29th Spencer J. Buchanan Lecture

Friday, November 12, 2021 at 2 PM

Virtually Via Zoom

https://briaud.engr.tamu.edu/buchanan.lecture/

Offshore Geotechnics: From Oil and Gas to Renewable Energy

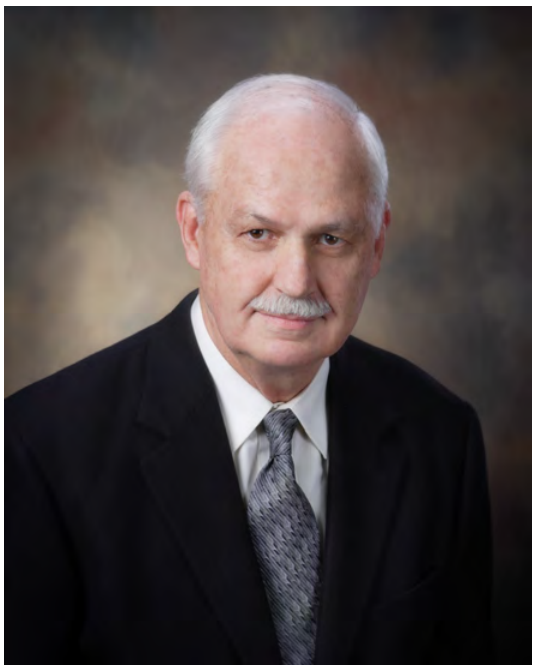
The 2021 Spencer J. Buchanan Lecture

By Philippe Jeanjean

Observing and Controlling Ground Behavior with Tunnel Boring Machines

The 2020 Terzaghi Lecture

By Edward J. Cording



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SPENCER J. BUCHANAN

Spencer J. Buchanan, Sr. was born in 1904 in Yoakum, Texas. He graduated from Texas A&M University with a degree in Civil Engineering in 1926, and earned graduate and professional degrees from the Massachusetts Institute of Technology and Texas A&M University.

He held the rank of Brigadier General in the U.S. Army Reserve, (Ret.), and organized the 420th Engineer Brigade in Bryan-College Station, which was the only such unit in the Southwest when it was created. During World War II, he served the U.S. Army Corps of Engineers as an airfield engineer in both the U.S. and throughout the islands of the Pacific Combat Theater. Later, he served as a pavement consultant to the U.S. Air Force and during the Korean War he served in this capacity at numerous forward airfields in the combat zone. He held numerous military decorations including the Silver Star. He was founder and Chief of the Soil Mechanics Division of the U.S. Army Waterways Experiment Station in 1932, and also served as Chief of the Soil Mechanics Branch of the Mississippi River Commission, both being Vicksburg, Mississippi.

Professor Buchanan also founded the Soil Mechanics Division of the Department of Civil Engineering at Texas A&M University in 1946. He held the title of Distinguished Professor of Soil Mechanics and Foundation Engineering in that department. He retired from that position in 1969 and was named professor Emeritus. In 1982, he received the College of Engineering Alumni Honor Award from Texas A&M University.

He was the founder and president of Spencer J. Buchanan & Associates, Inc., Consulting Engineers, and Soil Mechanics Incorporated in Bryan, Texas. These firms were involved in numerous major international projects, including twenty-five RAF-USAF airfields in England. They also conducted Air Force funded evaluation of all U.S. Air Training Command airfields in this country. His firm also did foundation investigations for downtown expressway systems in Milwaukee, Wisconsin, St. Paul, Minnesota; Lake Charles, Louisiana; Dayton, Ohio, and on Interstate Highways across Louisiana. Mr. Buchanan did consulting work for the Exxon Corporation, Dow Chemical Company, Conoco, Monsanto, and others.

Professor Buchanan was active in the Bryan Rotary Club, Sigma Alpha Epsilon Fraternity, Tau Beta Pi, Phi Kappa Phi, Chi Epsilon, served as faculty advisor to the Student Chapter of the American Society of Civil Engineers, and was a Fellow of the Society of American Military Engineers. In 1979 he received the award for Outstanding Service from the American Society of Civil Engineers.

Professor Buchanan was a participant in every International Conference on Soil Mechanics and Foundation Engineering since 1936. He served as a general chairman of the International Research and Engineering Conferences on Expansive Clay Soils at Texas A&M University, which were held in 1965 and 1969.

Spencer J. Buchanan, Sr., was considered a world leader in geotechnical engineering, a Distinguished Texas A&M Professor, and one of the founders of the Bryan Boy's Club. He died on February 4, 1982, at the age of 78, in a Houston hospital after an illness, which lasted several months.

The Spencer J. Buchanan '26 Chair in Civil Engineering

The College of Engineering and the Department of Civil Engineering gratefully recognize the generosity of the following individuals, corporations, foundations, and organizations for their part in helping to establish the Spencer J. Buchanan '26 Professorship in Civil Engineering. Created in 1992 to honor a world leader in soil mechanics and foundation engineering, as well as a distinguished Texas A&M University professor, the Buchanan Professorship supports a wide range of enriched educational activities in civil and geotechnical engineering. In 2002, this professorship became the Spencer J. Buchanan '26 Chair in Civil Engineering.

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Spencer J. Buchanan Lecture Series

1993	Ralph B. Peck	"The Coming of Age of Soil Mechanics: 1920 - 1970"
1994	G. Geoffrey Meyerhof	"Evolution of Safety Factors and Geotechnical Limit State Design"
1995	James K. Mitchell	"The Role of Soil Mechanics in Environmental Geotechnics"
1996	Delwyn G. Fredlund	"The Emergence of Unsaturated Soil Mechanics"
1997	T. William Lambe	"The Selection of Soil Strength for a Stability Analysis"
1998	John B. Burland	"The Enigma of the Leaning Tower of Pisa"
1999	J. Michael Duncan	"Factors of Safety and Reliability in Geotechnical Engineering"
2000	Harry G. Poulos	"Foundation Settlement Analysis – Practice Versus Research"
2001	Robert D. Holtz	"Geosynthetics for Soil Reinforcement"
2002	Arnold Aronowitz	"World Trade Center: Construction, Destruction, and Reconstruction"
2003	Eduardo Alonso	"Exploring the Limits of Unsaturated Soil Mechanics: the Behavior of Coarse Granular Soils and Rockfill"
2004	Raymond J. Krizek	"Slurries in Geotechnical Engineering"
2005	Tom D. O'Rourke	"Soil-Structure Interaction Under Extreme Loading Conditions"
2006	Cylde N. Baker	"In Situ Testing, Soil-Structure Interaction, and Cost Effective Foundation Design"
2007	Ricardo Dobry	"Pile response to Liquefaction and Lateral Spreading: Field Observations and Current Research"
2008	Kenneth Stokoe	"The Increasing Role of Seismic Measurements in Geotechnical Engineering"
2009	Jose M. Roesset	"Some Applications of Soil Dynamics"
2010	Kenji Ishihara	"Forensic Diagnosis for Site-Specific Ground Conditions in Deep Excavations of Subway Constructions"
2011	Rudolph Bonaparte	"Cold War Legacy – Design, Construction, and Performance of a Land-Based Radioactive Waste Disposal Facility"
2012	W. Allen Marr	"Active Risk Management in Geotechnical Engineering"
2013	Andrew J. Whittle	" Importance of Undrained Behavior in the Analysis of Soil-Structure Interaction"
2014	Craig H. Benson	"Landfill Covers: Water Balance, Unsaturated Soils, and a Pathway from Theory to Practice"
2015	William F. Marcuson III	"Katrina in Your Rearview Mirror"
2016	Edward Kavazanjian	"Bio-Geo-Alchemy: Biogeotechnical Carbonate Precipitation for Hazard Mitigation and Ground Improvement."
2017	Jonathan D. Bray	"Turning Disaster into Knowledge"
2018	Paul W. Mayne	"Versatility of Cone Penetration Tests in GeoCharacterization"
2019	Gregory B. Baecher	"Putting Numbers on Geotechnical Judgement"
2020	Lidija Zdravkovic	"Soil Characterisation for Advanced Geotechnical Design: Parameter Derivation"

The texts of the lectures and a DVD's of the presentations are available by contacting:

Dr. Jean-Louis Briaud
Spencer J. Buchanan '26 Chair Distinguished Professor
Zachry Department of Civil Engineering
Texas A&M University
College Station, TX 77843-3136, USA
Tel: 979-845-3795
E-mail: **briaud@tamu.edu**

2021	Philippe Jeanjean	"Offshore Geotechnics: From Oil and Gas to Renewable Energy"

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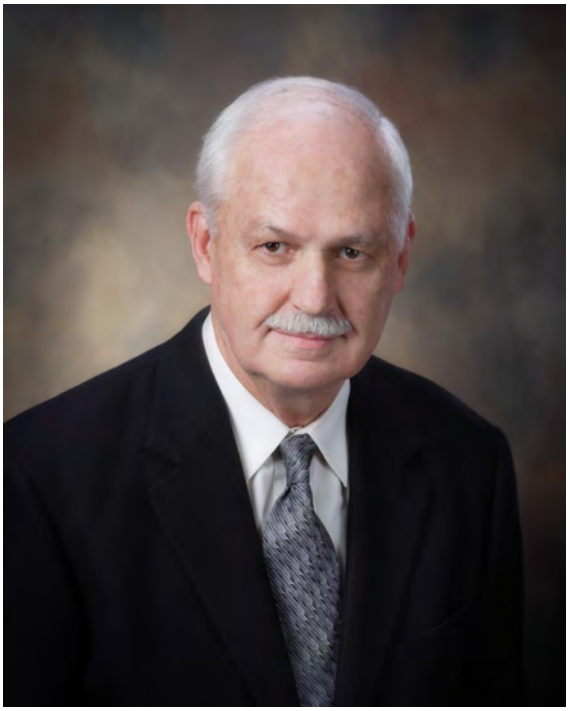
AGENDA

The Twenty-Ninth Spencer J. Buchanan
Lecture Friday, November 12, 2021
Virtually Via Zoom

2:00 p.m.	Introduction by Dr. Jean-Louis Briaud
2:15 p.m.	Introduction of Dr. Edward Cording by Dr. Jean-Louis Briaud
2:20 p.m.	"Observing and Controlling Ground Behavior with Tunnel Boring Machines" 2020 Terzaghi Lecture by Dr. Edward Cording
3:20 p.m.	Introduction of Dr. Philippe Jeanjean by Dr. Jean-Louis Briaud
3:25 p.m.	"Offshore Geotechnics: From Oil and Gas to Renewable Energy" 2021 Buchanan Lecture by Dr. Philippe Jeanjean
4:25 p.m.	Questions
4:30 p.m.	Closure with Dr. Jean-Louis Briaud

Biographies



Edward Cording

Professor Emeritus
Department of Civil and
Environmental Engineering
University of Illinois at Urbana
Champaign
Email: cordingconsult@gmail.com

Edward Cording is Professor Emeritus in the Department of Civil and Environmental Engineering at the University of Illinois at Urbana Champaign, where he taught and conducted research in geotechnical engineering concentrating in the areas of rock engineering and soil-structure interaction. His laboratory has been in the field, observing and analyzing the behavior of excavations, slopes and underground structures on projects ranging from large caverns in stress-fracturing tuff in Nevada, deep rock tunnels in squeezing shales in the Uintah Mountains, shallow subway station caverns in metamorphic rock along the East coast, shield tunnels in alluvial soils in Washington, D.C. and in the soft Chicago clay. Investigations on urban transit projects began with University of Illinois monitoring of station caverns, tunnels, and excavations for Washington Metro Phase 1 construction, and led to development of criteria used in practice for assessing excavation and tunnel-induced ground movements and their effect on building distortion and damage.

He is a member of the National Academy of Engineering and honors include the Moles Award for Outstanding Achievement in Construction, 2003, the Beaver's Engineering Award, 2013, and the Outstanding Educator Award in 2012 of the Underground Construction Association of SME. ASCE awards are the Middlebrooks Award, Martin S. Kapp Award, and Geo Institute Harry Schnabel, Jr Award for Career Excellence in Earth Retaining Structures. He presented the 2018 Muir Wood Lecture, World Tunnel Congress, Dubai, and the 2020 Terzaghi Lecture, ASCE Geo-Congress, Minneapolis

His practice as a geotechnical consultant includes projects with pressurized tunnel boring machines in Cleveland, Columbus, New York City, Sacramento, San Jose, San Francisco, Vancouver, Washington, D.C., Seattle, and Toronto. From 2010 to 2017 he was a consultant to Seattle Tunnel Partners JV on the Alaskan Way Viaduct Replacement Project, participating in construction monitoring and ground control during advance of the 17.5-m-diameter pressurized tunnel boring machine beneath downtown Seattle. Since 2010, he has been a member of the Tunnel Advisory Panel for Los Angeles Metro on the planning, design and construction of light and heavy rail subway stations and tunnels. Experiences from these projects are included in the presentation entitled "Observing and Controlling Ground Behavior with Tunnel Boring Machines.".

Philippe Jeanjean

Senior Advisor for Geotechnical Engineering
BP America in Houston, TX
Email: Philippe.Jeanjean@bp.com

Philippe graduated from the Ecole Centrale in Lyon, France in 1989 and Texas A&M University with a Master of Science in 1991 and a Ph.D. in 1993, both in Civil Engineering. He began his career with Amoco Production Company in Tulsa, Oklahoma, in 1993 and was transferred to Houston, Texas in 1995 where he still works for BP America. He currently is a Senior Advisor for geotechnical engineering, the highest-ranking geotechnical position in the company.
Philippe has 28 years of experience in leading activities such as the planning and execution of onshore and offshore geotechnical site investigations, geo-hazard assessment, designing and assessing shallow, intermediate, and deep foundations for fixed and floating offshore platforms subject to hurricanes or earthquake loads, design of flowlines and pipelines, and geotechnical support for offshore well drilling activities.

Philippe is very invested in many professional committees and activities. In particular, he is a member of the International Society of Soil Mechanics and Geotechnics and chaired its Technical Committee 209 on Offshore Geotechnics from 2009 to 2017. He has been a member of American Petroleum Institute and ISO committees on geotechnical and offshore foundations for 28 years and chaired the API committee from 1999 to 2014. He is also the co-chair of the 4th International Symposium on Frontiers in Offshore Geotechnics which will be held in August 2022.

Philippe has authored or co-authored more than 60 technical articles. He has been a guest speaker or keynote speaker at numerous conferences, delivered the 2019 Coulomb Lecture, and holds a patent on extrapolating geotechnical data using seismic reflection data.

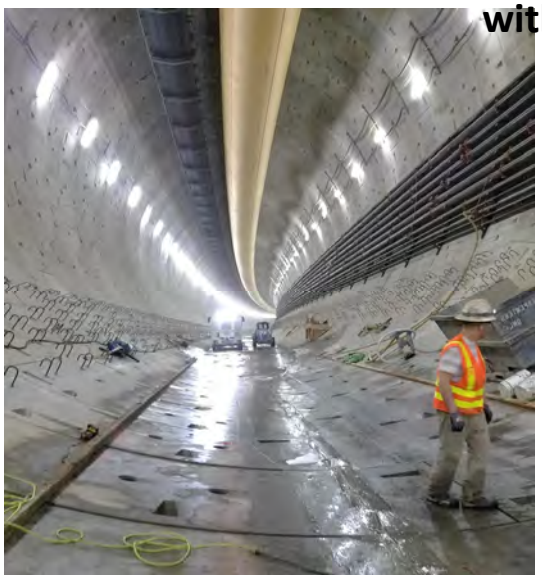
In 2015, the United States Board on Geographic Names agreed to name an underwater site in the Gulf of Mexico "Jeanjean Basin" to honor Philippe and his "substantial contribution to the advancement of geotechnical and geological knowledge of the seafloor of the Gulf of Mexico."

Observing and Controlling Ground Behavior with Tunnel Boring Machines

The 2020 Terzaghi Lecture

By Dr. Edward Cording

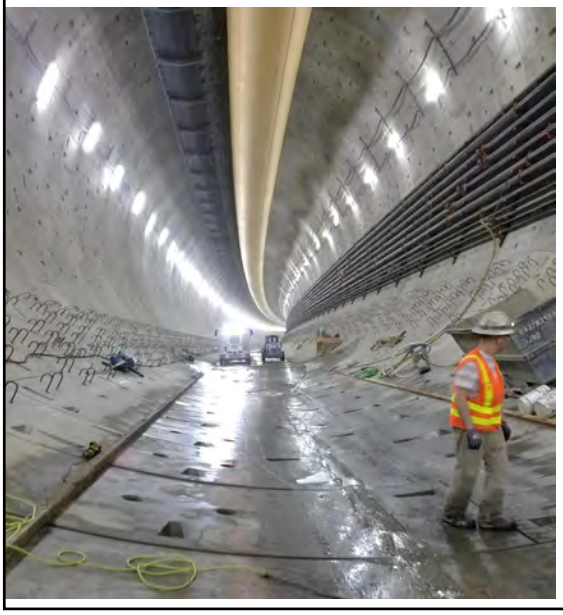
Observing and Controlling Ground Behavior with Tunnel Boring Machines



Edward Cording
University of Illinois at Urbana-Champaign

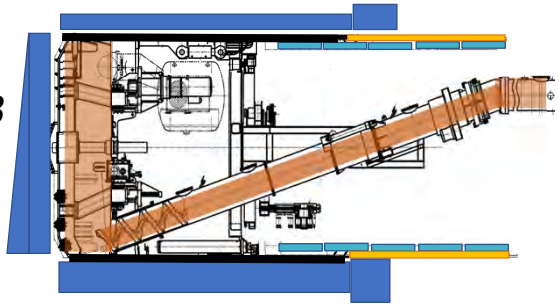
Texas A & M
November 12, 2021

Observing and Controlling Ground Behavior with Tunnel Boring Machines



- PRESSURE FACE TBMS 1970's
  - *SLURRY BALANCE*
  - *EARTH PRESSURE BALANCE (EPB)*
- **FULLY PRESSURIZED ENVELOPE**
  Face – Shield – Tail – Lining

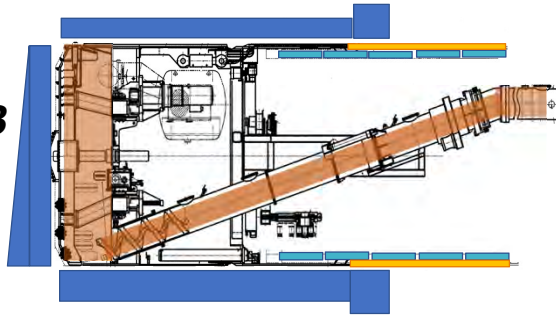
EPB



Observing and Controlling Ground Behavior with Tunnel Boring Machines

- PRESSURE-FACE TBMS 1970'S
  - *SLURRY BALANCE*
  - *EARTH PRESSURE BALANCE (EPB)*
- **FULLY PRESSURIZED ENVELOPE**
  Face – Shield Gap – Tail Gap– Lining

EPB



We are witnesses to a revolution…

- **In the ability to tunnel deep beneath waterways**
- **and to tunnel at shallow depth in urban areas without damaging settlement**

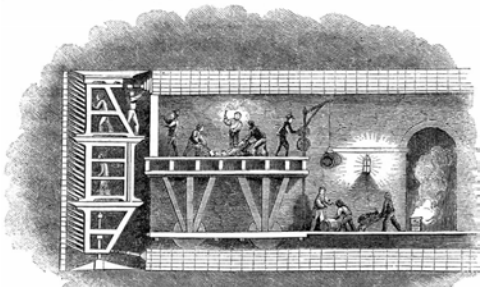
It began almost 50 years ago, and is continuing…

- **FIRST with pressurization of the face of the tunnel shield: (Slurry Balance and Earth Pressure Balance TBMs)**
- **SECONDLY, but importantly in urban areas, with pressurization around the entire shield: from the cutterhead, around the shield body, to the lining installed at the tail of the shield.**

First shield tunnel: Thames River, London 1843

Muir Wood, 1994,
"*WHERE SHIELD TUNNELING BEGAN*"

Skempton & Chrimes, 1994



1843 Thames River, London

Marc Isambard Brunel

1818 Patent Application: Objective:
"Open… the ground in such a manner that no more earth shall be displaced than is to be filled by the shell or body of the tunnel."

Ground behavior is observed and described at the source – around advancing pressurized TBMs (shield tunnels), ranging from 6-m transit tunnels to the 17.5 m Alaskan Way Viaduct replacement tunnel in Seattle.

We begin with some lessons learned on monitoring and control of ground behavior before the use of pressurized TBM shields.

The first shield tunnel, an 11.6-m wide rectangular tunnel advanced beneath the Thames River, was large even by today's standards.

- These 2 papers present the amazing story of the first subaqueous shield tunnel, completed in 1843 by Marc Isambard Brunel…
- Such papers are more than history, They are our engineering precedents, our tunneling experience, and provide a picture of how ground behavior **affects** the tunneling, and tunneling **controls** the ground…
- Sir Marc Isambard Brunel's design and construction was perhaps the most innovative shield tunnel project ever conceived and executed: bold, difficult & dangerous.

- The above Illustration shows the miners working in multiple pockets at the front of the shield, setting screw jacks to brace boards against the face, then advancing by removing one board at a time, excavating a foot, and then placing the board forward and resetting the jack. It is the classic method of breasting, used on open face shields to provide face support in unstable ground.

- The mlners encountered flowing ground conditions in sands and silts, inflows into the face and then break through of the thin cover of London Clay. With multiple irruptions… inundations of the Thames River into the tunnel.

- Thus, Brunel didn't achieve the objective stated in his 1818 patent application,
- Tunnelers have had the same objective for the last 200 years.
- This lecture describes how the objective is achieved with monitoring and control of pressurized TBMs.

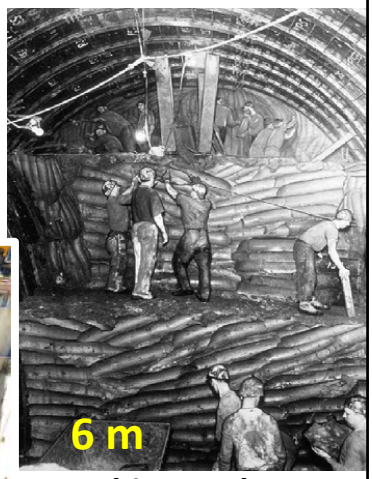
Observing and Controlling Ground Behavior during Tunneling

1. **Pioneering Observations 1939 – 1941**
   - Karl Terzaghi & Ralph Peck: *Chicago Subway*

2. **Observations at Source**
   - Open Face Shields
     - *Washington Metro: 1971*
     - *Chicago Clay Revisited: 2000*

3. **Observation & Control**
   - Pressurized TBMs
     - Increasing Control
     - Decreasing Ground Loss
     - Increasing Diameter
   - *Toronto, Seattle, Los Angeles*

Alaskan Way TBM 2017



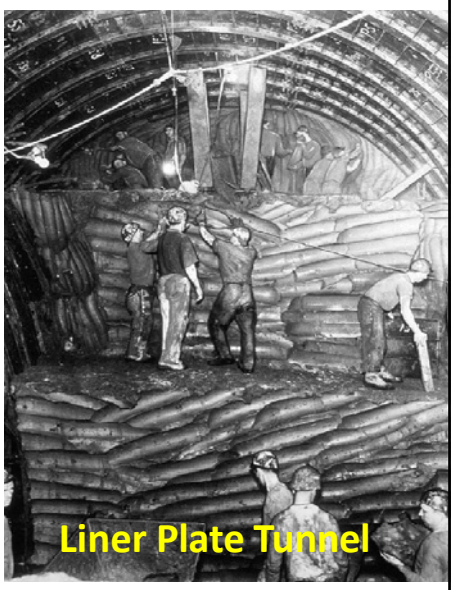
Chicago Clay Liner Plate Tunnel 1940

1. Pioneering Observations: 1939 - 1941

Karl Terzaghi & Ralph Peck: Chicago Subway

- ***Relate tunnel construction …to surface settlement***

- ***Squeeze tests: Rods driven into clay to measure volume of ground loss***
- ***Correlate with volume of surface settlement***
- ***Correlate with construction methods & ground conditions***
- ***Recommended changes that reduced settlement from 4 to 2 in.***



- **Soft "Knife" Clay, Heading & Bench**

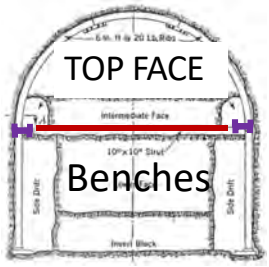
Tunneling deeper to go beneath the Chicago River: Why have settlements increased from 2 to 5 inches?

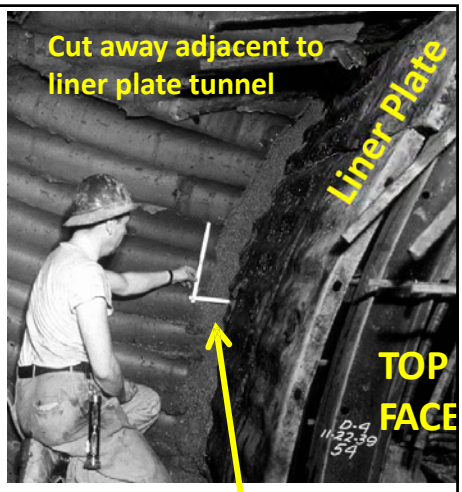
Dec 19. 1939

REPORT ON INVESTIGATION OF SETTLEMENTS
CONTRACT D3, S. HEADING,
N. B. TUBE.

R. B. PECK, ASS'T. SUB. ENG.

- Clay squeeze: Faces, Arch, Walls. ~ 1 inch
- Delay in filling 5-inch gap behind liner plate until <u>18 feet</u> behind face
- Only support is 14 psi air pressure.
- Delay in placing strut to support side walls





Gap Filled w/ Pea Gravel

Settlements reduced from 5 inches to 2 inches by eliminating delays

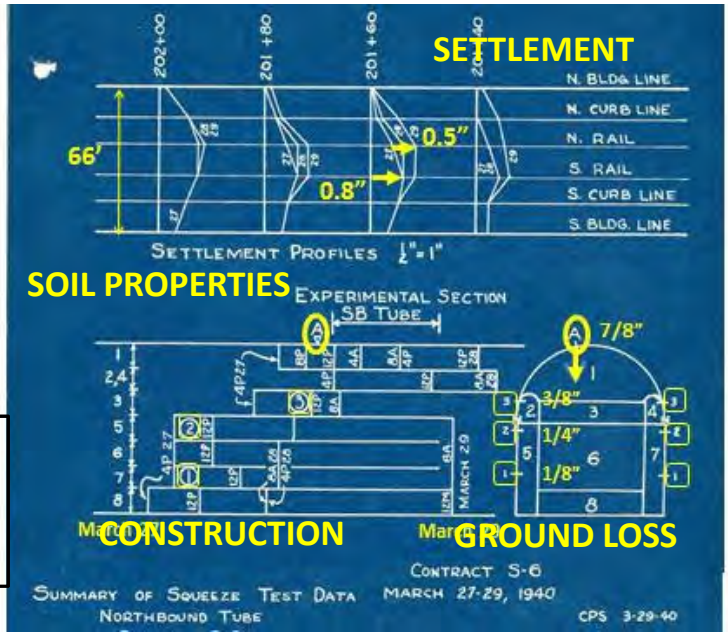
Monitoring and Controlling Ground Behavior at the Source

1941

- FILL THE GAPS
- *Correlate & Summarize on a Single Blue Print*

2017

- *FILL & PRESSURIZE THE GAPS*
- *Correlate ground movement with key TBM parameters*
- *Summarize from electronic data*



2. Observations at the Source

Open Face Shields

- **Ground loss – a continuing challenge**
- **Monitoring procedures developed to locate the source of the ground loss**

1972: OPEN FACE: DIGGER:

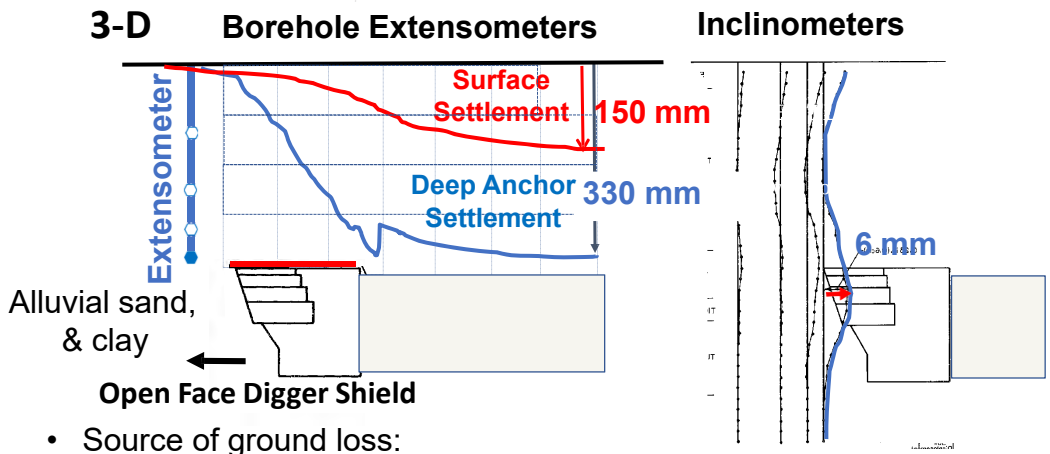
Alluvial Sand & Clay, Washington Metro

. *First use of extensometers & inclinometers to measure 3-D pattern of ground movements around a shield*

2000: OPEN FACE WHEEL: Soft Chicago Clay

Extensometers & piezometers show ground movement around shield & consolidation settlement due to stress changes.

1972: Washington Metro, Lafayette Square: Univ. of Illinois Test Section



- Source of ground loss:
  Over shield due to "plowing"
  Plumb bob shows pitch > grade
- 2nd drive: hood rebuilt, settlement reduced from 150 to 50 mm
- 6 mm displacement toward face
- Confirmed: face was not . . . source of ground loss

Cording & Hansmire, 1975

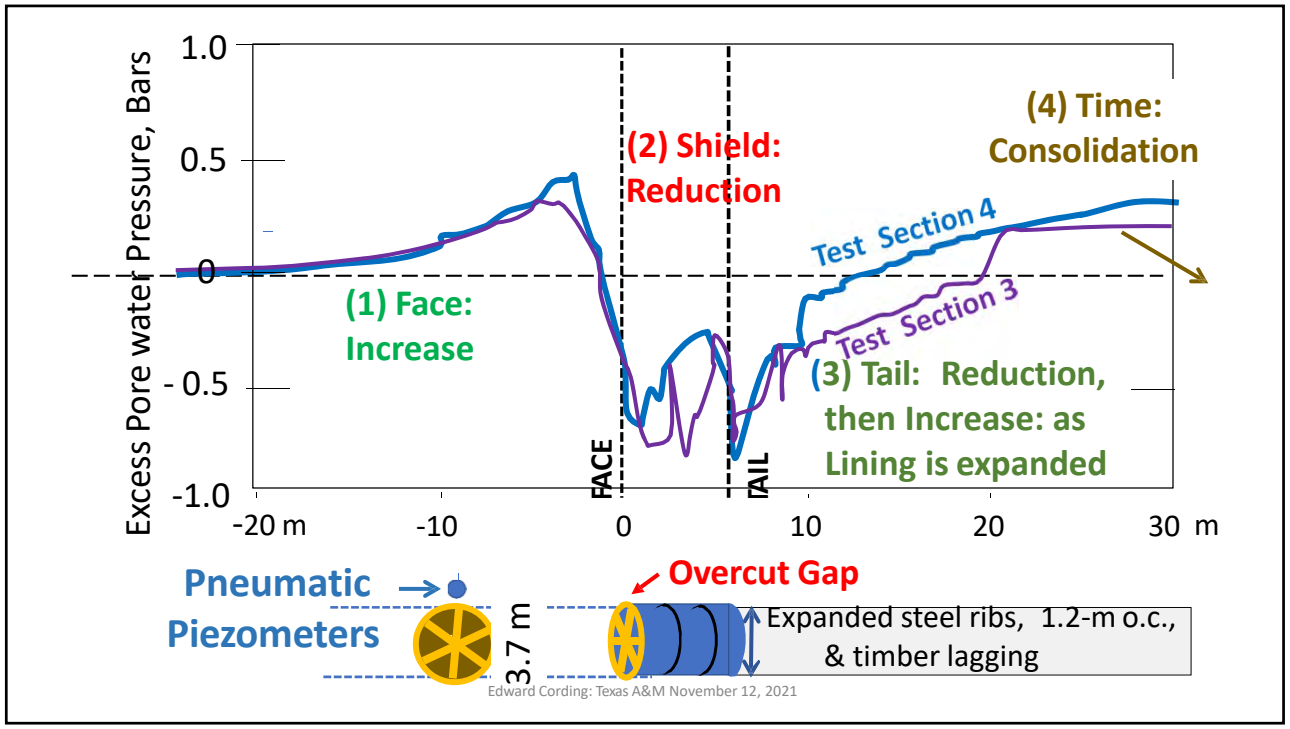
2000: Soft Chicago Clay

3.7-m-dia Lovat Wheel, articulated

McNally Tunneling & University of Illinois

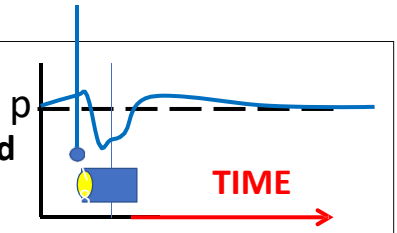
Edward Cording: Texas A&M November 12, 2021

After Srisirirojanakorn, 2004



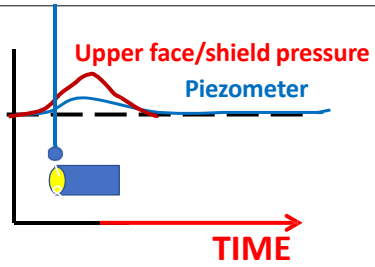
Piezometers in clay

- 1. Undrained stress changes: unloading over shield
- 2. Drainage & consolidation of soft clay with time
  - If piezometric pressures are above ambient after shield passes



Piezometers in Sand (Pressurized TBM)

- *Dynamic Increase in porewater pressure.*
- *Face/Shield pressure: Greater than porewater, to support effective weight of ground*



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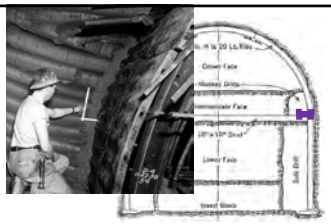
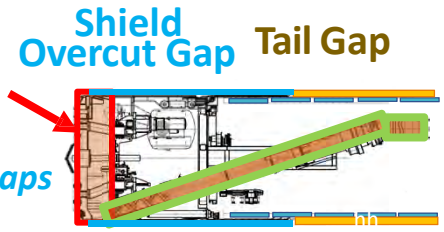
LESSONS: Support the Face & Fill the Gaps



1. Liner Plate Tunnels, Chicago,
   - **Face support:** ***Compressed air***
   - **Lining Gap:** ***Minimize Gap, Fill w/ Pea Gravel Immediately***

2. Open Face Shields
   - **Face:** ***Potential for large Ground Loss in Flowing & Running Sand***
   - **Overcut Gap:** ***Not filled: Inevitable Ground Loss***

3. Pressure-Face to Fully Pressurized TBMs
   - **Pressurize Face:** ***Prevent Inflow & Large Ground loss***
   - **Pressurize Gaps:** ***Prevent Ground Loss into Gaps***

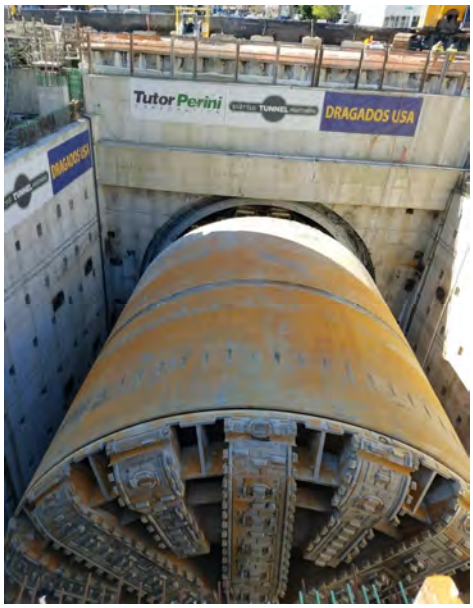


3. Pressure-Face TBMs: 1970's …. to Pressurized TBMs: 2000's

- **Increasing Control**
- **Decreasing Ground Loss**
- **Increasing Diameter**

Examples of Ground Control

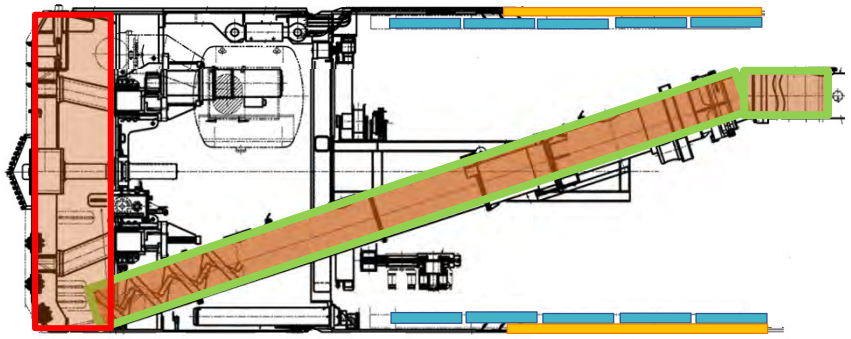
• Porto, Portugal, 2001:	8 m
• Barcelona Line 9, 2008:	12 m
• Sound Transit, Capitol Hill, 2011:	6 m
• **Toronto Transit, York Univ. , 2012:**	**6 m**
• **Alaskan Way: 2017:**	**17.5 m**
• **LA Metro: 2006 to present**	**6 m**



Alaskan Way, Seattle
World's largest EPB TBM

Chamber filled with Pressurized Muck - Conditioned w/foam, polymer, bentonite

PRESSURE-**FACE** TBM
Earth Pressure Balance (EPB)



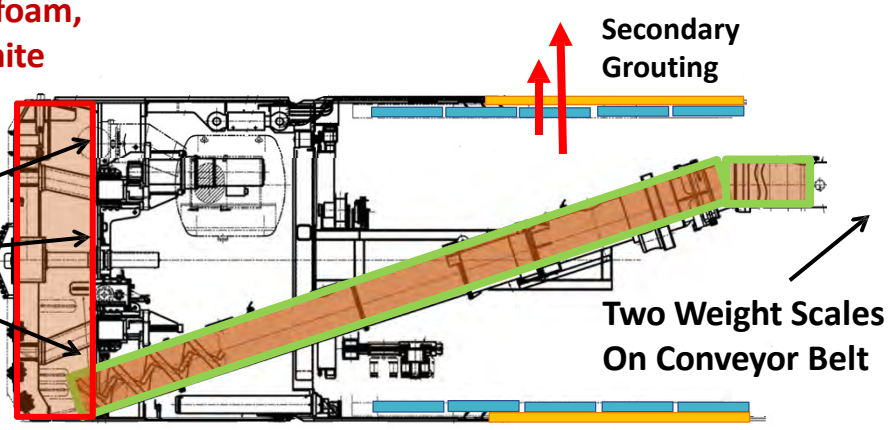
BALANCE GROUNDWATER PRESSURE & ACTIVE EARTH PRESSURE

CUTTERHEAD CHAMBER

Screw removes muck & provides back-pressure

Chamber filled with Pressurized Muck - Conditioned w/foam, polymer, bentonite

PRESSURE-FACE TBM: EPB
Monitoring

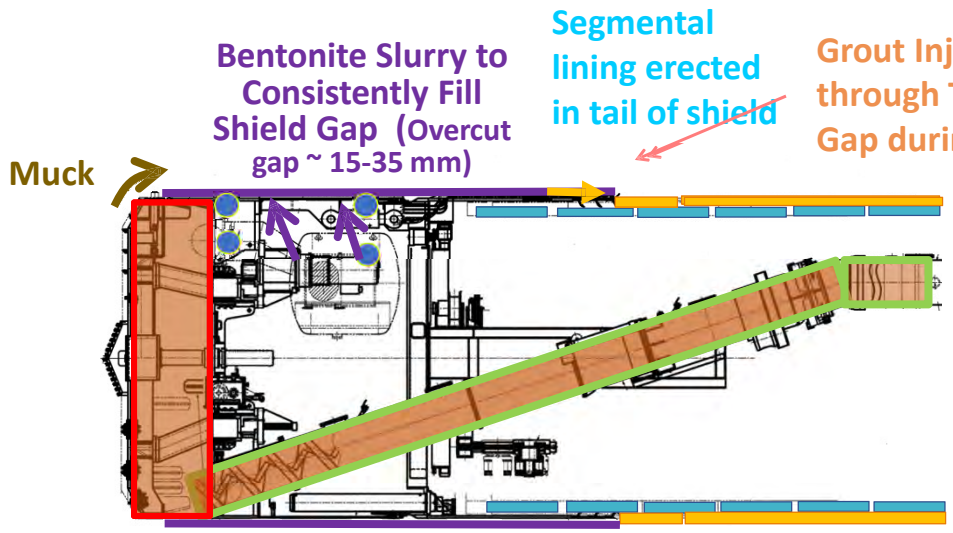


CUTTERHEAD CHAMBER

Screw removes muck & provides back-pressure

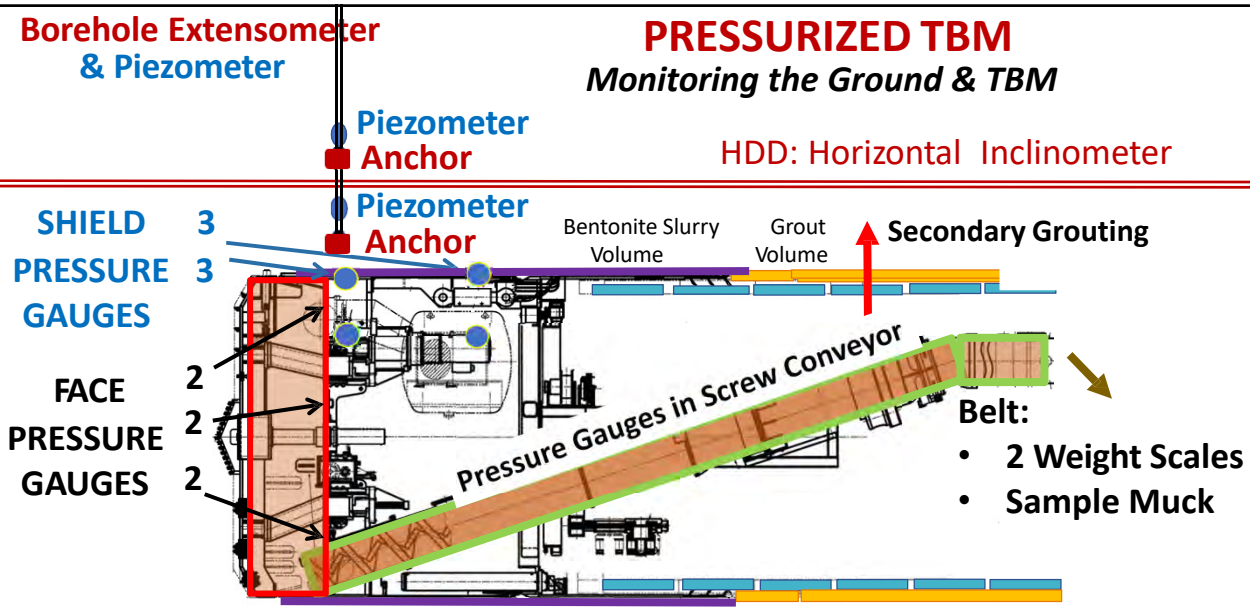
Reconciliation: Muck Weight = Theoretical Excav. Volume X Density + Weight of additives

PRESSURIZED TBM --- Envelope around Face-Shield-Tail



PRESSURIZED TBM

Monitoring the Ground & TBM



Monitoring TBM & Ground

2012: Toronto Transit Commission
Toronto-York Spadina Subway Extension

- Two Earth Pressure Balance TBMs
- Tunnels to pass 6-m below spread foundations of Schulich Building, York Univ.
- Compensation grouting required beneath foundations as TBMs passes: Settlement/heave < 10 mm

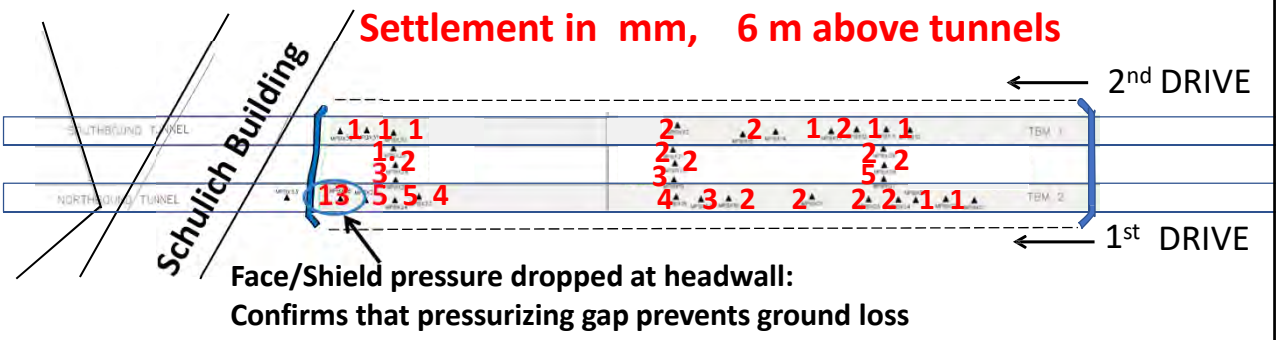
Schulich Building
York University, Toronto

Glacial Till with Sand & Gravel

- **Grout pipes not in place as TBMs advances within 150 m of building**
- **Contractor requests advancing beneath building without compensation grouting**
- **Test Section proposed by Toronto Transit**

  Requirements:
  - **< 10-mm settlement**
  - **Inject bentonite around shield body to prevent ground loss into gap**
  - **Consistent Control of TBM**

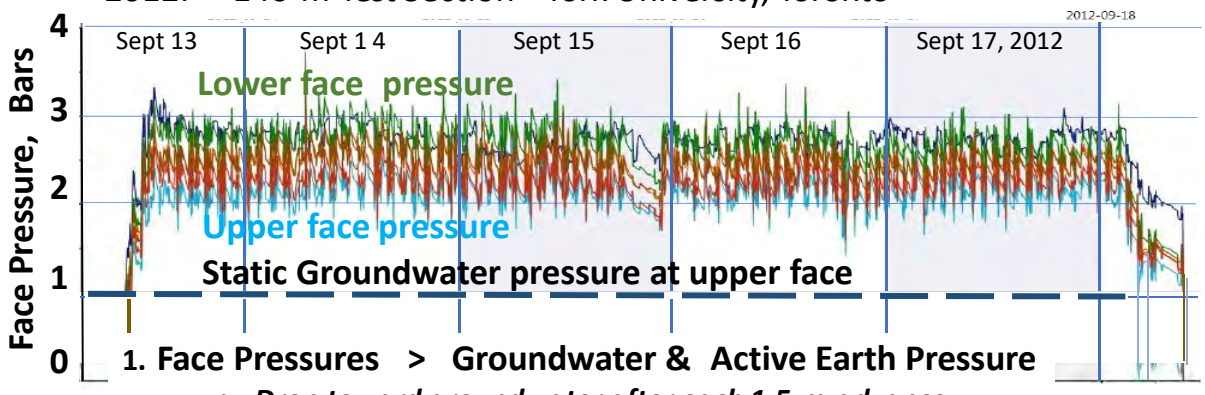
140-m Test Section at York University (future station excavation)



Face/Shield pressure dropped at headwall:
Confirms that pressurizing gap prevents ground loss

- **Gaps filled & pressurized: Small settlement; No ground loss**
- **2nd Drive: Face/shield pressure jncreased, reducing differential pressure between overburden (2.5 bar) & shield pressure (2.1 bar)**
  - **Reduced elastic settlement to < 2 mm**

2012: 140-m Test Section York University, Toronto

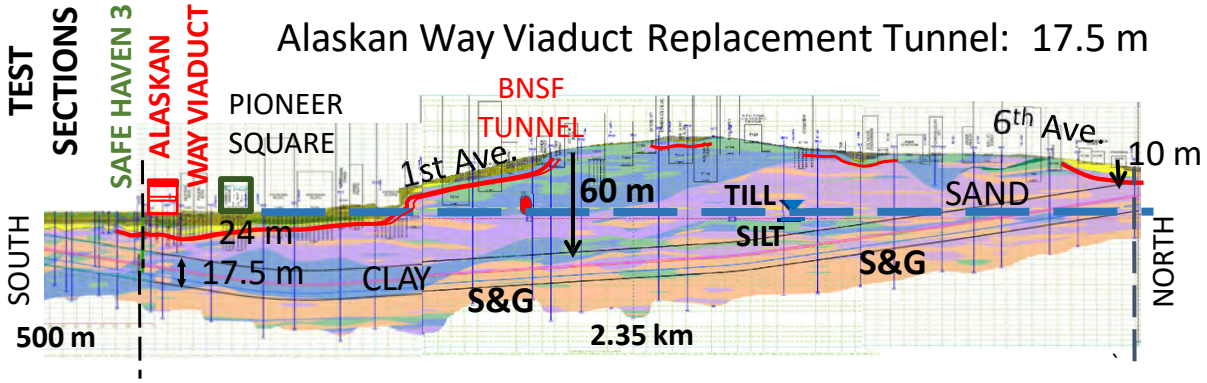


1. **Face Pressures > Groundwater & Active Earth Pressure**
   - ***Drop toward groundwater after each 1.5-m-advance.***

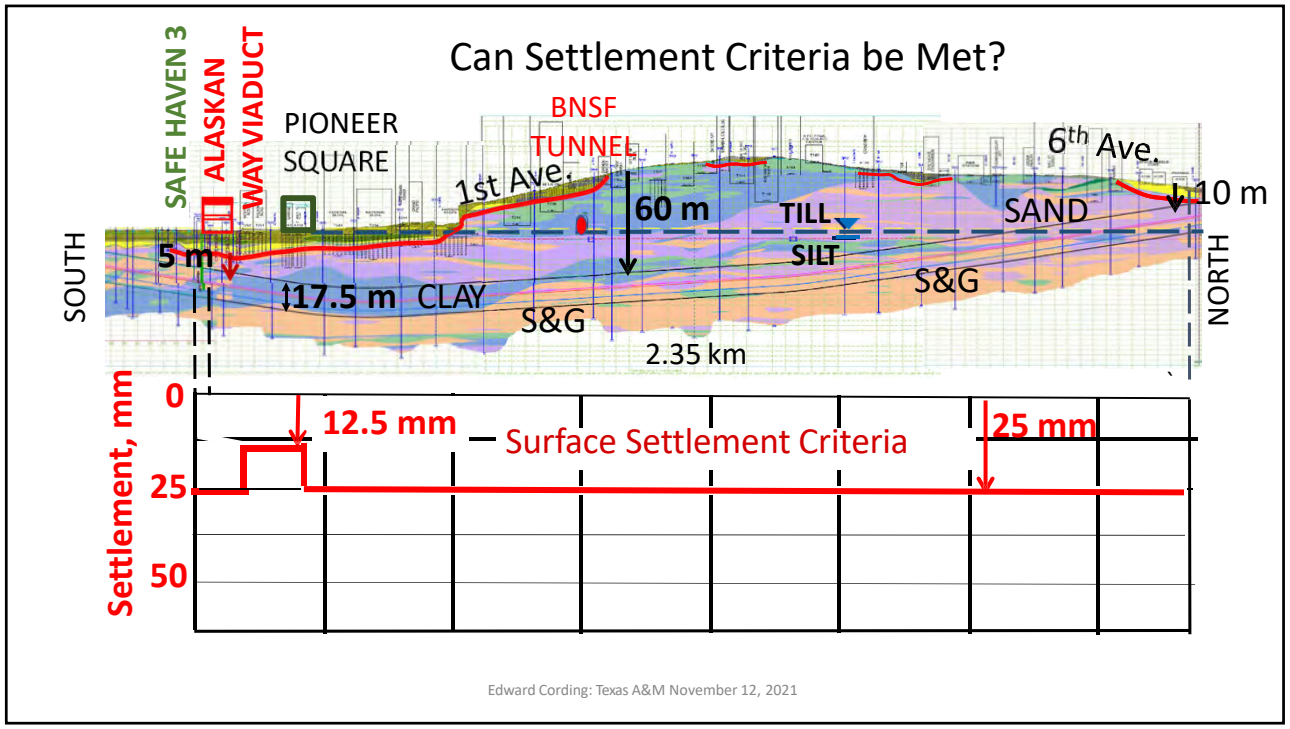
2. **Consistent Control**
   - ***Maintained upper face pressure 1 bar above groundwater pressure***
   - ***Even pressure distribution across height of TBM: Muck density = 1.4***

Advanced beneath building: 2 mm heave, 1 mm settlement

Alaskan Way Viaduct Replacement Tunnel Washington State DOT



- Pioneer Square: **Tide Flats:** 10 m of Hydraulic Fill & Recent Alluvium
- Glacially Overridden Lacustrine **Clays & Silts**
- **Till**
- Pro-glacial, **Outwash Sand & Gravel** over entire length:
- **As a result: Groundwater @ Tide level**



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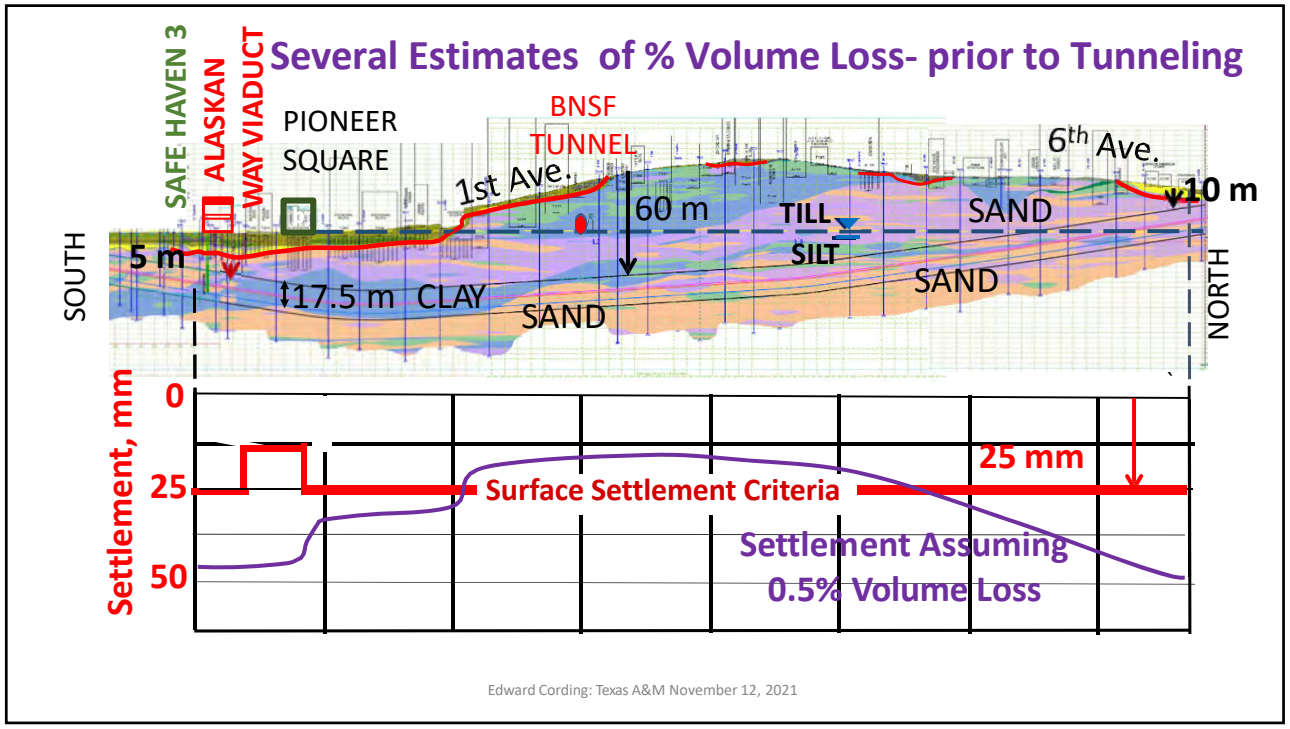
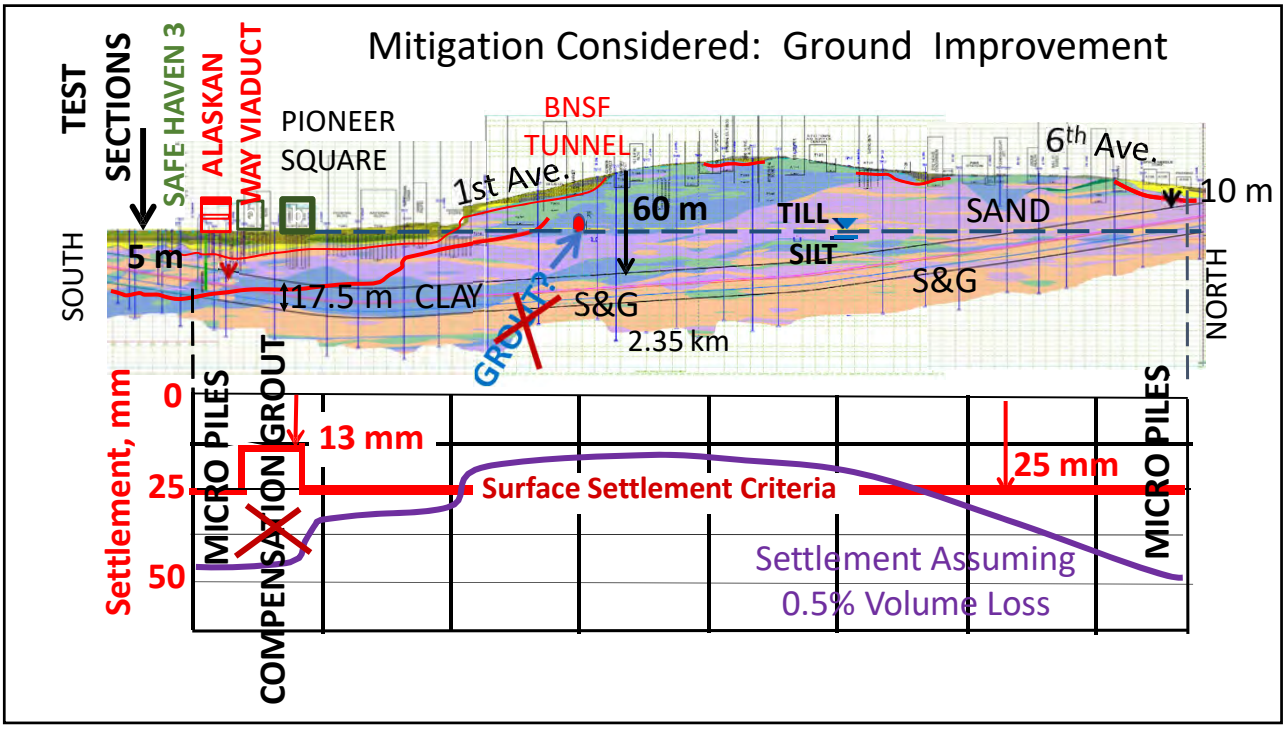
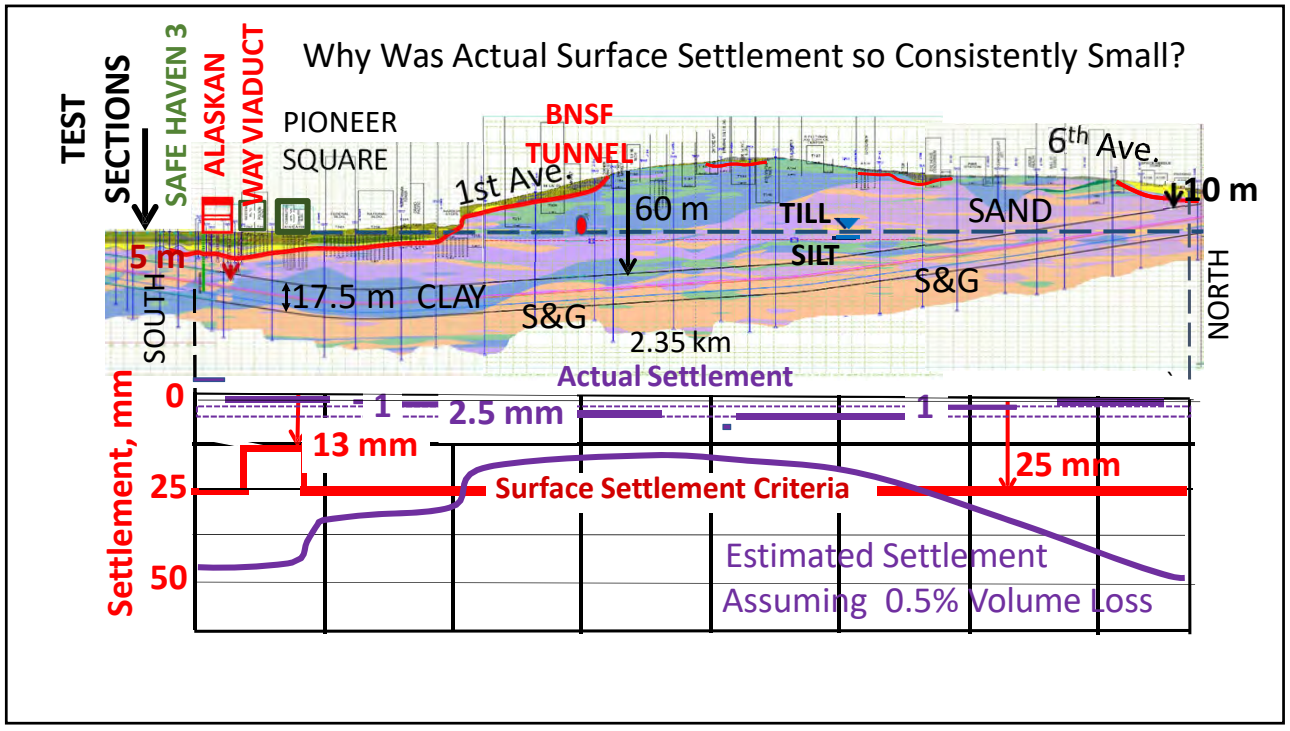


Edward Cording: Texas A&M November 12, 2021

SOUTH END SETTLEMENT MONITORING PLAN (SESMP) SECTION

Edward Dowding, Texas A&M, November 12, 2021





Washington State Department of Transportation (WSDOT)

Construction Monitoring Task Force DAILY

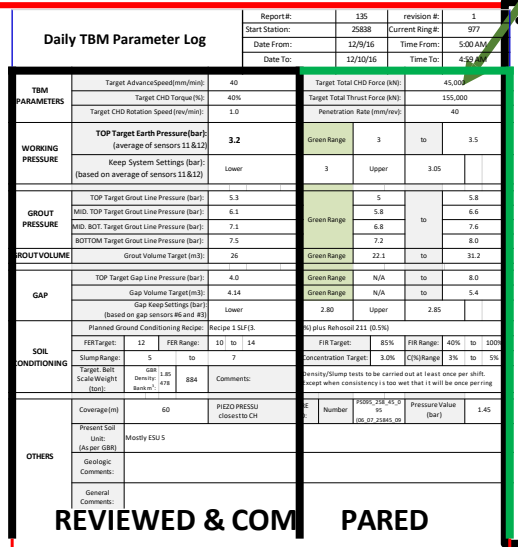
SEATTLE TUNNEL PARTNERS (STP) Tunnel Manager

Tunnel Work Plan

Daily TBM Parameter Log

TBM OPERATION

DAILY TBM Parameter Log 12/9/2016



Daily TBM Parameter Log

	Report#:	135	revision #:	1			
	Start Station:	25838	Current Ring#:	977			
	Date From:	12/9/16	Time From:	5:00 AM			
	Date To:	12/10/16	Time To:	4:59 AM			
TBM PARAMETERS	Target AdvanceSpeed(mm/min):	40	Target Total CHD Force (kN):		45,000		
	Target CHD Torque (%):	40%	Target Total Thrust Force (kN):		155,000		
	Target CHD Rotation Speed (rev/min):	1.0	Penetration Rate (mm/rev):		40		
WORKING PRESSURE	TOP Target Earth Pressure(bar): (average of sensors 11&12)	3.2	Green Range	3	to	3.5	
	Keep System Settings (bar): (based on average of sensors 11&12)	Lower	3	Upper	3.05		
GROUT PRESSURE	TOP Target Grout Line Pressure (bar):	5.3	Green Range	5	to	5.8	
	MID. TOP Target Grout Line Pressure (bar):	6.1		5.8		6.6	
	MID. BOT. Target Grout Line Pressure (bar):	7.1		6.8		7.6	
	BOTTOM Target Grout Line Pressure (bar):	7.5		7.2		8.0	
GROUT VOLUME	Grout Volume Target (m3):	26	Green Range	22.1	to	31.2	
GAP	TOP Target Gap Line Pressure (bar):	4.0	Green Range	N/A	to	8.0	
	Gap Volume Target (m3):	4.14	Green Range	N/A	to	5.4	
	Gap Keep Settings (bar): (based on gap sensors #6 and #3)	Lower	2.80	Upper	2.85		
SOIL CONDITIONING	Planned Ground Conditioning Recipe:	Recipe 1 SLF(3.	%) plus Rehosoil 211 (0.5%)				
	FERTarget: 12	FER Range: 10 to 14	FIR Target:	85%	FIR Range:	40% to 100%	
	Slump Range: 5 to 7		Concentration Target:	3.0%	C(%)Range	3% to 5%	
	Target. Belt Scale Weight (ton):	GBR Density: 1.85 t/m³ Bank 478	884	Comments:	ensity/Slump tests to be carried out at least once per shift. except when consistency i s too wet that it will be once per ring		
OTHERS	Coverage(m)	60	PIEZO PRESSU closest to CH	E:	Number	P0065_258_45_0 95 (06_07_25845_09	Pressure Value (bar) 1.45
	Present Soil Unit: (As per GBR)	Mostly ESU 5					
	Geologic Comments:						
	General Comments:						

REVIEWED & COM PARED

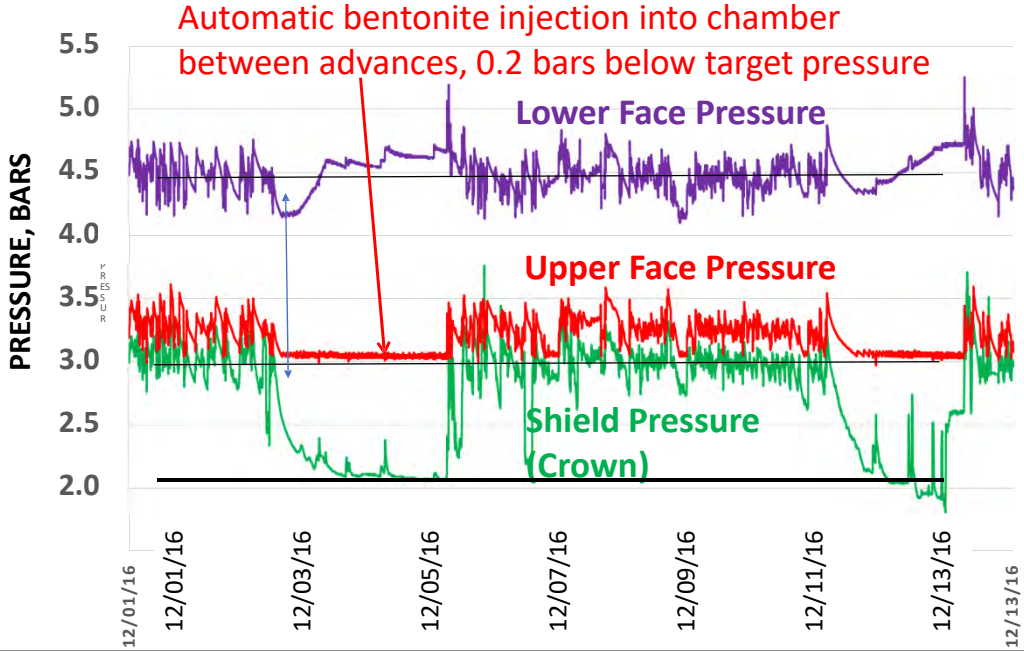
WITH PREVIOUS DAYS SHOVES

BY CONST. MON. TASK FORCE

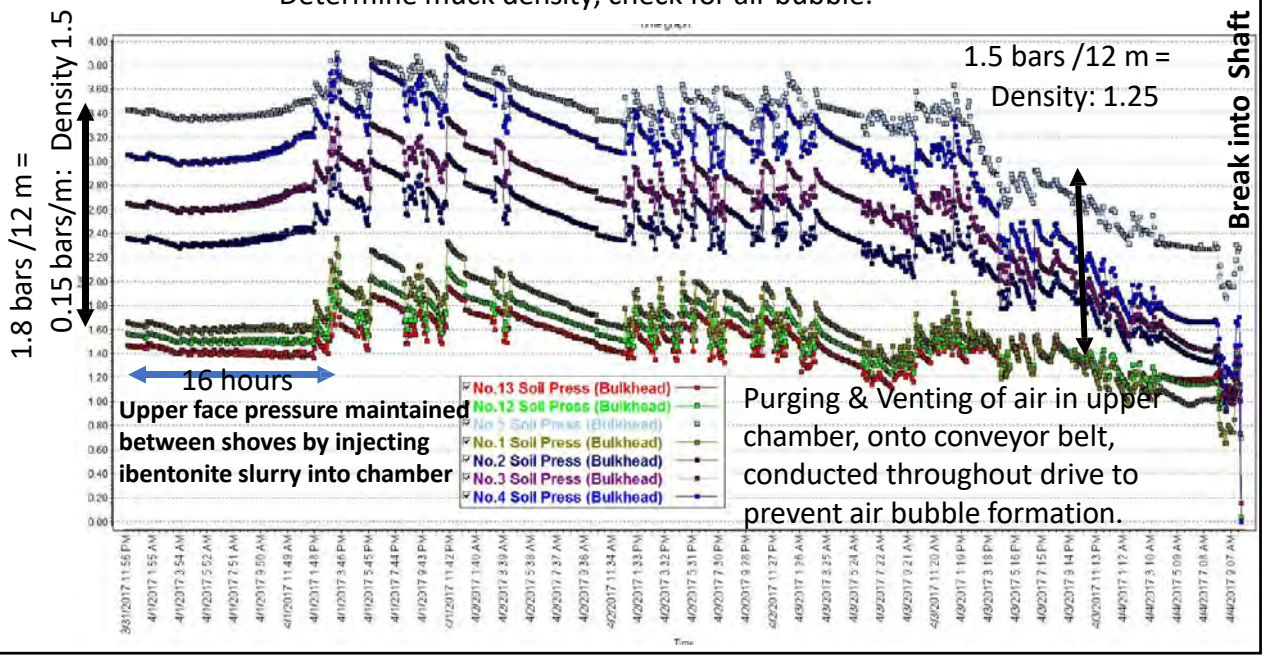
TARGETS & GREEN RANGE 12/9/16

- **TBM** Torque, Thrust, Rotation, Penetration Rate.
- **FACE** PRESSURE, UPPER
  - During 2-m Advances: 3.2 bar
  - **•Between pushes: Automatic bentonite injection into chamber** 3.0 bar
- **SHIELD GAP Injection during push: 4.14 cu m**
  - Pressure between pushes: 2.8 bar
- **TAIL** GROUT PRESSURE: 5.3 – 7.5 bar
  VOLUME 26 cu m
- **CONDITIONING of Muck: foam, polymer**
  FIR: 85%, FER: 12, Concentration: 3 %
- **MUCK WEIGHT**, SLUMP: 5-7,
  - Reconciliation with theoretical volume during advance: +2 to -4%
- **Automatic Venting** top of chamber to prevent air accumulation

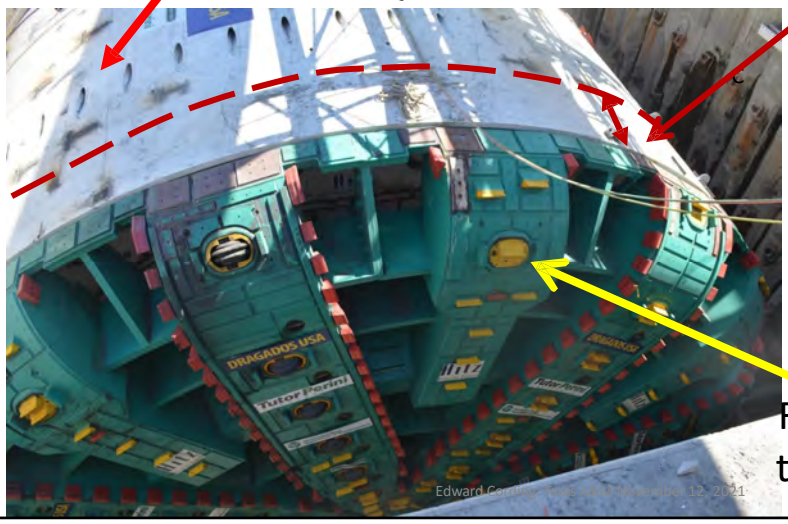
17.5-m EPBM: FACE & SHIELD PRESSURE 12/1/16 – 12/13/16



April 1-4, 2017: Approaching Exit Shaft: Pressure distribution across height of chamber (Face). Determine muck density, check for air bubble.



- BENTONITE SLURRY INJECTED THROUGH PORTS INTO 34-mm SHIELD GAP
- 6 SHIELD PRESSURE GAUGES ON UPPER SHIELD BODY



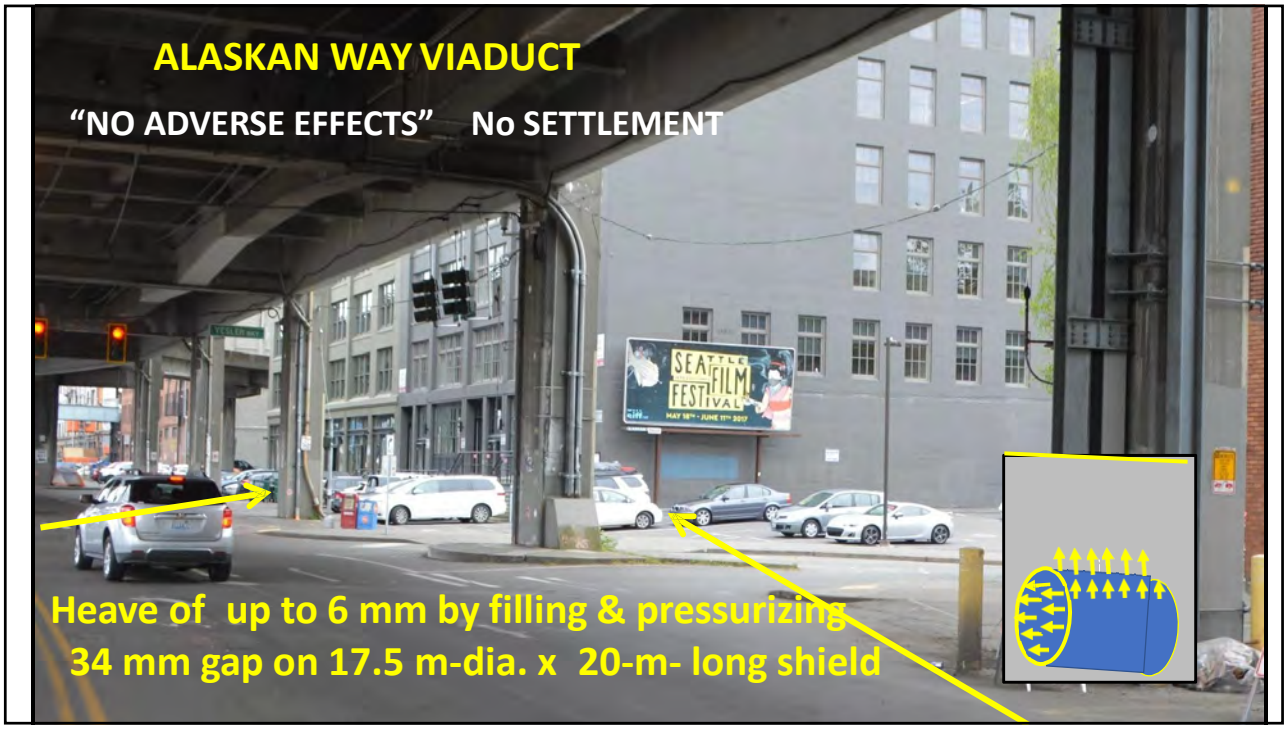
CHAMBER PRESSURIZED WITH CONDITIONED MUCK

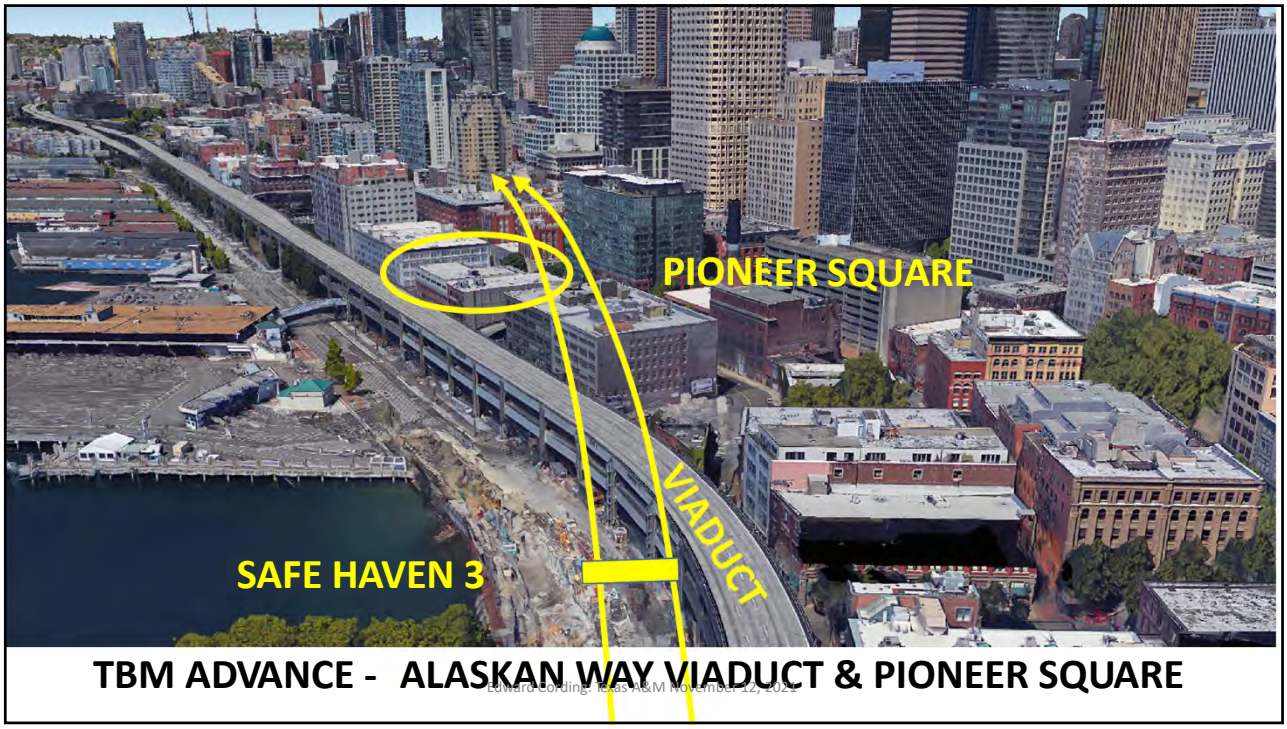
13 "FACE"PRESSURE GAUGES on back wall of Chamber

17.5-m EPBM

Free Air Access into Spokes to Change Cutters (Rippers)


PIONEER
SQUARE
VIADUCT
April 29, 2016
SAFE HAVEN 3
VIADUCT
TEST SECTIONS 1 & 2

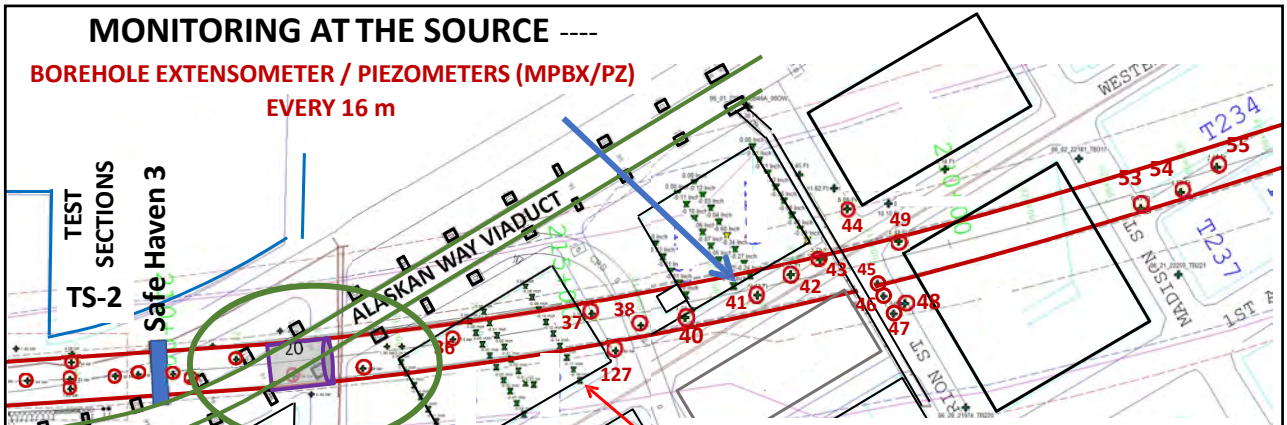




TBM ADVANCE - ALASKAN WAY VIADUCT & PIONEER SQUARE

MONITORING AT THE SOURCE ----

BOREHOLE EXTENSOMETER / PIEZOMETERS (MPBX/PZ)
EVERY 16 m

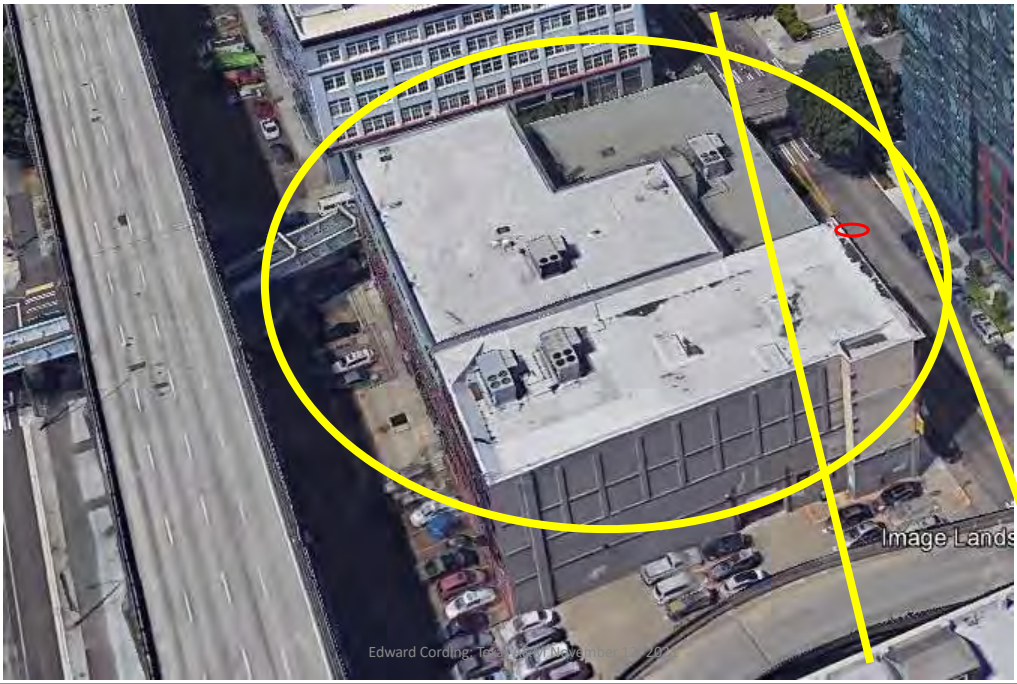


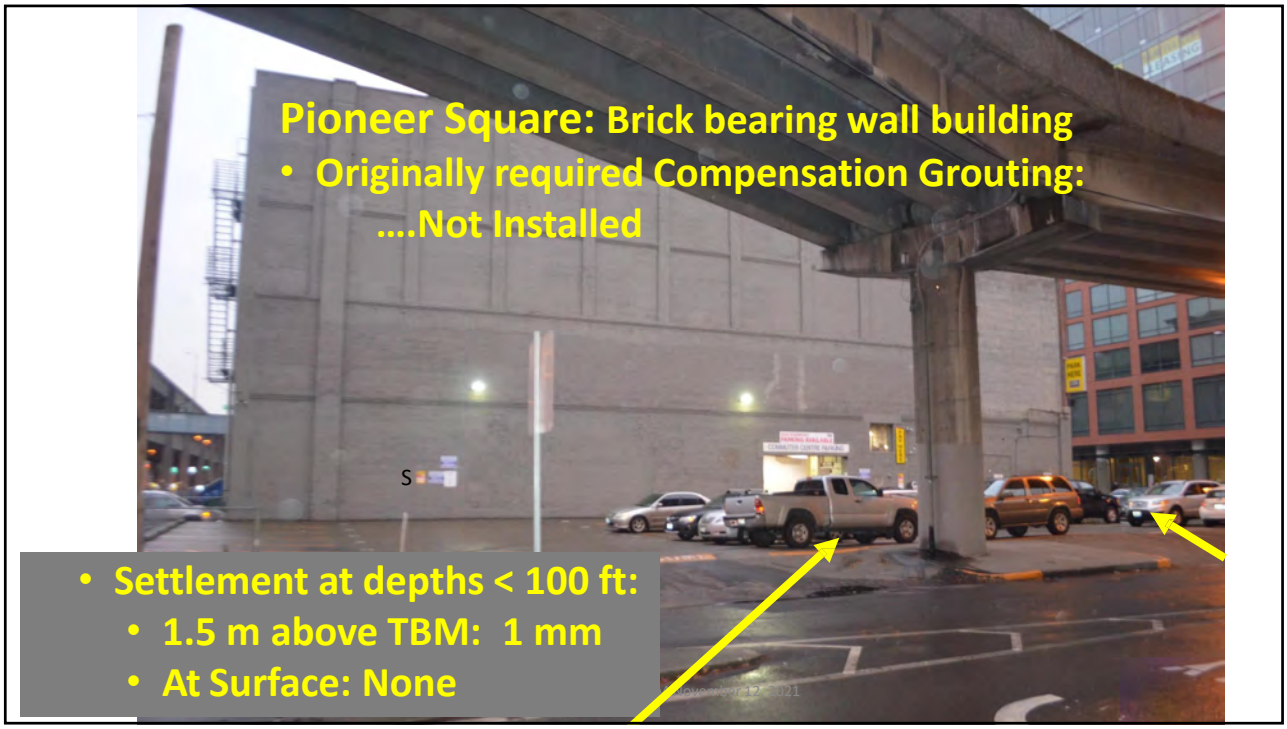
STP: GEOTECHNICAL MONITORING REPORT, EVERY 2-M ADVANCE

Construction Monitoring Task Force: Daily Meetings

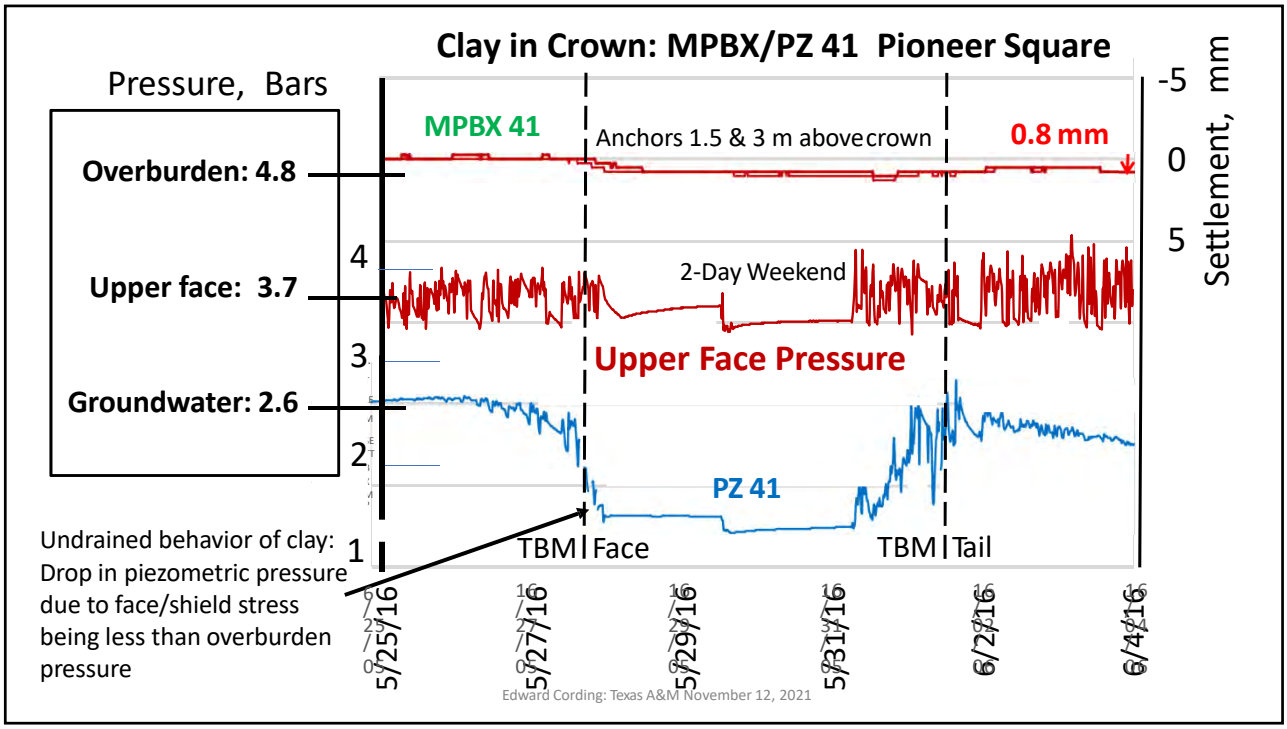
MONITORING AT THE SURFACE ---

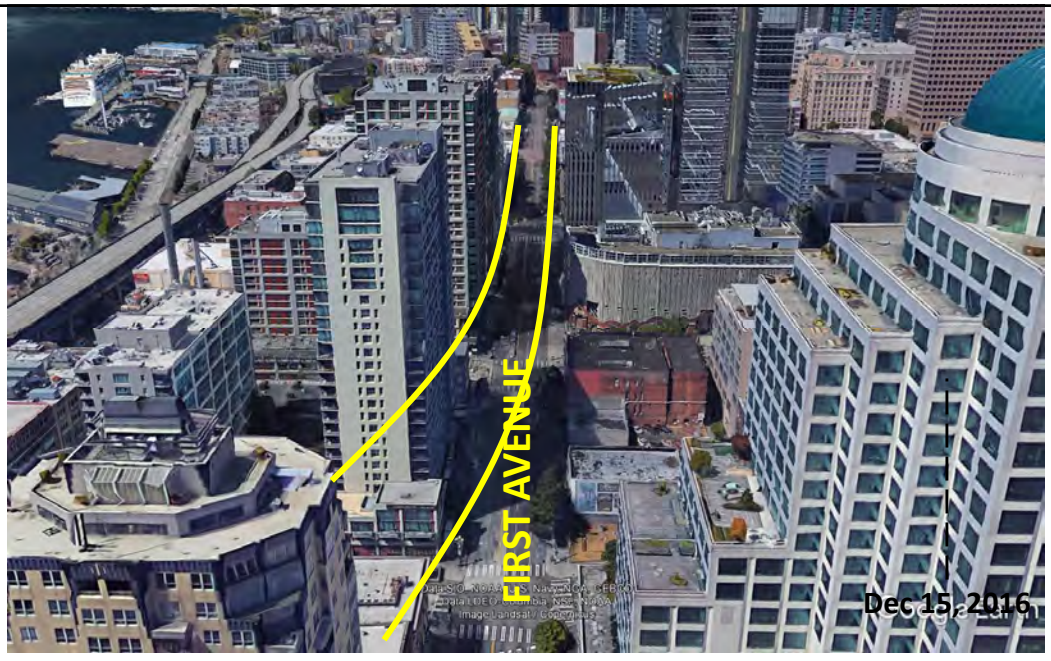
AUTOMATIC STRUCTURE MONITORING POINTS
(TOTAL STATION)
LIQUID LEVELS, LEVEL SURVEYS
---- TO CONFIRM NO IMPACTS



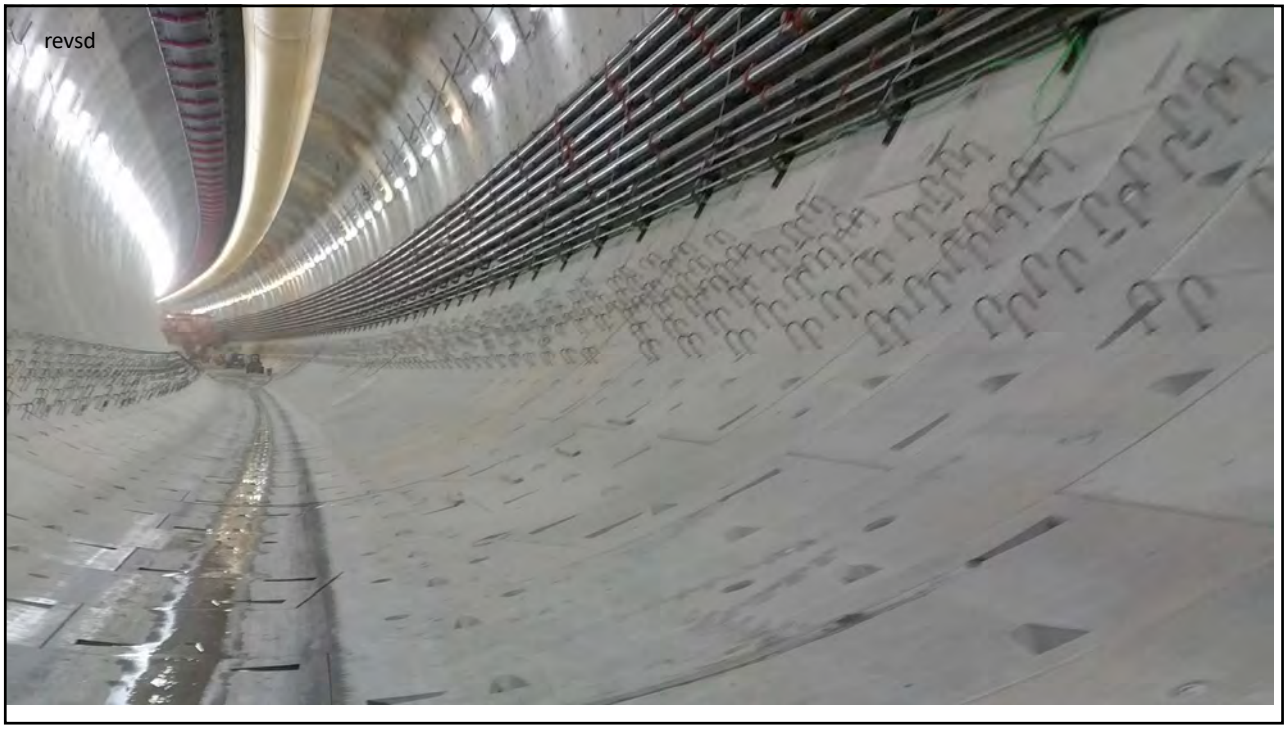

Pioneer Square: Brick bearing wall building
• Originally required Compensation Grouting:
….Not Installed
• Settlement at depths < 100 ft:
• 1.5 m above TBM: 1 mm
• At Surface: None

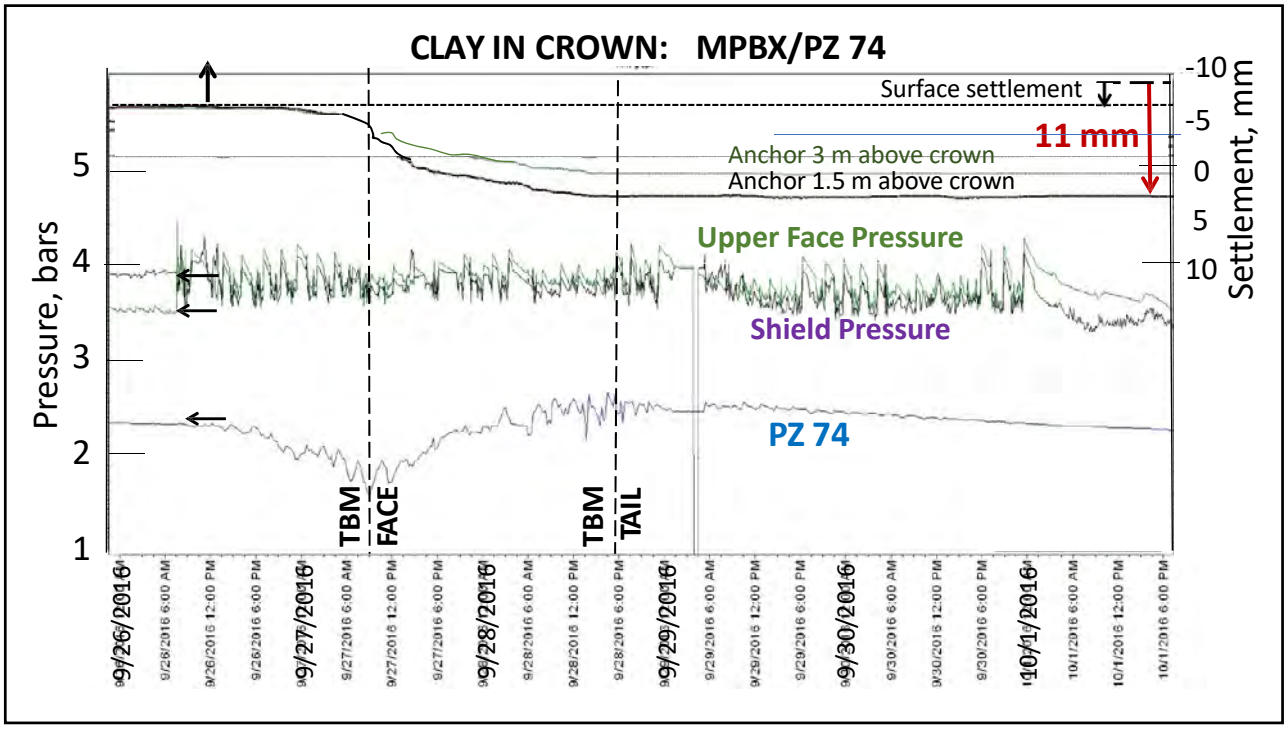

PARK HERE
MPBX/PZ 41

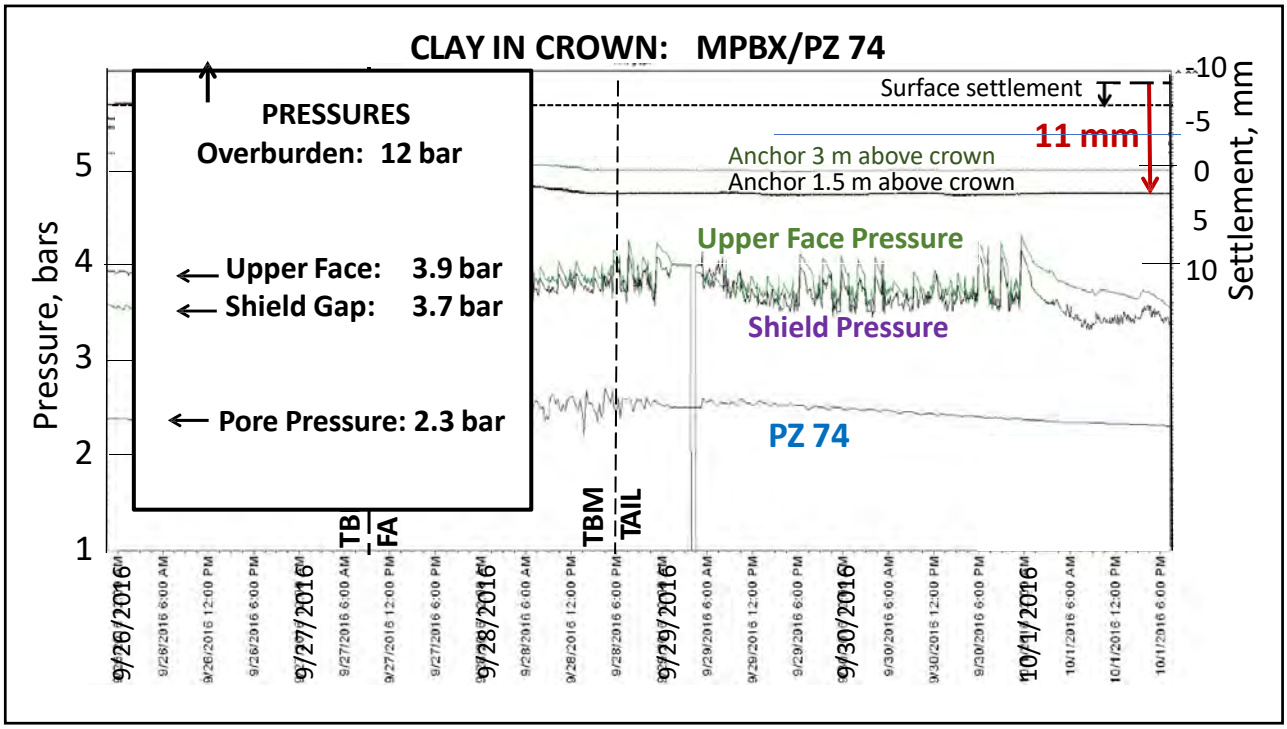




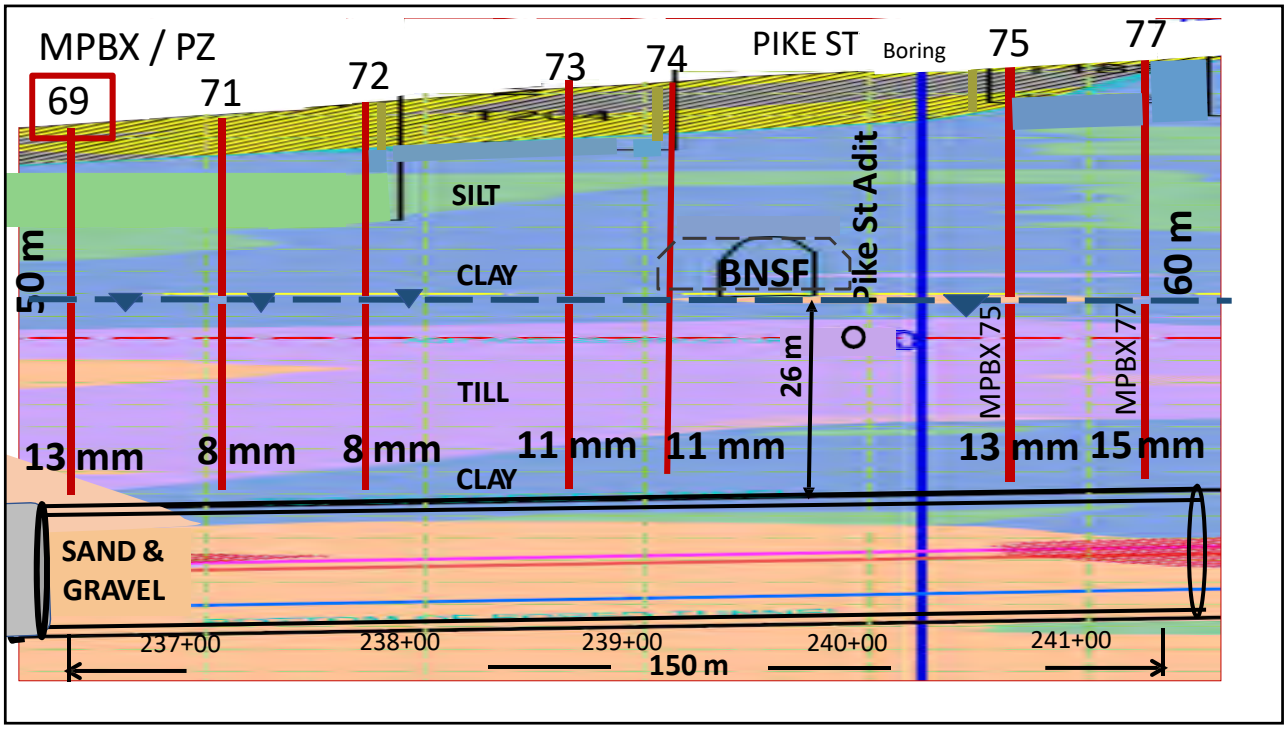
FIRST AVENUE HILL: INCREASING TUNNEL DEPTH

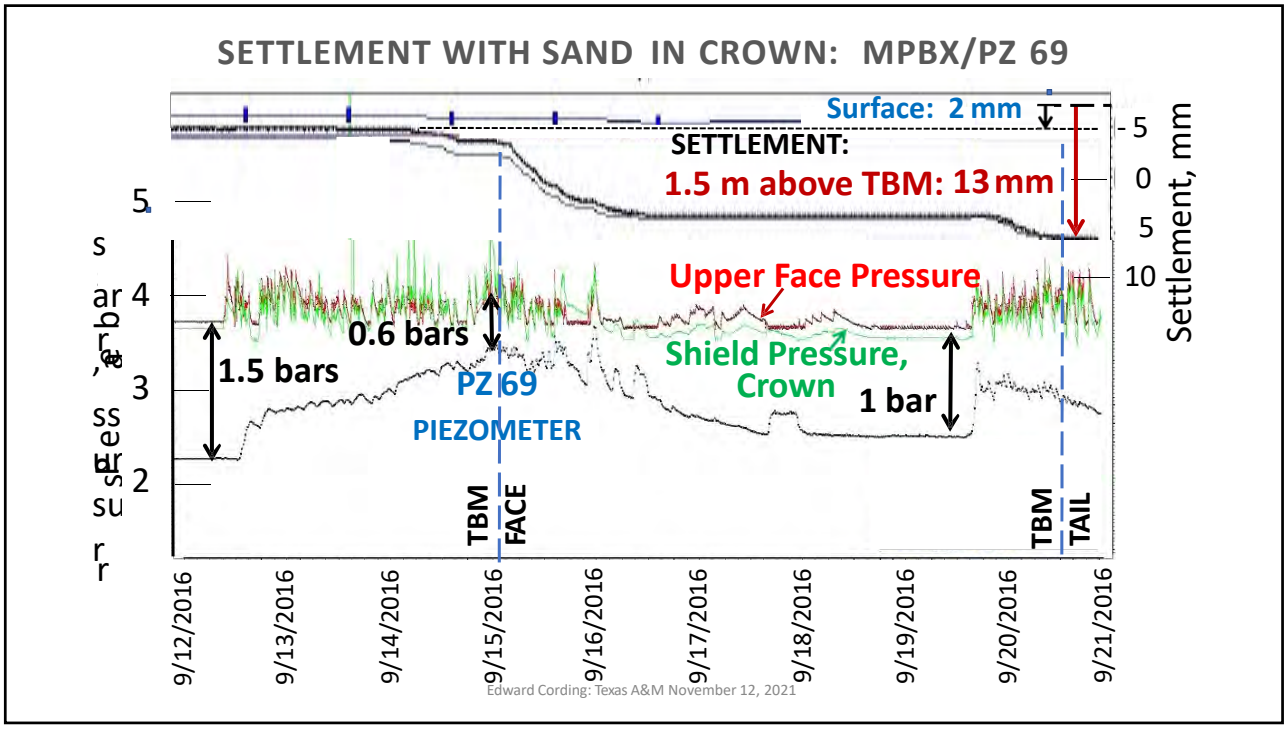

revsd

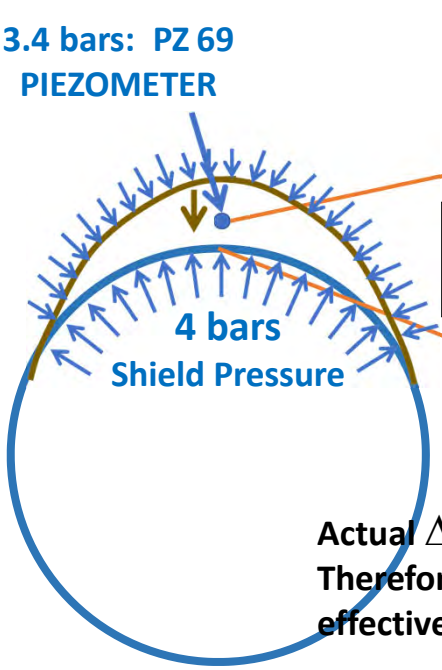

CLAY IN CROWN: MPBX/PZ 74
Surface settlement
11 mm
Anchor 3 m above crown
Anchor 1.5 m above crown
Upper Face Pressure
Shield Pressure
PZ 74
TBM FACE
TBM TAIL
Pressure, bars
Settlement, mm
1
2
3
4
5
-10
-5
0
5
10
9/26/2016
9/27/2016
9/28/2016
9/29/2016
9/30/2016
10/1/2016



Can Overcut Gap be pressurized & support the sand?
…. Added Polymer to the Bentonite Slurry


MPBX / PZ
69
71
72
73
74
PIKE ST Boring
75
77
SILT
CLAY
BNSF
Pike St Adit
50 m
60 m
26 m
MPBX 75
MPBX 77
TILL
13 mm
8 mm
8 mm
11 mm
11 mm
13 mm
15 mm
CLAY
SAND &
GRAVEL
237+00
238+00
239+00
240+00
241+00
150 m


SETTLEMENT WITH SAND IN CROWN: MPBX/PZ 69
Surface: 2 mm
SETTLEMENT:
1.5 m above TBM: 13 mm
Upper Face Pressure
0.6 bars
1.5 bars
PZ 69
PIEZOMETER
Shield Pressure, Crown
1 bar
TBM FACE
TBM TAIL
Settlement, mm
9/12/2016
9/13/2016
9/14/2016
9/15/2016
9/16/2016
9/17/2016
9/18/2016
9/19/2016
9/20/2016
9/21/2016



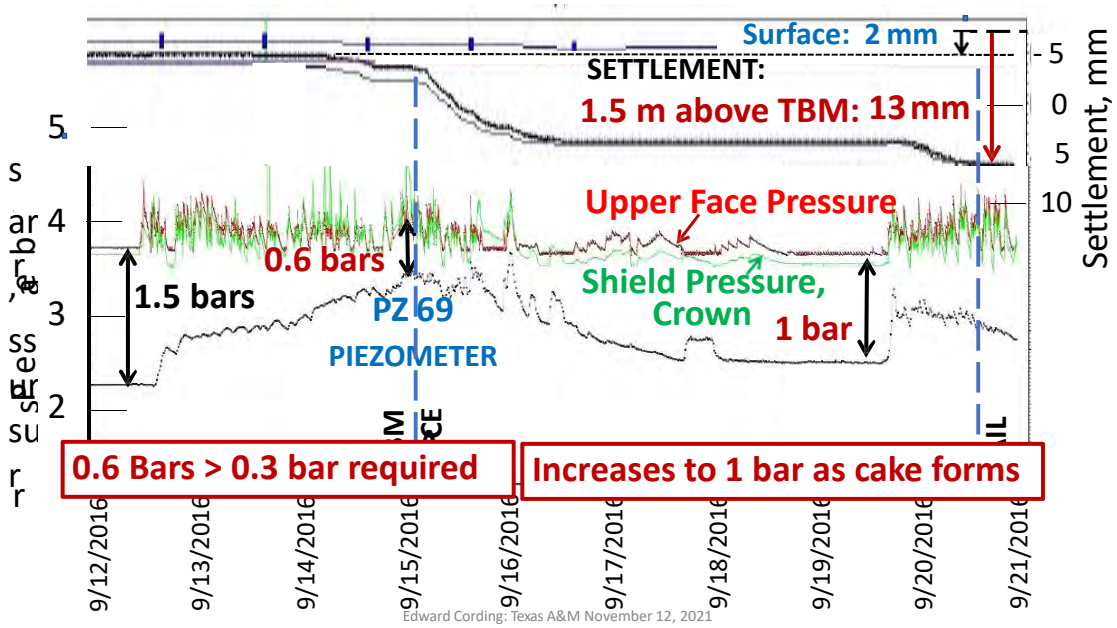
Piezometric Pressure 3.4 bars

TBM Shield Pressure: 4 bars

0.15 bars: hydrostatic
+
0.15 bars: Effective
= 0.3 bars: Minimum Differential

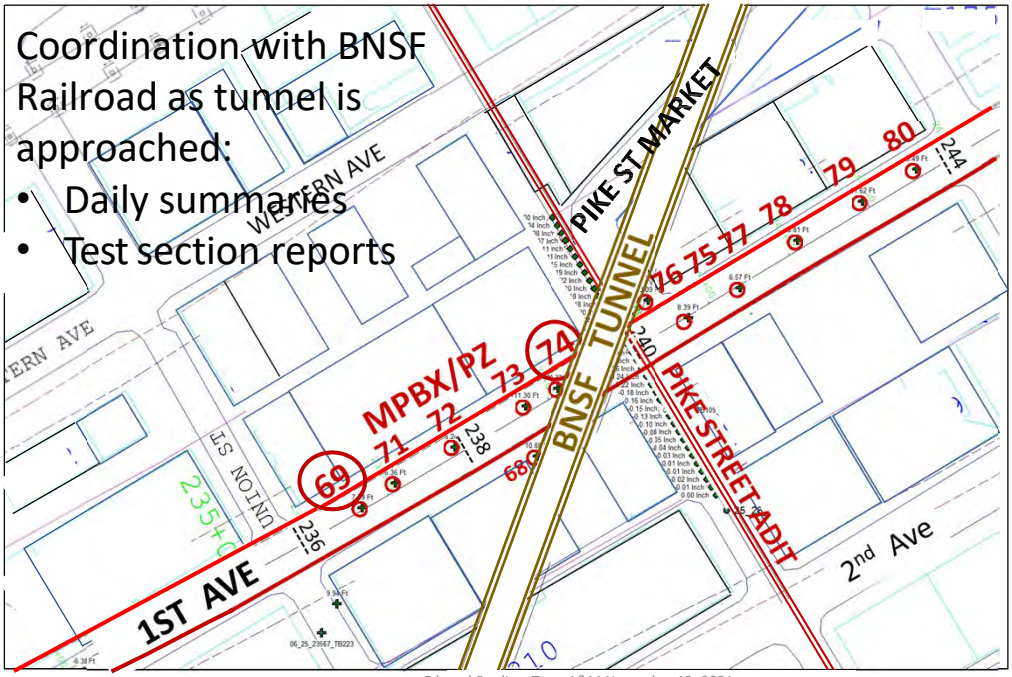
Actual Δ = 0.6 bars > 0.3 Minimum Differential
Therefore shield pressure supports both the effective soil weight and the weight of water

SETTLEMENT WITH SAND IN CROWN: MPBX/PZ 69



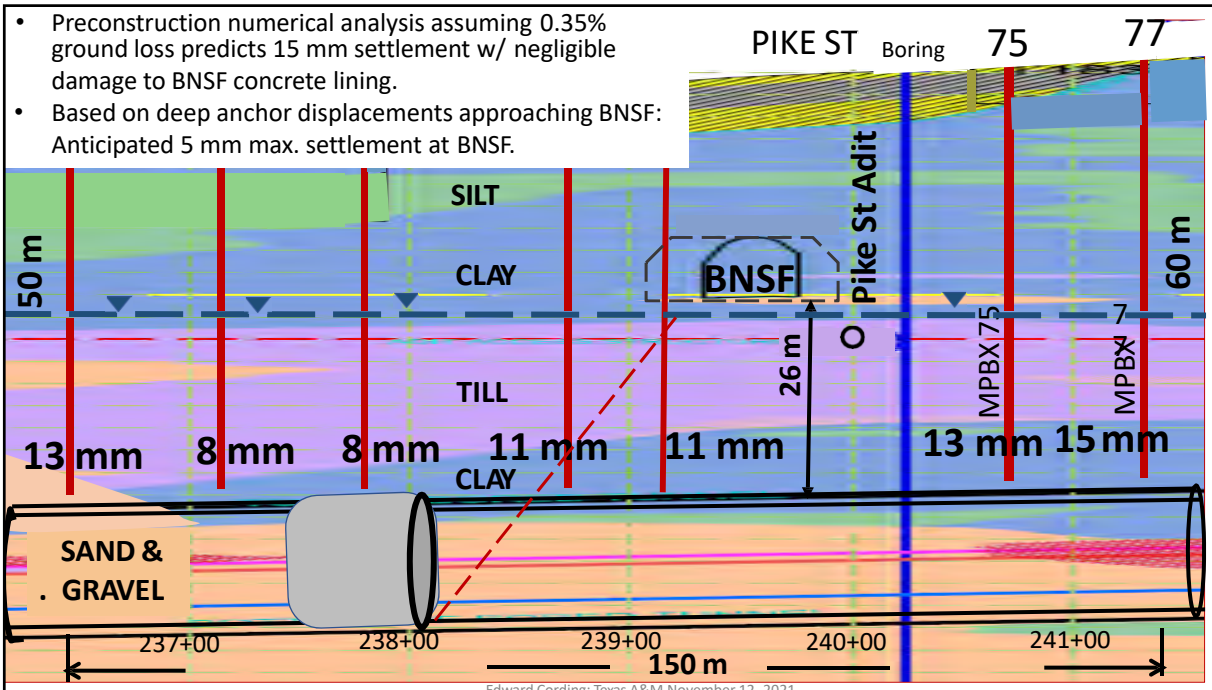
Coordination with BNSF Railroad as tunnel is approached:

- Daily summaries
- Test section reports



Edward Cording: Texas A&M November 12, 2021

- Preconstruction numerical analysis assuming 0.35% ground loss predicts 15 mm settlement w/ negligible damage to BNSF concrete lining.
- Based on deep anchor displacements approaching BNSF: Anticipated 5 mm max. settlement at BNSF.



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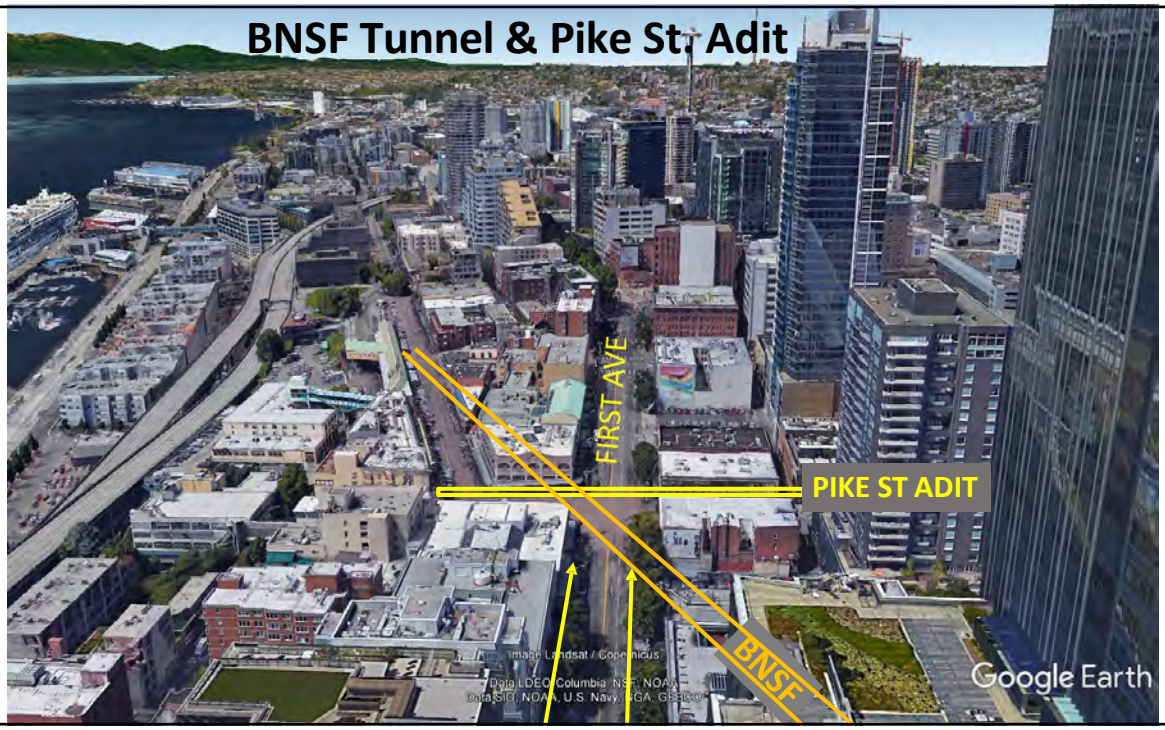
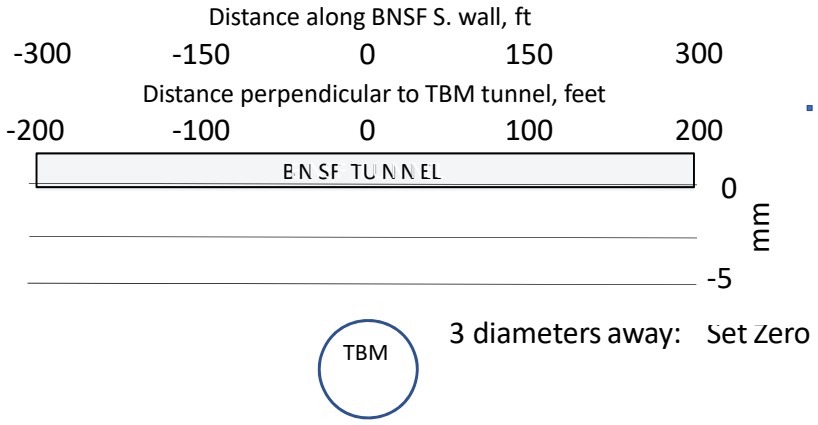
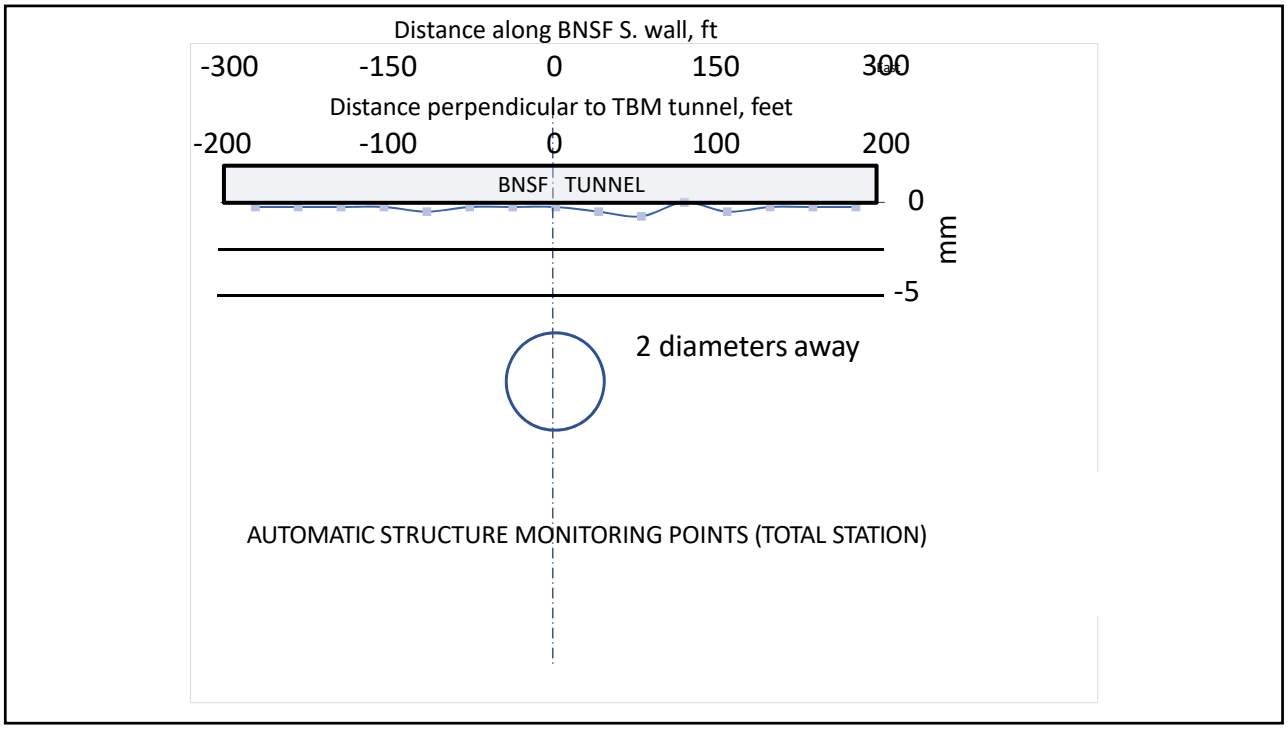
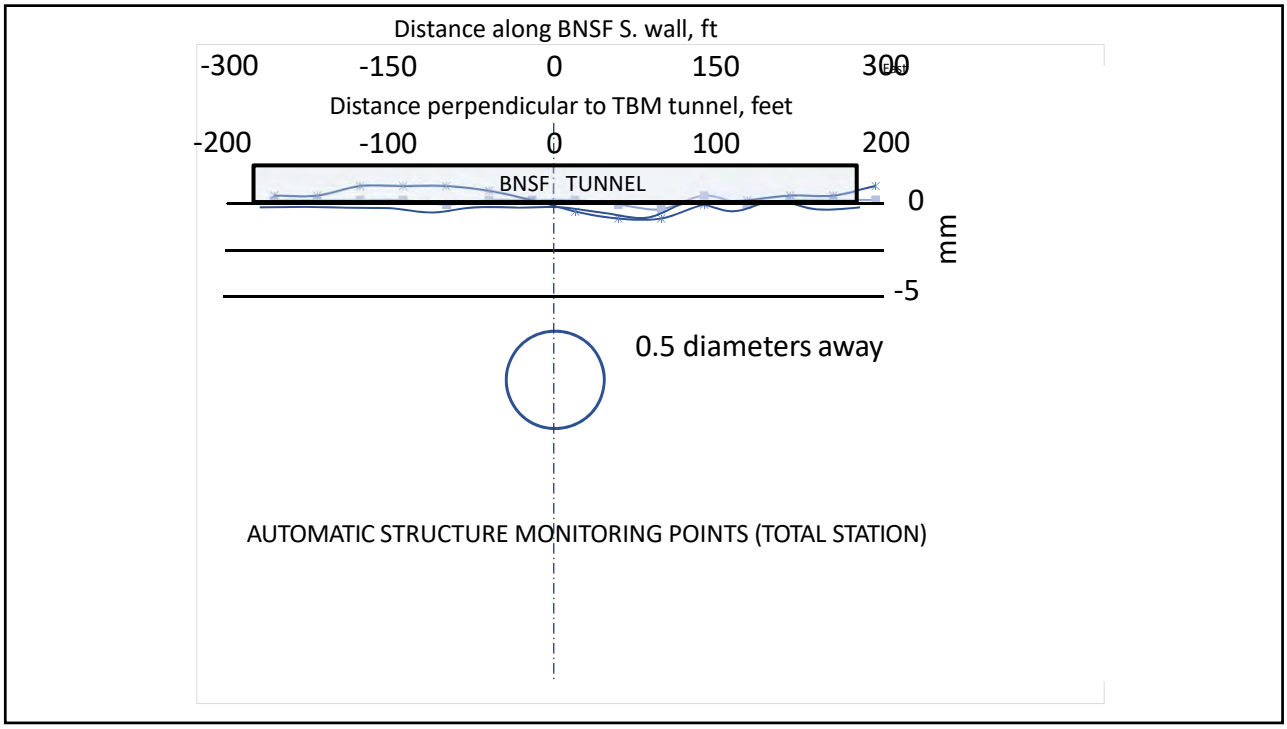
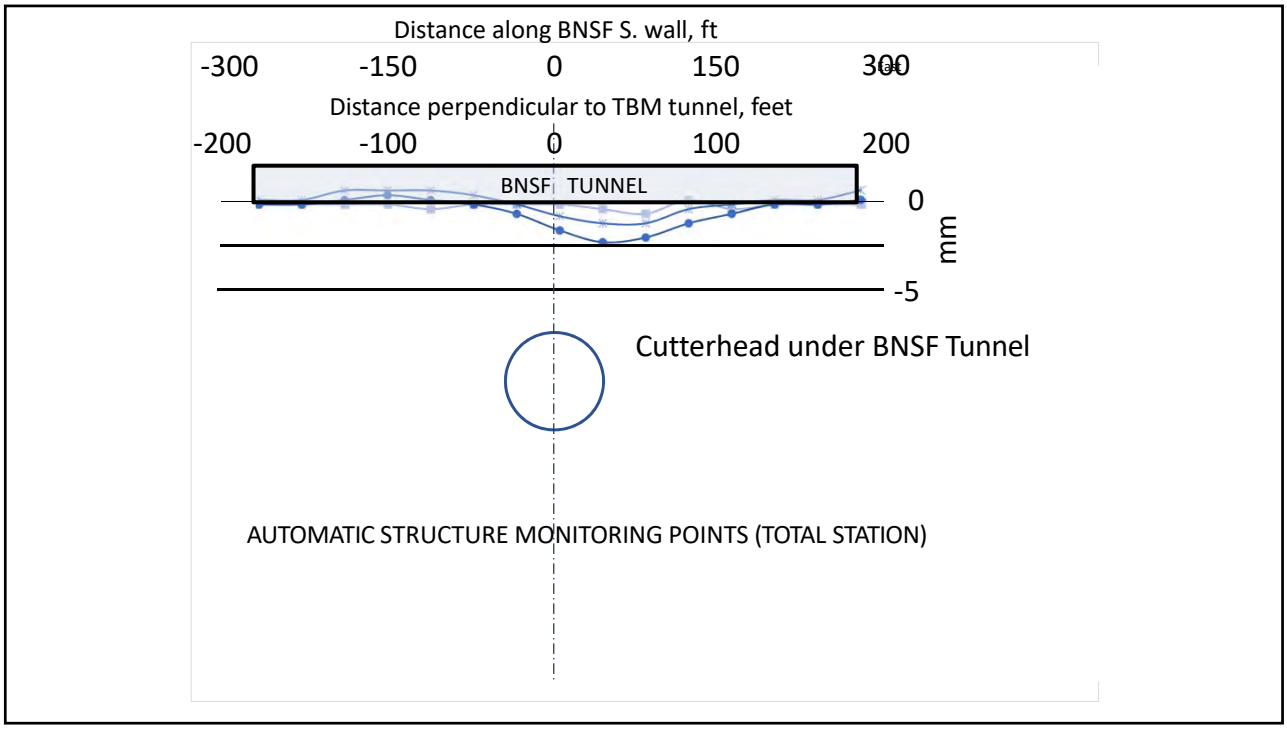


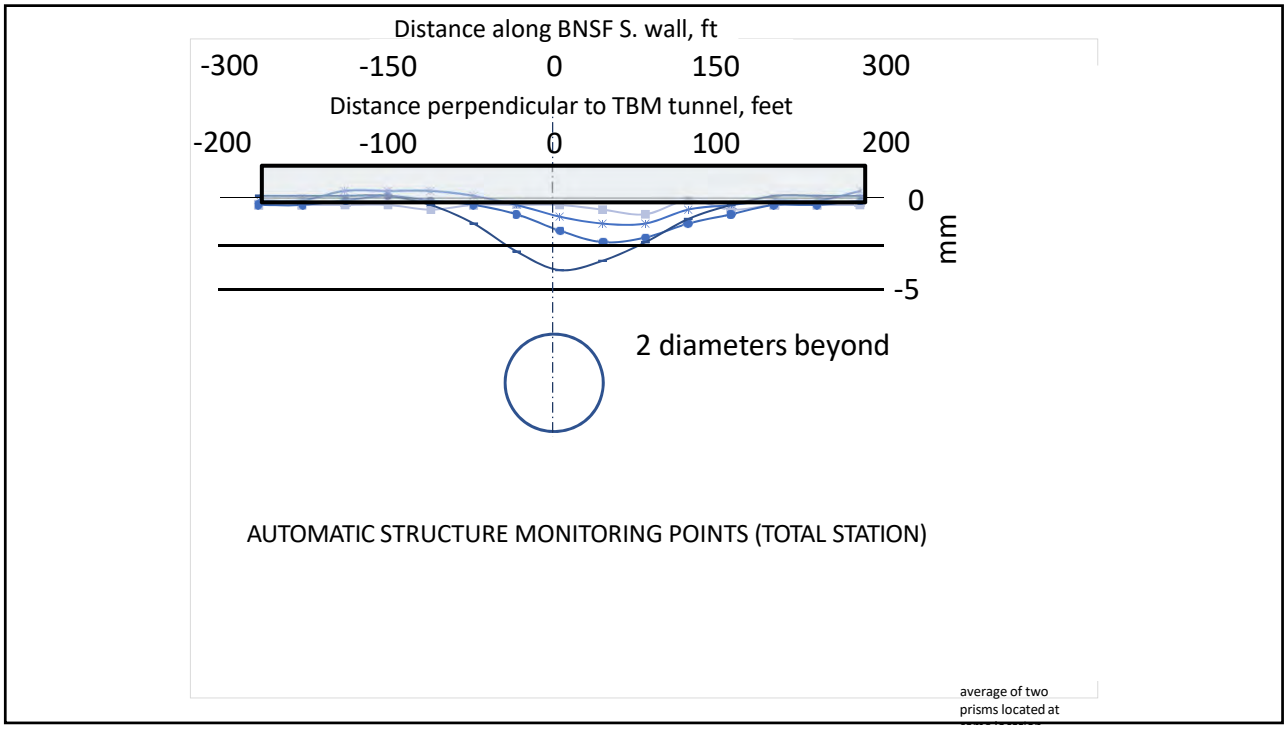


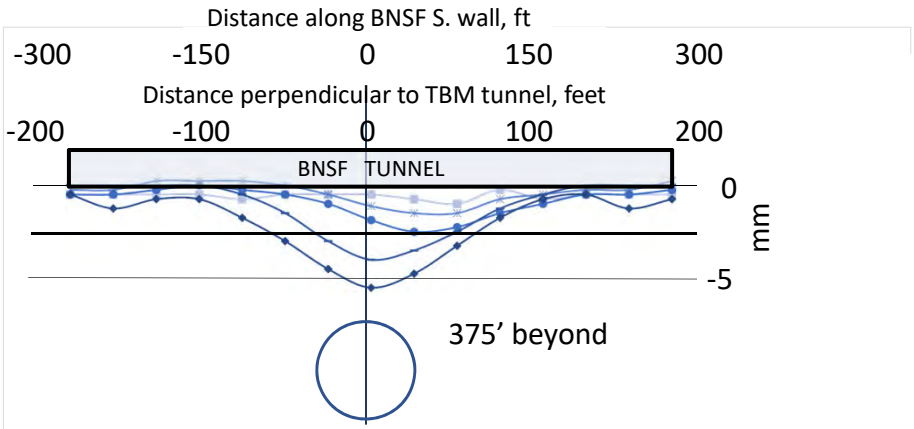
AUTOMATIC STRUCTURE MONITORING POINTS (TOTAL STATION)
LINES ON TIES & WALLS OF BNSF TUNNEL


Distance along BNSF S. wall, ft
-300
-150
0
150
300
Distance perpendicular to TBM tunnel, feet
-200
-100
0
100
200
BNSF TUNNEL
0
mm
-5
2 diameters away
AUTOMATIC STRUCTURE MONITORING POINTS (TOTAL STATION)

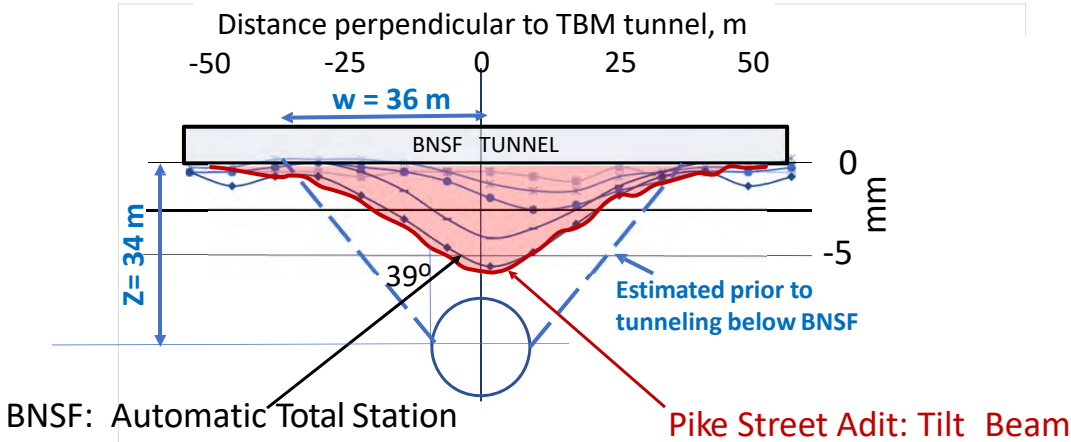

Distance along BNSF S. wall, ft
-300
-150
0
150
300
Distance perpendicular to TBM tunnel, feet
-200
-100
0
100
200
BNSF TUNNEL
0
mm
-5
0.5 diameters away
AUTOMATIC STRUCTURE MONITORING POINTS (TOTAL STATION)


Distance along BNSF S. wall, ft
-300
-150
0
150
300
Distance perpendicular to TBM tunnel, feet
-200
-100
0
100
200
BNSF TUNNEL
0
-5
mm
Cutterhead under BNSF Tunnel
AUTOMATIC STRUCTURE MONITORING POINTS (TOTAL STATION)

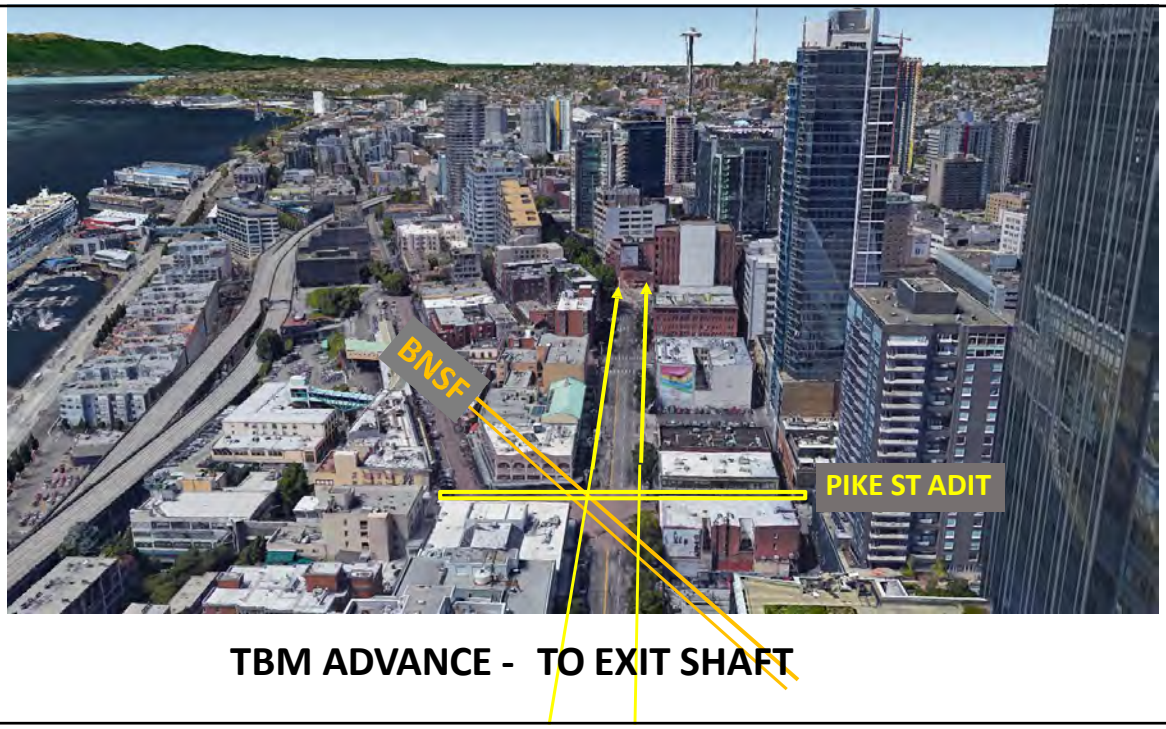

Distance along BNSF S. wall, ft
-300
-150
0
150
300
Distance perpendicular to TBM tunnel, feet
-200
-100
0
100
200
0
-5
mm
2 diameters beyond
AUTOMATIC STRUCTURE MONITORING POINTS (TOTAL STATION)
average of two
prisms located at



Excellent Precision with small settlement



- Excellent precision in both tunnels for such small displacements
- Useful for evaluating numerical analyses and empirical relations
- Trough half width: w=2.5i = radius+z tan 39° = 8.75 m + 0.8 Z = 36 m
- Volume: 36m x 0.005m = 0.180 cu m/m ; % Vol = 0.07%



TBM ADVANCE - TO EXIT SHAFT

DEPTH DECREASES to 10 m **Clean Sand in crown**

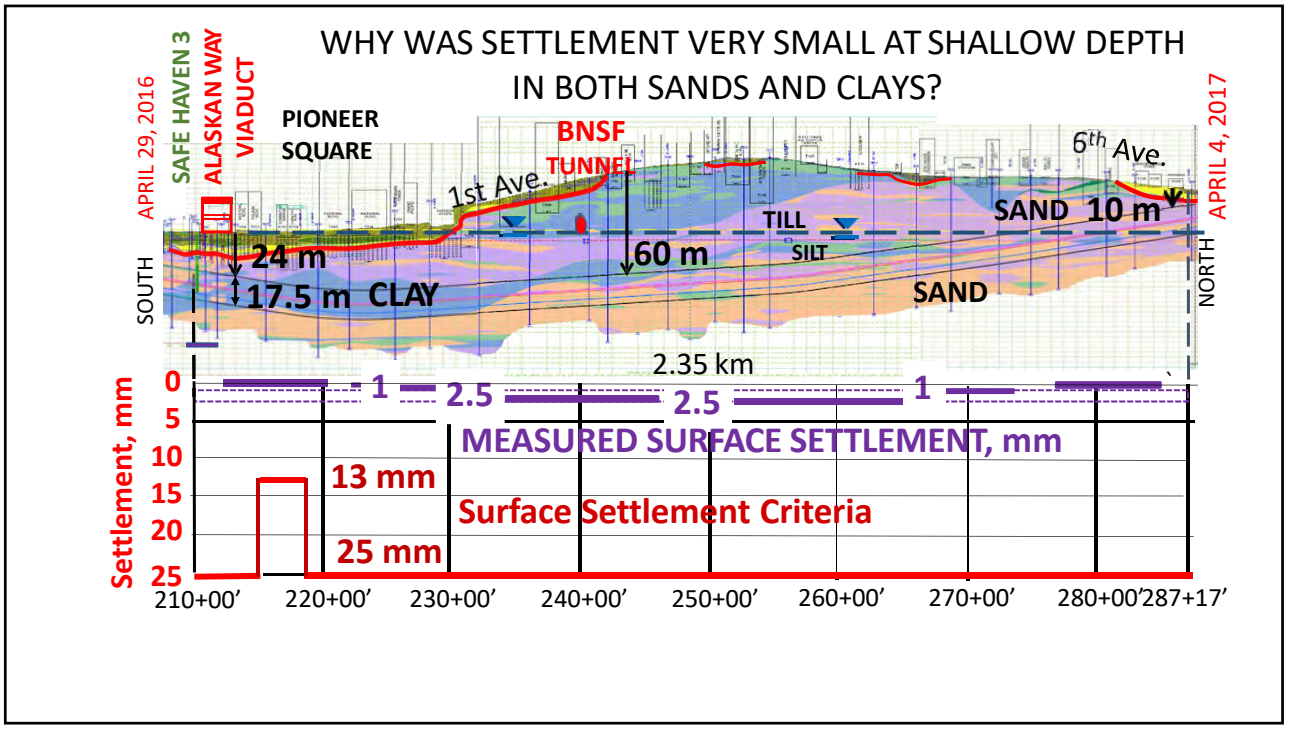
Will Injection & Pressurization of Shield Gap (7.5-m-dia. x 20-m-long) support the sands & prevent ground loss?

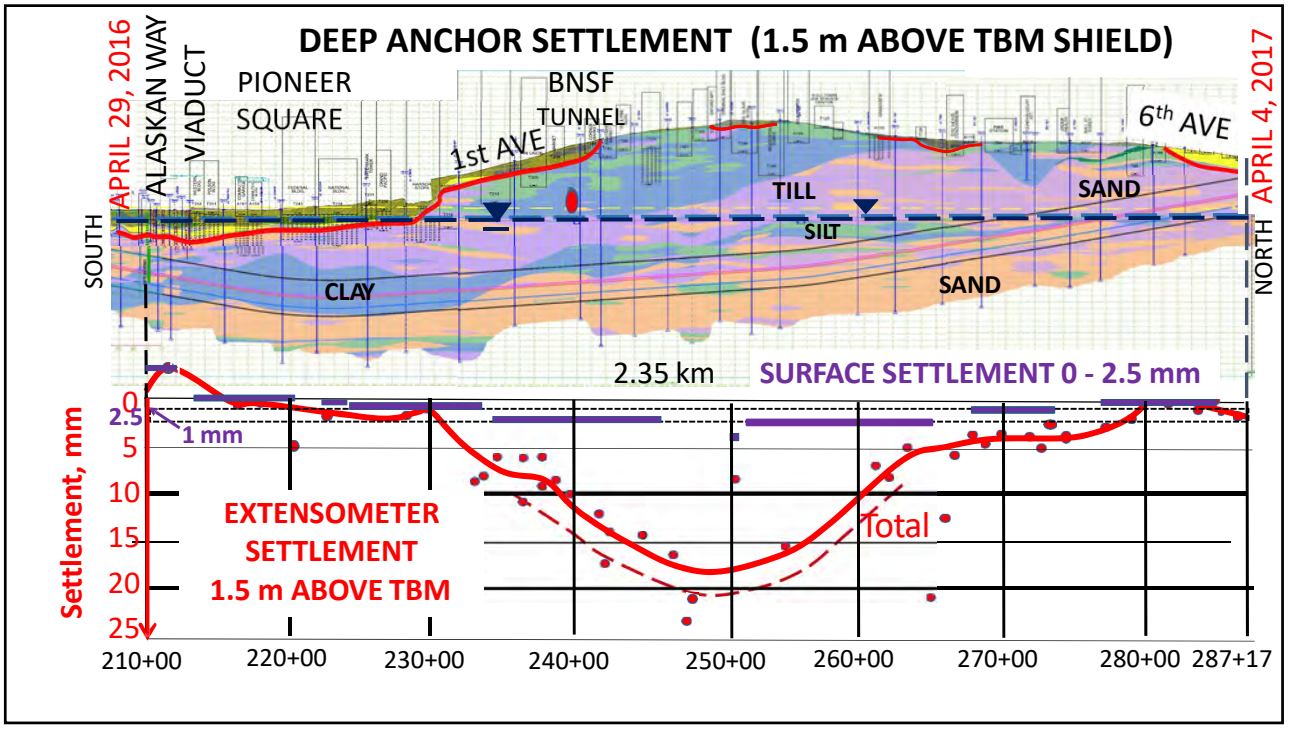
revsd

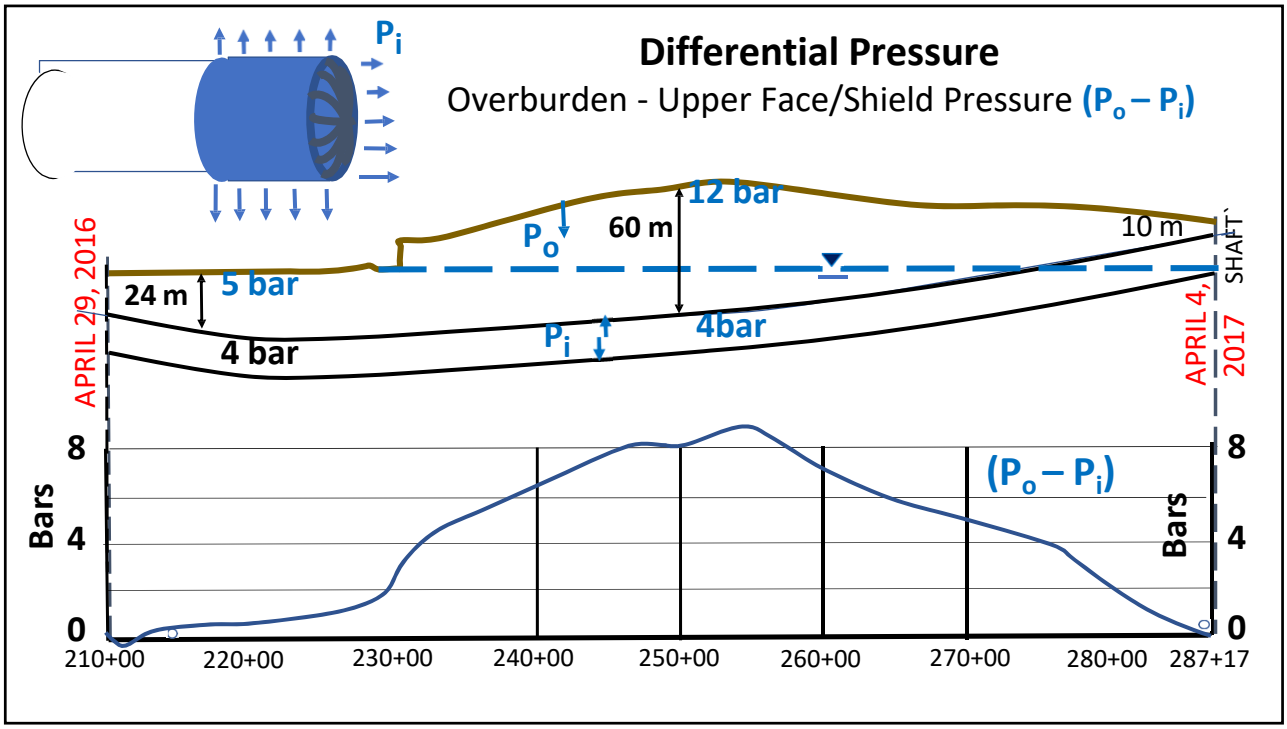
DEPTH DECREASES to 10 m **Clean Sand in crown**

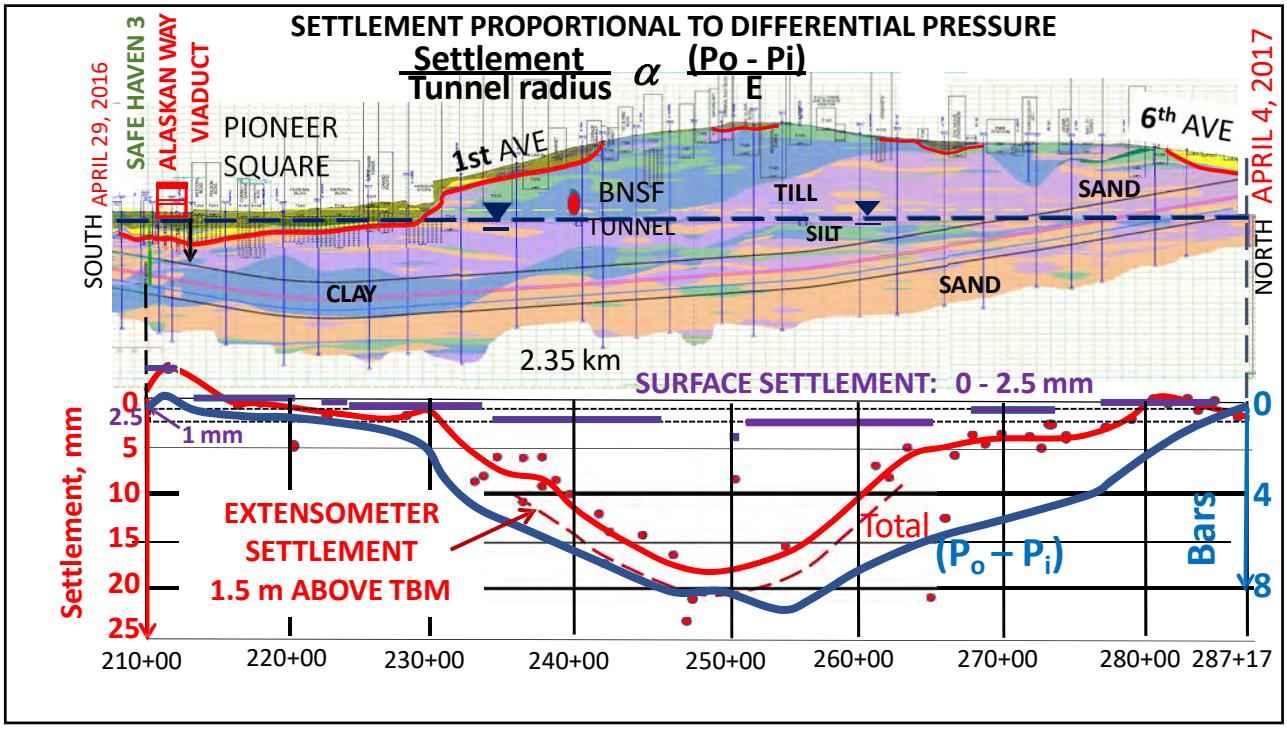
Bentonite w/polymer injected in overcut

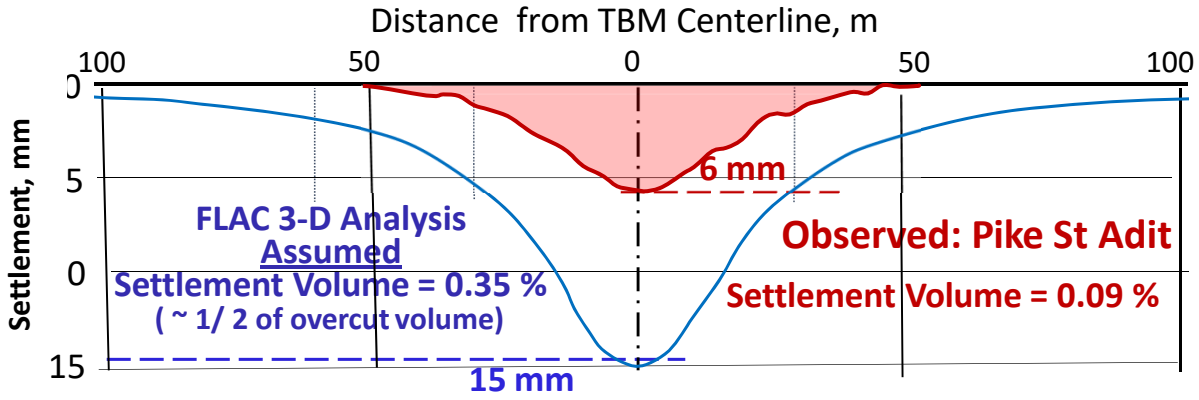
- **No significant surface or deep settlement**


WHY WAS SETTLEMENT VERY SMALL AT SHALLOW DEPTH
IN BOTH SANDS AND CLAYS?
APRIL 29, 2016
SAFE HAVEN 3
ALASKAN WAY VIADUCT
PIONEER SQUARE
BNSF TUNNEL
1st Ave.
6th Ave.
APRIL 4, 2017
SOUTH
NORTH
TILL
SILT
SAND 10 m
24 m
17.5 m CLAY
60 m
SAND
2.35 km
Settlement, mm
0
5
10
15
20
25
1
2.5
2.5
1
MEASURED SURFACE SETTLEMENT, mm
13 mm
Surface Settlement Criteria
25 mm
210+00'
220+00'
230+00'
240+00'
250+00'
260+00'
270+00'
280+00'
287+17'


DEEP ANCHOR SETTLEMENT (1.5 m ABOVE TBM SHIELD)
APRIL 29, 2016
ALASKAN WAY VIADUCT
PIONEER SQUARE
BNSF TUNNEL
1st AVE
6th AVE
APRIL 4, 2017
SOUTH
NORTH
TILL
SILT
SAND
CLAY
SAND
2.35 km
SURFACE SETTLEMENT 0 - 2.5 mm
Settlement, mm
0
2.5
5
10
15
20
25
1 mm
EXTENSOMETER SETTLEMENT
1.5 m ABOVE TBM
Total
210+00
220+00
230+00
240+00
250+00
260+00
270+00
280+00
287+17

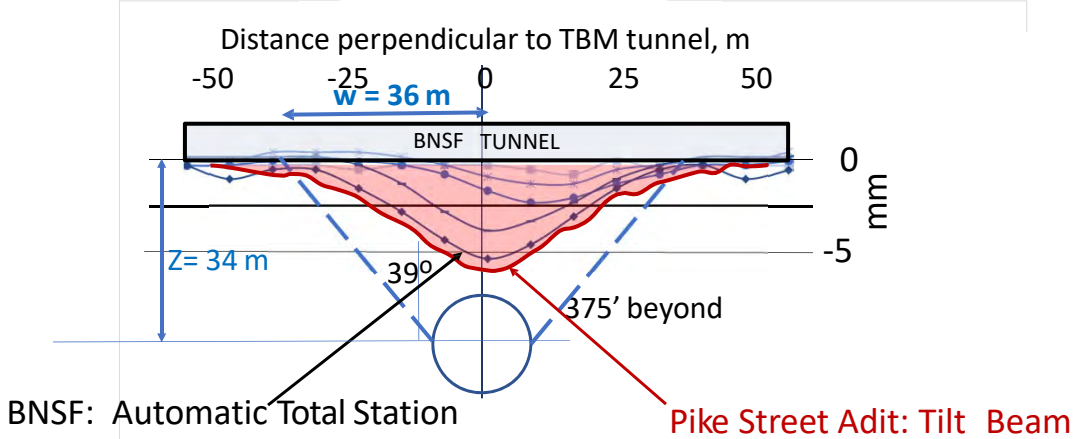

Differential Pressure
Overburden - Upper Face/Shield Pressure (Po – Pi)
Pi
Po
12 bar
60 m
10 m
SHAFT
APRIL 29, 2016
APRIL 4, 2017
24 m
5 bar
4 bar
Pi
4bar
Bars
8
4
0
(Po – Pi)
210+00
220+00
230+00
240+00
250+00
260+00
270+00
280+00
287+17


SETTLEMENT PROPORTIONAL TO DIFFERENTIAL PRESSURE
Settlement / Tunnel radius α (Po - Pi) / E
SOUTH APRIL 29, 2016
SAFE HAVEN 3
ALASKAN WAY VIADUCT
PIONEER SQUARE
1st AVE
BNSF TUNNEL
6th AVE
TILL
SILT
SAND
CLAY
SAND
NORTH APRIL 4, 2017
2.35 km
SURFACE SETTLEMENT: 0 - 2.5 mm
Settlement, mm
0
2.5
5
10
15
20
25
1 mm
EXTENSOMETER SETTLEMENT 1.5 m ABOVE TBM
Total
(Po – Pi)
Bars
0
4
8
210+00
220+00
230+00
240+00
250+00
260+00
270+00
280+00
287+17



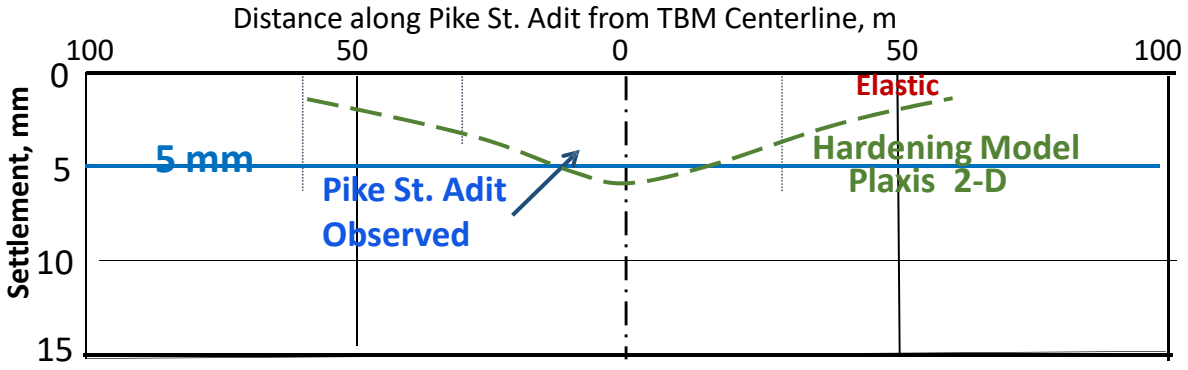
Observed Settlement Volume: 0.07% to 0.09% at BNSF & Pike St Adit because gaps were filled preventing ground loss.

Preconstruction: FLAC 3-D Analysis assuming 0.35% Ground Loss,



- Trough half width: w=2.5i = radius+z tan 39° = 8.75 m + 0.8 Z = 36 m
- Volume: 36m x 0.005m = 0.180 cu m/m ; % Vol = 0.07%

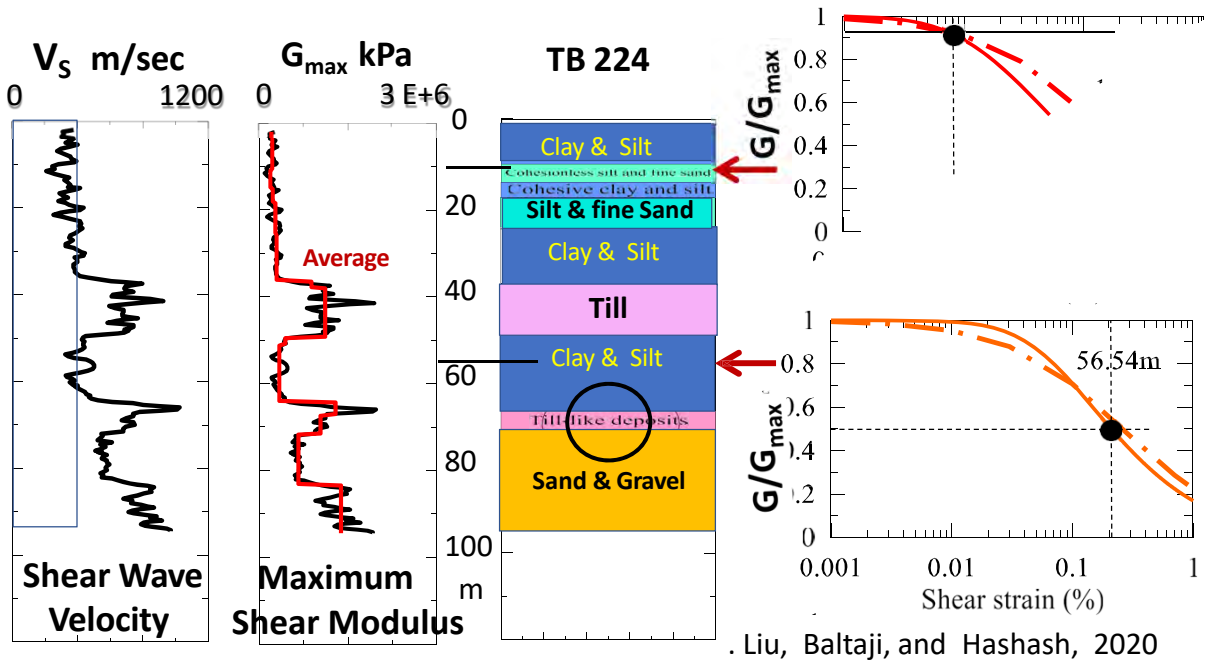
2020: Post Construction Analysis



Numerical Analysis: Baselining with **in situ moduli** obtained from:

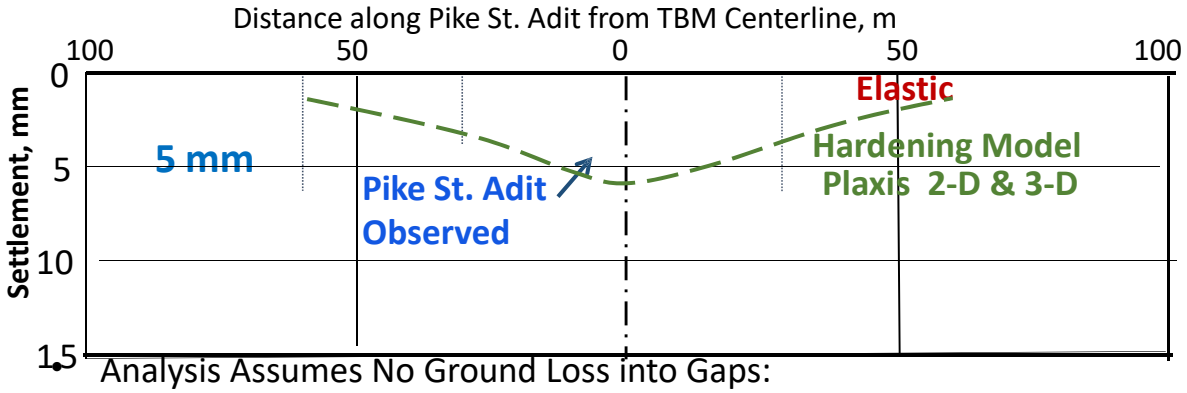
- Borehole Shear Wave Velocities - Maximum Shear Modulus, Gmax
- Normalized Shear modulus, G/Gmax, vs Strain

. Liu, Baltaji, and Hashash, 2020



. Liu, Baltaji, and Hashash, 2020

2020 Post Construction Analysis . Liu, Baltaji, and Hashash, 2020



Analysis Assumes No Ground Loss into Gaps:

- Correlates closely with Settlements at BNSF & Pike st. Adit, & and Surface Settlement throughout tunnel drive
- Confirms: No ground loss into gaps

Deep Anchor

- Demonstrates Validity of Shear Wave Velocity to determine In Situ Static Modulus.

Throughout Alaskan Way (SR 99) TBM drive:

- *No ground loss*
  - *Confirmed by Observations & by Numerical Analyses with Stiffness baselined from Shear Wave Velocity*
- *Shield gap consistently filled with slurry; Shield pressures maintained above dynamic groundwater pressures; Balanced with upper face pressures.*
- *Face pressures maintained 24/7*
  - *both during & between 2-m advances for 11 months, 5 days.*

MONITOR GROUND BEHAVIOR AT THE SOURCE…

- **Ground movement**
  - Centerline settlement
  - Extensometers
  - Horizontal (HDD) & Vertical Inclinometers
- **Dynamic Groundwater Pressures**
  - Piezometers In Clay: Respond to stress changes, drain with time
  - Piezometers In Sand: Respond to dynamic groundwater pressures
- **Upper face/upper shield pressures balanced**
  - Bentonite slurry fills & pressurizes shield gap
  - > dynamic groundwater pressures with:
    - Consistent outward hydraulic gradient
    - Use additives such as polymers in sands
- **Face (Chamber) pressure: Monitor**
  - Distribution across face; check for air bubbles
  - Vent

Edward Cording: Texas A&M November 12, 2021

2017-2021: LA METRO Horizontal Directional Drilling (HDD)

- Magnetometer surveys
  - Prior to tunneling
  - Locate anomalies, remove obstructions, abandon oil wells
- Horizontal Inclinometers
  - Continuous monitoring above advancing TBM.
  - Extend hundreds of feet from shafts & beneath critical structures.

Borehole Extensometer: urface & Anchor Settlement
/Piezometer
Pore Pressure response

Piezometer Anchor

HDD: Magnetometer & Inclinometer

Piezometer Anchor

SHIELD PRESSURE GAUGES 3 3

FACE PRESSURE GAUGES 2 2 2

Bentonite Slurry Volume

Grout Volume

Secondary Grouting

Pressure Gauges in Screw Conveyor

Belt:
- 2 Weight Scales
- Sample Muck

20-ft-dia. EPBM

Monitoring TBM & Ground

Face/Shield Pressures, bars

2 0 - 2.4	1.7	2	1.5 to 1.2	0.9	0.7

Settlement, mm

Settlement, mm

-10
0
10

125 m 100 m 75 m 50 m

10 mm

SHAFT

Horizontal Inclinometer

MPBX 1

FIRST DRIVE

SHAFT

E 2ND STREET

2017: LA Metro; Regional Connector, 6-m EPBM, RCC

6-m to 10-m cover, Alluvial Sand & Gravel

Hansmire, et. al. 2017

2021: Update on Current Experience on Transit Projects

- To ensure consistent filling and pressurization of the overcut gap, injection of bentonite and additives is required during the pressurized TBM advance.
- Pressures are maintained on the TBM face and perimeter during and between advances.
- Upper Face and shield body pressures are balanced.
- Horizontal Inclinometers and Extensometers above the advancing TBM show that small settlements develop over the shield, and are not due to ground loss into the gaps
- Settlements can be estimated from difference in overburden pressure and upper face/shield pressure, using In Situ Moduli determined from Shear Wave Velocities.
- TBM face/shield pressures: can be increased as needed to reduce elastic settlement beneath critical structures.
- Settlements are being consistently controlled to values << 0.5 inches and below damaging levels in Alluvial and Glacial Soils.
- Demonstrating consistent control that can be relied upon by Project and 3rd Parties

Coordinate Building Protection Studies with TBM Capabilities and Requirements

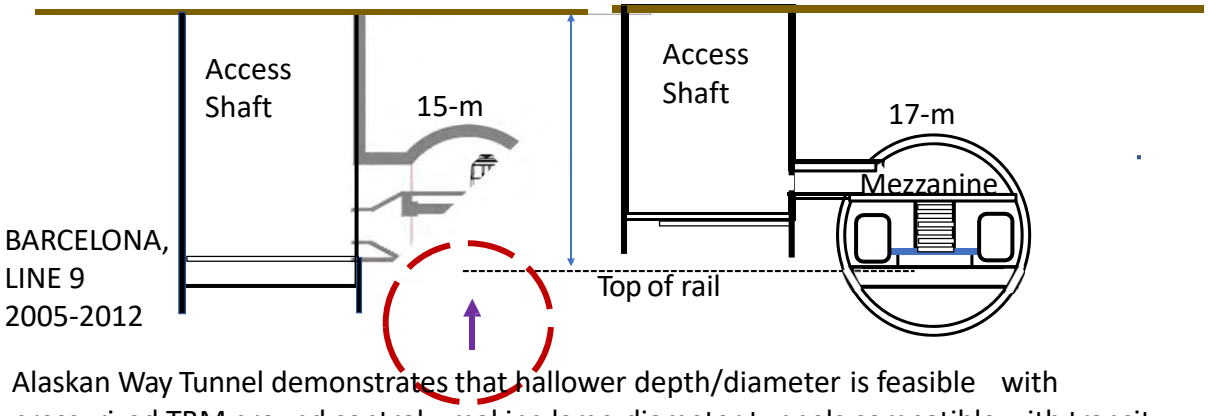
- Building Protection Studies need to be coordinated with the TBM ground control and monitoring measures required on the project, rather than assuming percent ground losses based on projects where ground control and monitoring information is not known.
- Primary measure for preventing ground loss and for protecting buildings is the control and monitoring of the pressurized TBM.
- Ground improvement measures are used where the TBM cannot be fully pressurized. Special building protection measure are evaluated in combination with the required TBM control and monitoring measures.

Correlate Geotechnical Observations with TBM Monitoring

- Geotechnical observations around the advancing TBM should be correlated with the key TBM parameters, e.g.. Face and shield pressures, volumes injected.
- Projects need to have team/lead to coordinate and link geotechnical instrumentation with TBM operations.

LARGE DIAMETER TBM: TRANSIT STATION PLATFORM WITHIN TUNNEL

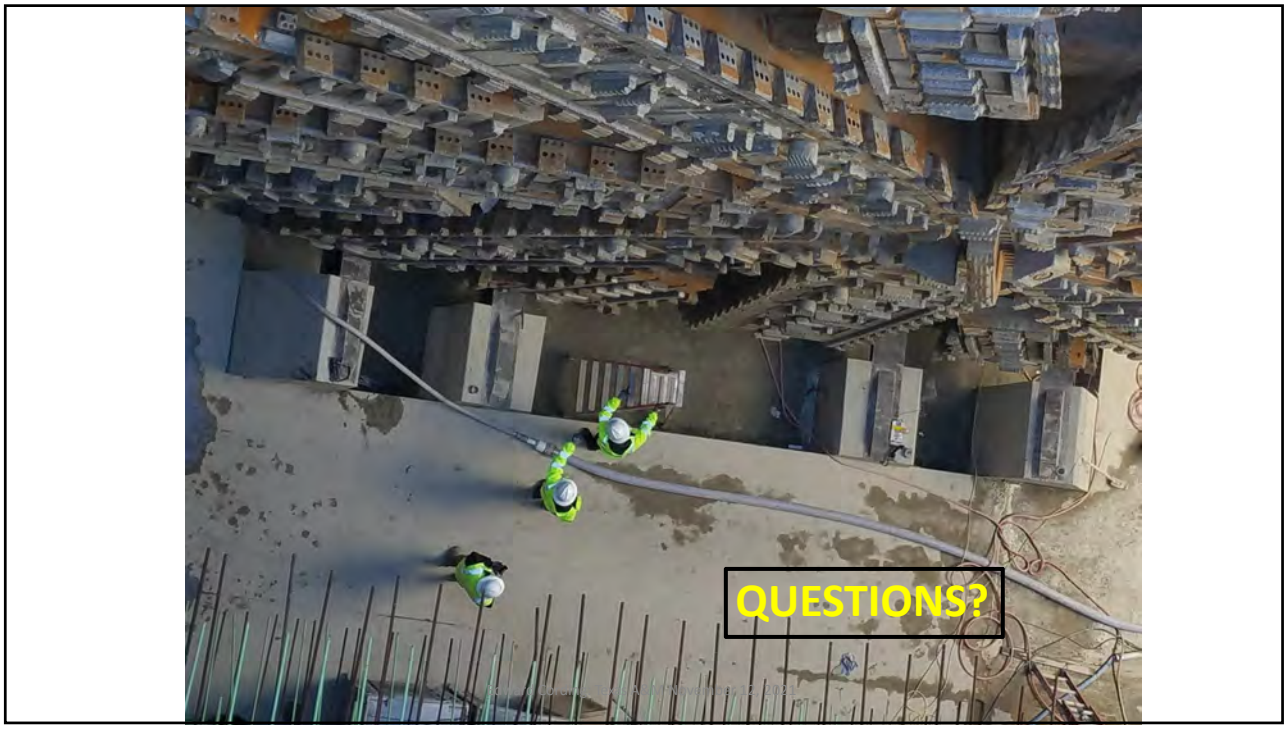
- **ELIMINATE CUT & COVER EXCAVATION IN STREET**
- **PARTICULARLY SUITED FOR CLOSELY SPACED STATIONS**



Alaskan Way Tunnel demonstrates that hallower depth/diameter is feasible with pressurized TBM ground control, making large-diameter tunnels compatible with transit station depths and access with escalators.

12 m Cover
+ 5 m Mezz.
+ 6 m T.R.

- *Achieving Brunel's Objective: "open… the ground in such a manner that no more earth shall be displaced than is to be filled by the shell or body of the tunnel."*


QUESTIONS?

Offshore Geotechnics: From Oil and Gas to Renewable Energy

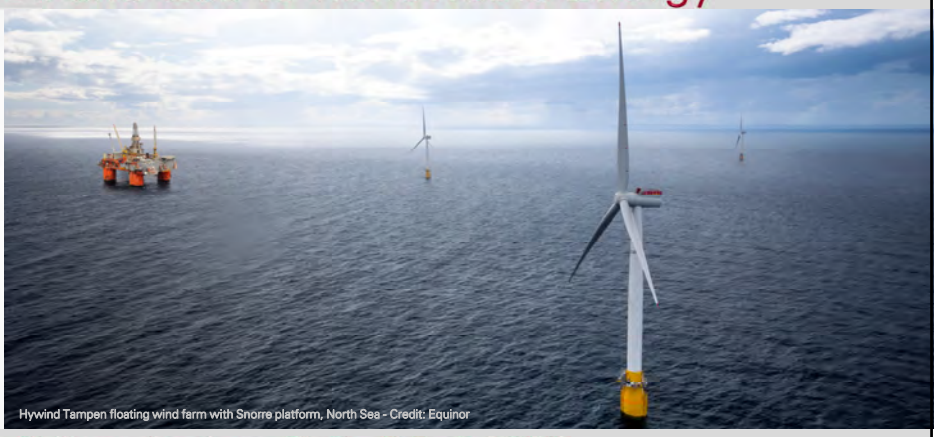
The 2021 Spencer J. Buchanan Lecture

By Dr. Philippe Jeanjean

The 29th Spencer J. Buchanan Lecture: Offshore Geotechnics From Oil and Gas to Renewable Energy

bp I&E innovation & engineering

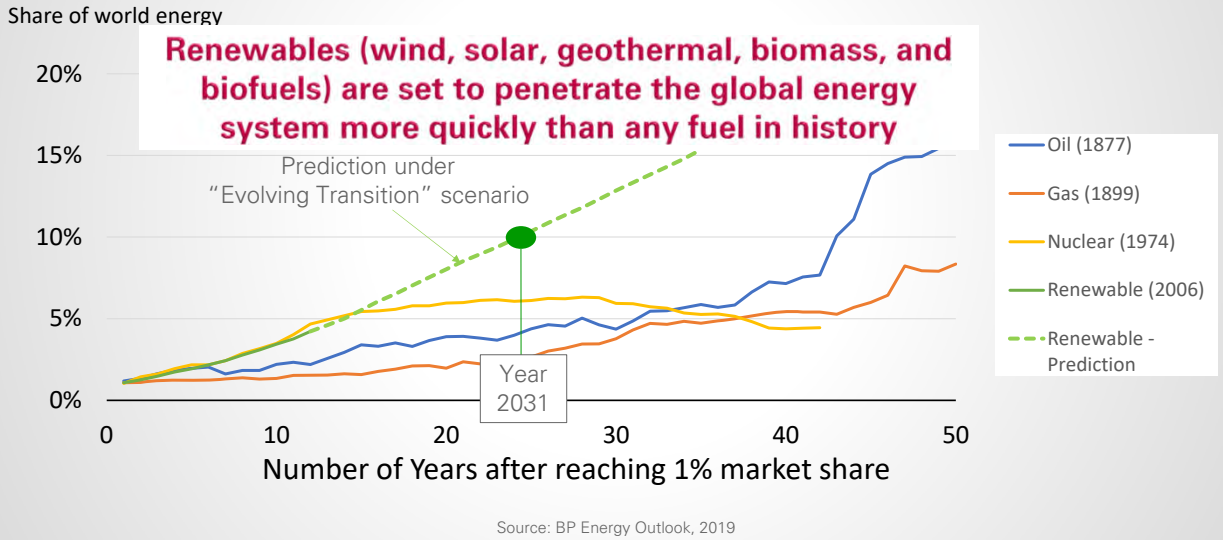
ZACHRY DEPARTMENT OF CIVIL ENGINEERING TEXAS A&M UNIVERSITY



Philippe Jeanjean, Ph.D., P.E., F.ASCE
BP America, Houston TX
Texas A&M University (via Zoom), November 12, 2021

1

The World Energy Transition



Source: BP Energy Outlook, 2019

2

Monopod and Multipod Structures

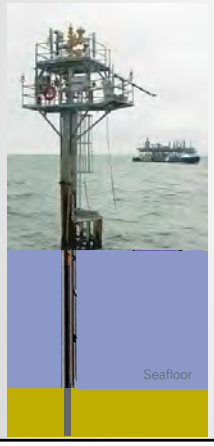
Oil & Gas (O&G):

- Represent a few % of all platforms
- Used in water depths < ≈ 50 m

Offshore Wind (OW):

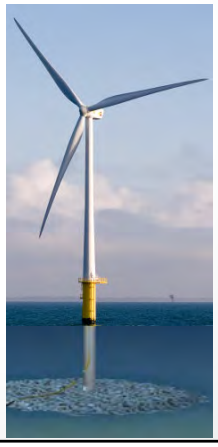
- Monopiles represent ≈ 82% of all turbine foundations
- Tripods represent ≈ 2% of all turbine foundations
- Used in water depths < ≈ 35 m

Free standing caisson
Gulf of Mexico



Multipod structure,
North Sea

Wind Turbine Generator
with monopile foundation



Tripod foundation
Alpha Ventus wind farm, Germany

3

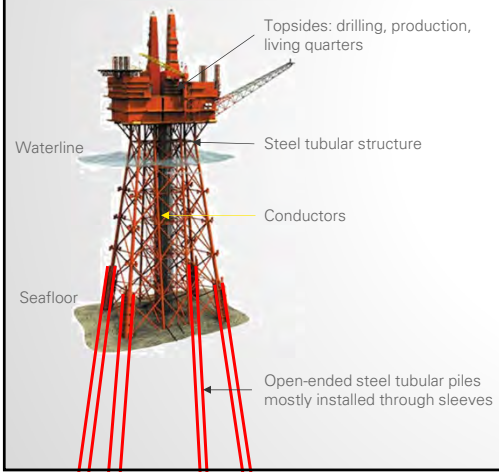
Jacket Structures

Oil & Gas:

- Represent ≈ 95% of all platforms
- Used in water depths up to 411m
- Foundation: mostly use driven piles

Offshore Wind:

- Represent ≈ 10% of all platforms
- Used in water depths up to ≈ 60m
- Foundation: mostly use driven piles



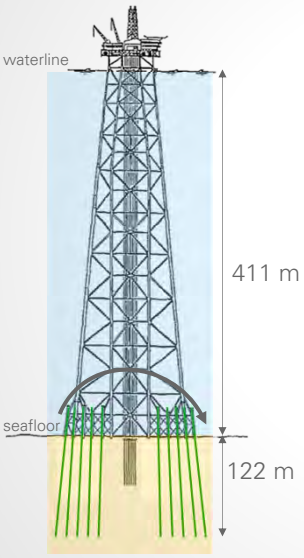
Credit: German Offshore Wind Energy Foundation - DOTI
Matthias Ibeler, 2009

Artist impression, Bjarne Stenberg
CeSOS, NTNU, Norway

4

Bullwinkle (1988): transportation to site

Eiffel Tower
Height: 324 m



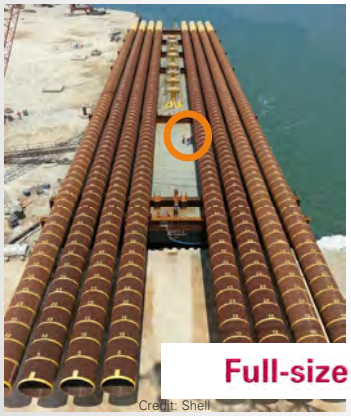
5

Typical Pile Dimensions

Oil & Gas:

- Diameter, D: 1.2 m to 2.5 m
- Length, L: 50 m up to 140 m
- L/D: > 30 – long & flexible
- Calculated capacity: up to ~100 MN

Piles for *Olympus* Tension Leg Platform



Credit: Shell

Offshore Wind Monopiles:

- Diameter, D: 4 m to 10+ m
- Embedded Length, L: 10 m to 40 m
- L/D: 2 to 6 – short & rigid
- Calculated capacity: less than 40 MN

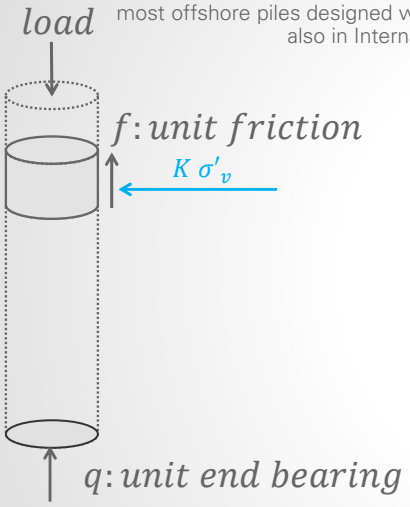
Monopile for Gode Wind offshore wind farm - Diameter: 7.5 m

From Kallehave et al. (2015)

Full-size pile load tests not feasible!

6

Design of Axially Loaded Piles in Silica Sands



most offshore piles designed with the American Petroleum Institute (API) method, used since 1969: also in International Organization for Standardization (ISO) codes

$$f = K\,\sigma'_v \tan\delta \le f_{lim}$$

- K : coef. of lateral earth pressure at failure,
- σ'_v : vertical effective stress,
- δ : friction angle at soil-pile interface at failure,
- f_{lim} : limit unit friction.

$$q = N_q\,\sigma'_{v\_tip} \le q_{lim}$$

- N_q : bearing capacity factor,
- σ'_{v_tip} : vertical effective stress at pile tip,
- q_{lim} : limit unit end bearing.

- K, δ, f_{lim}, N_q, q_{lim} : given as function of sand relative density, periodically updated

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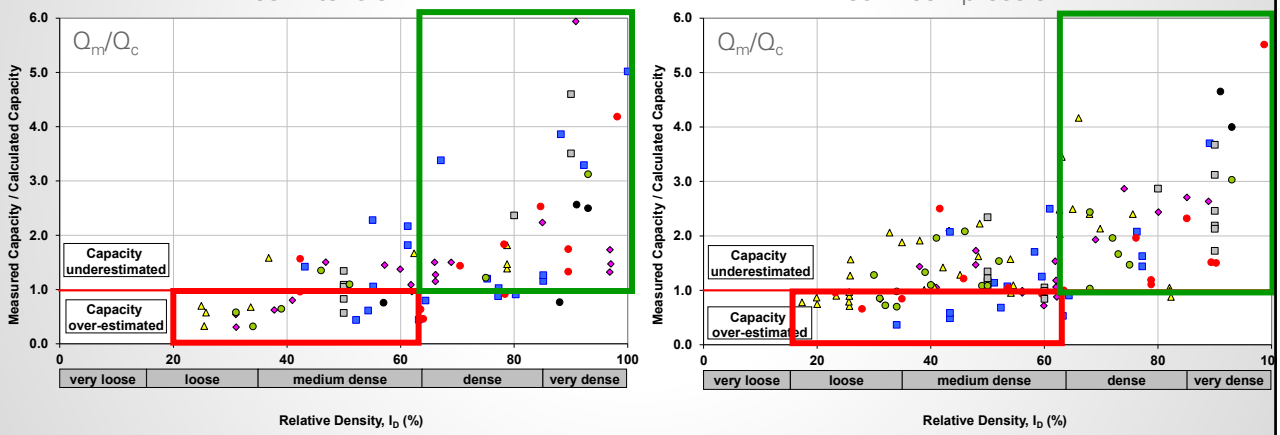
Bias of the API method:

Open and closed ended, steel and concrete piles

- Dense and very dense sands: capacity under-estimated, sometimes grossly
- Loose and medium dense sands: Capacity potentially over-estimated

- Piles in tension
- Piles in compression

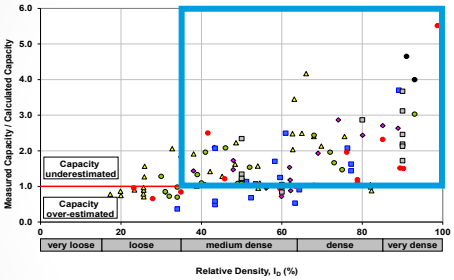


Adapted from Jeanjean (2012) with data from Hossain and Briaud (1992), NGI (2001), Fugro (2005), Lehane et al. (2005), Jardine et al. (2005)

8

Thoughts on API method

- Why have we been using it for so long?:
  - It does provide reliable, albeit sometimes overly conservative (i.e. under-predicts capacity) capacity for open-ended tubular steel piles loaded in compression in medium dense to very dense sands, which are the typical offshore design cases.



- It does not capture the physics of friction and end bearing:
  - Difficult to justify linear increase of friction and end bearing with depth (i.e. σ'_v)
  - Difficult to defend the existence of limit values, both for friction and end bearing
- It is difficult to extrapolate outside the database of pile load tests and to large diameter piles
- Increased efficiencies needed for pile design and re-assessment

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Some Direct CPT Methods for Axial Pile Capacity

(from Mayne, 2018 Buchanan Lecture)

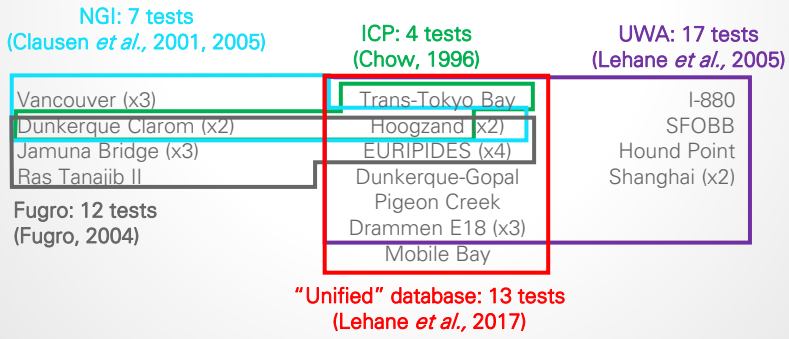
- Almeida et al. (1996, BRE-NGI)
- Eslami and Fellenius (1997)
- Jardine & Chow (1996, MTD)
- Takesue et al. (1998, KTRI)
- Lee & Salgado (1999, ASCE JGGE)
- Powell, et al. (2001, 15th ICSMGE)
- Jamiolkowski (2003 BAP - Ghent)
- Fugro-05 Kolk et al. 2005
- ICP-05 Jardine et al. 2005
- Abu-Farsakh & Titi (2004, JGGE)
- UWA-05 Lehane et al. 2005
- Xu et al. (2006, JGGE)
- Van Dijk and Kolk (2011, ISFOG-II)
- Aoki & de Alencar (1975 Pan-Am)
- Schmertmann (1978, FHWA Rept)
- de Ruiter & Beringer (1979)
- Bustamante & Gianeselli (1982)
- Zhou et al. (1982, ESOPT)
- Tumay & Fakhroo (1982)
- Price & Wardle (1982, ESOPT)
- Van Impe (1986, 4th IGS)
- Robertson et al. (1988, ISOPT-1)
- Alsamman (1995, PhD, UIUC)
- NGI-05 Clausen et al. 2005
- Fioravante et al. (1995, 10th ARC)
- Schneider et al. (JGGE 2008)
- Lehane et al. (UWA 2012)
- Niazi & Mayne (2015, 2017)

10

"Unified" database of pile load tests in sands

- Four key CPT methods developped in 1995 – 2005: Fugro, ICP, NGI, UWA, each from different databases
- No agreement in 2005 in industry as to which method was "the best"
- 2005 – 2014: agree to disagree
- JIP initiated in 2014 to develop a "unified" "non-associated" CPT-based design method for piles in sands
- First step: develop a *unified* database of pile load tests (2014 – 2017)

Compression tests on open-ended steel tubular piles



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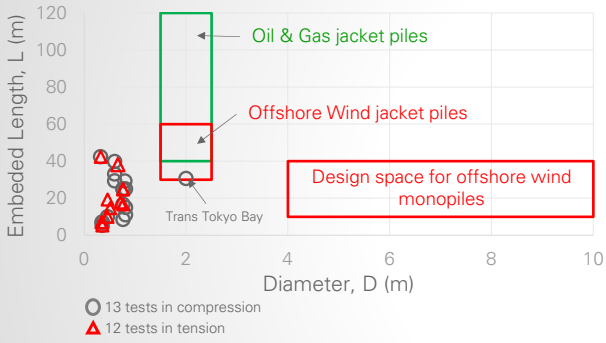
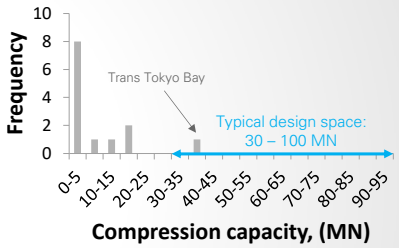
Extrapolating outside the database

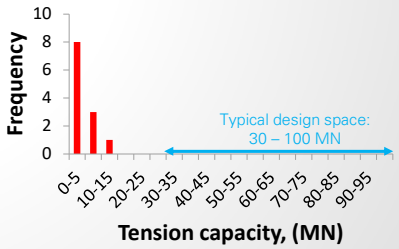
Open-ended steel tubular piles in *Unified* database vs typical offshore design

Pile dimensions



Pile capacity





Key Challenges

- Only one test resembles an offshore design condition
- *Unified* method needs to capture the physical processes that control friction and end bearing and understand how they extrapolate to offshore pile design conditions

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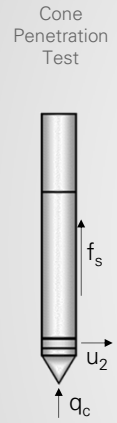
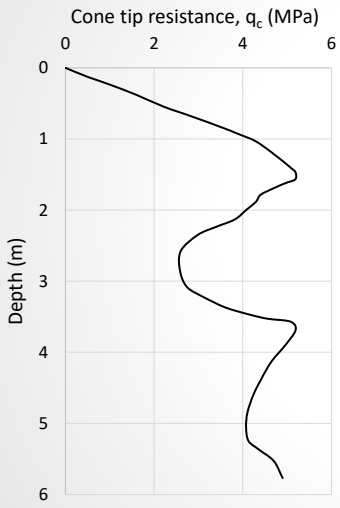
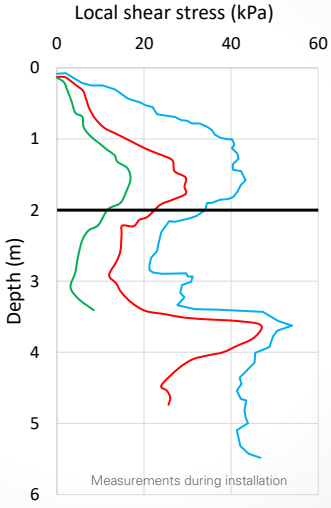
Imperial College Pile 1993 tests in Labenne, France







h: height above pile tip

Friction fatigue:

At a given depth, the shear stress at failure decreases as more pile is driven past that depth because of a continuing decrease in the normal stress against the pile at that depth

Adapted from Lehane et al. 1993

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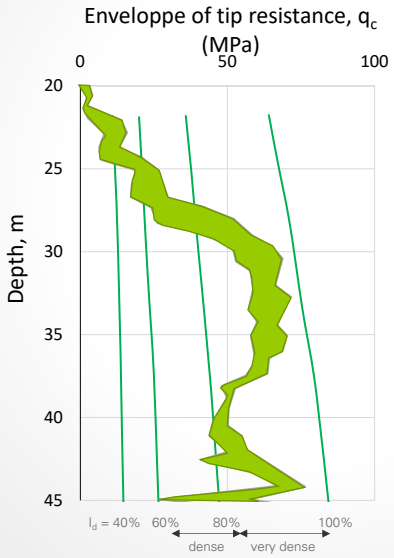
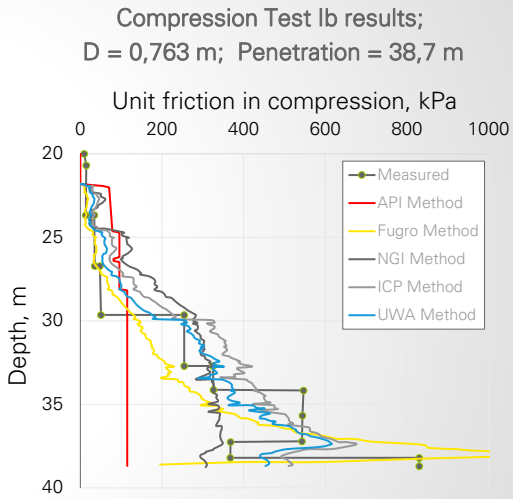
EURIPIDES pile tests

EURIPIDES: EUROpean Initiative on PIles in DEnse Sands

Fugro / Geodia

1992-1996





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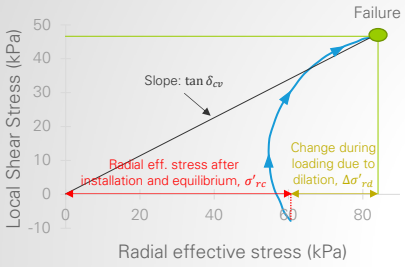
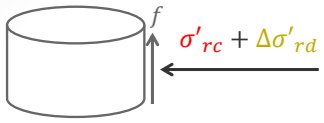
CPT *Unified* method for offshore piles in sands





Unit friction, f:

$$f = f_L(\sigma'_{rc} + \Delta\sigma'_{rd}) \tan \delta_{cv}$$

f_L = 1.0 in compression; 0.75 in tension

$$\sigma'_{rc} = \left(\frac{q_c}{44}\right)\underbrace{\left(1 - \left(\frac{D_i}{D}\right)^2\right)^{0.3}}_{\text{Area ratio.}}\underbrace{\left(Max\left[1, \left(\frac{h}{D}\right)\right]\right)^{-0.4}}_{\text{Friction fatigue term}}$$

D_i: internal diameter

$$\Delta\sigma'_{rd} = \left(\frac{q_c}{10}\right)\left(\frac{q_c}{\sigma'_v}\right)^{-0.33}\left(\frac{d_{ref}}{D}\right)$$

Calibrated from cavity expansion theory.

d_{ref} = 0.0356 m

Angle of interface friction δ_{cv} :

Little to no dependency on sand D_{50}

due to grain crushing with repeated shearing

δ_{cv} = 29 degrees

- Unit base resistance: $q = \left(0.12 + 0.38\left(1 - \left(\frac{D_i}{D}\right)^2\right)\right) q_c$

Adapted from Lehane et al. (2020)

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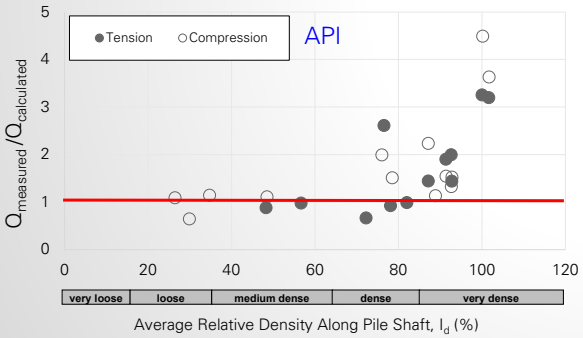
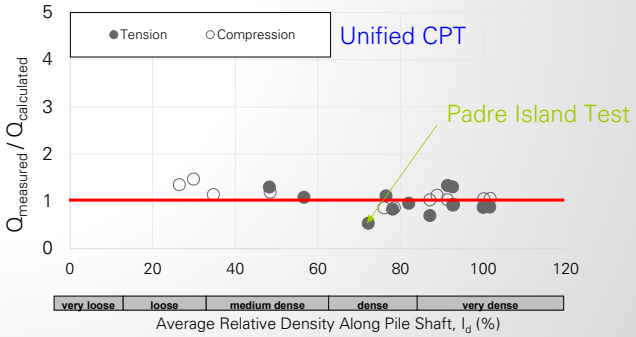
Statistics for Q_m / Q_c for API and CPT methods using the *Unified* database

Method	All open-ended and closed ended piles (total capacity); 71 piles		
	Mean, μ	Standard deviation, σ	C.O.V
API	1.66	0.93	0.56
Fugro-05	0.99	0.39	0.40
ICP-05	1.04	0.28	0.27
NGI-05	0.99	0.34	0.34
UWA-05	1.06	0.28	0.26
Unified	1.05	0.25	0.24

Removal of bias - Example for open-ended steel tubular piles (Adapted from Lehane et al. 2020)





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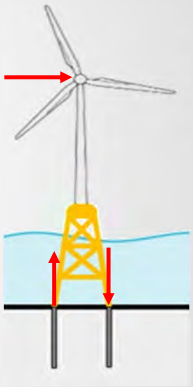
Key benefits of *Unified* CPT method of piles in sand

- Improved understanding of physical mechanisms and confidence in extrapolating outside the database to large and very large diameters.
- Achieving offshore industry consensus which allow codification in ISO and API standards as main method.
- Improved efficiencies in design (i.e. often shorter piles) with same reliability.
- Improved efficiencies in assessment (i.e. ability to justify higher capacity for existing piles) with same reliability.
- Removal of un-conservatism of API method for piles in loose and medium dense sands.
- Applicable to Oil & Gas and Offshore Wind structures



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Timeline of Development and Codification of Offshore CPT methods

- 1990's: Key instrumented pile load tests
- 1995 – 2005: Development of 4 CPT methods: Fugro, NGI, UWA, ICP
- 2007: Inclusion of above 4 methods into ISO and API codes in the annex without mandating or stating preferred method
- 2017: Publication of *Unified* database (through JIP work 2014 – 2017)
- 2020: Publication of *Unified* method
- 2022: Expected inclusion of *Unified* method as main method in ISO standard 19901-4, 3rd Edition.

A 30-Year journey!

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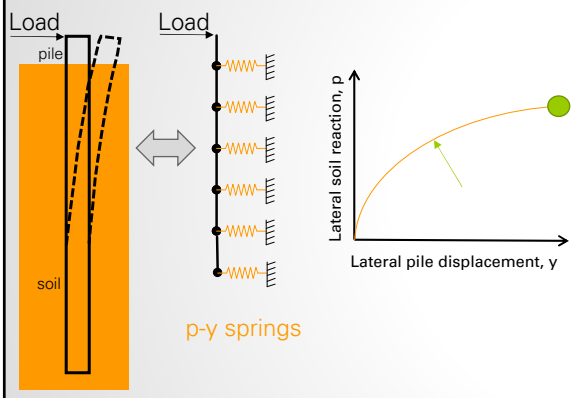
Laterally loaded piles in clays

Beam column analyses – p-y curves
Pioneered by McClelland and Focht (1956)



Appropriate for analysis of long flexible piles:

- Soil assumed to move only laterally
- Pile tip does not move

p-y curves in current API and ISO codes:

- Based on limited tests performed in 1958 – 1961
- Published by Matlock in 1970 and included in API practice in 1972
- Widely documented to underpredict soil stiffness and strength

Used for 50 years because they typically over-predict the maximum bending moment in the pile

Upcoming updates to API and ISO codes (2022?):

- Updated p-y curves for clays for three load conditions: monotonic, cyclic, and fatigue limit states;
- Based on work by Jeanjean (2009), Jeanjean et al. (2017), Zhang et al. (2019) and Zakeri, et al. (2016);

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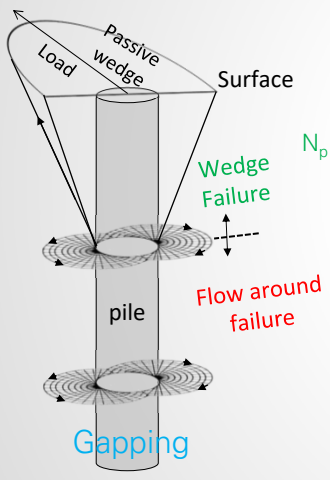
Failure mechanism for long flexible piles

Ultimate soil reaction: $p_u = s_u D N_p$

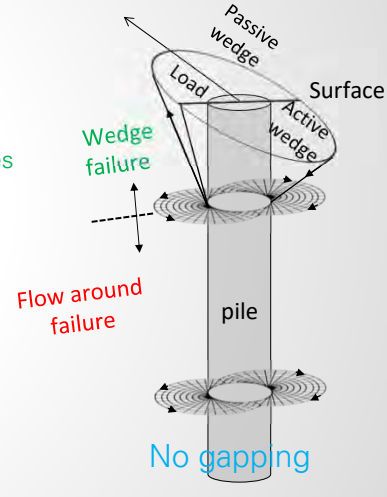
- s_u : undrained shear strength
- D : diameter
- N_p : lateral bearing capacity factor



N_p from finite element & upper bound limit analyses
Yu *et al.* (2015); Zhang *et al.* (2016)

N_p from upper bound limit analyses
Randolph and Houlsby (1984)

- $N_p > N_p$ from Matlock

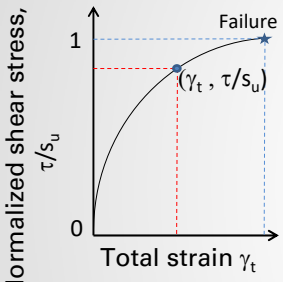


See Jeanjean et al. (2017) for equations and details

20

Scaling DSS stress-strain curves to derive p-y curves

Direct Simple Shear (DSS)

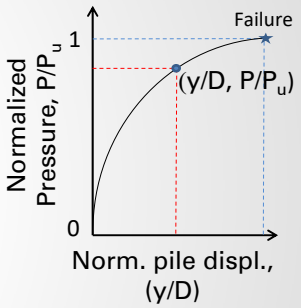


Used to define soil strain - hardening law

Scaling

Each point (γ_t , τ/s_u) on the normalized DSS curve is transformed into an equivalent point (y/D, P/P_u) on the normalized p-y curve

p-y



y: Lateral displacement;
D: pile diameter

Six default p-y curves will be given in the next ISO code as a function of I_p and OCR to replace the curves from Matlock (1970)

See Jeanjean et al. (2017), Wu et al. (2020) for equations and details

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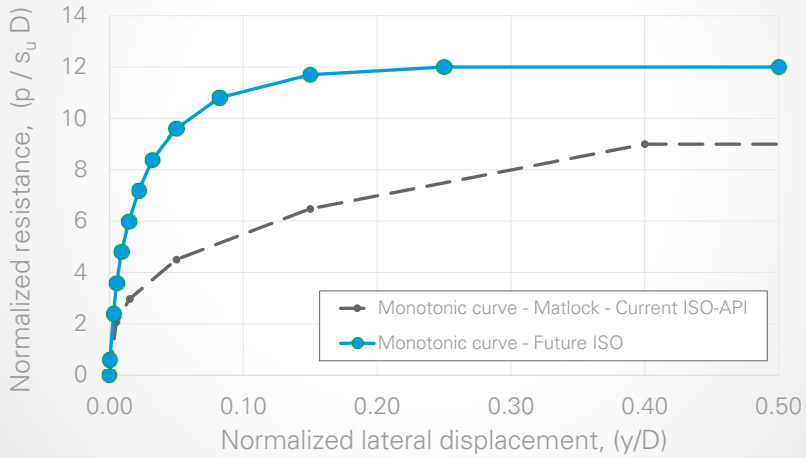
Typical monotonic p-y curves: Matlock (1970) vs new ISO curves

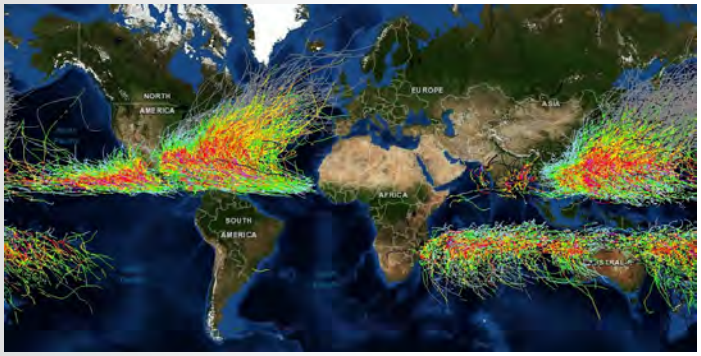
For flow-around mechanism and adhesion factor $\alpha = 1$



22

Design Against Hurricanes – Typhoons - Cyclones

Typical height of 100-year design wave: H_{100} > 30 m



Paths of hurricanes – typhoons - cyclones over last 50 years

Source: https://coast.noaa.gov/hurricanes/

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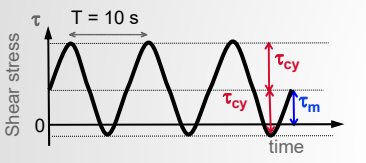
Behavior of clays under Direct Simple Shear (DSS) cyclic loads

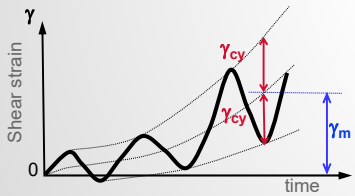
Cyclic effects = rate effects + cyclic degradation

s_u ↑ if $\frac{\partial \gamma}{\partial t}$ ↑ **s_u ↓ if the number of cycles, N ↑**



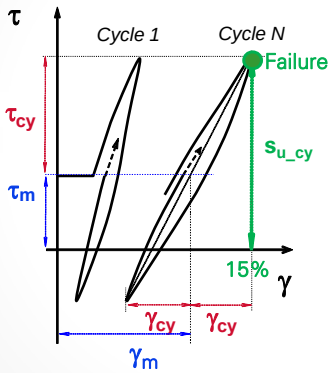
τ_m : mean stress

τ_{cy} : cyclic stress



γ_m : mean shear strain

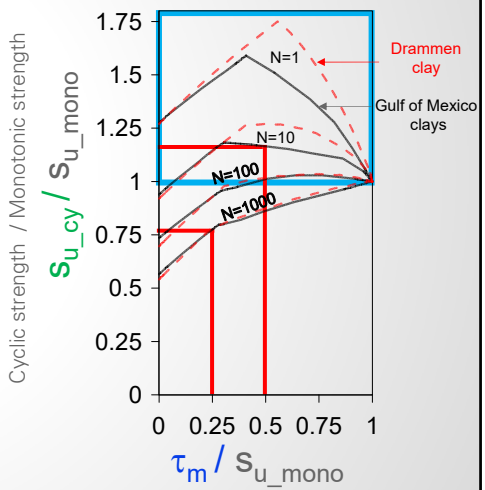
γ_{cy} : cyclic shear strain



s_{u_cy} : cyclic shear strength

$s_{u\_cy} = \tau_m + \tau_{cy}$ at failure (γ = 15%)

Illustrations after Andersen (2015) and Liedtke et al. (2019)



s_{u_mono}: Monotonic (static) shear strength

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Cyclic loads due to Gulf of Mexico hurricanes

Measurement of s_{u_mono} with monotonic DSS test:

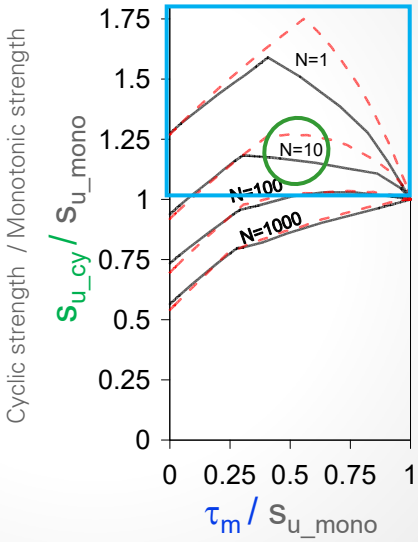
Sample is sheared at a shear rate $\frac{\partial \gamma}{\partial t} \approx$ 5%/hr

and reaches failure typically in 1 to 3 hours

Measurement of s_{u_cy} with cyclic DSS test:

Sample sheared with a load period of 10 s

and reaches failure in ≈ 2 mins if N = 10.



Typical conditions along jacket pile under GoM hurricanes or North Sea winter storms loads

$N_{eq} < 25$

See Liedtke et al. (2019) for details

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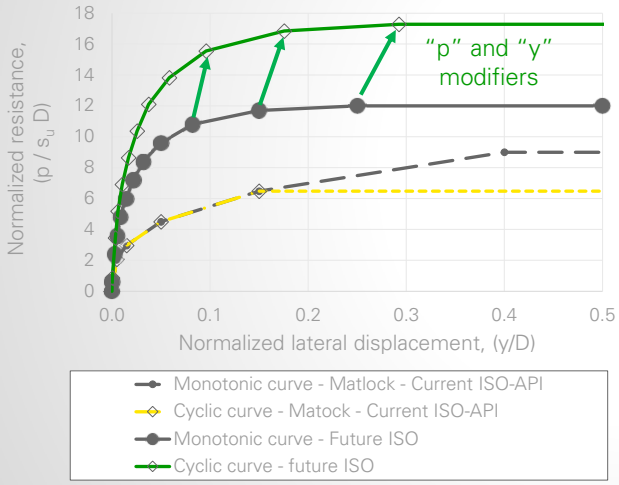
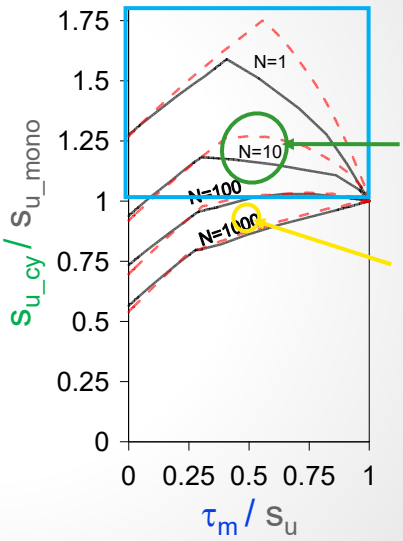
Current and Future p-y curves in ISO-API

Typical curves for flow around mechanism
Fully rough pile (adhesion = 1)





Typical of GoM hurricanes and North Sea winter storms

Matlock tests conditions (API curves) 300-400 cycles not representative of offshore hurricane loads

See Zhang et al. (2019), Jeanjean et al. (2022) for equations and details

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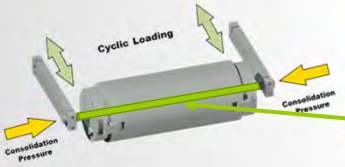
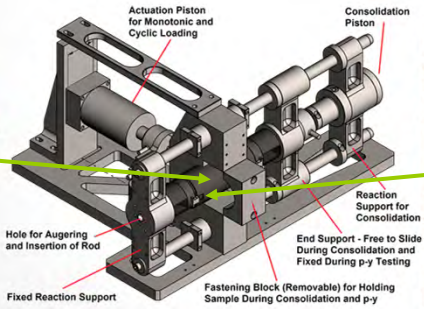
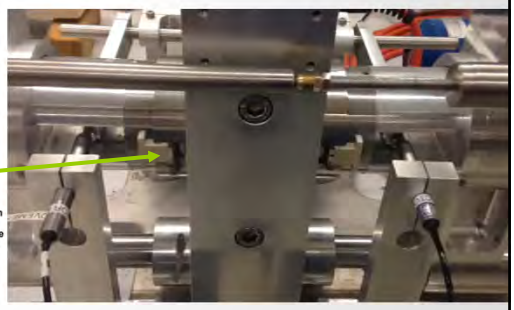
p-y curves for fatigue limit state

- Developped from centrifuge tests and direct measurements with novel laboratory p-y testing apparatus
- Represent steady-state soil interaction after a few thousand cycles
- Loading period T is between 2 s and 4 s to represent typical sea-states that cause most fatigue
- Validated by numerous well monitoring data



Soil sample:
Ø: 68mm by 100mm





p-y testing apparatus developped through a collaboration between BP and NGI

See Zakeri et al. (2019) for details

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Hurricanes in the Gulf of Mexico

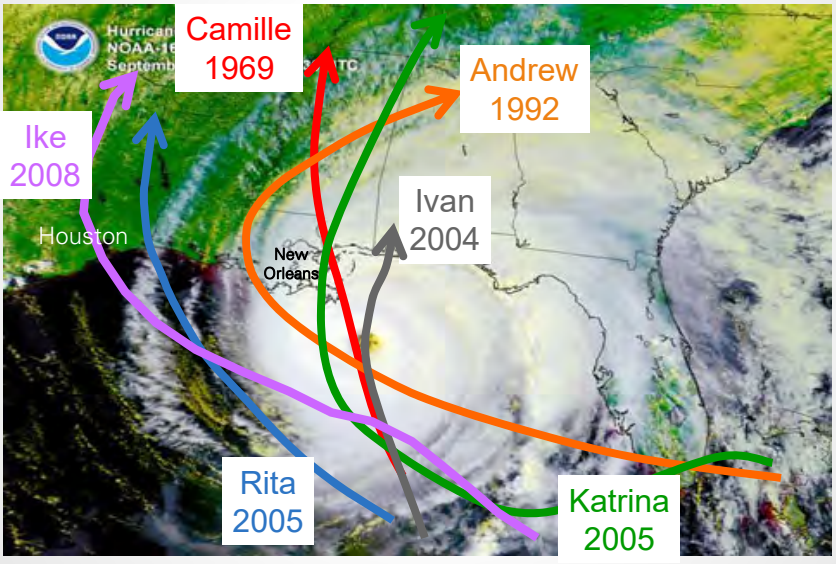

Photo of Hurricane Ivan Sept. 15, 2004 Credit: NOAA

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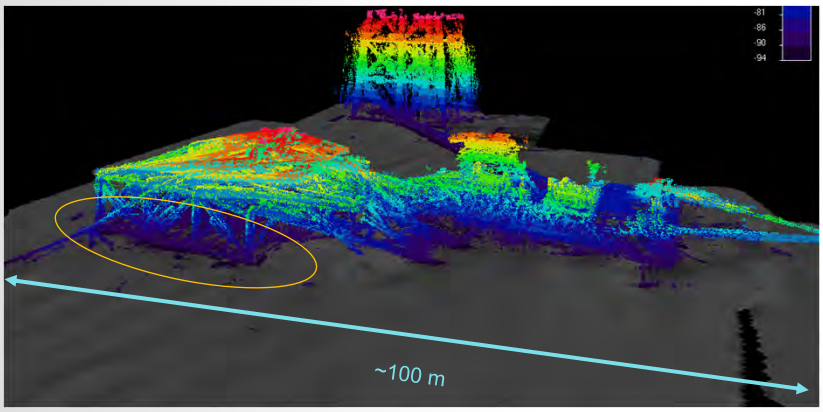
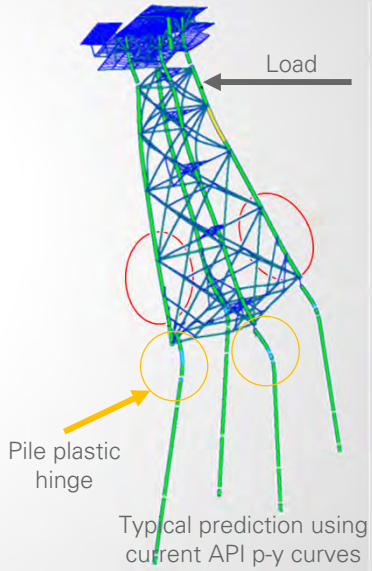
Failure Mechanism: Observations and predictions



Echoscope survey of GoM platform destroyed during hurricane



Typical prediction using current API p-y curves

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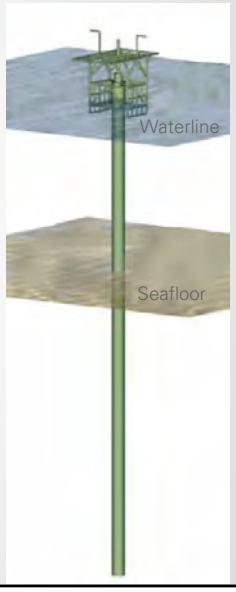
API/KBR study (Wu et al. 2020)

Hindcast of performance of free-standing caisson during Hurricane Andrew (1992)

- Water depth: 16.2 m
- Pile: 1.2 m diameter, 29 m penetration in soft clay
- Caisson damaged during Hurricane Andrew in August 1992, found leaning 15 degrees at waterline

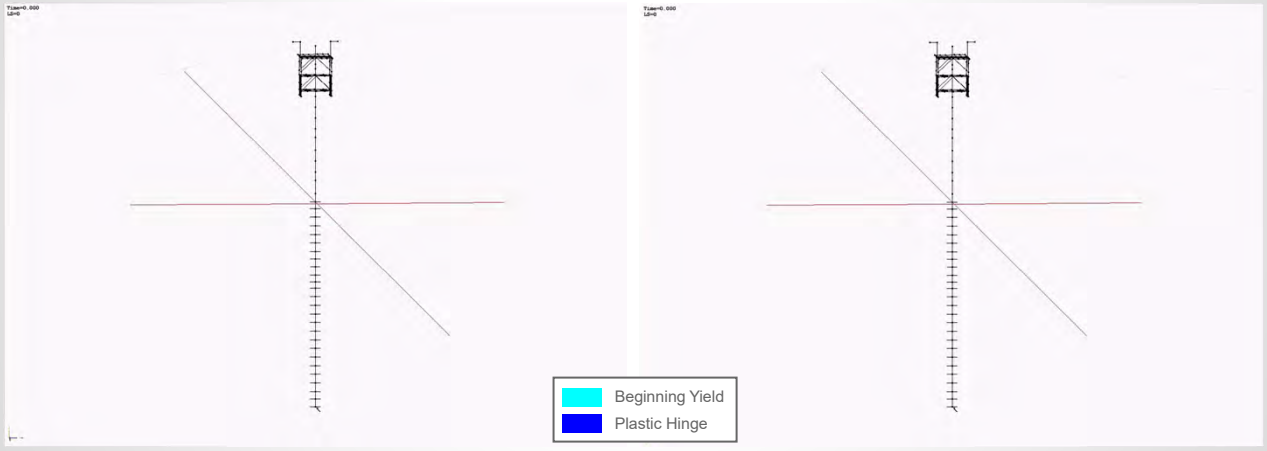
Illustrative caisson damage after hurricane



30

SS caisson – dynamic push-over analysis results

- Future API/ISO **cyclic** curves:
- Current API/ISO **static** p-y curves:



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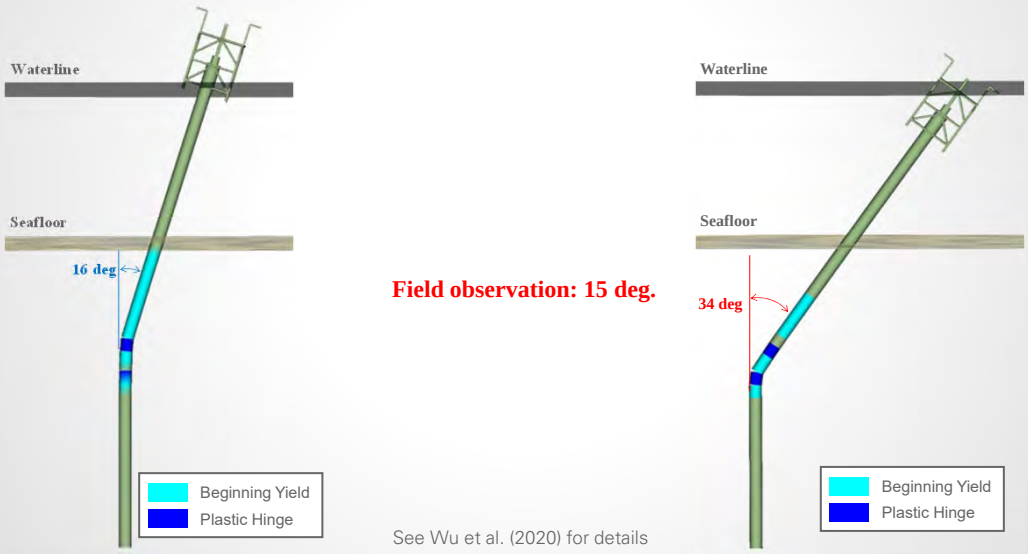
SS caisson – Predicted vs Measured Performance – Hurricane Andrew

- Future API/ISO cyclic curves:
- Current API/ISO static p-y curves:



Field observation: 15 deg.

See Wu et al. (2020) for details

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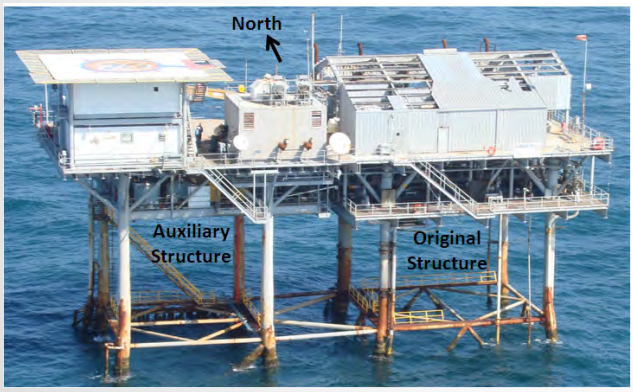
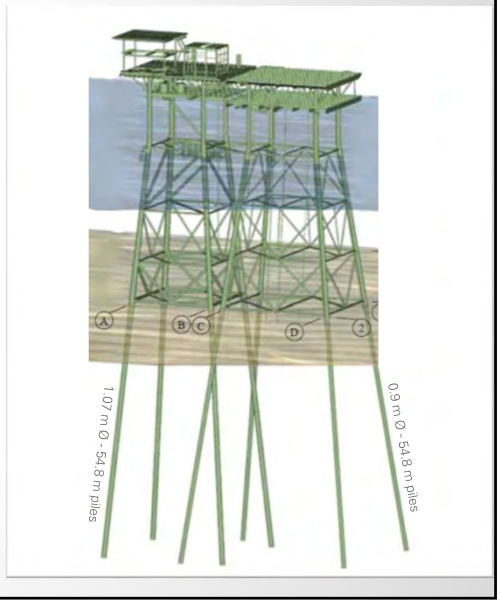
API/KBR study (Wu et al. 2020)
Hindcast of performance of SS jacket during Hurricane Ike

- Two 4-pile jacket platforms
- Pile penetration: 54.8 m into soft to stiff clay
- Platform damaged during Hurricanes Gustav & Ike (2008)





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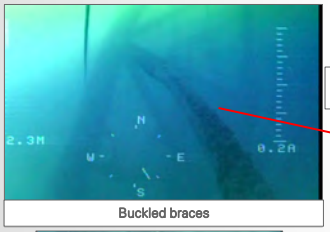
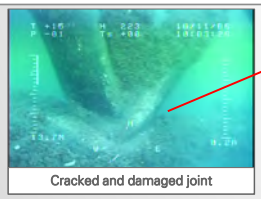
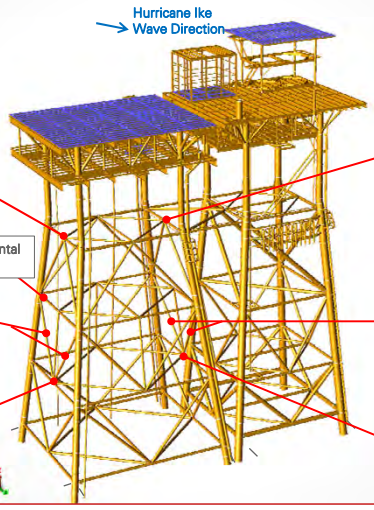
SS platform: Damage Post Hurricane Ike (Sept. 2008)


Joint failure and separated members


Buckled braces


Cracked and damaged joint




Joint failure and separated members

Buckled braces


Cracked and damaged joint

PILE FOUNDATION
NO INDICATION OF PILE FOUNDATION FAILURE OR LATERAL DISPLACEMENTS IN UNDERWATER INSPECTIONS

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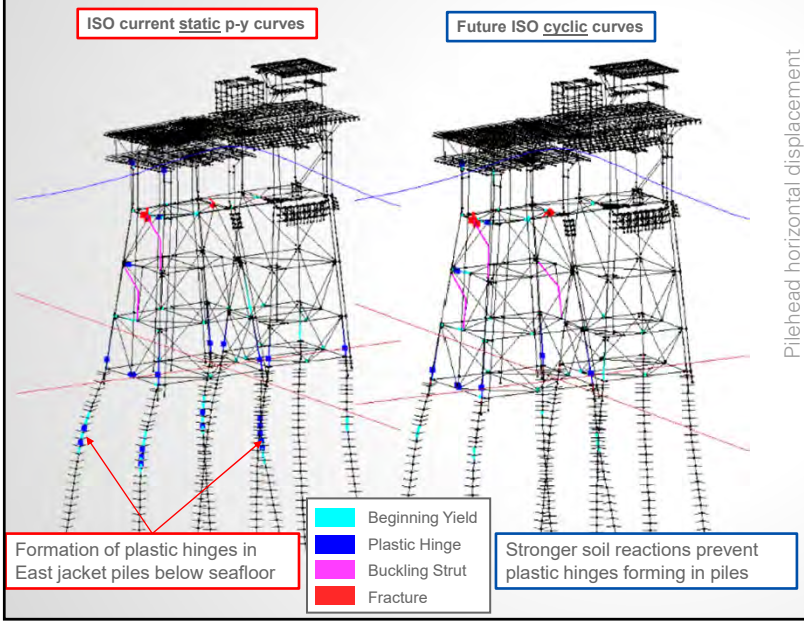
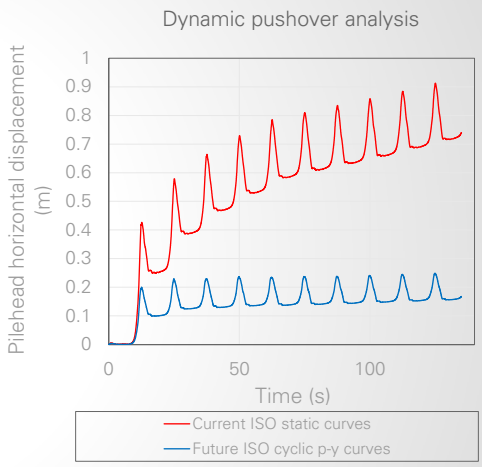
SS platform: Performance during Hurricane Ike after 2 waves





The hindcast with the future cyclic p-y curves is more consistent with the fact that no noticeable pile lateral displacement was reported (0.4 m displacement vs 0.1 m)

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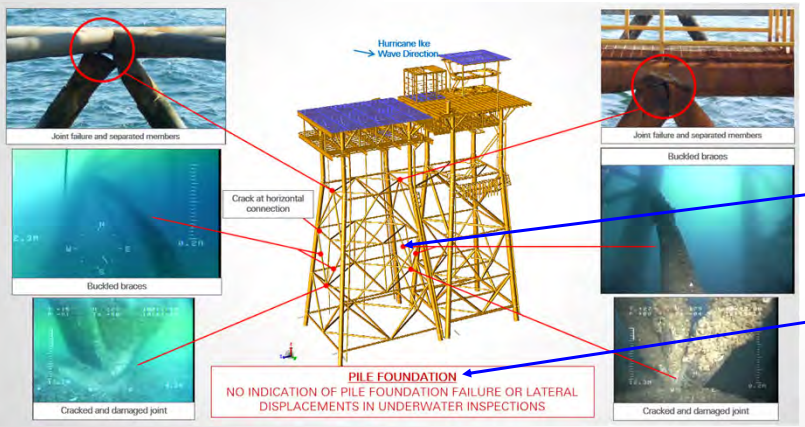
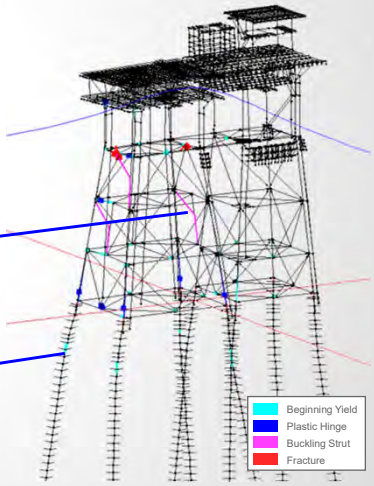
SS platform: Predicted vs observed damage during Hurricane Ike

"Experience is the hardest kind of teacher. It gives you the test first and the lesson afterward" - Oscar Wilde

- ISO future cyclic p-y curves





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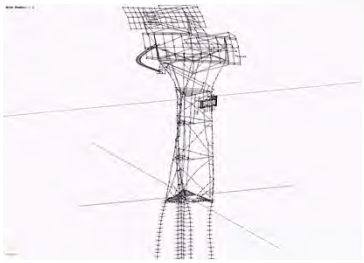
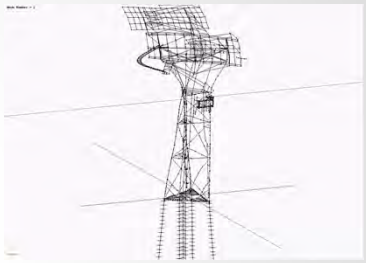
EI tripod: Predicted vs measured natural period

- Water depth: 37 m
- Piles: 84.1 m long in clay
- Vibration data recorded in 2018 under low sea-states

➢ Current ISO static curves



➢ Future ISO fatigue curves



Data Source	Natural period (s)			Difference from measurements (%)		
	N-S (X-sway)	E-W (Y-sway)	Torsion	N-S (X-sway)	E-W (Y-sway)	Torsion
Current static p-y curves	1.13	1.13	0.83	12.1	13.0	9.2
Future fatigue p-y curves	1.08	1.07	0.75	1.0	7.0	-1.3
Field measurements	**1.07**	**1.00**	**0.76**	-	-	-

See Wu et al. (2020) for details

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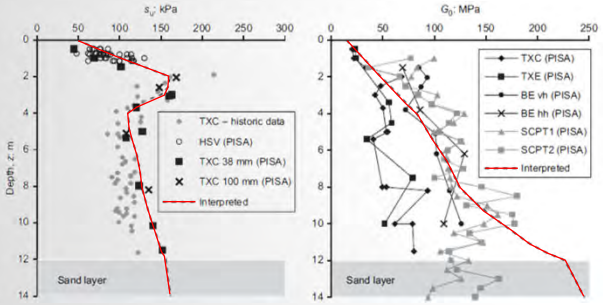
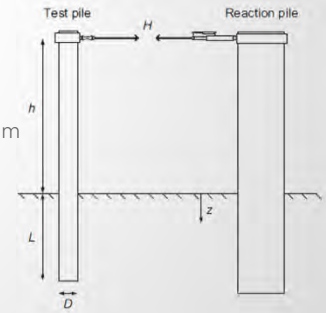
Project PISA (PIle Soil Analysis) (2014 – 2018)
New Approach for Offshore Wind Monopile Design Applications

Testing at Cowden site in glacial till

Shear strength and small-strain stiffness profiles





- D = 0,243 m; 0,762 m; 2 m
- L/D = 3; 5,25; 10
- h = 5 m; 10 m

From Byrne et al. (2020)

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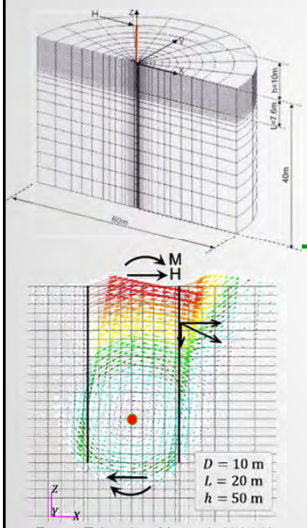
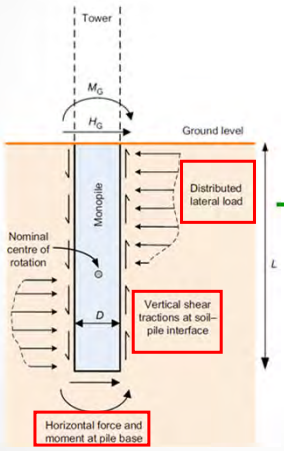
PISA design method – monotonic loads

3D FEA for site soil condition and range of D, L, h, t

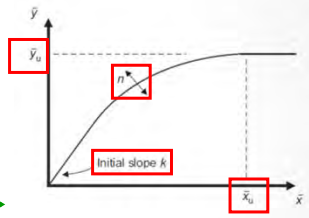


From Zdravković et al. (2020)

Extraction of 4 soil reaction curves from each FEA



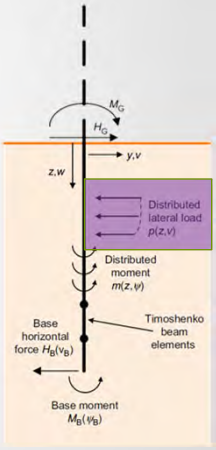
Curve-fitting and parametrization of all reaction curves with conic function



$\bar{y}$	$\bar{x}$
Distributed lateral load	Lateral displ.
Distributed moment	Pile cross section rotation
Base shear	Lateral displ. at base
Base moment	Rotation at base

From Byrne et al. (2020)

Generation of 1D beam column model over design space



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Ultimate lateral bearing pressure p_u: PISA vs new ISO

$$p_u = N_p \, s_u \, D$$

New ISO rule:

$N_p = N_{p0} + \frac{\gamma' z}{s_u} \leq N_{pd}$ if gapping is assumed on the back side of the pile

$N_p = 2N_{p0} \leq N_{pd}$ if no gapping is assumed on the back side of the pile

$$N_{p0} = N_1 - (1 - \alpha_{ave}) - (N_1 - N_2)\left[1 - \left(\frac{z}{dD}\right)^{0.6}\right]^{1.35} \leq N_{pd}$$

$N_1 = 12$

$N_2 = 3.22$

$d = 16.8 - 2.3\, log_{10}(\lambda) \geq 14.5$

$\lambda = s_{u0} / (s_{u1} D)$

$N_{pd} = 9 + 3\alpha_{ave}$

PISA rule for Cowden site:

$$N_p = \bar{p}_u = 10.7 - 7.101\, e^{-0.3085\frac{z}{D}}$$

From Byrne et al. (2020)

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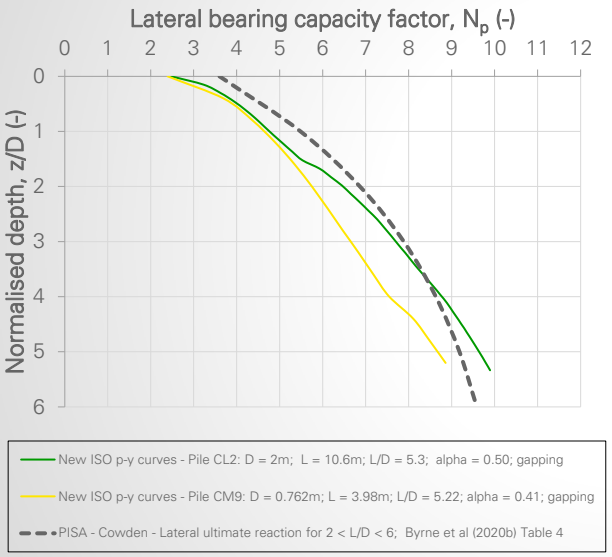
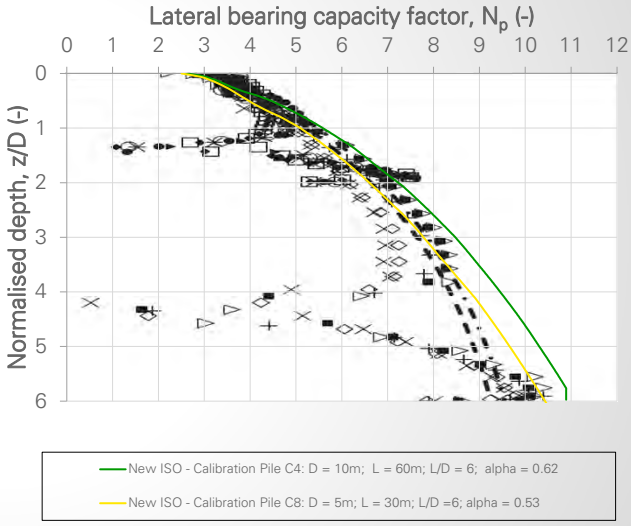
Lateral Bearing Capacity Factor, N_p: PISA Cowden vs new ISO

PISA Cowden tests CL2 and CM9



PISA FEA Calibration Piles C4 & C8



See Jeanjean et al. (2022) for details

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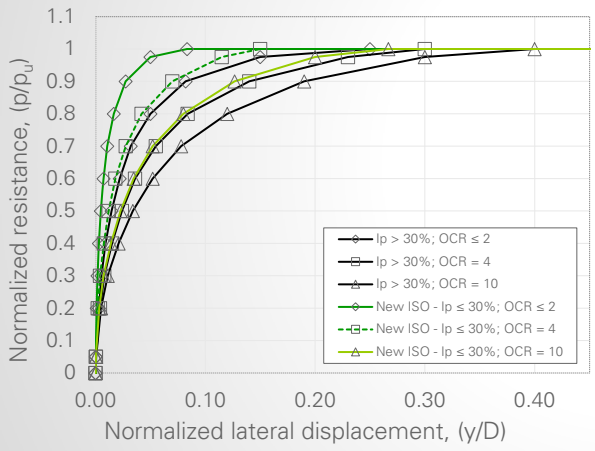
Shape of p-y curves: PISA Cowden vs new ISO

New ISO default normalized curves:



PISA rule for Cowden site:

$$-n_p\left(\frac{p}{p_u}-\frac{v}{v_u}\right)^2+(1-n_p)\left(\frac{p}{p_u}-\frac{v\,k_p}{p_u}\right)\left(\frac{p}{p_u}-1\right)=0$$

Parameter	Symbol	Definition	Value for Cowden till for $2 \le \frac{L}{D} \le 6$ Byrne et al. (2020)
Normalized distributed lateral load	$\bar{p}$	$\bar{p}=\frac{p}{s_u D}$	-
Normalized lateral displacement	$\bar{v}$	$\bar{v}=\frac{v\,G_0}{D\,s_u}$	-
Ultimate normalized lateral displacement	$\bar{v}_u$	$\bar{v}_u=\frac{v_u\,G_0}{D\,s_u}$	241.4
Initial stiffness	k_p	-	$10.6-1.65\frac{z}{D}$
Curvature	n_p	-	$0.939-0.03345\frac{z}{D}$
Ultimate normalized lateral load	$\bar{p}_u$	$\bar{p}_u=\frac{p_u}{s_u D}$	$10.7-7.101\,e^{-0.3085\frac{z}{D}}$
With			
Distributed lateral load	p		
Lateral displacement	v		
Undrained shear strength at depth z	s_u		
Small-strain shear modulus at depth z	G_0		
Pile diameter	D		

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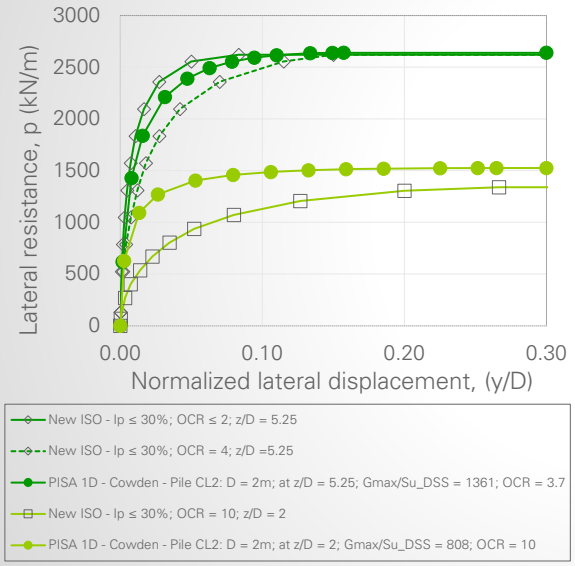
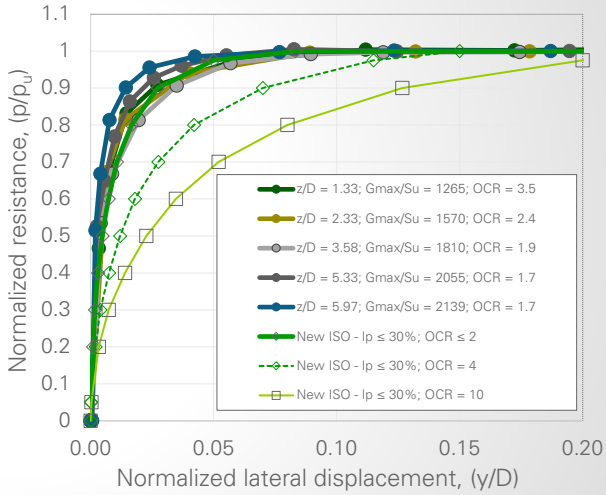
Shape of p-y curves: PISA Cowden vs new ISO

PISA Cowden test CL2



PISA FEA Calibration Piles C4 & C8



See Jeanjean et al. (2022) for details

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Contributions

- Unified CPT design method for piles in sands:
  - Developped by Oil and Gas industry
  - Equally applicable to Offshore Wind jacket piles and monopiles design
  - Key reference: Lehane et al. (2020)
- New ISO p-y curves for long flexible piles in clays:
  - Developped by Oil and Gas industry
  - Applicable to Offshore Wind jacket piles design but not for monopiles with L/D < 10
  - Monopiles: more soil reaction curves needed (PISA method uses 4)
  - Key reference: Jeanjean et al. (2022)
- PISA design method for clays:
  - Developped by the Offshore Wind industry
  - Distributed lateral load reaction curves for Cowden are entirely consistent with the new ISO p-y curves
  - Potentially applicable to Oil & Gas industry, for foundation of similar geometry
  - Key reference: Byrne et al. (2020)

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Main references:

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Wu F, Chen C-H, and Litton R, (2020): "Effect of Best-Estimate Clay P-Y Curves on Performance of Offshore Structures", Offshore Technology Conference, Houston, TX, Paper 30889

Zakeri, A., Sturm, H., and Jeanjean, P. (2019): "Validation and extension of soil response framework for fatigue analysis of offshore wells and piles." Proceedings, Offshore Technology Conference, Houston, TX, Paper 29236.

Zhang, Y., Andersen, K.H., Jeanjean, P. (2020) " Verification of a framework for cyclic p-y curves in clay by hindcast of Sabine River, SOLCYP and centrifuge laterally loaded pile tests" Applied Ocean Research, Vol. 97, April, https://doi.org/10.1016/j.apor.2020.102085

Zhang, Y., Andersen, K.H., Jeanjean, P. (2019). "Cyclic p-y curves in clays for offshore structures." Proceedings, Offshore Technology Conference, Houston, TX, Paper 29346.

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- Proceedings published in Aug. 2020: 202 technical papers; 6 keynote lectures; 5th ISSMGE McClelland Lecture
- In-person (?) conference: Austin, Texas | August 28-31, 2022
- Six webinars will be offered in 2022 – free of charge - streamed live – uploaded to YouTube

Date	Topic	Organizer
January 2022	Data Science 101	Bruno Stuyts Ghent University
February 2022	Fundamentals of Cyclic Loading in Offshore Geotechnics	Phil Watson University of Western Australia
March 2022	New proposed ISO/API p-y curves for piles in clays	Philippe Jeanjean BP America
April 2022	New proposed ISO/API unified CPT-based method for axial pile capacity in sand.	Farrokh Nadim Norwegian Geotechnical Institute
May 2022	New integrated design models for offshore wind turbines foundations (REDWIN JIP)	Ana Page Norwegian Geotechnical Institute
June 2022	New Design Tools for Laterally Loaded Wind Turbine Monopiles (PISA JIP)	Byron Byrne University of Oxford

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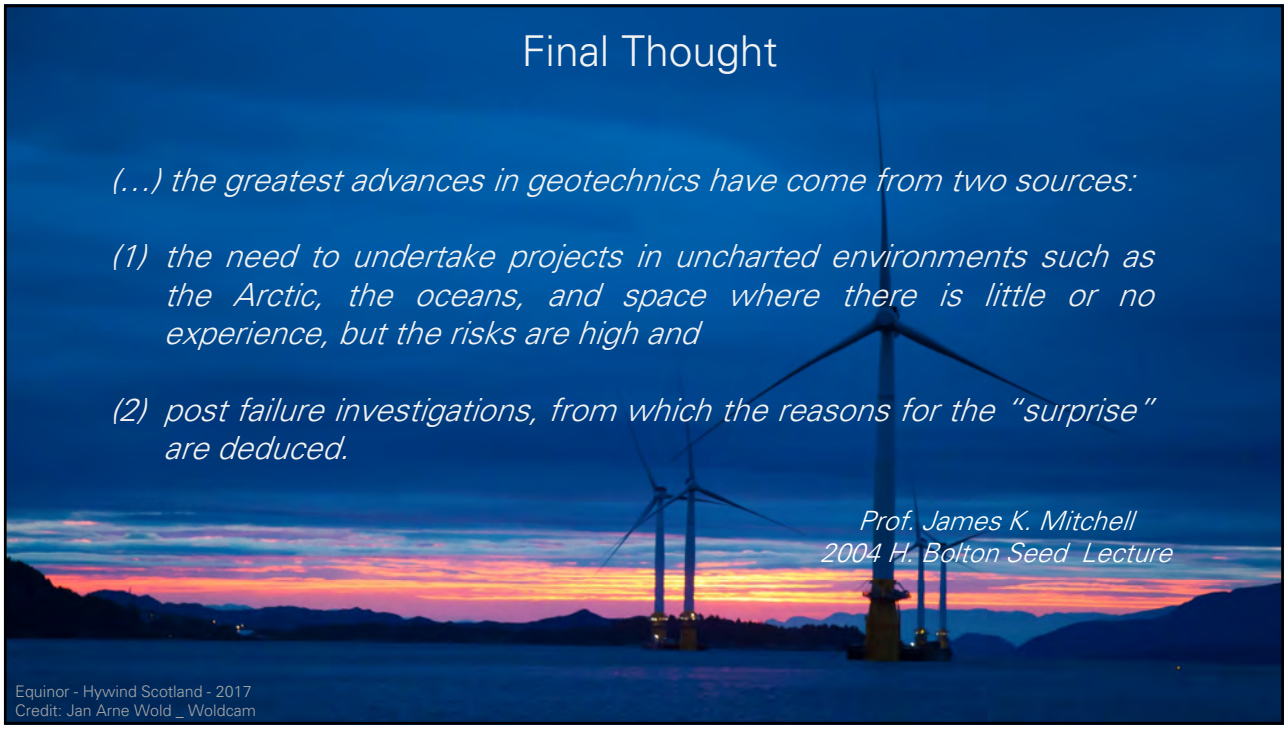
Final Thought
(…) the greatest advances in geotechnics have come from two sources:
(1) the need to undertake projects in uncharted environments such as the Arctic, the oceans, and space where there is little or no experience, but the risks are high and
(2) post failure investigations, from which the reasons for the "surprise" are deduced.
Prof. James K. Mitchell
2004 H. Bolton Seed Lecture
Equinor - Hywind Scotland - 2017
Credit: Jan Arne Wold _ Woldcam

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