 inch diameter: Maintained Test with 48 hour hold with maximum test load of 575 tons

- Piles were driven by an open ended diesel hammer delivering about 80,000 ft*lbs to about 90% of f_y

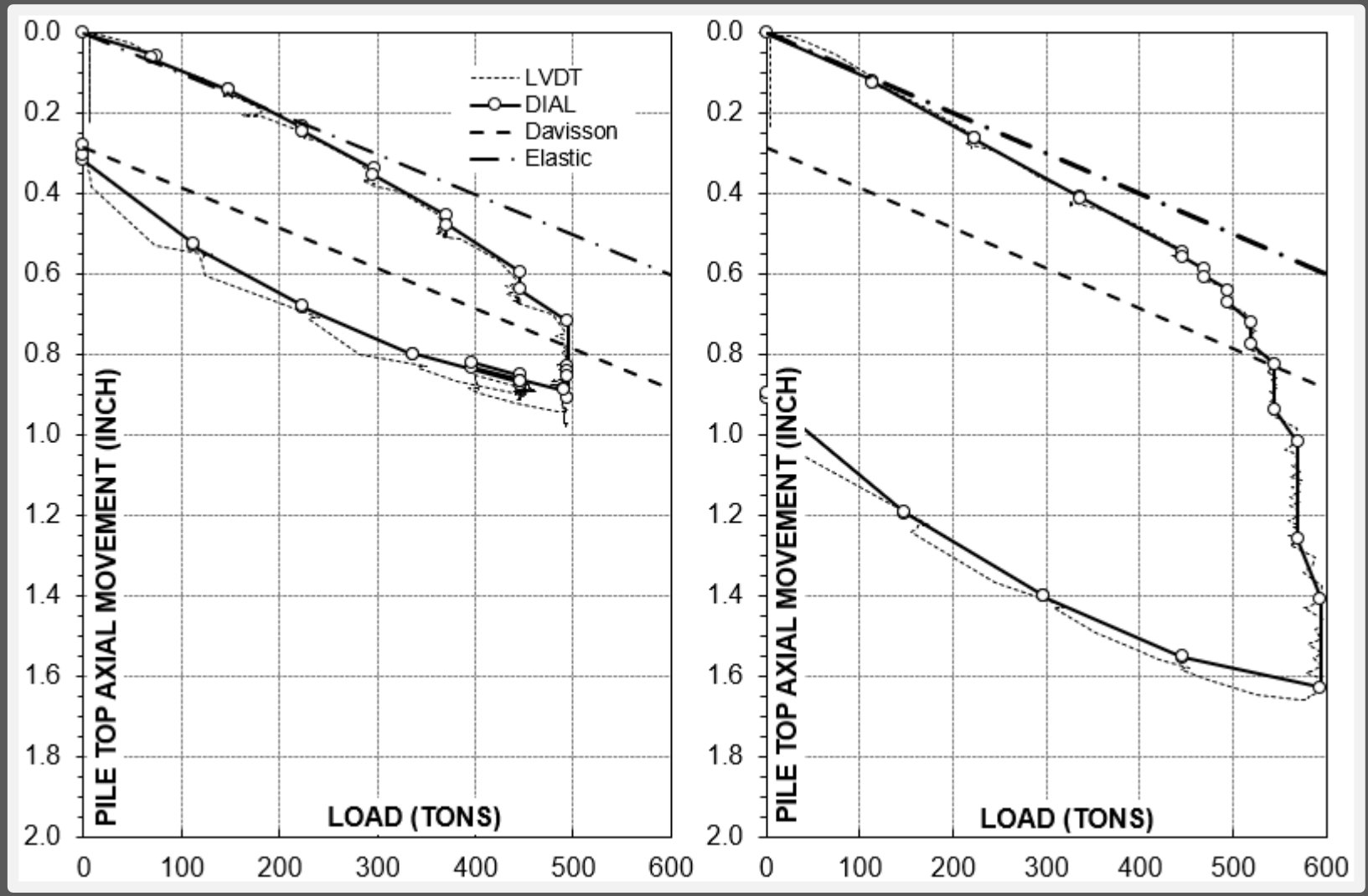
Re-Development – VI
Offsite Pile Load Test Results

Pile Type	PDA Ultimate Capacity, tons	Drive Time, minutes	Length of Pile, feet	Weight of Steel per Pile, tons	Splice Required
16-inch Pipe	500	40	50	2.1	No
18-inch Pipe	575	40	50	2.4	No
HP14x117	600	60	90	5.3	Yes
HP18x135	650	60	90	6.1	Yes

Selected based on benefit of reduced install time and steel

Re-Development – VII

- Example load static axial load test (ASTM D 1143-07) results 16-inch Diameter Pipe Pile Maintained, Method B (Left) and Excess, Method C (Right).

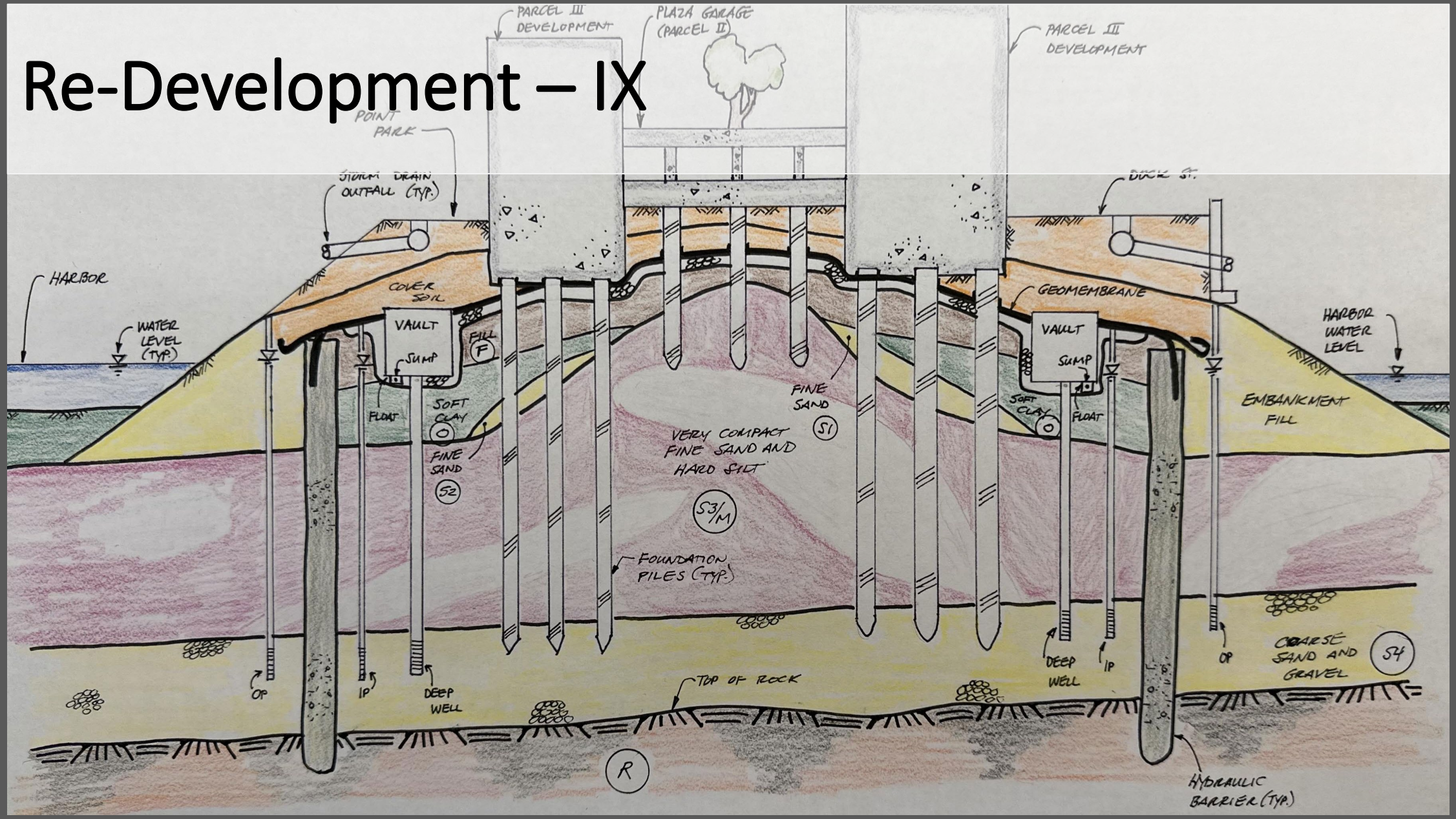


Re-Development – VIII





Re-Development – IX



Future Development

- At present, one parcel remains for re-development.
- The remaining parcel encompasses some of the most densely populated abandoned foundations and harsh groundwater chemistry at the site. Namely:
  - Abandoned steel sheet pile coffer dams
  - Abandoned five-foot thick concrete mat
  - Groundwater with pH ranging from 3 to 12 across a distance of less than 50 feet
  - Driven steel and timber piles
  - Steel rails
  - Timber relieving platform

Images courtesy of Honeywell International and Beatty Development Group



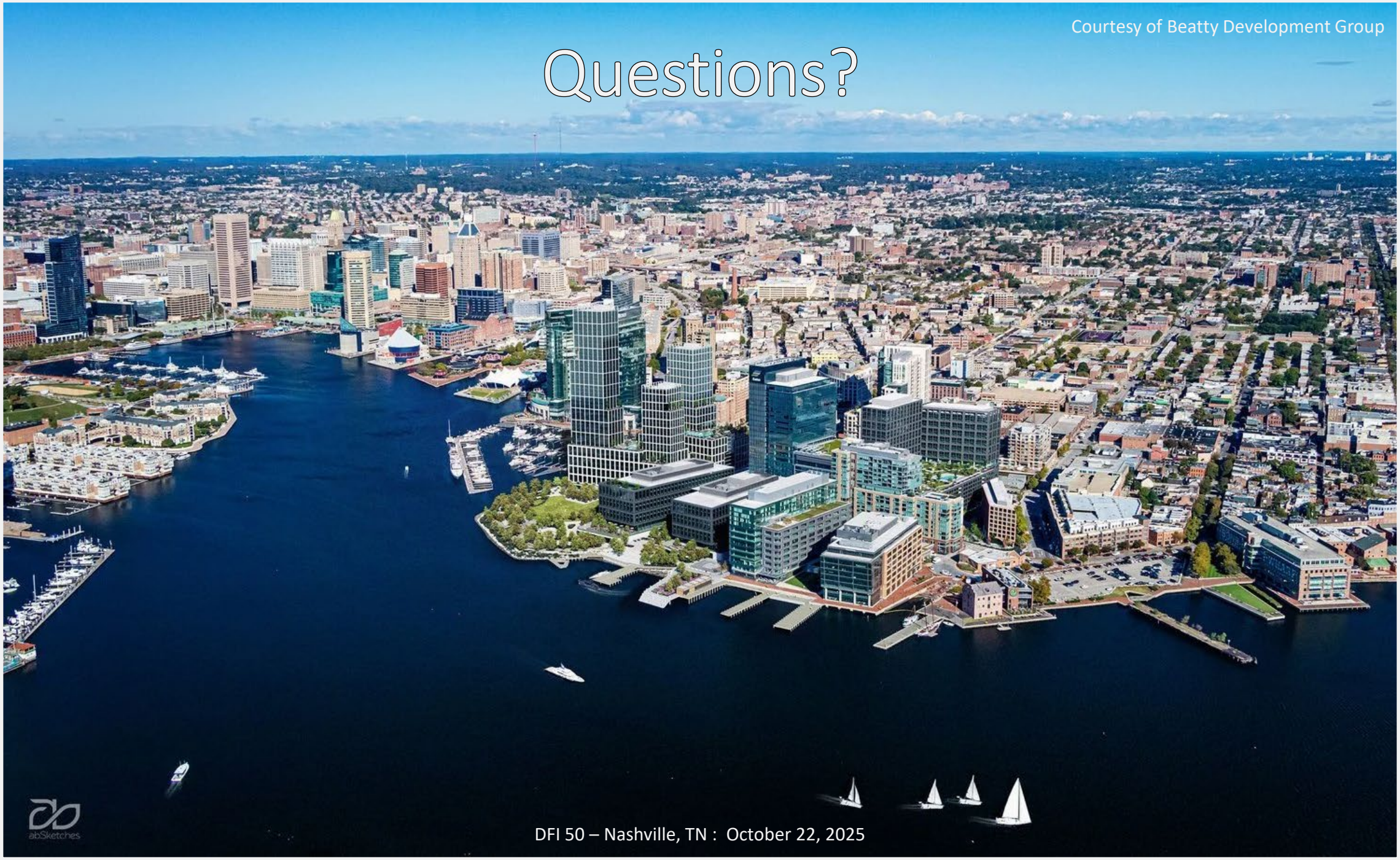
July 2025

Closing

- Work at the site over the past 40 years has been around to witness the full evolution of commonly used digital tools such as:
  - Slope/W, MODFLOW, and AutoCAD
- Implementation of these tools and methodical documentation of as-built conditions has provided the opportunity to take notice of the state of practice and the potential for beneficial adaption and inclusion of the available tools.

MRCE
built on firm foundations

Questions?



Courtesy of Beatty Development Group

PLENARY SESSION 4

Going for the Gold



Calculated Load-Transfer Responses from Atypical Strain Readings Using the Incremental Rigidity Method

Van E. Komurka, P.E., BC.GE, F.ASCE, Life Member (presenter)

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Seth Robertson, Ph.D., P.E. (coauthor)

GRL engineers, inc.

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Topics Outline

- Theory: From measured strains to calculated internal forces
  – BRIEF overview of the Incremental Rigidity method.
- Case history
  - Test shaft and instrumentation
  - Strain readings
  - Interpreted internal forces using the Incremental Rigidity method
- Conclusions

Converting Measured Strain to Calculated Internal Force

$$\Delta L = \frac{FL}{EA} \qquad \frac{F}{EA} = \frac{\Delta L}{L} = \varepsilon$$

$$F = EA\varepsilon$$

For a static loading test, the values of E and A may be unique at each strain-gauge level.

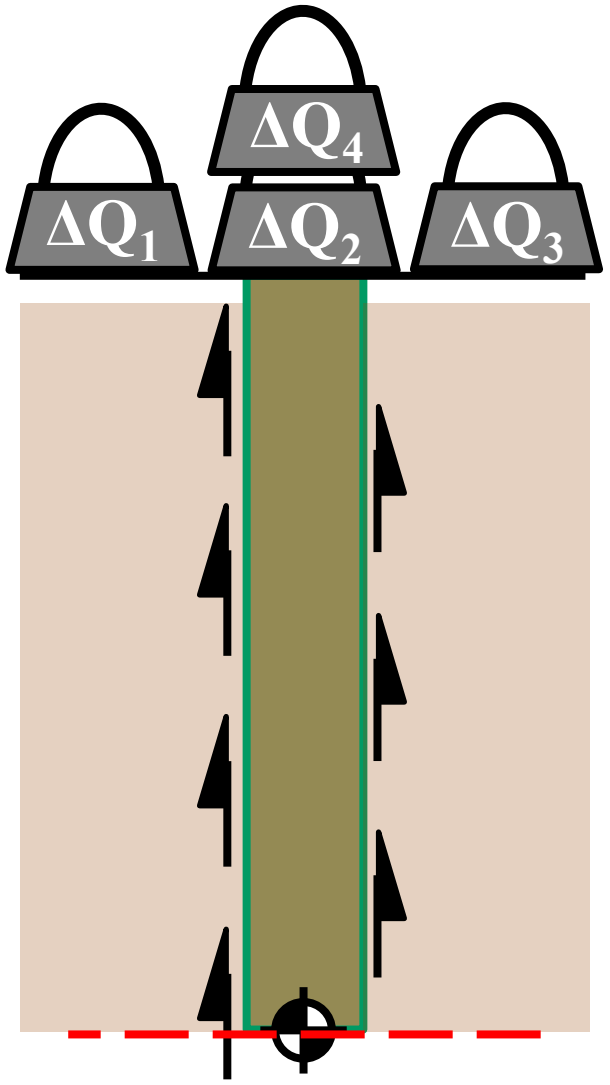
Conventional Method for Converting Measured Strain to Calculated Internal Force

$$F = EA\varepsilon$$

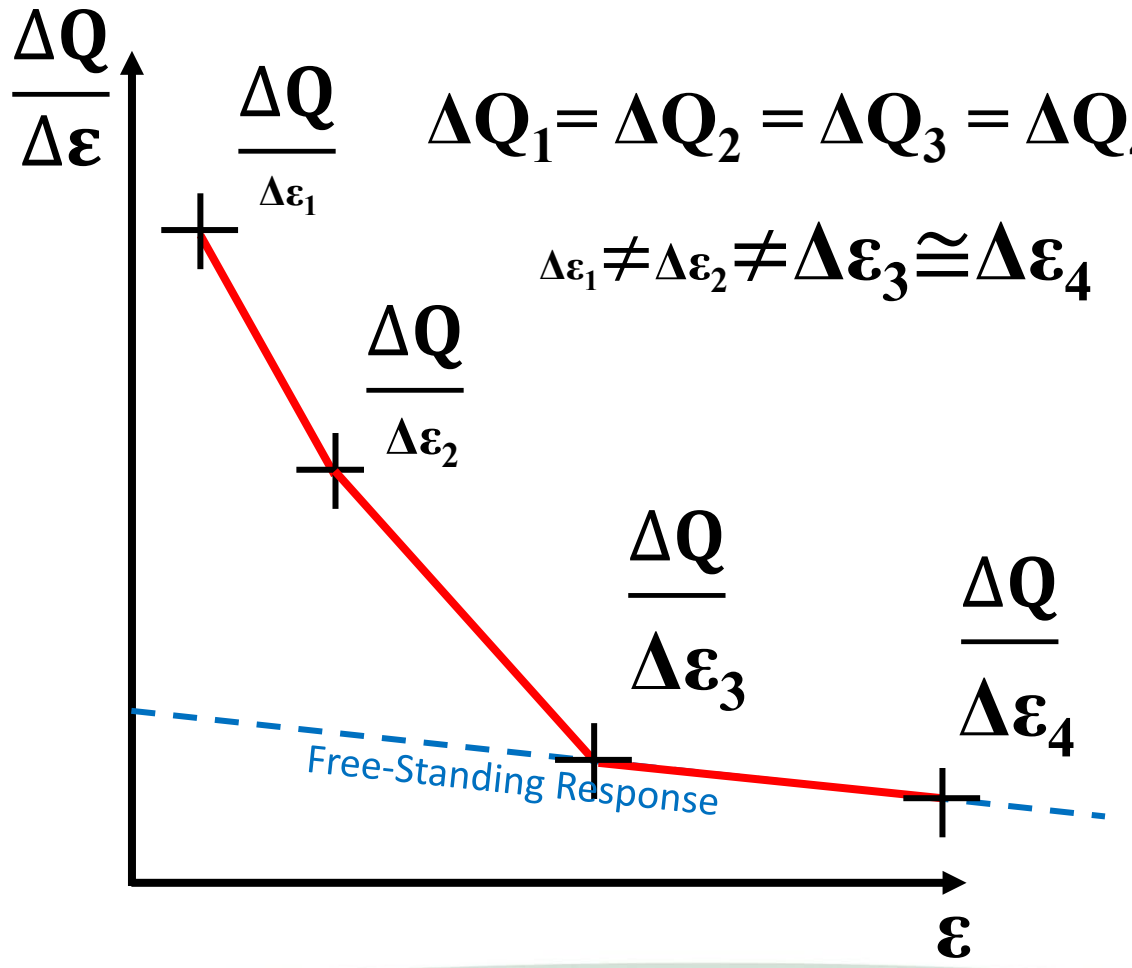
Noteworthy:

- Individual values of E and A are really not required, only their product EA is required.
- ***EA = Axial Rigidity***
- The Incremental Rigidity method determines the product EA.

Composite Pile Loaded Axially in *SOIL* – Load-Strain Behavior



$$\Delta\varepsilon_1 + \Delta\varepsilon_2 + \Delta\varepsilon_3 + \Delta\varepsilon_4$$



$$\Delta Q_1 = \Delta Q_2 = \Delta Q_3 = \Delta Q_4$$

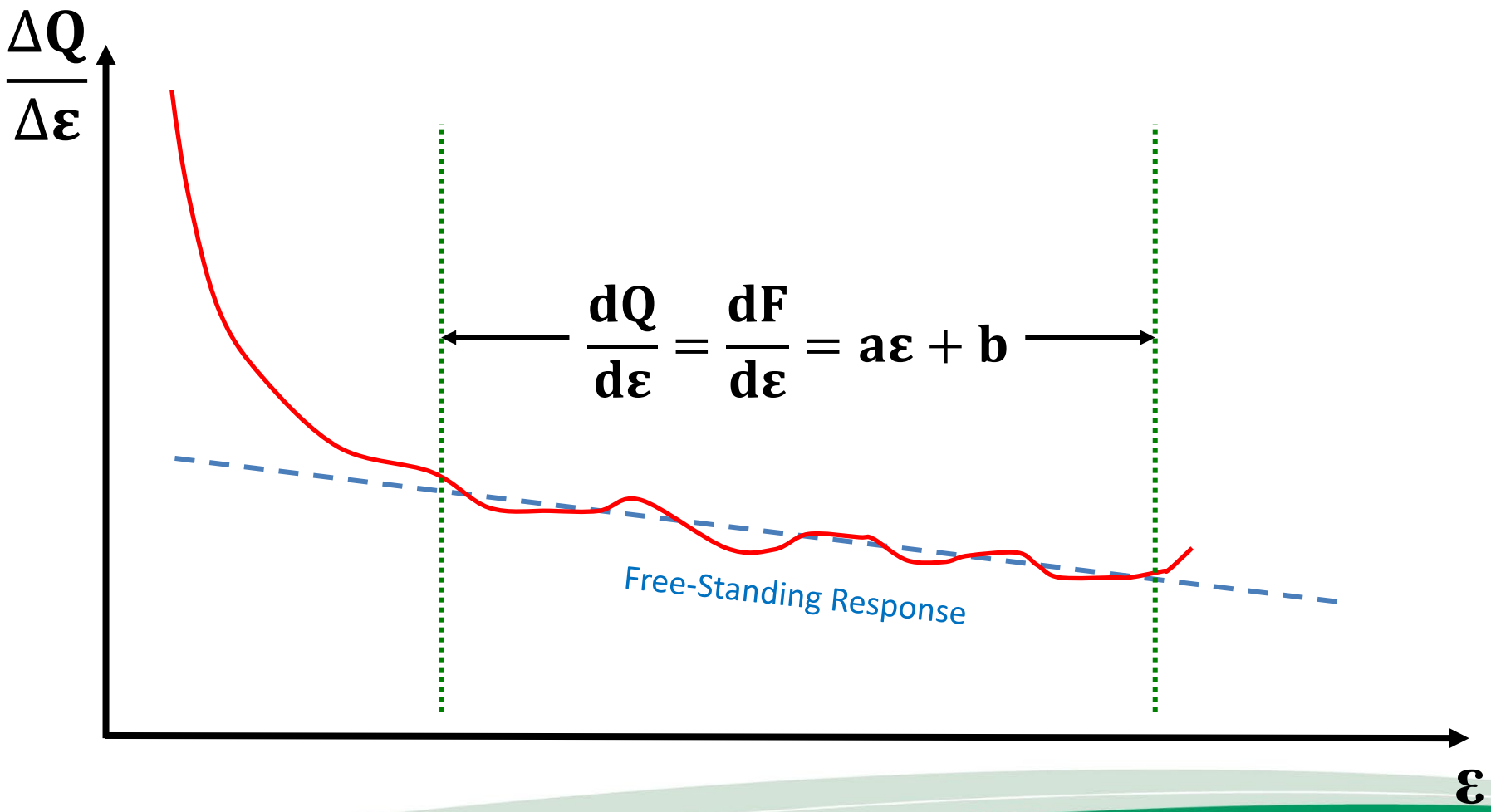
$$\Delta\varepsilon_1 \neq \Delta\varepsilon_2 \neq \Delta\varepsilon_3 \cong \Delta\varepsilon_4$$

Internal Force to Applied Load Relation Conditions

After all the shaft resistance above a strain-gauge level has been mobilized, the pile responds as a free-standing column above that strain-gauge level, and change in applied load is equal to change in internal force. Only data points after shaft resistance has been mobilized (i.e., after the ΔQ/Δε plot exhibits linear response) are used in the Incremental Rigidity method.

$$\frac{dQ}{d\varepsilon} = \frac{dF}{d\varepsilon}$$

Example ΔQ/Δε Plot for One Strain-Gauge Level



$$\frac{dQ}{d\varepsilon} = \frac{dF}{d\varepsilon} = a\varepsilon + b$$

$$F = \int (a\varepsilon + b) d\varepsilon$$

$$F = 0.5a\varepsilon^2 + b\varepsilon \quad [+c = 0]$$

Recall:

$F = EA\varepsilon$

$$F = \underbrace{(0.5a\varepsilon + b)}_{EA}\varepsilon$$

EA (if recorded ε is accurate)

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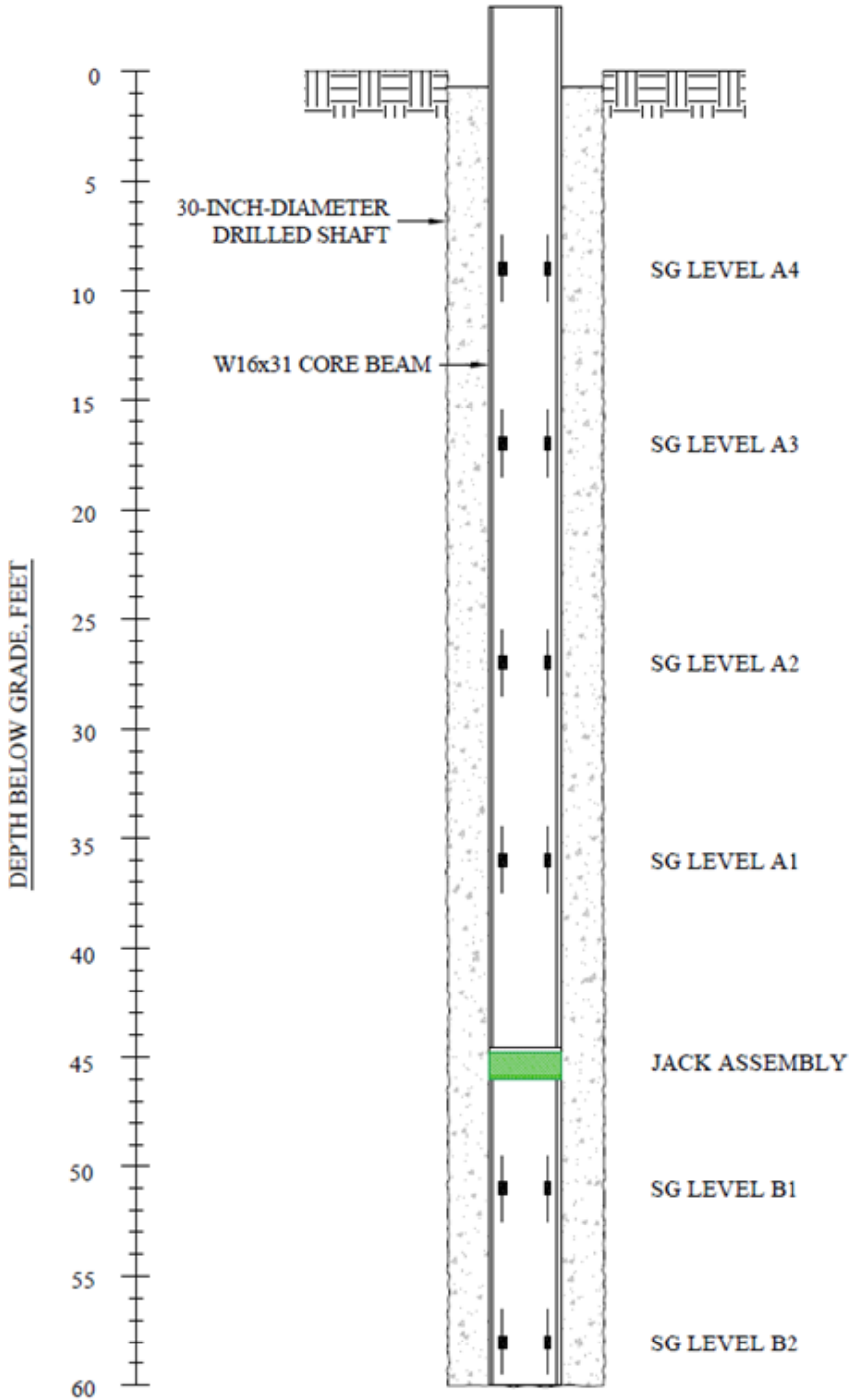
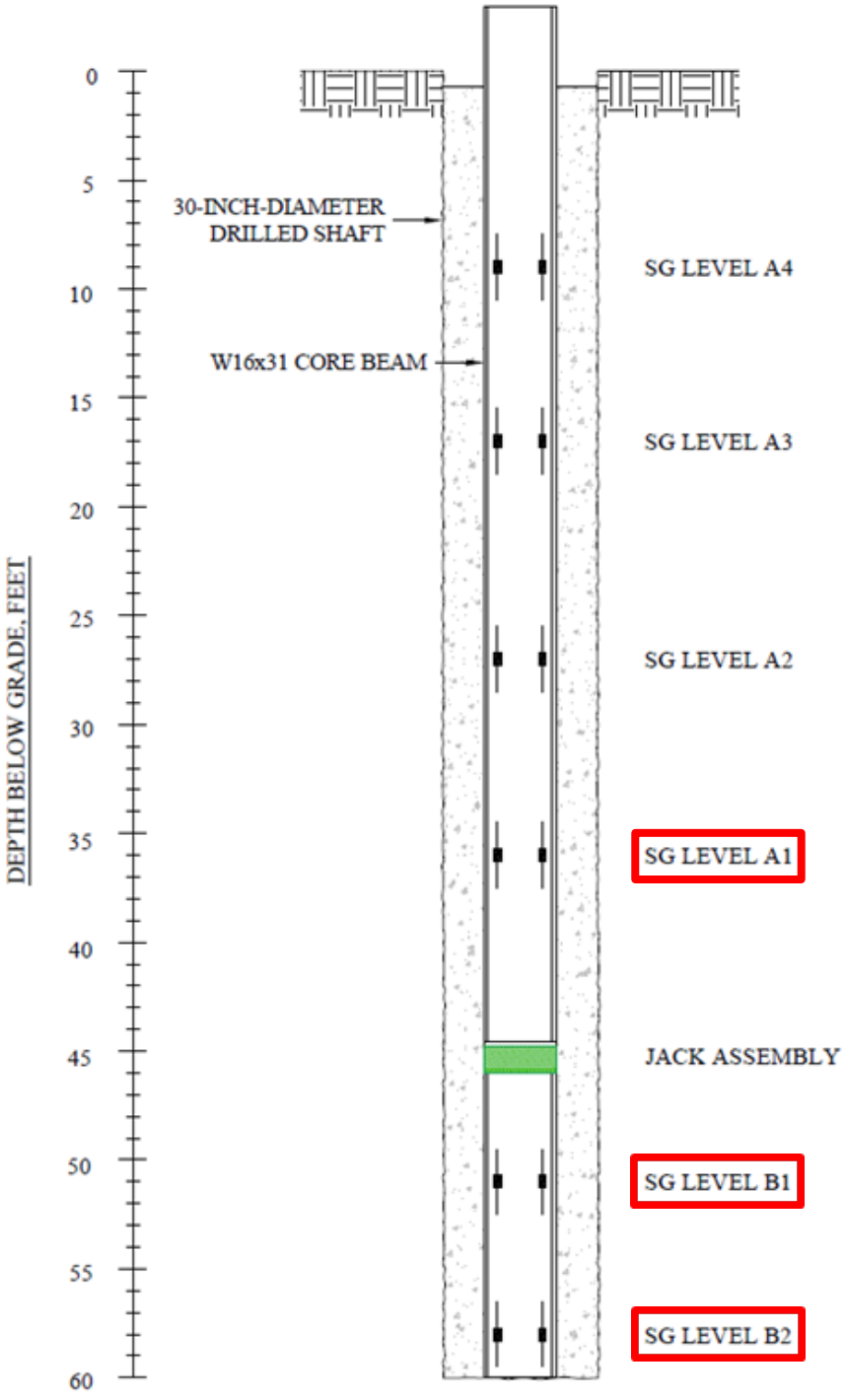
Bi-Directional Test Pile Details



- 30-inch-diameter concreted drilled shaft.
- Embedded length = 60 feet.
- Six embedded vibrating-wire “sister-bar” strainmeter strain-gauge levels (four above & two below jack assembly).
- Two strainmeters per level.
- W16x31 core beam welded to top and bottom of jack assembly.
- Strainmeters installed between flanges of W16x31 core beam.

Bi-Directional Test Pile Details





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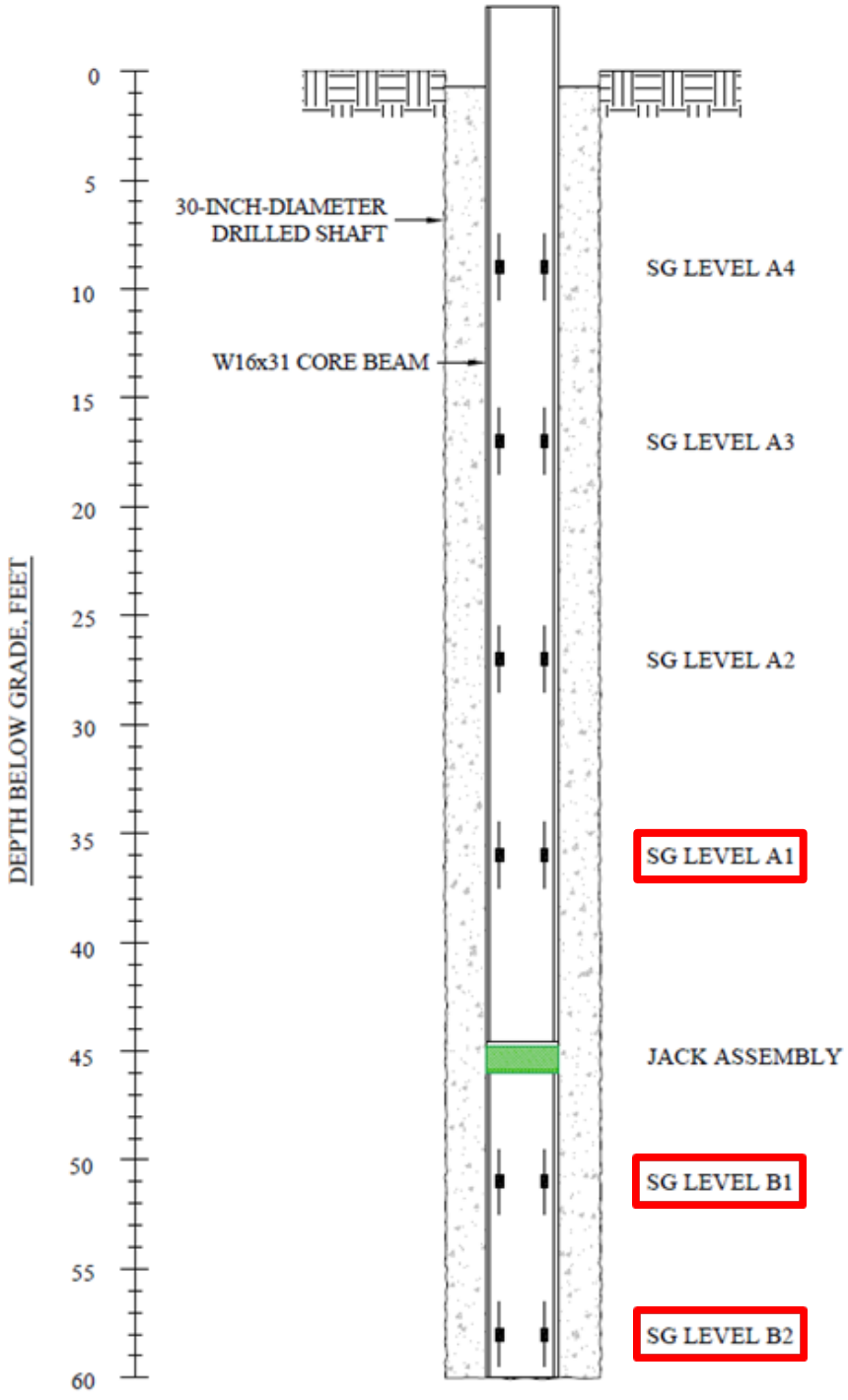
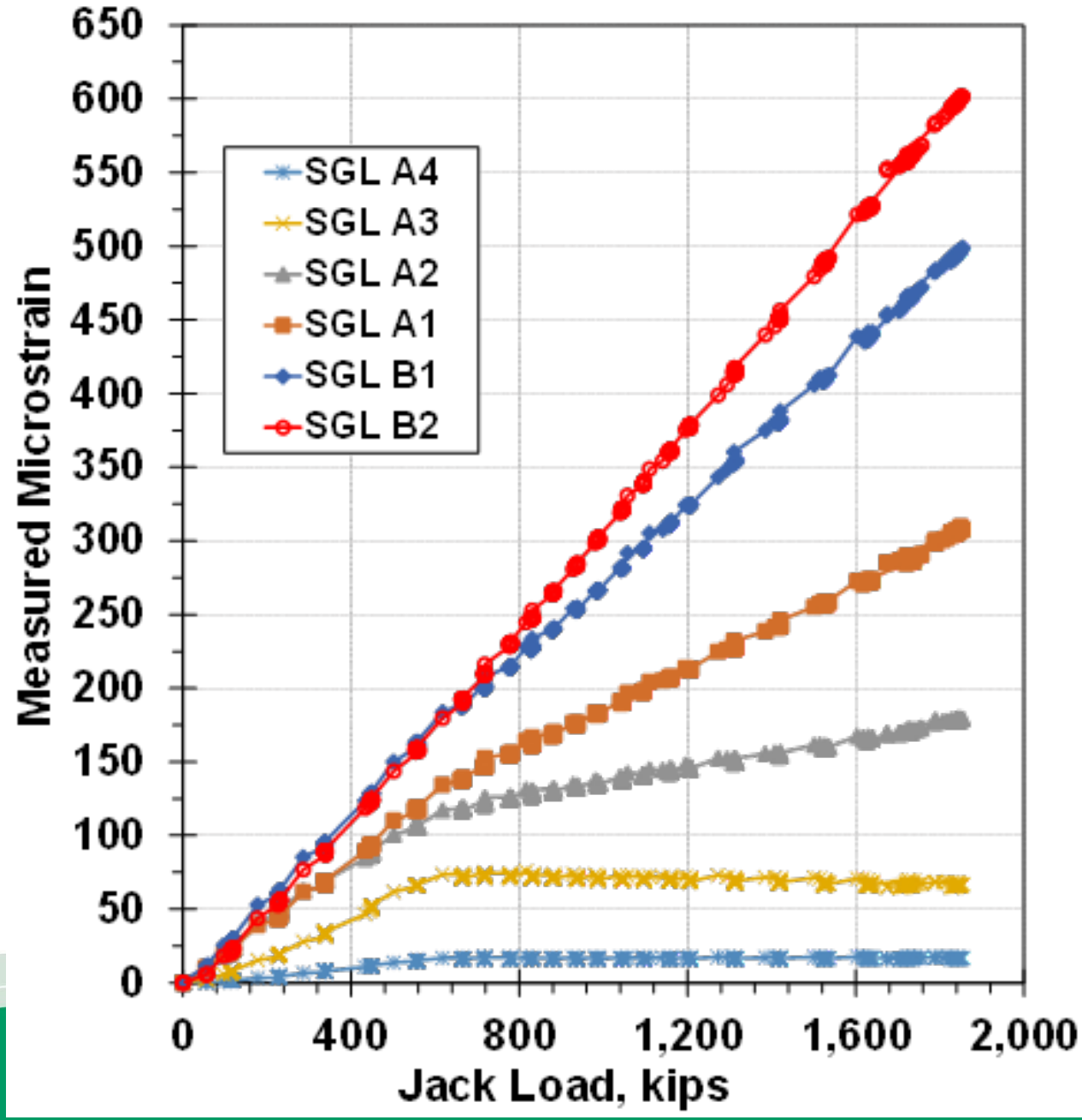
Internal Strain vs. Time of Day





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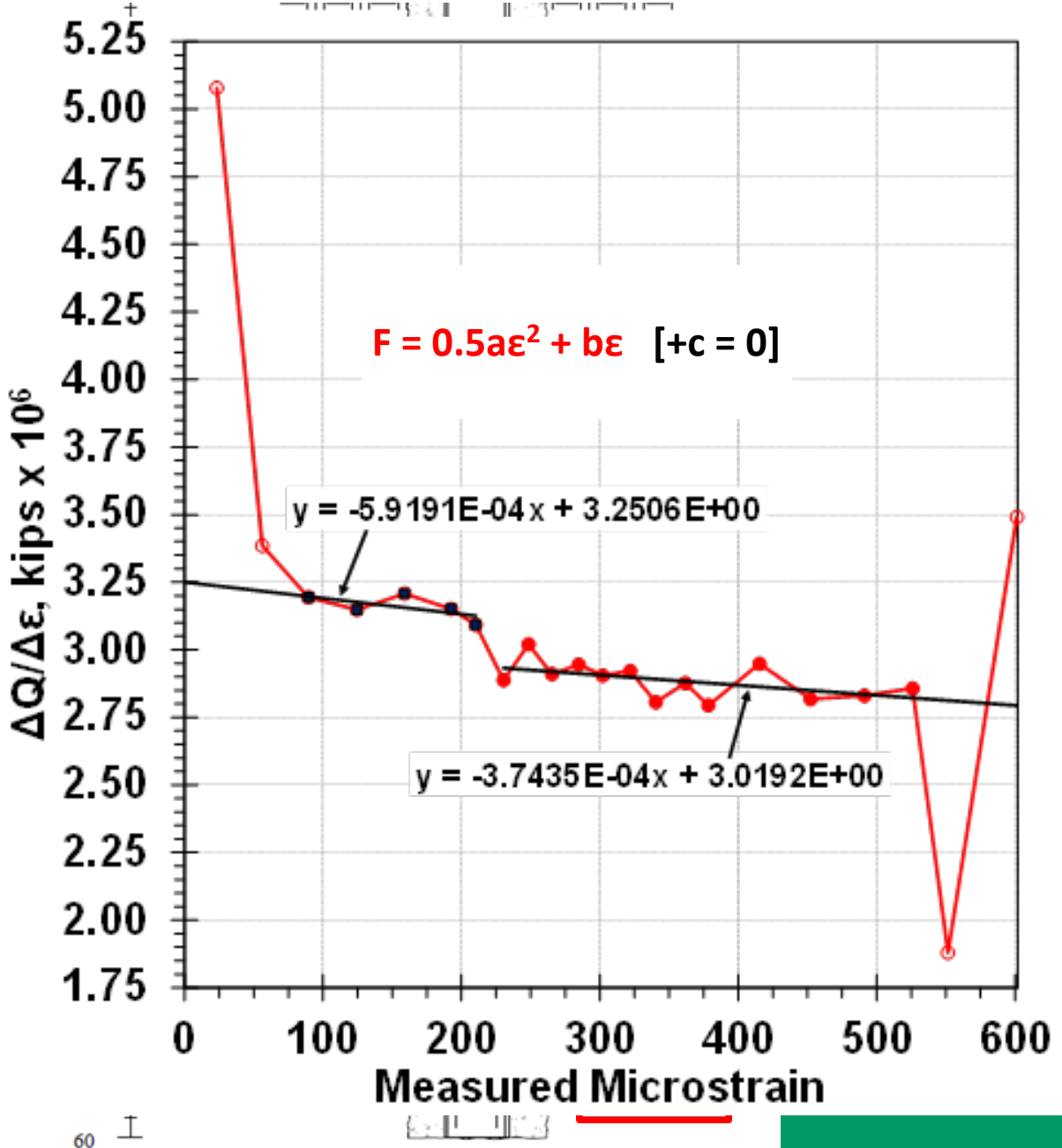
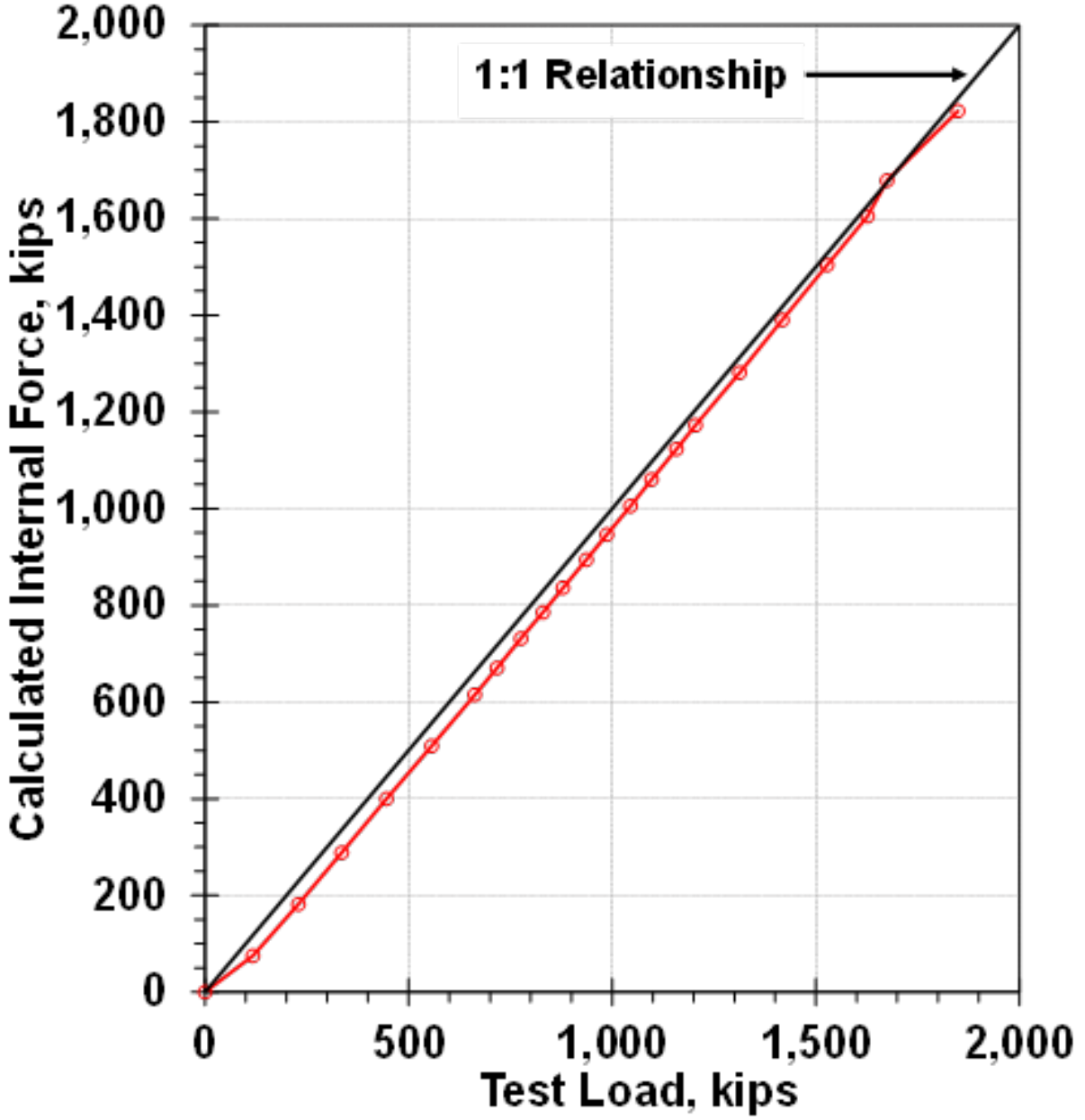
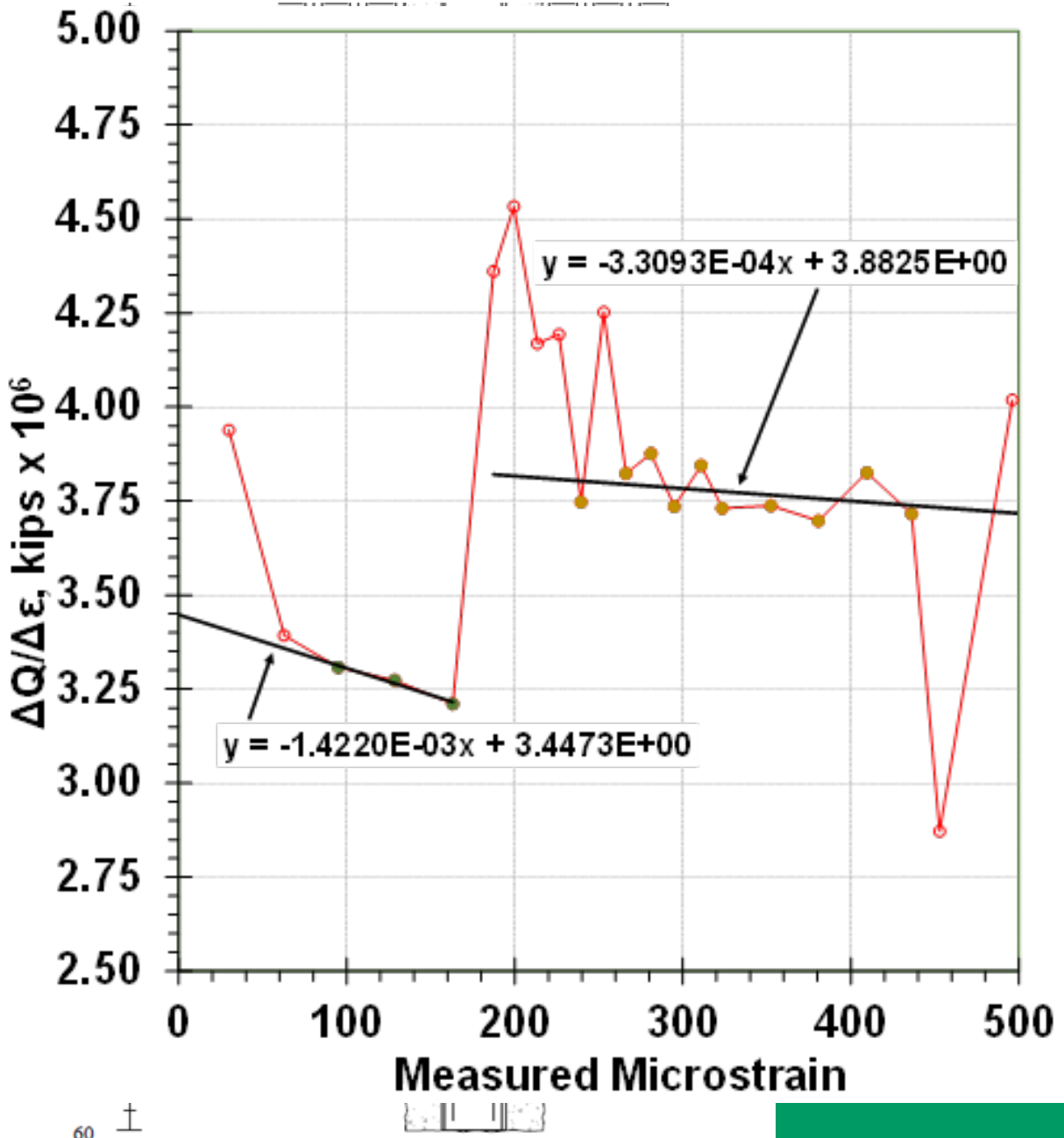
SGL B2 ΔQ/Δε vs. ε



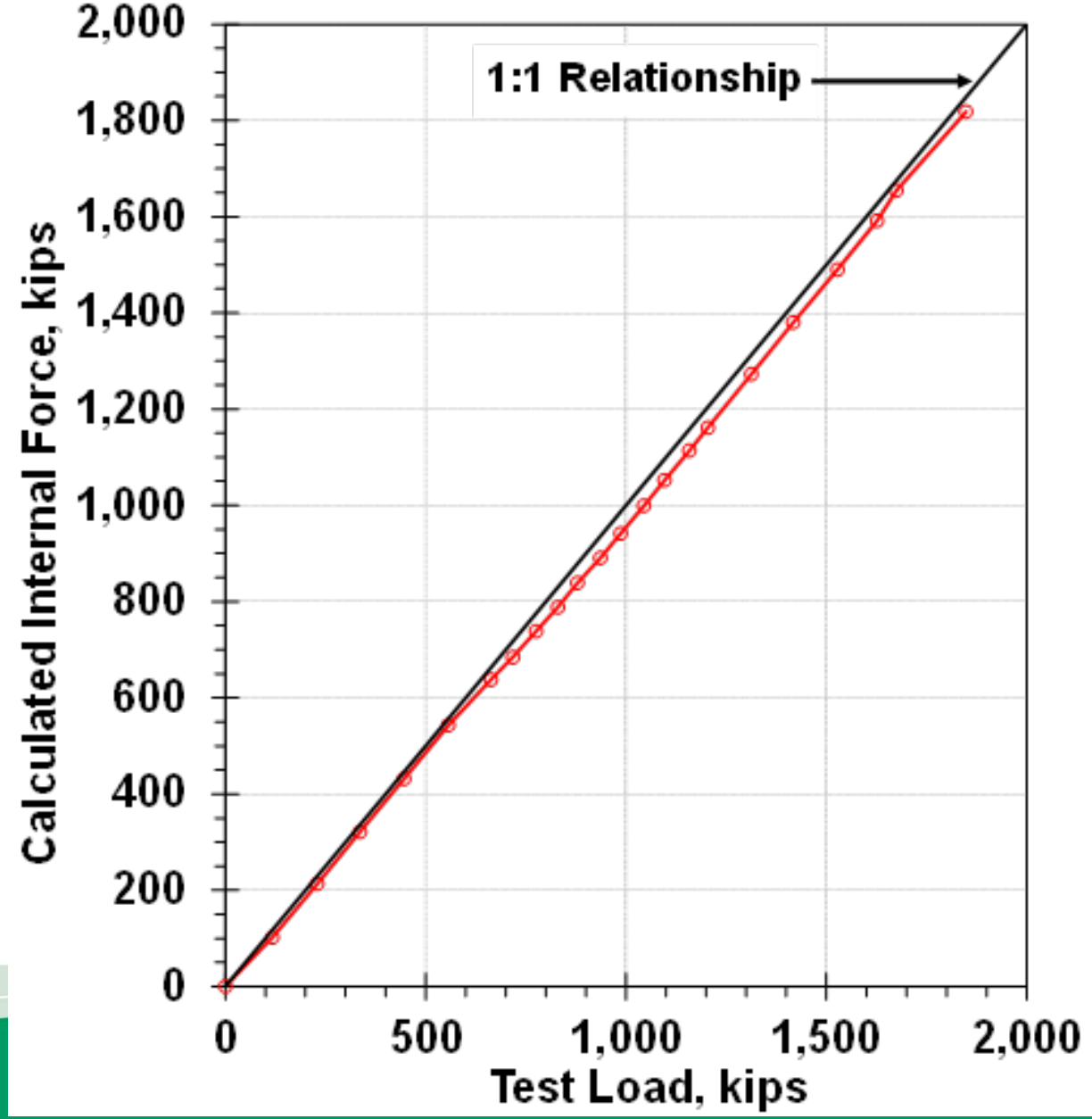
SGL B2 Internal Force vs. Test Load



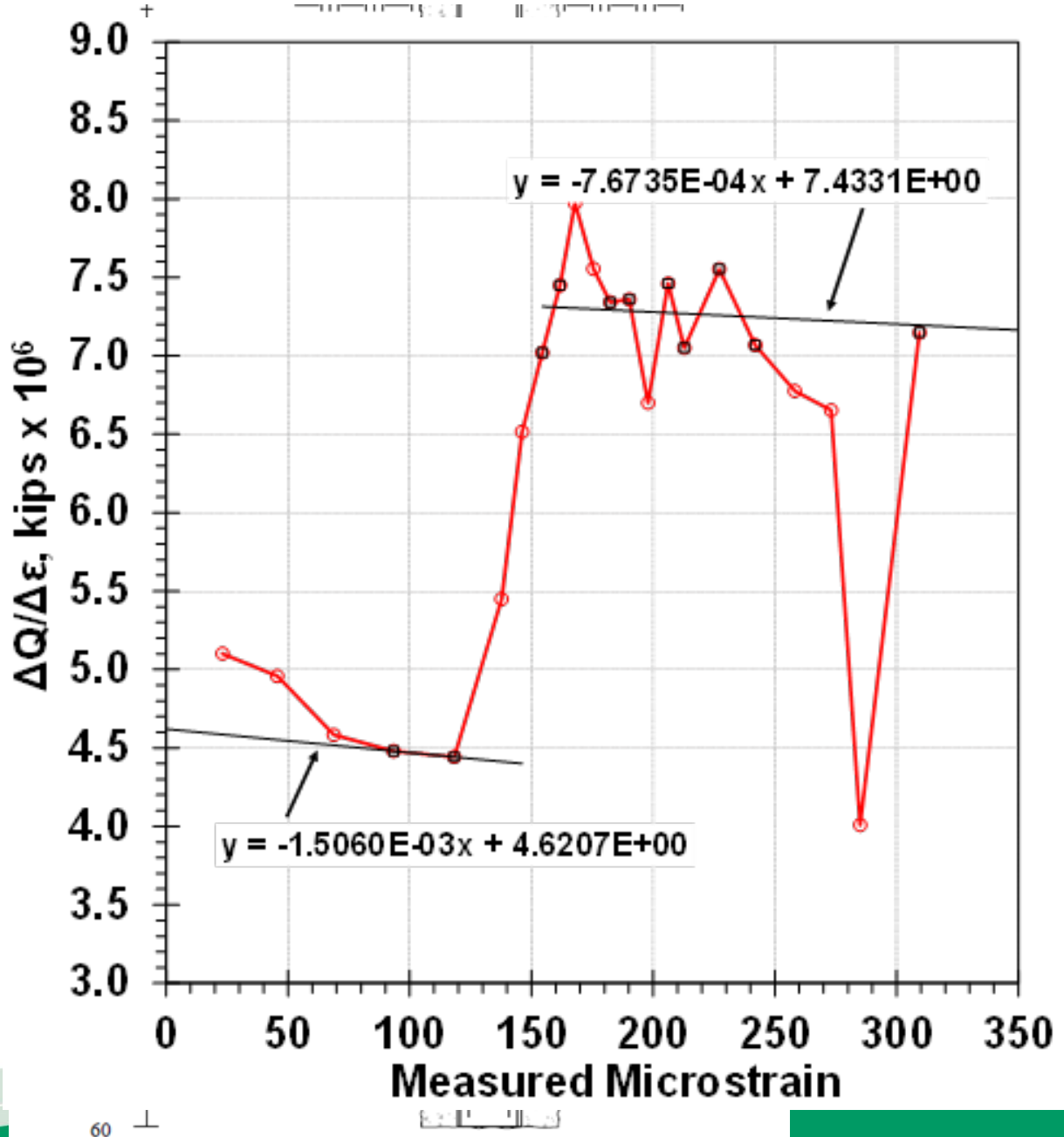
SGL B1 ΔQ/Δε vs. ε



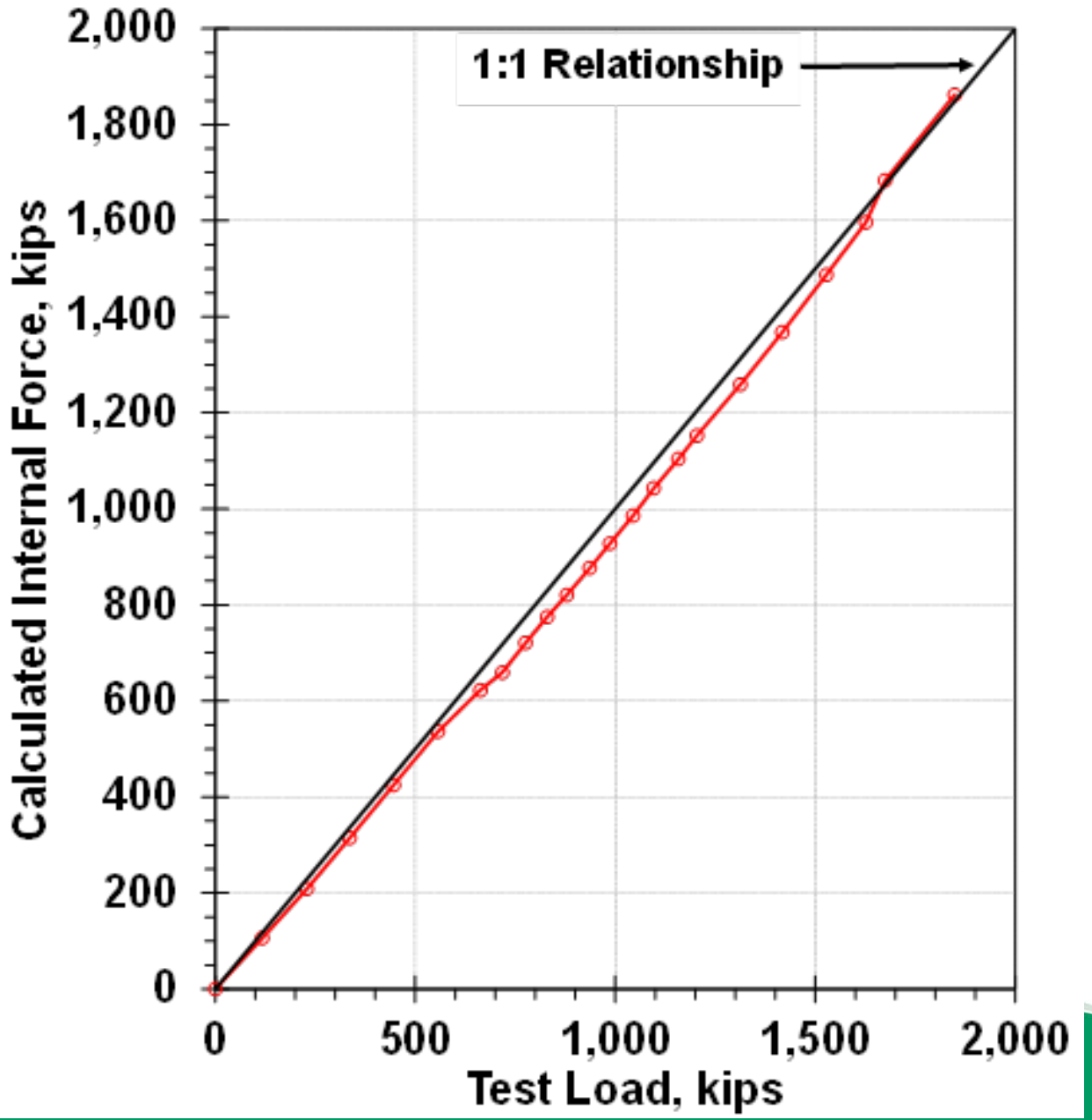
SGL B1 Internal Force vs. Test Load



SGL A1 ΔQ/Δε vs. ε



SGL A1 Internal Force vs. Test Load



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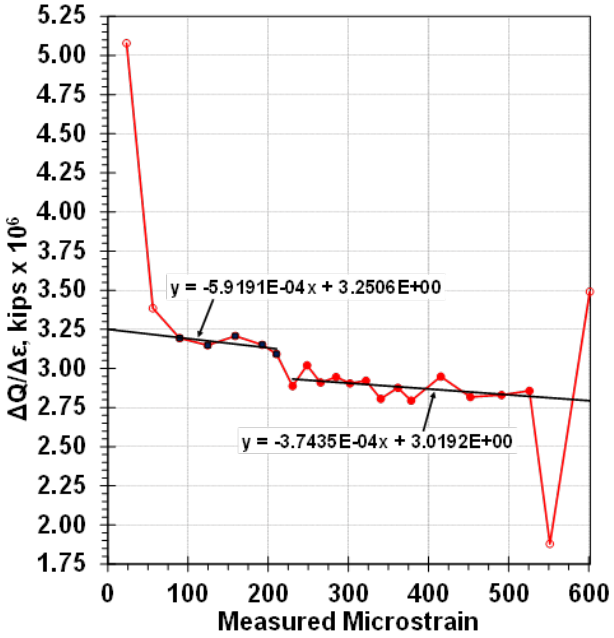
Conclusions

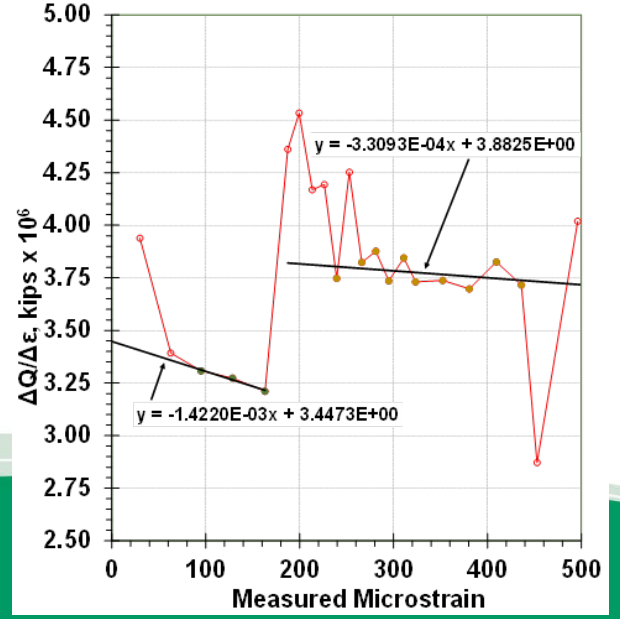
- **The Incremental Rigidity method can identify and account for a pile's change in rigidity (i.e., can model bilinear ΔQ/Δε response), and account for a strain-dependent cementitious elastic modulus, during a static load test to provide reasonable calculated internal forces. Note: Using the ACI formula to estimate cementitious elastic modulus can do neither.**

- **The Incremental Rigidity method can provide reasonable calculated internal forces from relatively "noisy" ΔQ/Δε vs. ε response (resulting from uncertainties in test loads, measured strains, resistance mobilization, and potentially the stress history of both the pile itself and surrounding material).**

- **The Incremental Rigidity method can provide reasonable calculated internal forces when other methodologies cannot.**

- **Mounting sister-bar strainmeters on a core beam for a bi-directional static load test risks obtaining atypical strain readings, and so should be avoided.**





Thank You!

Questions, Comments, Discussion?

Van E. Komurka, P.E., BC.GE, F.ASCE, Life Member

Senior Engineer, GRL Engineers, Inc.

vkomurka@grlengineers.com

GRL engineers, inc.



Bauer Maschinen GmbH – Cementation Skanska

Low Carbon Initiatives at Euston Approaches, HS2 –Environmentally Friendly Pile Installation in London, UK

Deon Louw, Operations Director, Cementation Skanska

Robert West, Technical Manager, Cementation Skanska

Franz-Werner Gerressen, Director Method Development, Bauer Maschinen GmbH

2025-10-23

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Agenda

No.	Agenda topic
1	Introduction
2	Low Carbon Guide Walls
3	High Replacement Concrete
4	Electric Driven Rig
5	Zero Trim



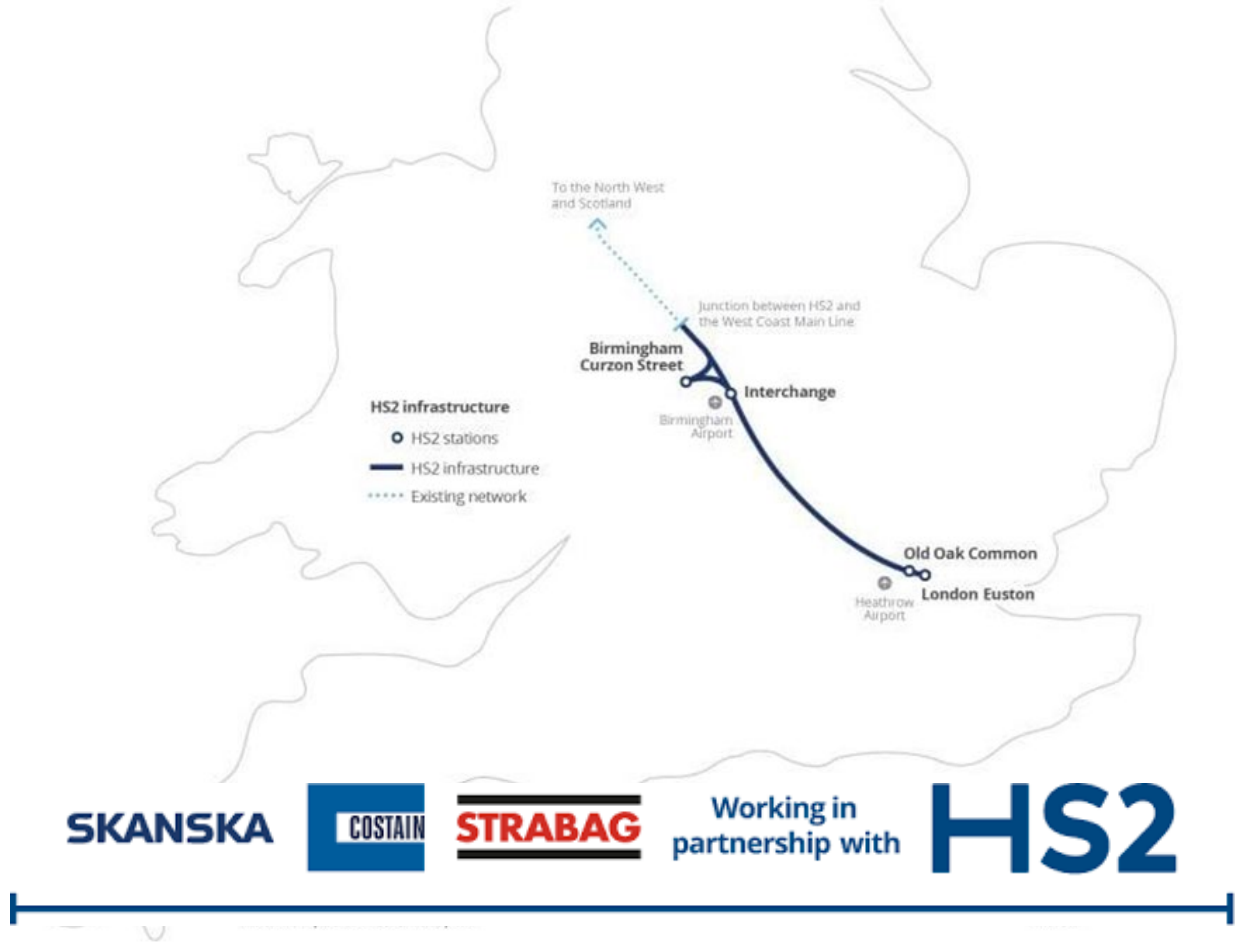
1

Introduction

Introduction

HS2 Overview

- The HS2 project will deliver:
  - 140 miles of new high-speed line
  - 4 new state of the art stations
  - Two depots
  - 32 miles of tunnel
  - 130 bridges
- Connects the West Midlands, London and the Southeast
- Expected to open early 2030s
- *“HS2 will leave a legacy of environmentally responsible travel, economic regeneration and technological innovation for generations to come”*

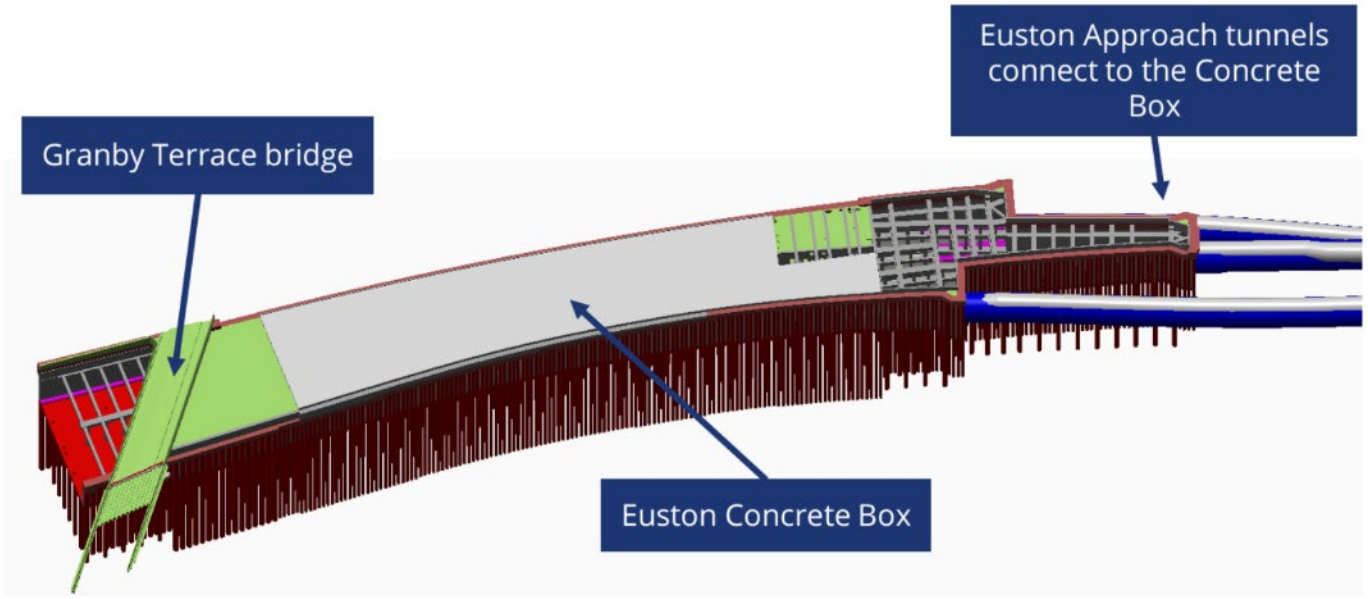


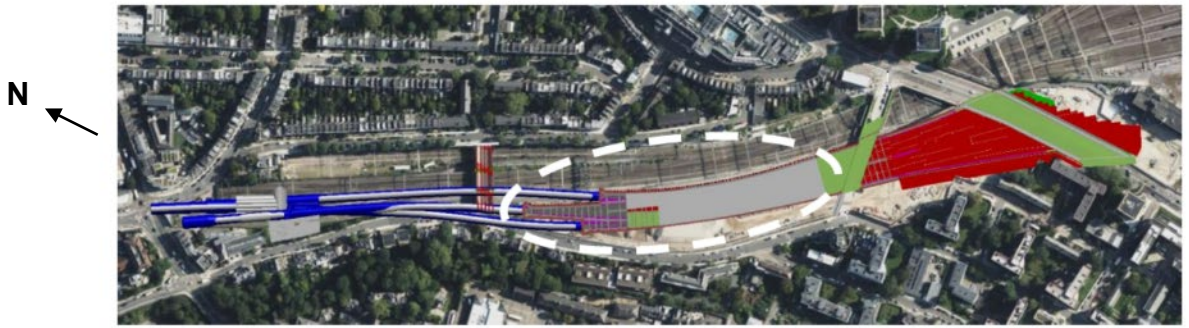
https://assets.hs2.org.uk/wp-content/uploads/2024/02/DIGITAL_HS2_route-map_020224-1-900x600.jpg

Introduction

Euston Approaches – Piling Contract

- 5 Structures
  - Granby Terrace Bridge (GTB)
  - Euston Scissor Box (ESB)
  - Euston Throat Retained Cut (ETRC)
  - Cavern Vent Shaft (CVS)
  - Hampstead Road Bridge (HRB)
- Approximately 2000 rotary bored piles
- Pile diameters of between 750 – 1800mm
- ~1300l/m of contiguous bored pile retaining walls
- Majority of piles installed under a polymer support f depths of >50m
- Striving to achieve low carbon solutions to typical s operations, several innovations were conceived…





https://assets.hs2.org.uk/wp-content/uploads/2021/08/Design-Update-Mar-2022-online-version.pdf



2

Low Carbon Guide Walls

Low Carbon Guide Walls

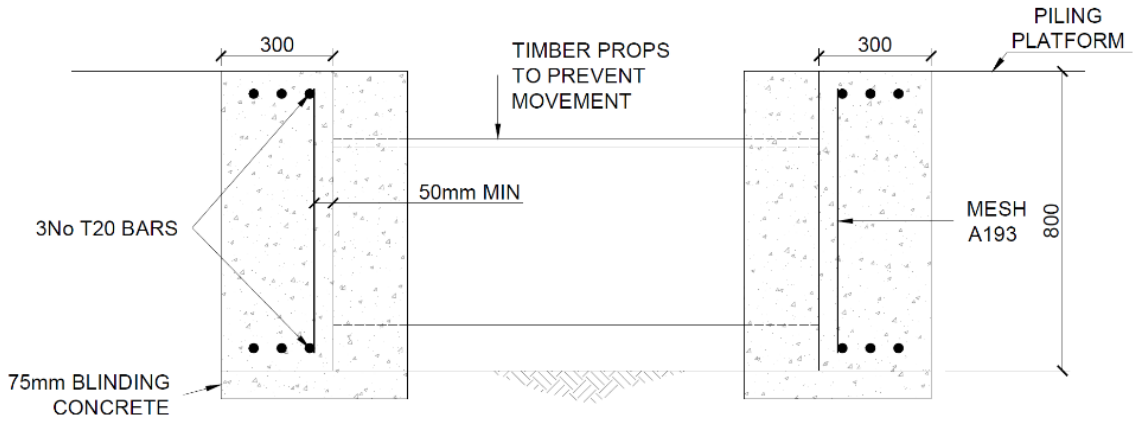
Traditional Guide Wall

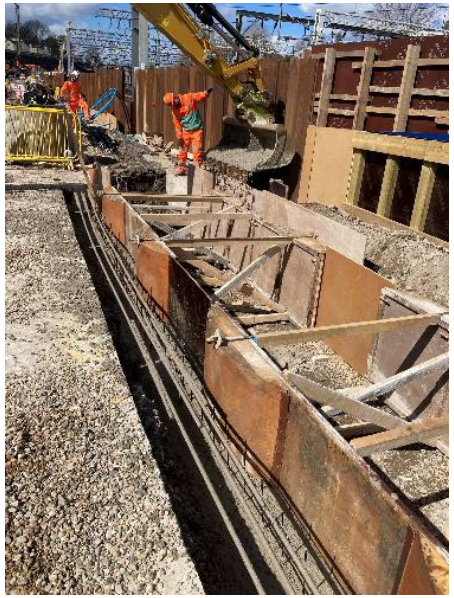
What is a guide wall?

- Guides the piling equipment and improves the accuracy.
- Improves the plan tolerance, ±75mm > ± 25mm
- Guides the pile casing helping to achieve the verticality tolerance

Traditional construction method:

- Low strength concrete (C20/25)
- Longitudinal steel reinforcement
- Steel mesh along the faces







Low Carbon Guide Walls

Low Carbon Guide Wall

Basalt Fibre Reinforced Polymer (BFRP), what are the headline messages?

- BFRPs global warming potential can be up to 74% lower than steel
- Boasts tensile strengths of 870MPa to 1150MPa
- Lightweight, easier to handle
- Can be cut with conventional tools, no sparking
- Its versatile, can be produced to typical shape codes
- Costs more than steel
- Poor compressive strength



Low Carbon Guide Walls

What were the savings?

- 50% of embodied carbon life cycle is present in the A1 to A3 or cradle to the gate.
- Simply swapping reinforcement bars for BFRP can save around 30% of embodied carbon
- Not explicitly covered in current European guidance, limited to temporary works?

Type	Area of Steel (m^2/m)	Density (kg/m^3)	Weight (kg/m)	A1-A3 ECF ($kgCO_2e/kg$)	Embodied Carbon
Steel	0.469	7750	36.37	0.787[1]	28.62
Basalt, BFRP	0.469	2000	9.39	1.820[2]	17.08

Notes:
[1] CARES, 2023
[2] Basalt Technologies UK Ltd, 2022

Low Carbon Guide Walls

Concrete Mix

Type	Volume per m run of guidewall (m^3/m)	Density (kg/m^3)	Weight (kg/m)	Embodied Carbon of concrete, A1-A3 [1](kg/m)	Embodied Carbon A1 – A3 ($kgCO_2e$)
CEMIII/A 50% GGBS (C20/25)	0.48	2293	1100	0.0478[1]	51.8
CEMIII/B 73% GGBS (C20/25)	0.48	2364	1134	0.0355[1]	40.3

Notes:

[1] ICE Database – Embodied Carbon Model of Cement, Mortar and Concrete (Jones & Hammond, 2011)

Small changes…significant gains.



3

High Replacement Concrete

High Replacement Concrete

Concrete Development Process @ Euston Approaches

- Approach concrete suppliers Technical Teams
- Developed initial mix designs in three cement content categories
- Schedule laboratory trials
- Mix selection for progression to plant trials
- Plant trials, and further proof-trialling on adjustments to mixes (retention etc)
- Site trials (trial box and 4 No anchor piles)
- Confirmatory durability trial further to minor adjustment (more on this later)
- MCR Submitted for approval



High Replacement Concrete

High Replacement Concrete Mix

- Initial trials:

- Different cement contents
  - 380 – 420 kg/m3
  - 435 kg/m3
  - 485 kg/m3

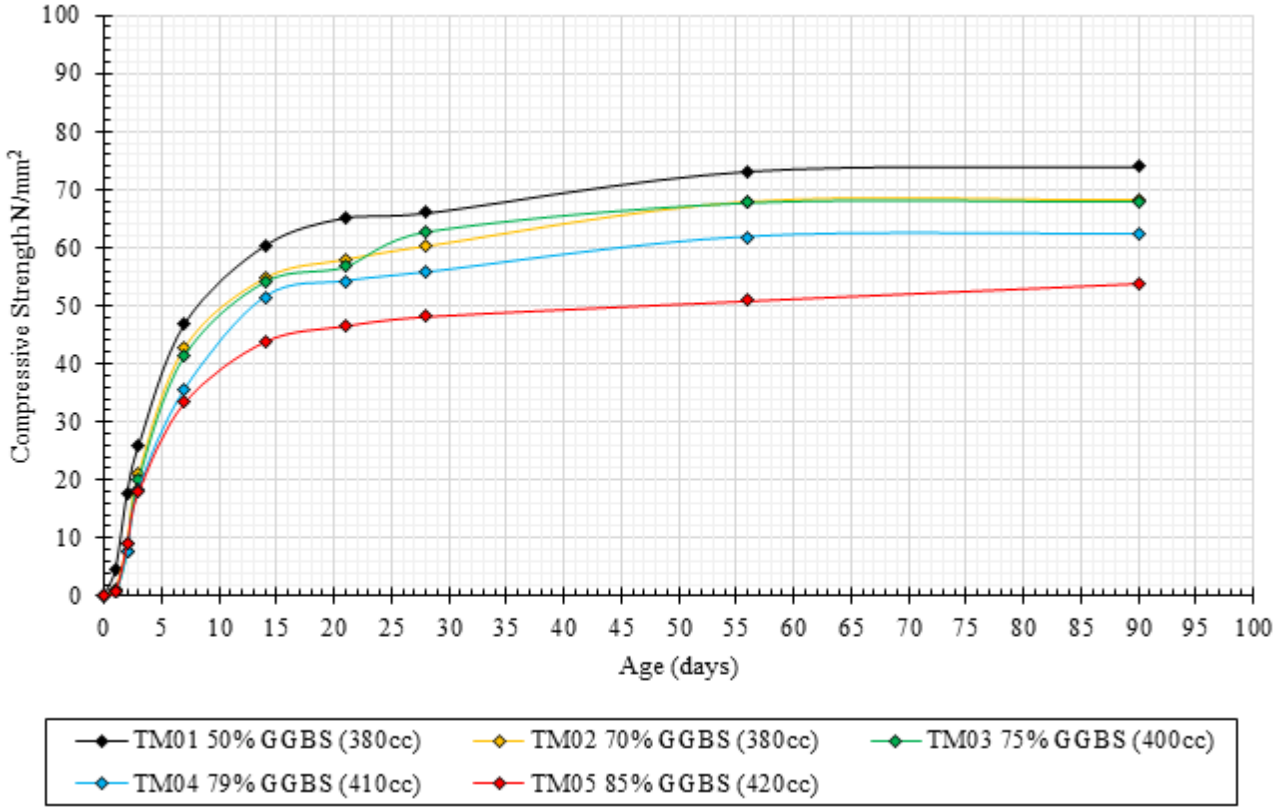
- Variable GGBS Content 70 – 90%

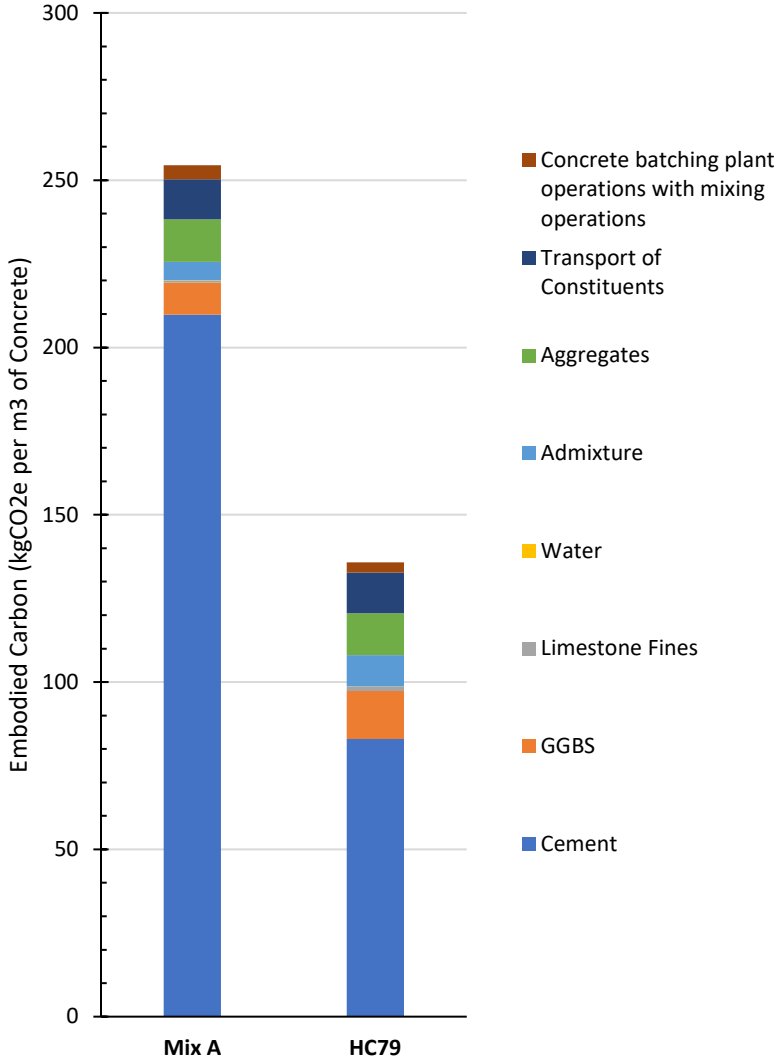
Mix Specification:

Strength:	**C40/50**
Cement Class:	**CIIIA + SR**
Consistence:	**F5**
MCC:	**380**
Maximum Water/Cement Ratio:	**0.38**
Carbonisation Class:	**XC2**
Chloride Class:	**XD2**
Chemical Class:	**DC2**

High Replacement Concrete

HC79 Concrete Mix







4

Electric Driven Rig

Electric Driven Rig



eBG 33

Electrification for high-demand applications

- Development in consideration of high demand applications such as FDP or CCFA
- Power consumption required not warrant use with a battery pack
- -> development of wired solution to meet full shift requirement with zero loss of power

Electric Driven Rig

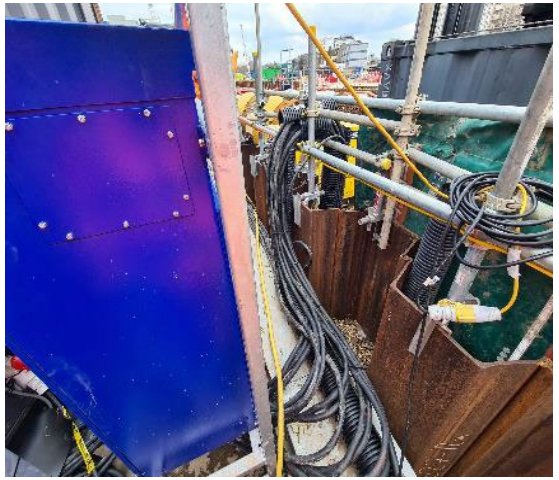
Scope of work eBG

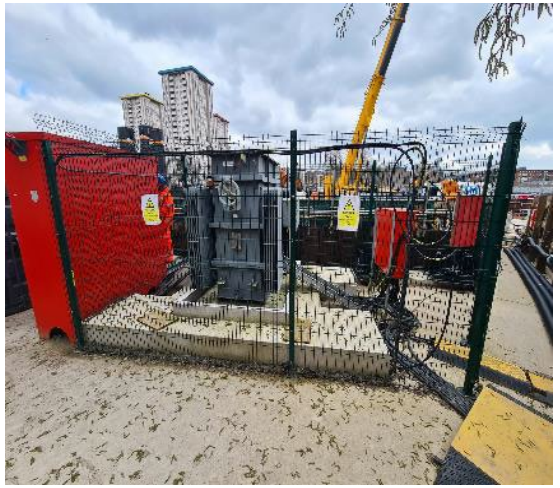
- Drilling Diameter:
  - 1800 mm
- Drilling depth:
  - 46 m
- Drilling procedure:
  - Starter casing for 6 m, diameter 2000 mm
  - 42 m in London Clay, dry drilling
  - slurry supported to final depth in Sand

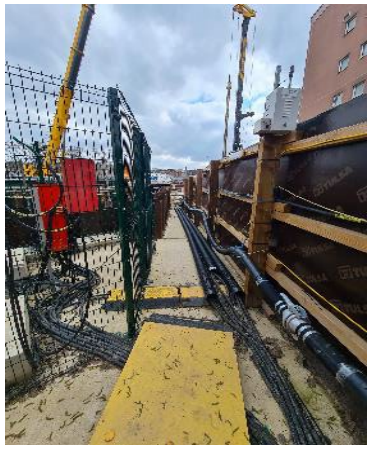


Electric Driven Rig

Cable organization on site

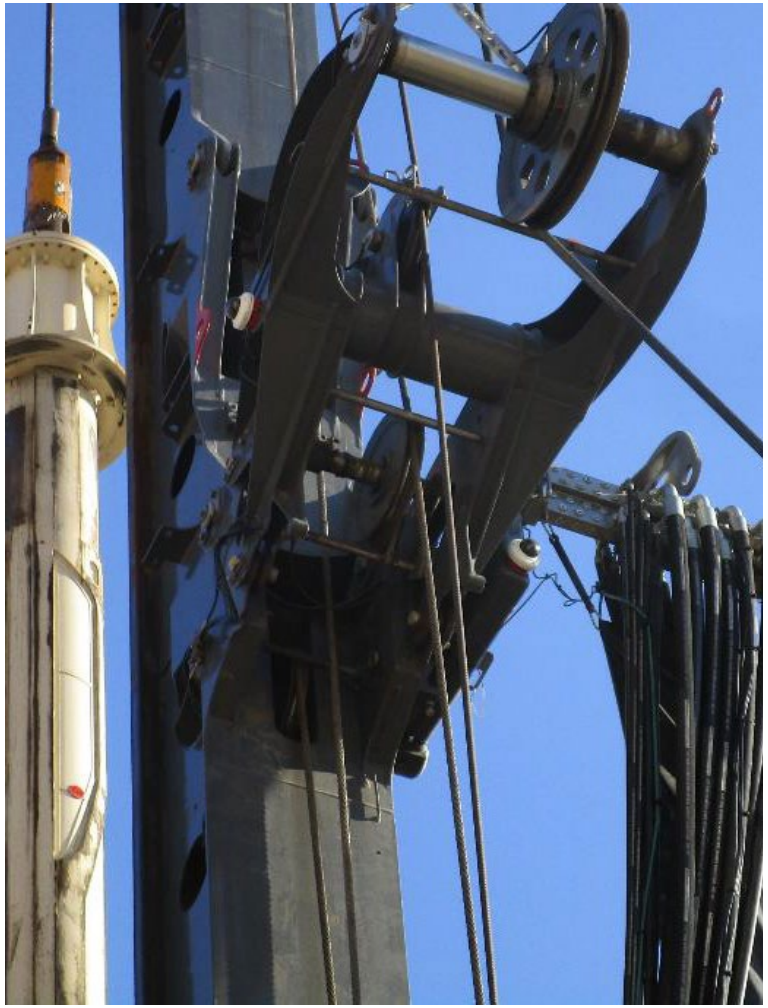


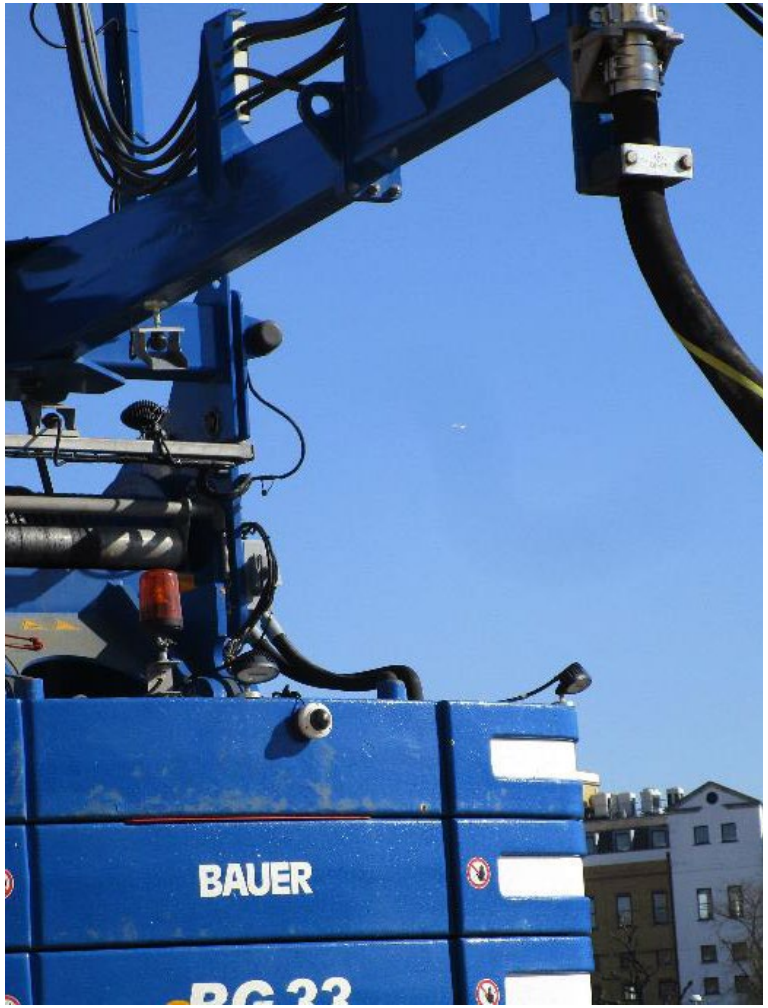




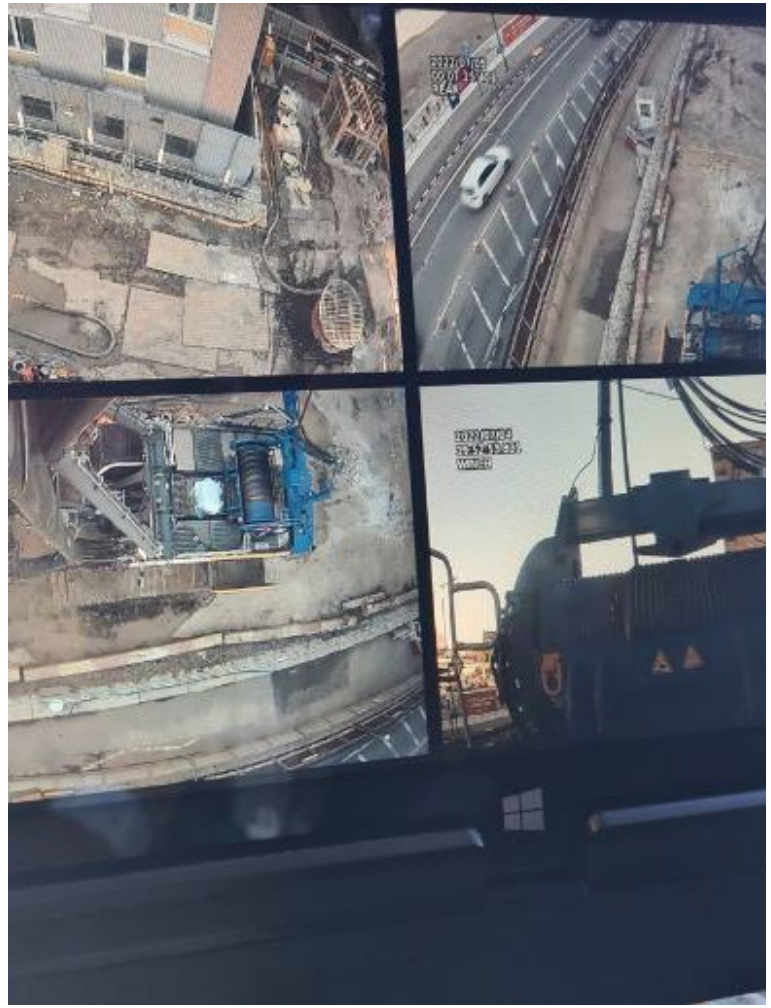
Electric Driven Rig - Periphery monitoring



Three cameras on mast



One camera at counterweight



Real time control

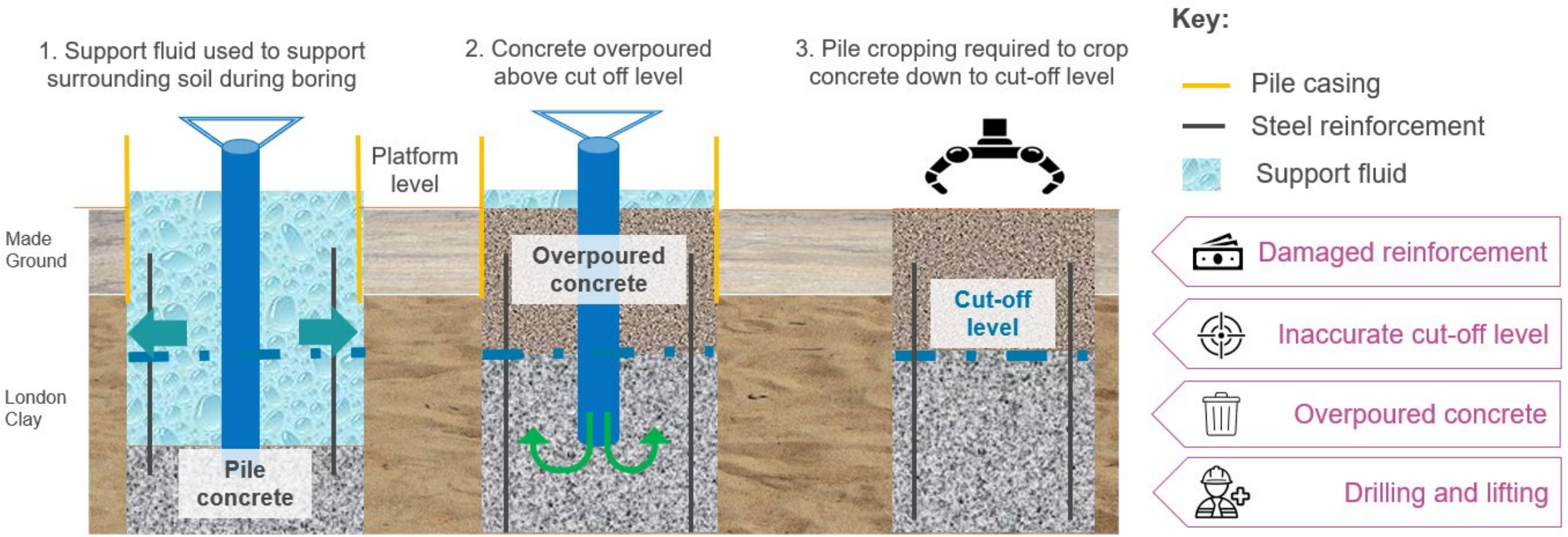


5

Zero Trim

Zero Trim

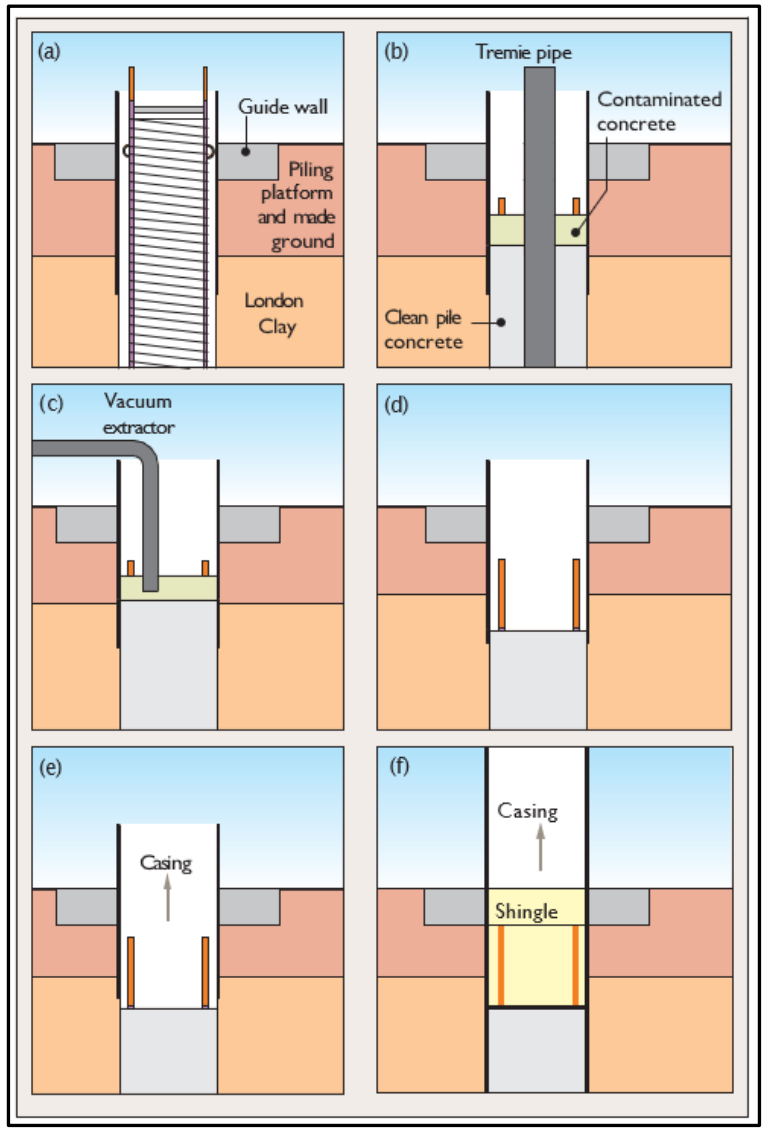
Traditional process and challenges



Zero Trim

Procedure

- Concrete poured corresponding to traditional methods
- Overbreak is removed using vacuum excavation to within 40mm of pile casting level
- Bore filled with shingle
- Pile complete.



Zero Trim

Social and Environmental impact

1.1 million kg Co2 saved from reduced over pour

60,000 manual hand breaking hrs eliminated

*"HS2 contractors SCSjv were committed to reassessing the piling techniques that have been in use for the past two decades." …"From a noise perspective, the "zero trim pile technique" has been a **success** and hopefully it will encourage other contractors to incorporate this technique into future projects."*

London Borough of Camden – Camilo Castrol-Llach (Noise officer)

Risk	Zero Trim Pile	Pile trimming using hand tools	Pile breakdown using excavator	Pile cropping using drill and burst
Silica / dust				
Noise				
Hand-arm vibration				
Musculo-skeletal disorders				
Lifting				





Zero Trim

Commercial and program impact

Zero Trim Piles	Traditional drill and burst method
• Zero-Trim = 30 minutes per pile • 1 vacuum excavator + 2 operatives • Used across multiple assets	• Pile cropping = 1.5 day per pile • Multiple gangs and plant required to work in parallel

Significant commercial savings expected across Area East

Approx. one year programme savings in cropping activity

Safe guarded railway blockade





Zero Trim

Which one will you choose?





Conclusion

Element	Type	Total Carbon Saving (tCO_2e)
Low carbon guide walls (A1 – A3)	Actual & forecasted	38110
Electric driven rig (A5a)	Actual[1]	28
Low carbon concrete (A1 – A4)	Actual & forecasted	4272
Zero Trim	Actual & forecasted	800

The project has demonstrated that small, considered, changes to everyday operations can have a significant impact on reducing the overall carbon emissions of a project.



Cementation SKANSKA

BAUER

Passion for Progress

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HALEY ALDRICH

Modeling to Monitoring: Evaluating Adjacent Structures Feet from a Boston Excavation

Tyler Poggiogalle, P.E.
Damian Siebert, P.E.
Wystan Carswell, P.E.

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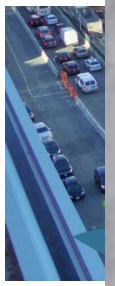
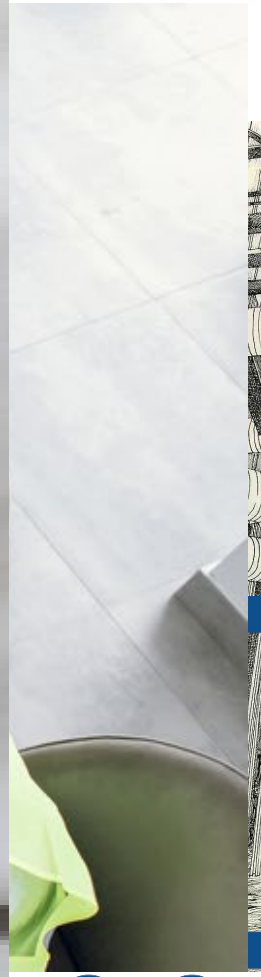
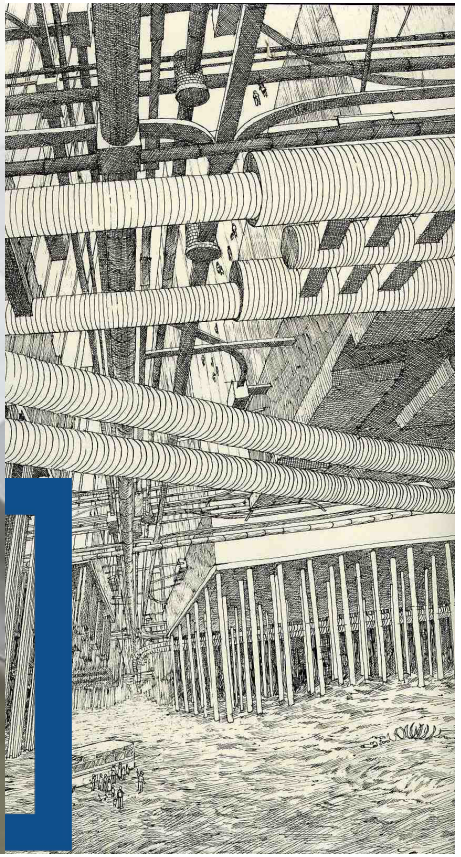
You want to build what where?



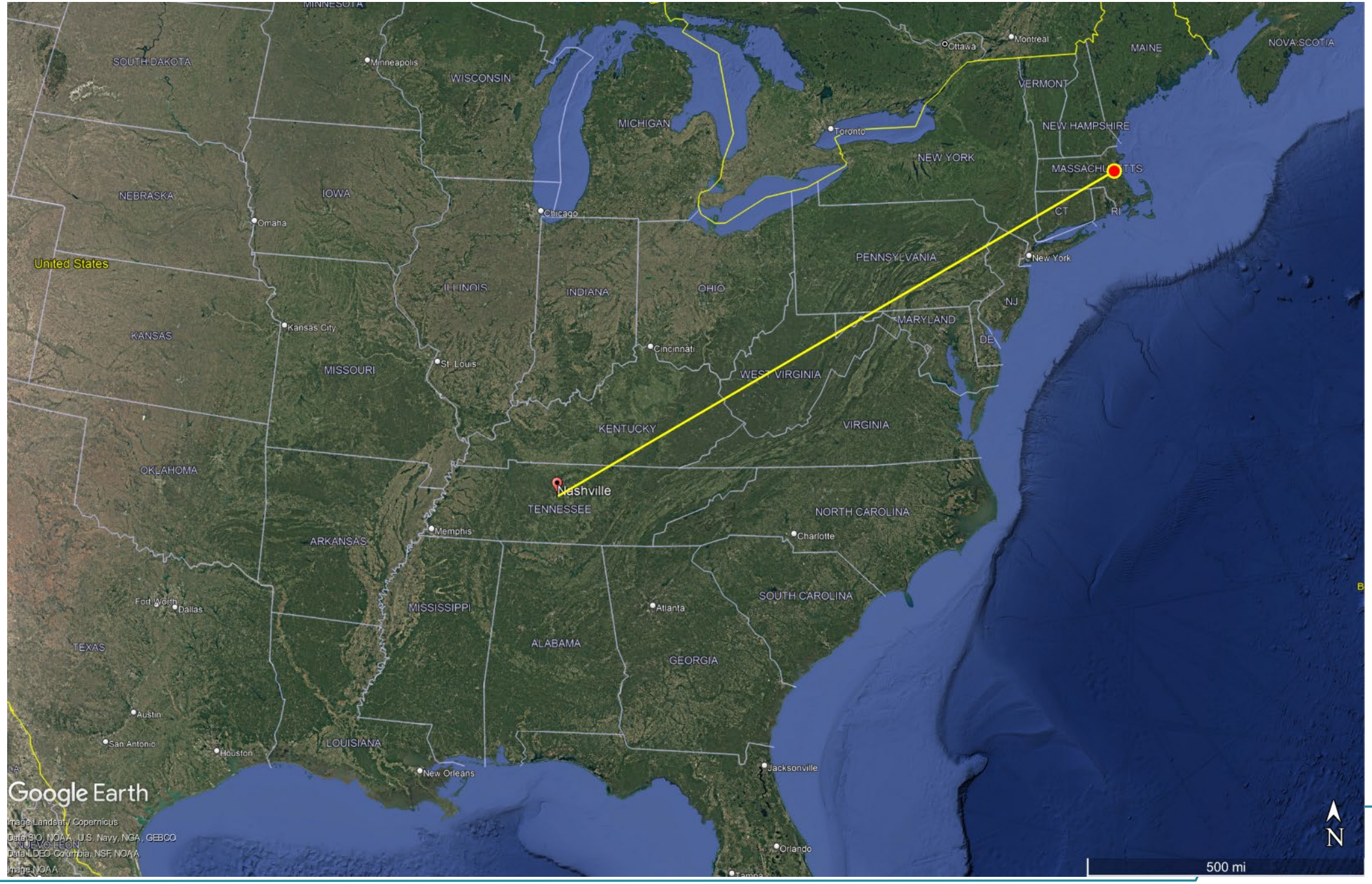








TM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VIRGINIA
OKLAHOMA
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TENNESSEE
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Atlanta
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San Antonio
Houston
LOUISIANA
New Orleans
Jacksonville
Orlando
Google Earth
Image Landsat / Copernicus
Data SIO, NOAA, U.S. Navy, NGA, GEBCO
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Somerville
ADMIRALS HILL
EAST BOSTON
CHARLESTOWN
EST CAMBRIDGE
DORKER STREET
CENTRAL-MAVERICK SQUARE / PARIS STREET
Cambridge
KENDALL SQUARE
JEFFRIES POINT
NORTH END
OWER ALLSTON
AREA 2/MIT
ALLSTON
Boston
BACK BAY
FENWAY-KENMORE
SOUTH END
SOUTH BOSTON
Google Earth
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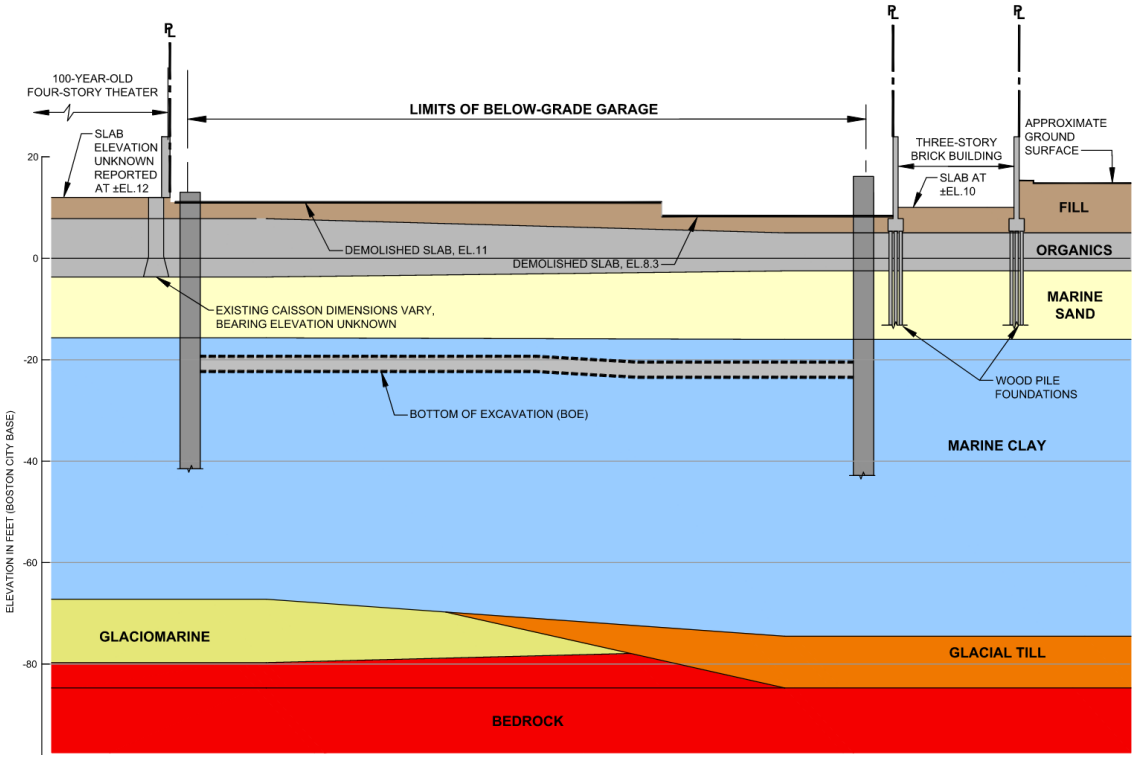
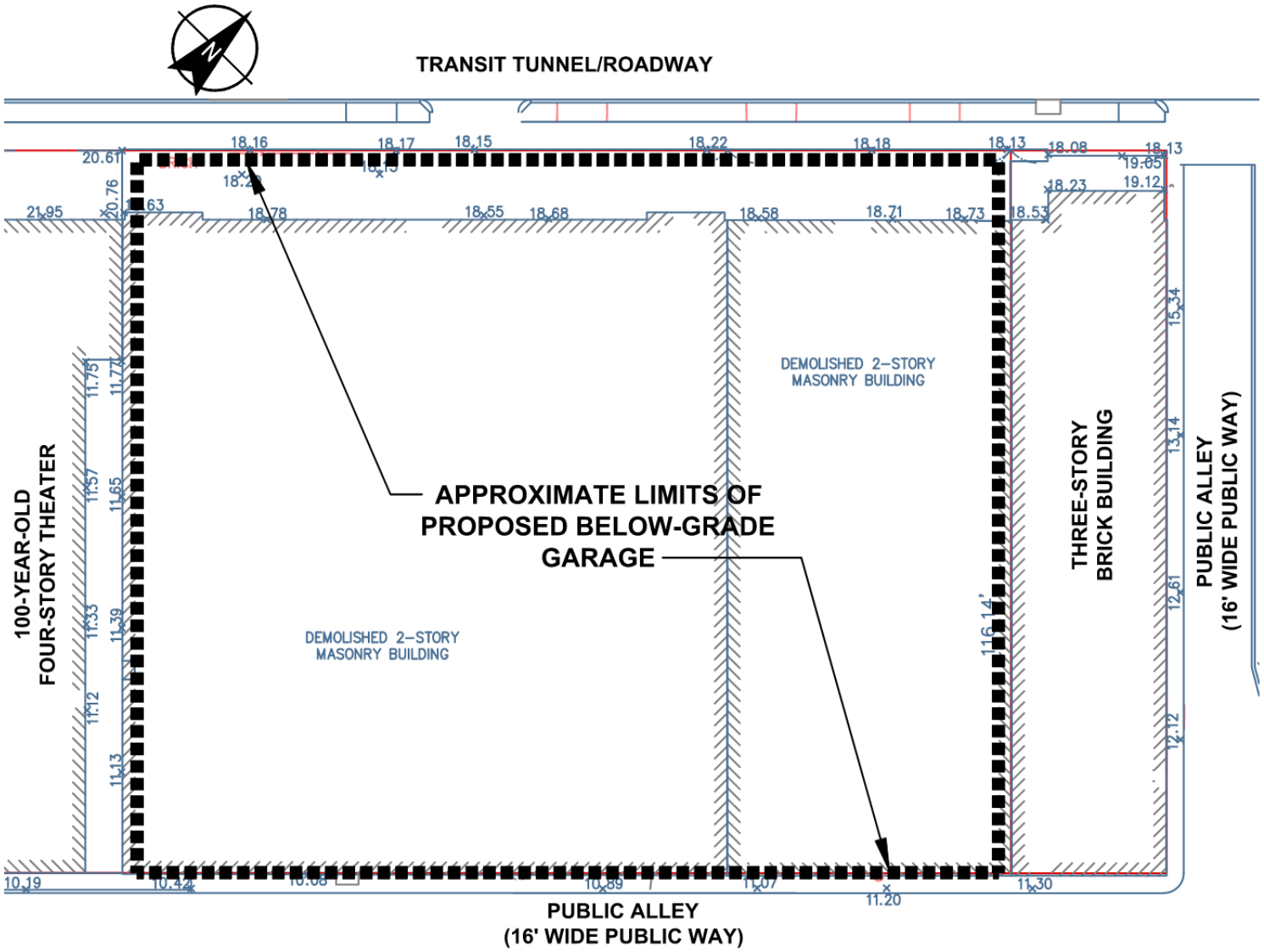
FRENCH
FRENCH

Project background

- Boston's Fenway neighborhood
- 34-story residential tower with two to three levels of below-grade parking
- 15,500 square foot site to the property lines
- Sensitive abutters
  - North: heavily traffic roadway atop transit tunnel
  - East: three-story brick building
  - South: narrow public alley
  - West: four-story historic theater

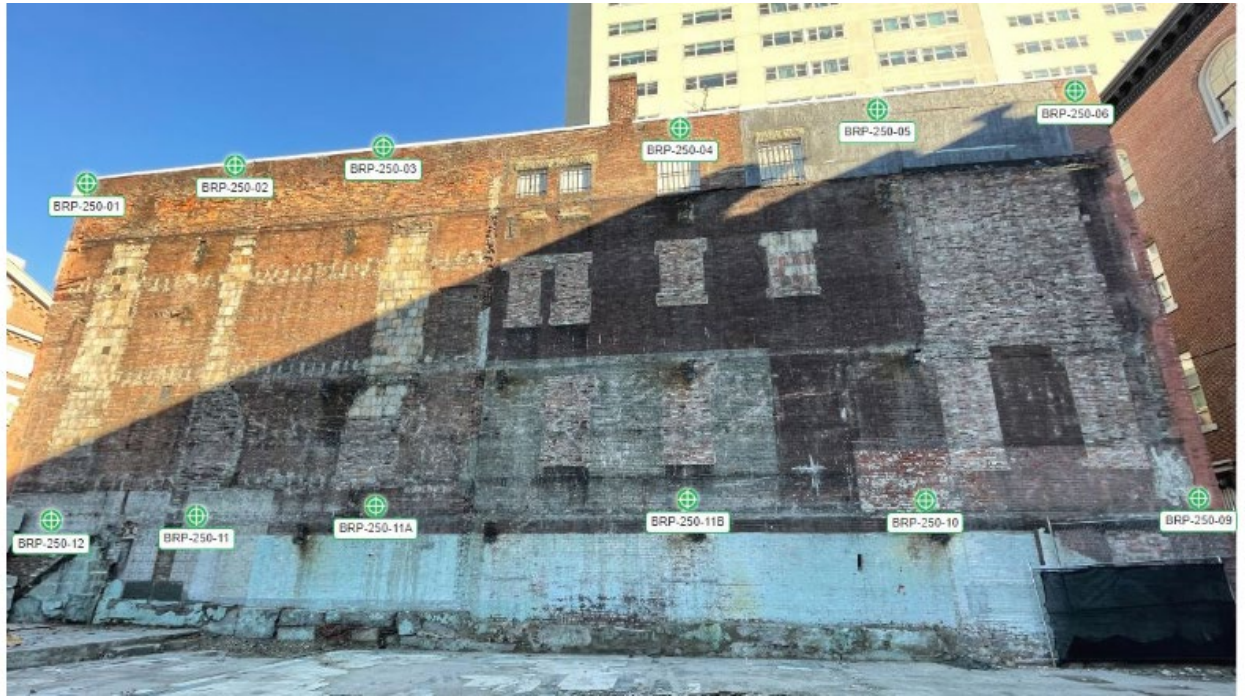


Project background



Instrumentation

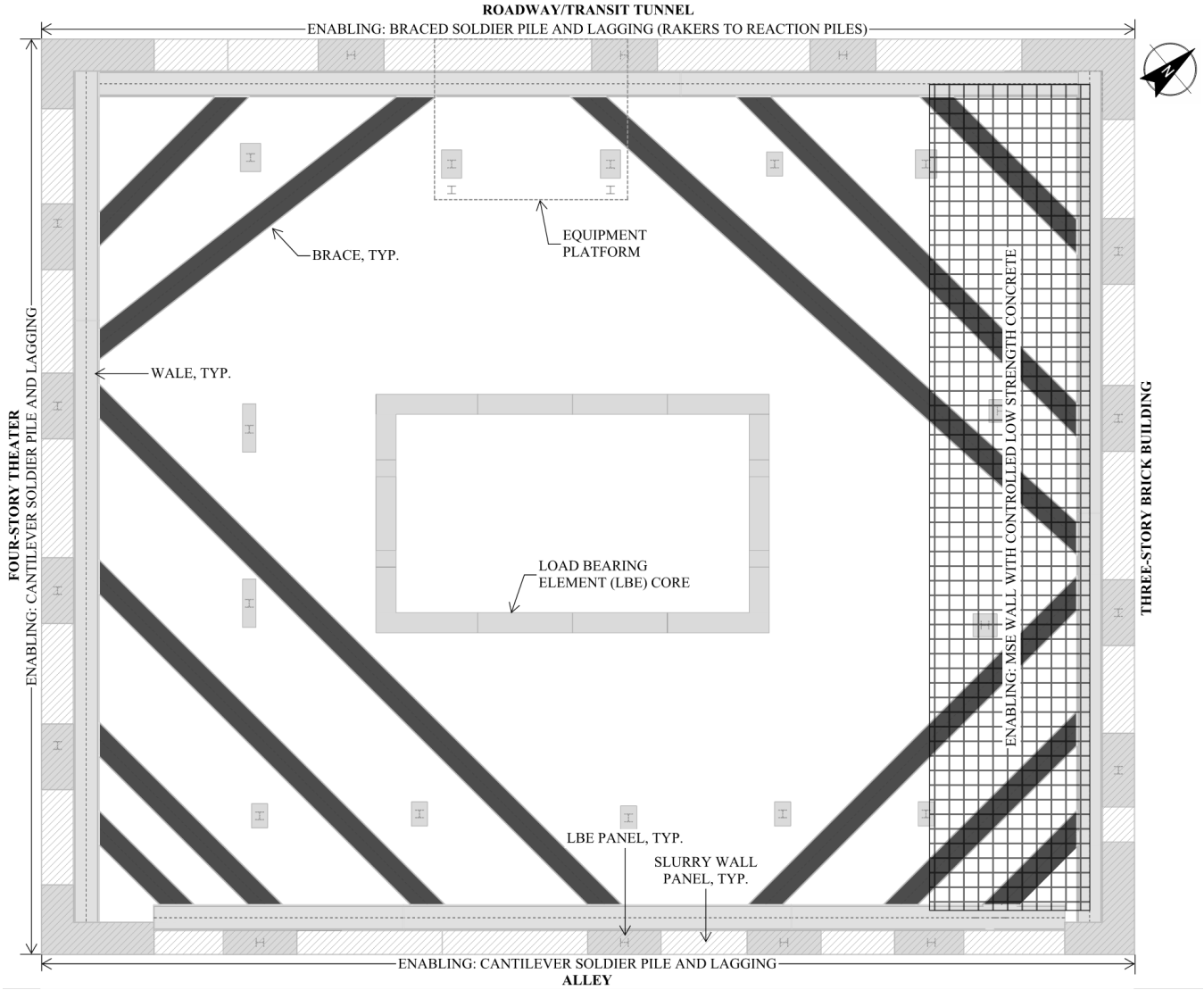
Brick building to the east ⟶





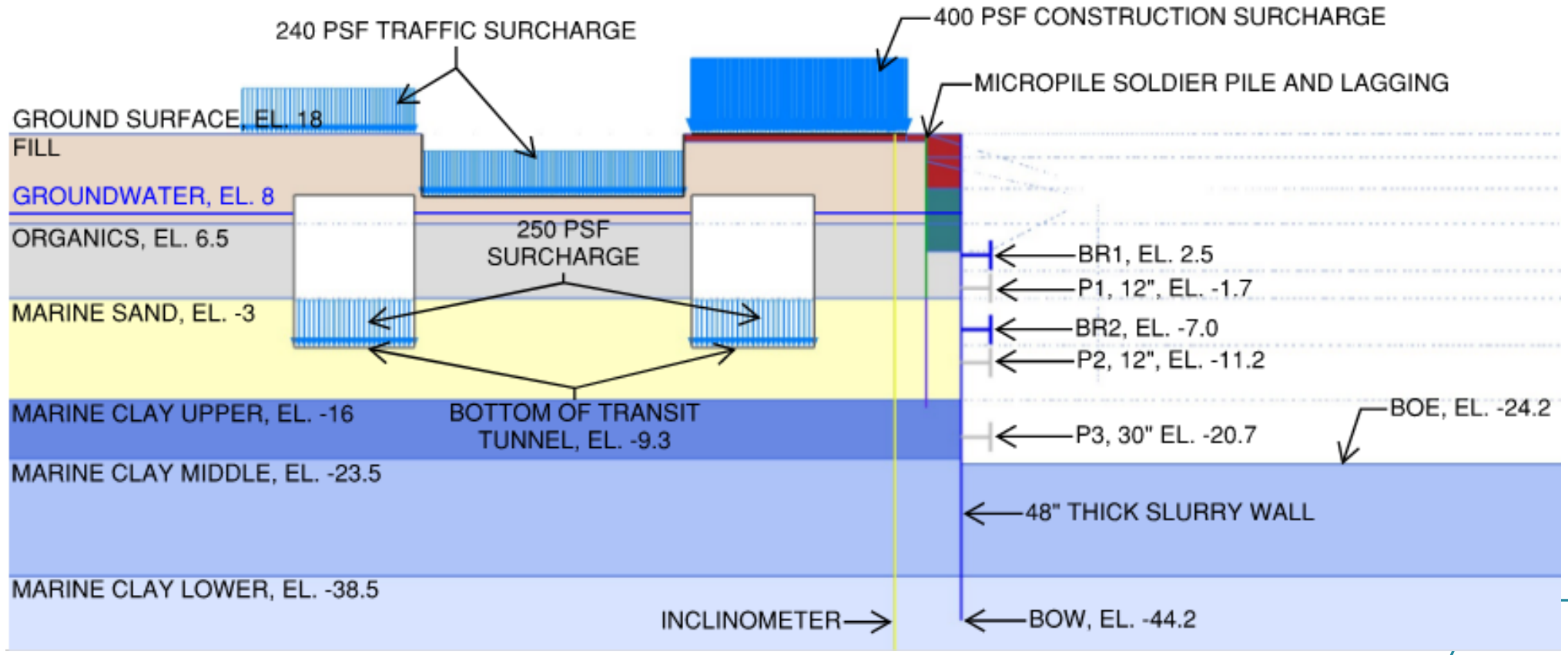
⟵ Theater to the west

Support of excavation



Engineering analysis and modeling

- Finite element analysis to predict potential movement of adjacent structures during mass excavation



Monitoring and performance assessment

- Movements of adjacent structures incurred during enabling work means less movement allowed during mass excavation

Structure	Conclusion of Enabling and Pre-Excavation for Slurry Wall Installation		Conclusion of Slurry Wall Installation		Conclusion of Cap Beam Construction	
	Max. Vert. (in.)	Max. Horiz. (in.)	Max. Vert. (in.)	Max. Horiz. (in.)	Max. Vert. (in.)	Max. Horiz. (in.)
Transit Tunnel	0.1	0.1	0.1	0.2	0.2	0.3
Theater	0.2	0.2	0.4	0.5	0.5	0.5
Brick Building	0.3	0.5	0.6	1.3	0.7	1.4

Instrumentation Measured

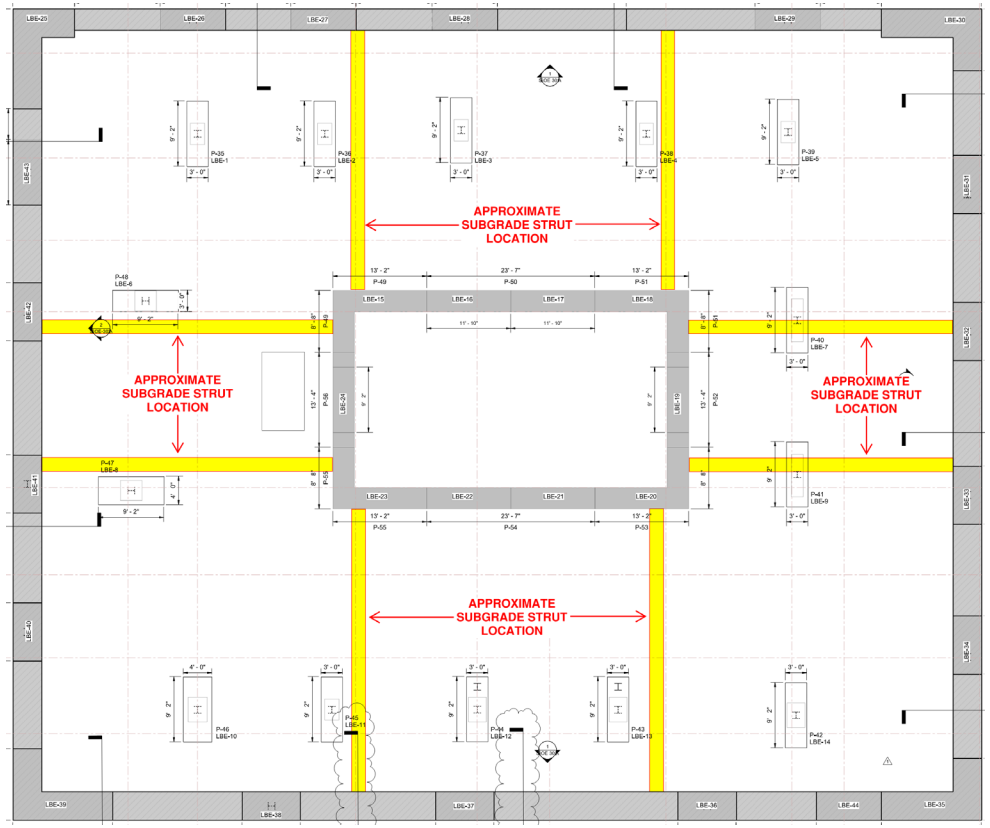
Structure	Conclusion of Below-Grade Construction	
	Vert. (in.)	Horiz. (in.)
Transit Tunnel	0.5	0.5
Theater	1.0	0.7
Brick Building	0.9	0.6

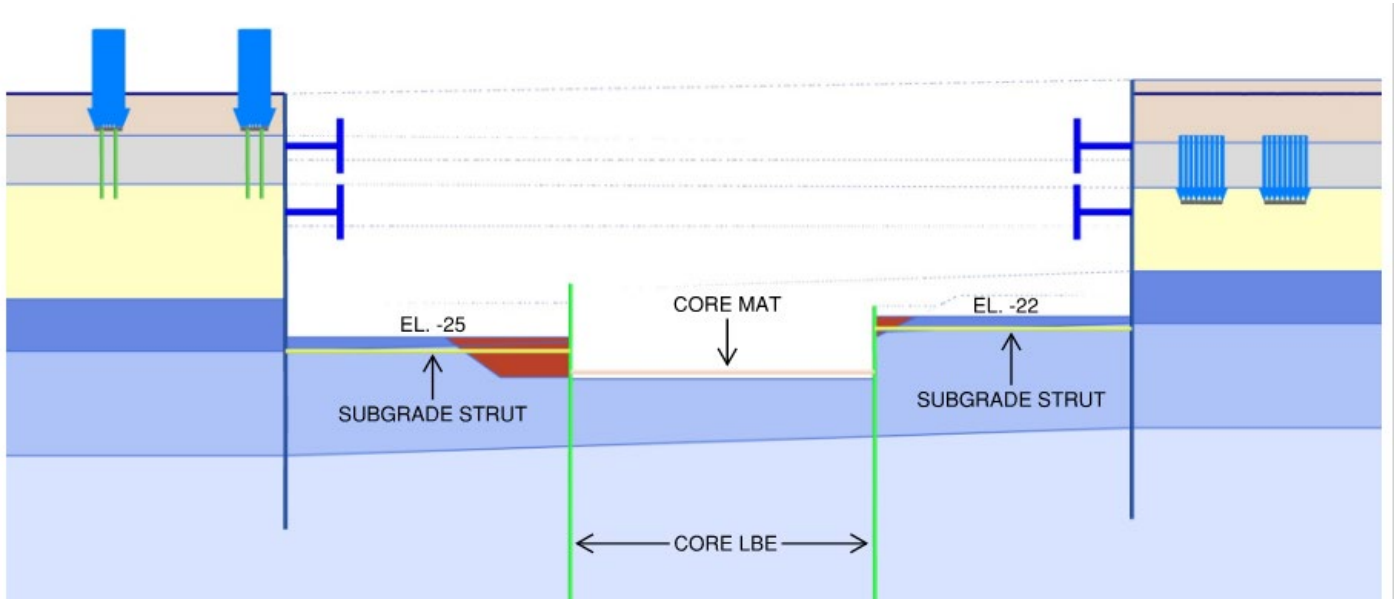
FE Predicted

Monitoring and performance assessment

- Given the movement of the buildings prior to any mass excavation, project stakeholders met to assess potential mitigation strategies
- Bracing preloads increased from 30% to 50% and “observational approach”
- Other strategies discussed:
  - Additional bracing levels/elements
  - Subgrade struts
  - Sequenced slab construction
  - Accelerated construction schedule

Monitoring and performance assessment





Monitoring and performance assessment

Structure	Instrumentation Measured Movement due to Mass Excavation		FE Predicted Movement due to Mass Excavation	
	Vert. (in.)	Horiz. (in.)	Vert. (in.)	Horiz. (in.)
Transit Tunnel	0.0	0.0	0.5	0.5
Theater	0.2	0.0	1.0	0.7
Brick Building	0.1	0.3	0.9	0.6

	Slurry Wall Deflection (in.)			
	Alley	Historic Theater	Transit Tunnel	Brick Building
Inclinometer Measured	0.9	0.5	0.9	0.8
Finite Element Predicted	1.4	1.2	0.9	1.2

What have we learned?

- Important to consider the primary support AND the enabling support
- Consider impacts of difficult to model items (pre-trenching, slurry wall installation)
- Instrumentation monitoring is a vital component to risk management
- Communicate early and often among stakeholders
- Proactive risk management and tailored SOE solutions resulted in successful project execution



Questions?

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HALEY ALDRICH

CLOSING PLENARY

Golden Opportunities



The Importance of Collaborative Project Delivery – Looking Back While Looking Forward

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The Panelists

Luca Barison

Nicholson Construction Co.



Tom Pullen

Brierley Associates



Chris Saucier

Tennessee Valley Authority



Derrick Dasenbrock

Minnesota Department of Transportation



FINDING COMMON GROUND

The Evolution of Collaboration in Procurement Models

- We each play a role - Owner, Designer, Contractor—early alignment is vital
- Establishing trust in relationships
- The role of DFI
- Traditional models (e.g., DBB, Lump Sum) lack flexibility for modern needs
- Trends: complexity, risk sensitivity, innovation, progressive contracts
- Focus on collaboration in geotechnical procurement, not new methods
- Rise of collaborative models: CM/GC, CMAR, DB, PDB but only at Prime level
- Shift from risk-transfer to integrated, risk-conscious/risk sharing delivery
- Drivers: tech advances, complexity, cost/time efficiency
- Features: early contractor involvement, shared risk, flexible procurement

Limitations of Conventional Price-Driven Procurement Models

- Separates design and construction, limiting collaboration
- Designers face late-stage constraints due to codes and standards
- Geotechnical Contractors often engaged post-design, reducing value
- Designs based on assumptions without site-specific input
- Real-world conditions can cause delays, cost increases, and disputes
- Delayed engagement limits flexibility and risk mitigation
- Prescriptive designs hinder adaptability in subsurface work
- Lowest bid focus overlooks innovation and constructability
- Contractors bid with limited data, increasing risk and contingencies
- Owner often reacts to issues instead of proactively managing risks

Why Collaborative Delivery Can Benefit All Parties

- Shifts focus from cheapest to smartest project delivery
- Encourages early contractor involvement and shared goals
- Allows for joint site investigations minimize assumptions
- A more realist appreciation of risk and how the different parties view risk - Shared risk pricing based on real-world experience.
- Unified project team improves outcomes
- Owners gain control and confidence in budget and schedule
- Transparent priorities and performance requirements
- Designers avoid prescriptive methods, focus on outcomes
- Geotechnical Contractors add value through early reviews
- Ground-related risks addressed collaboratively
- Innovation is encouraged and rewarded

How Owners Can Influence Early Involvement

- Require Geotechnical Contractor engagement in collaborative teams
- Stipulate their role in team selection and design development
- Use qualifications-based selection for specialty Contractors as key subs
- Evaluate geotechnical expertise beyond cost
- Include Contractors in design meetings and risk workshops
- Foster collaboration through early value engineering
- Incentivize early risk identification and mitigation
- Use contracts with shared contingencies and phased scopes

How Designers Can Influence Early Involvement

- Influence Owner involvement
- Maintain guidelines: Life Safety, Operational Safety, Design Standards, Codes, Ethics
- Bridge technical insight with constructability
- Enable accurate assessment of subsurface risks
- Refine design parameters and select tailored methods
- Collaborate with Contractors during early design phases
- Identify challenges like soil variability and groundwater
- Early risk identification through site investigations and data interpretation
- Determine realistic quantities in collaboration with Contractor
- Incorporate practical, cost-effective solutions
- Reduce costly design changes and delays
- Foster innovation, safety, and optimized delivery

How Geotechnical Contractors Can Influence Delivery

- Understand obligations in a collaborative model
- Be available for early involvement and have the adequate resources to follow through
- Shared understanding of site conditions to reduce disputes and claims
- Recommend innovative foundation, shoring and ground treatment systems and construction methods
- Enable data-driven decision-making for risk and schedule reliability
- Influence critical path activities for schedule and cost control
- Help avoid overdesign and unnecessary ground treatments
- Improve constructability and risk allocation accuracy for the project as a whole
- Reduce change orders and construction-phase disputes

Is Collaborative Delivery Always Right and Appropriate?

- Collaborative delivery offers advantages but isn't universal
- Conventional models may suit simpler, well-defined projects
- Best suited for high-risk, complex, evolving projects and new to Owners
- Requires Owner involvement, experienced teams, and transparency
- Strategic application unlocks value; misapplication adds risk/schedule/cost
- Early involvement demands significant time and resources
- Not all Contractors are willing to invest in preconstruction phases
- Firms must assess fit based on capabilities and project needs

Conclusion: Stronger Foundations Start with Smarter and Collaborative Procurement

- Early contractor involvement has shown both success and challenges
- Collaborative procurement is standard for CMs and GCs, less so for Geotechnical Contractors
- Shift toward collaboration is reshaping the construction industry
- Geotechnical Contractors can become risk managers and strategic partners
- Collaborative models enable proactive risk management and innovation
- DFI's evolution depends on active participation from all stakeholders
- DFI expertise is essential for the next 50 years and beyond

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FINDING COMMON GROUND

Moderator Questions

Why write a paper about collaboration in construction – isn't this common?

Moderator Questions

What does success look like?

Moderator Questions

What unique value do geotechnical contractors bring to early project planning and risk mitigation?

AND

In what ways do designers influence constructability and cost efficiency when geotechnical contractors are engaged early in the delivery process?

Moderator Questions

Who owns the risk of (something) going wrong?

Moderator Questions

What are the real obstacles to allow the involvement of geotechnical contractors into an early procurement model?

Moderator Questions

What is the owner concern(s) for early collaboration? And what fosters this concern(s)?

AND

What advice can be given to owners or advisors about maximizing the value of these roles in future collaborative projects?

Moderator Questions

What are some best practices for fostering trust and transparency between design teams, geotechnical contractors, and owners?

Moderator Questions

Can you share examples where collaborative design led to innovative solutions in geotechnical challenges?

AND

Temporary design and Permanent design interface is always a challenge. How can the interface between temporary design and permanent design be effectively managed in a collaborative procurement process to ensure seamless integration and project success?

Moderator Questions

What lessons have you learned from past projects about the timing and depth of involvement of designers and geotechnical contractors?

Moderator Questions

What can each of you influence

to shape the next 50 years?