GRIP MEETING 2022, Final Report

Project: Prediction model of vibration-induced settlement due to pile driving (BDV24 977-33)

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Outline

Selected Topics:

- Brief project overview (Benefits, Objectives, Scope)
- Technical review of case studies (Task 1)
- Survey to practitioners (Task 2)
- Field testing in pile installation sites (Task 3)
- Numerical modeling of pile driving induced deformations (Task 4)
- Empirical prediction eqns./formulae/charts for ground def. (Task 5)
- Implementation and recommendations

Review of benefits and objectives

Qualitative:

- Better estimation of infrastructure damage as a result of pile-driving induced deformations.
- Understanding pile driving induced deformation mechanisms can improve design practices in Florida.
- Infrastructure damage will be minimized as a result of pile driving and potentially avoid unnecessary countermeasures in FDOT projects.

Quantitative:

- Produce pile driving induced ground deformation charts (or correlations or equations) relating PPV, D_r, distance from source, and input energy to be used in FDOT projects.

Objectives:

- To understand mechanisms of near-field and far-field settlement and determine influence zones.
- To measure field vibration-induced deformations in predetermined locations in Florida.
- To develop calibrated numerical models of dynamic ground deformations due to pile driving.
- To develop pile driving induced deformation prediction models (e.g., equations or charts).

Tasks and research activities

Task 1: Technical Review

- Case histories database
- Existing ground deformation prediction methods
- Previously reported numerical modeling approaches

Task 2: Survey

- Statewide survey to geotechnical engineers
- Questions about pile driving effects

Task 3: Field testing

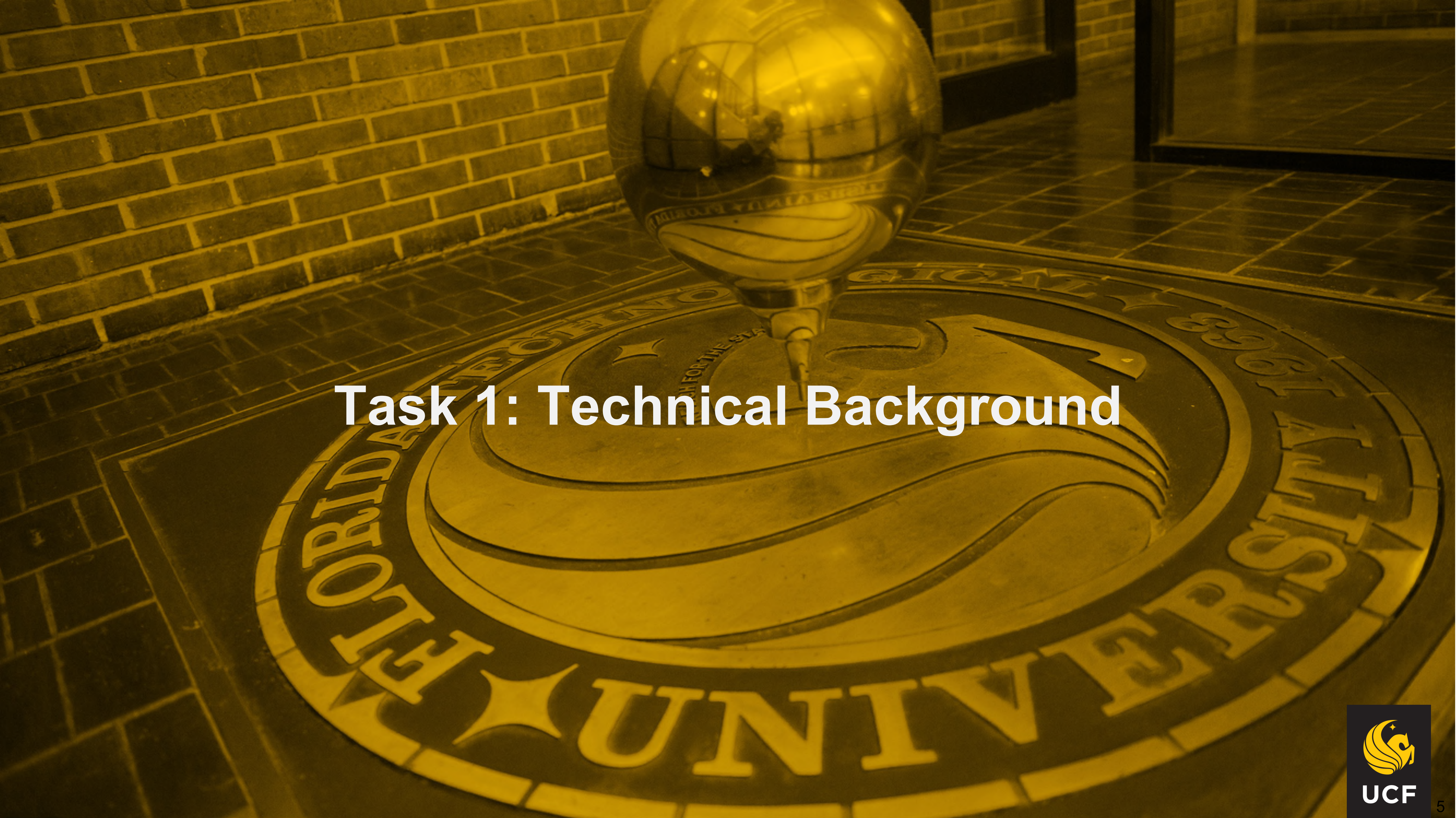
- 11 construction sites in Central Florida
- Site-specific characterization, subsurface conditions, and geotechnical mechanisms
- Measurement of ground deformations and vibrations

Task 4: Numerical modeling

- Comparison of modeling approaches
- Model validation and comparison with field data
- Parametric study analyzing main variables involved

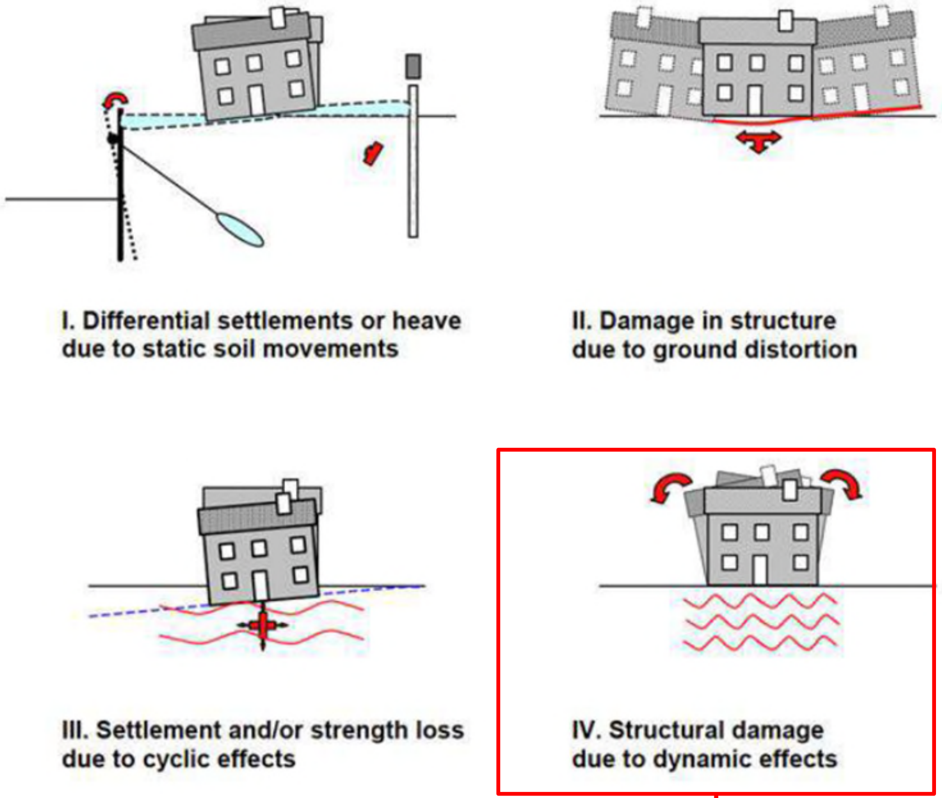
Task 5: Empirical correlations

- Propose charts and equations to predict ground deformations caused by pile driving



Task 1: Technical Background

Introduction and motivation of the study


I. Differential settlements or heave due to static soil movements

II. Damage in structure due to ground distortion

III. Settlement and/or strength loss due to cyclic effects

IV. Structural damage due to dynamic effects

Most design standards focus on this category!

Massarsch and Fellenius (2014)

Pile driving effects (Dowding, 1996):

i) Vibration-induced particle rearrangement: **ground deformations!**
ii) Excess pore water pressure build-up, that when dissipated: **ground deformations!**
iii) Soil re-sedimentation after localized liquefaction around pile: **ground deformations!** (Pile driving may cause settlement due to densification and liquefaction of vulnerable soils)
iv) Damage nearby infrastructure as a product of **ground deformations!**

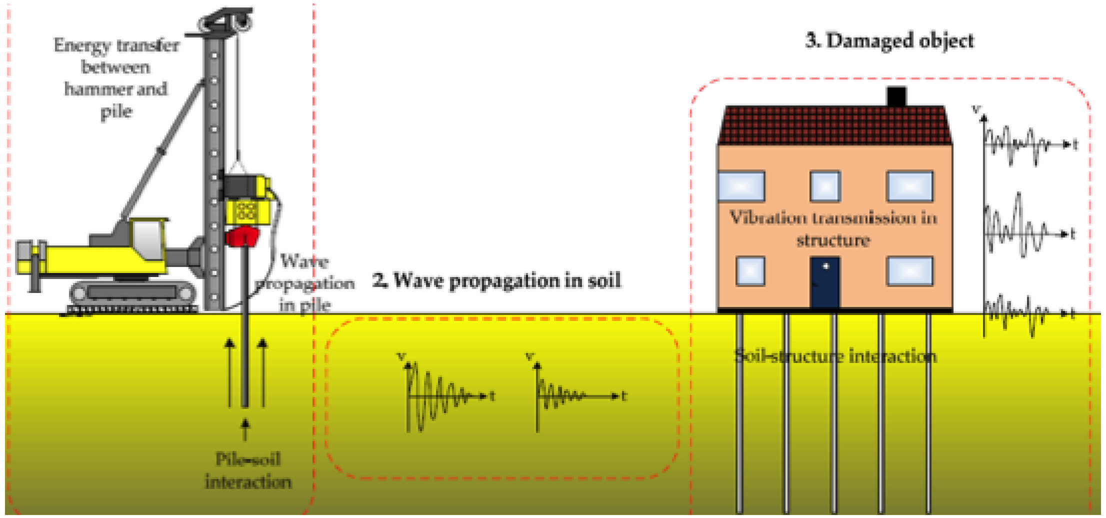
Comments:

- Vibrations caused by pile driving can generate PPV up to *4 in/s,* but even for PPV of only *0.1 in/s* settlements in sands can still occur.
- PPV limit in Florida: *0.5 in/s*. Some recommendations on monitoring zones, but not much about ground deformations.
- FDOT monitoring zone: 0.5 $ft/\sqrt{lb-ft}$ (i.e., 16 $ft/\sqrt{kips-ft}$)
- Dowding (1996) and Lacy and Gould (1985): *0.08 in/s* is the limit beyond which dynamic settlements may occur.

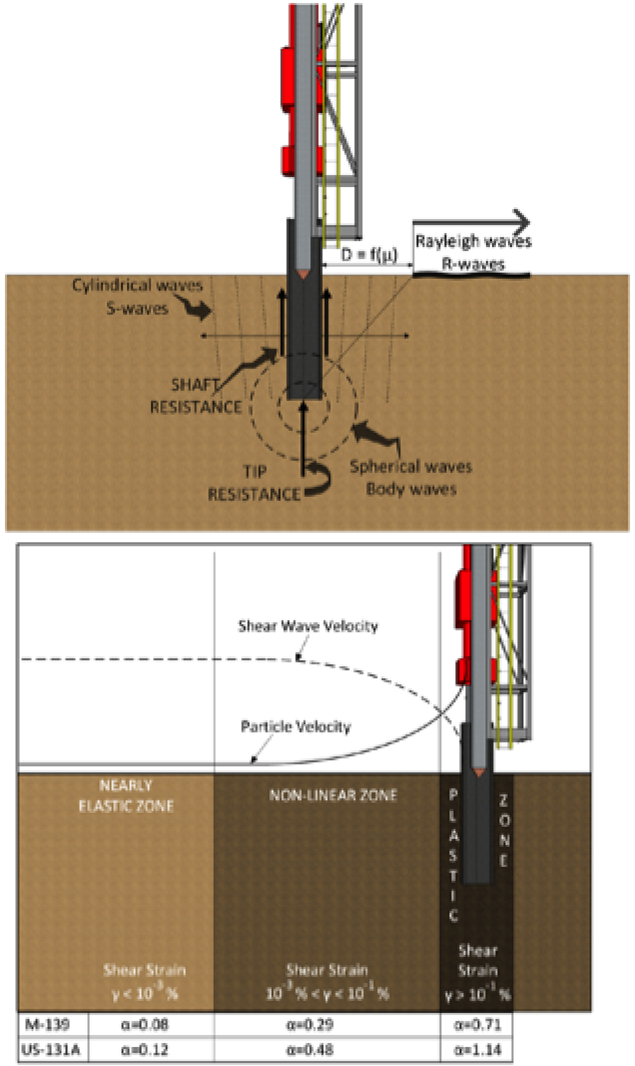
Technical background: Main variables involved in the problem

76 case histories and 55 papers reviewed to study variables involved:

- ➢ Vibration characteristics and input energy: vibration type, amplitude, frequency, and duration of the source
- ➢ Soil characteristics: soil gradation and type, relative density, and moisture content
- ➢ Attenuation characteristics: geometric and material damping



Pile-driving induced vibration in urban environments (Hintze et al. 1997 and Deckner 2013)

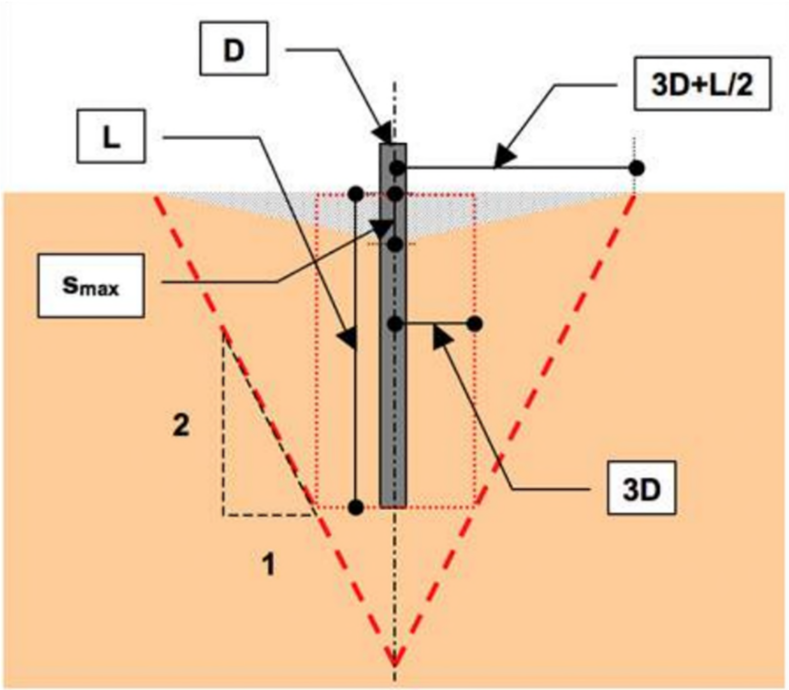


M-139	α=0.08	α=0.29	α=0.71
US-131A	α=0.12	α=0.48	α=1.14

Energy transfer from pile to soil (top)
Hypothetical soil behavior zones in terms of shear strains and attenuation coefficients (bottom)

Technical background: pile driving induced settlement estimation methods

Massarsch (2004)



Settlements adjacent to a single pile in homogeneous sand

- From that research, they claimed that densification due to pile driving occurs within a zone of three pile diameters around the pile.

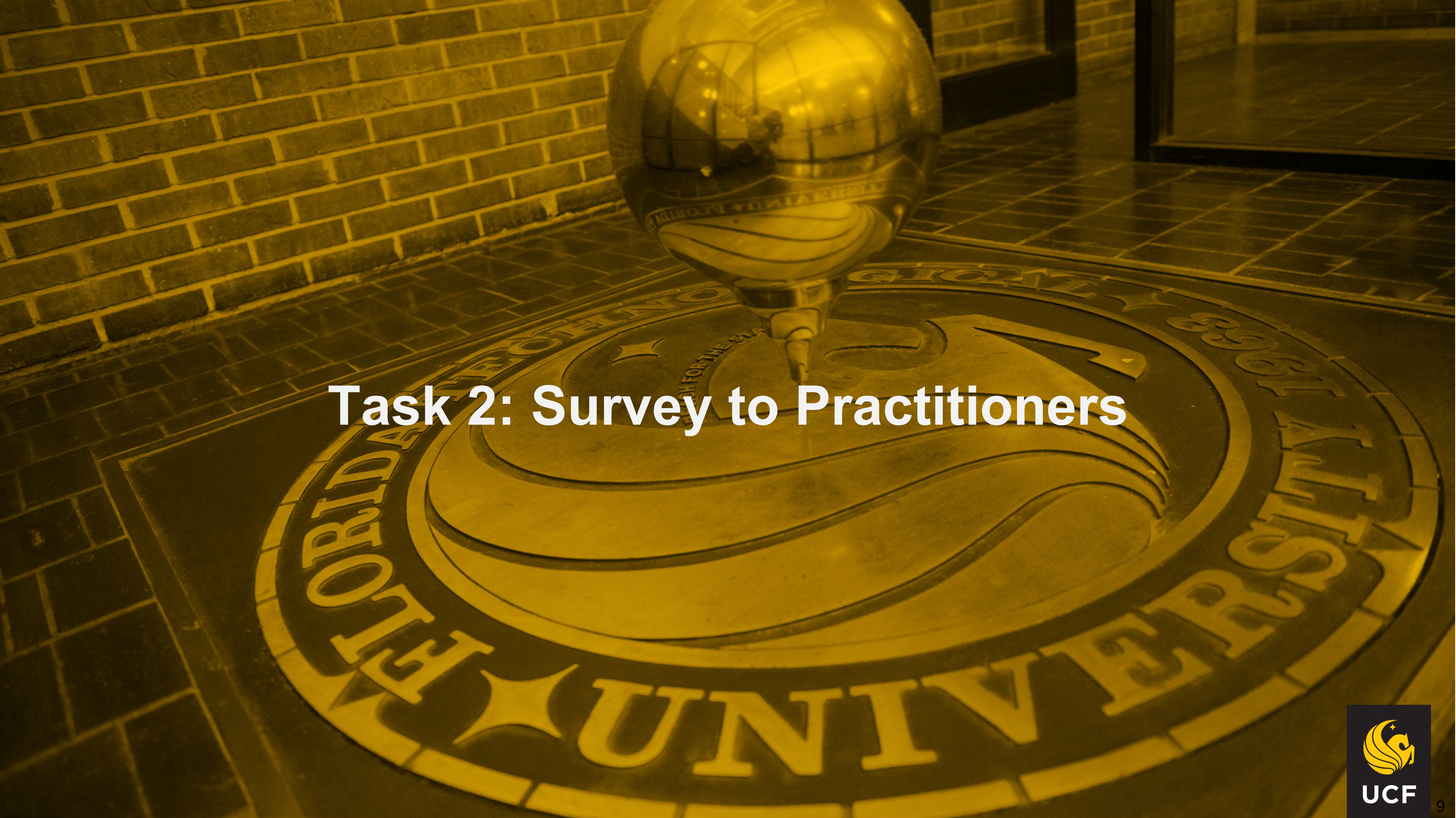
$$\boxed{\begin{aligned} S_{max} &= \alpha(L+6D) \\ S_{avg} &= \alpha\frac{(L+3D)}{3} \end{aligned}}$$

Drabkin et. al (1996)

Factor	Factor Code	Tested Ranges	Coding of Factors
Peak Particle Velocity (PPV)	x_1	0.1-0.7 in/sec	$x_1 = -1 + \frac{PPV - 0.1}{0.3}$
Deviatoric Stress (s)	x_2	2-15 psi	$x_2 = -1 + \frac{s-2}{6.5}$
Confining Pressure (p)	x_3	10-30 psi	$x_3 = -1 + \frac{p-10}{10}$
Sand Mixture	x_4	Coarse, Medium or Fine	x_4 ranges from -1 for coarse sand to 1 for fine sand
Number of vibration cycles (N)	x_5	60-500,000 cycles	$x_5 = -1 + \frac{N-60}{26{,}997}$
Moisture content	x_6	Dry, Saturated	x_6 ranges from -1 for dry sand to 2 for saturated sand
Initial relative density	x_7	Loose, Medium Dense	x_7 ranges from -1 for loose sand to 2 for medium dense sand

- Estimate/measure PPV.
- Compute x_i <u>only</u> if within the tested ranges.
- Calculate settlement.

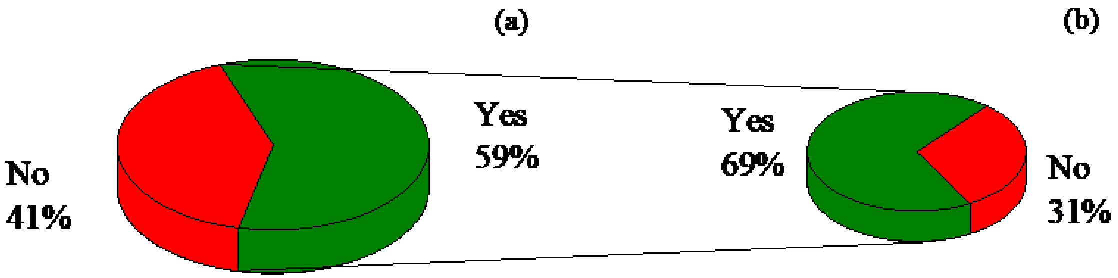
$$\ln Y = 2.27 + 1.19x_1 - 0.71{x_1}^2 + 0.49x_2 - 0.68{x_2}^2 - 0.80x_3 + 1.09{x_3}^2 - 0.46x_4 + 0.06{x_4}^2 + 0.45x_5 - 0.38{x_5}^2 - 0.19x_6 - 0.10x_7$$



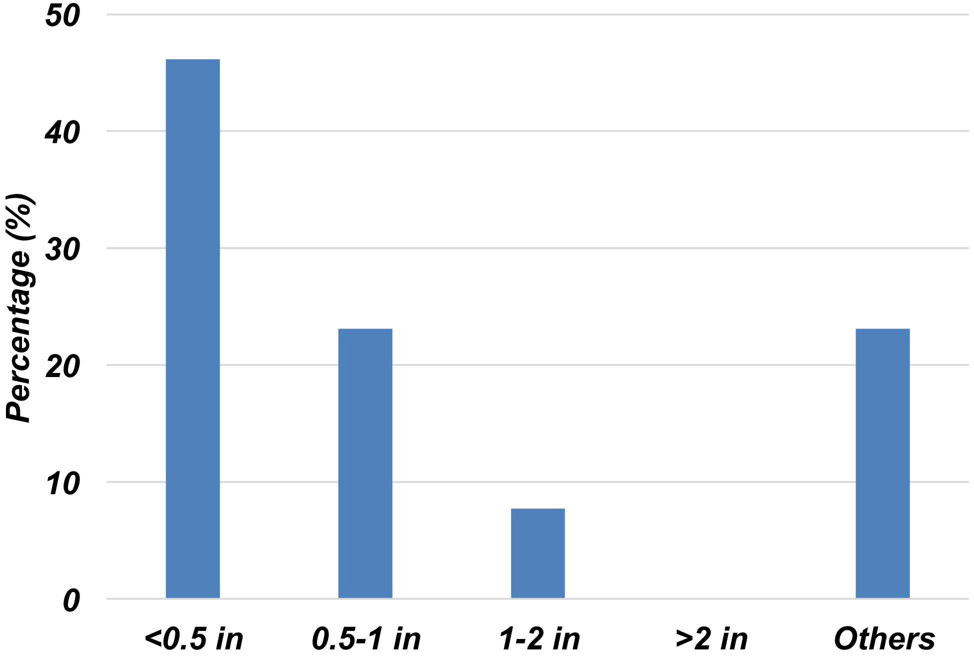
Task 2: Survey to Practitioners

Survey to practitioners (selected responses)

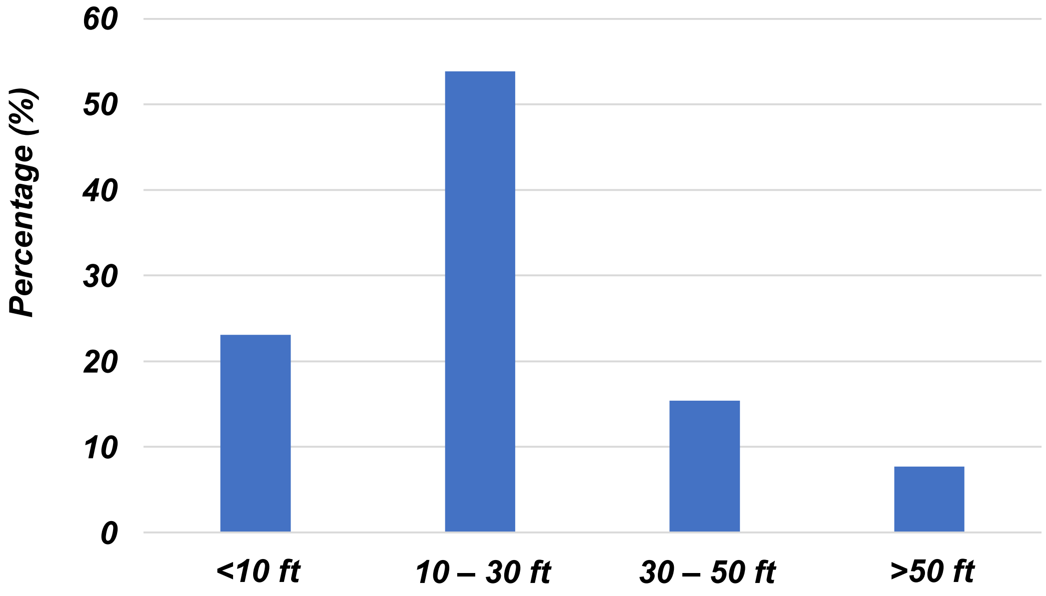
Did you observe or experience (a) any ground surface settlement and (b) any type of damage to adjacent infrastructure during pile driving because of high vibration levels?

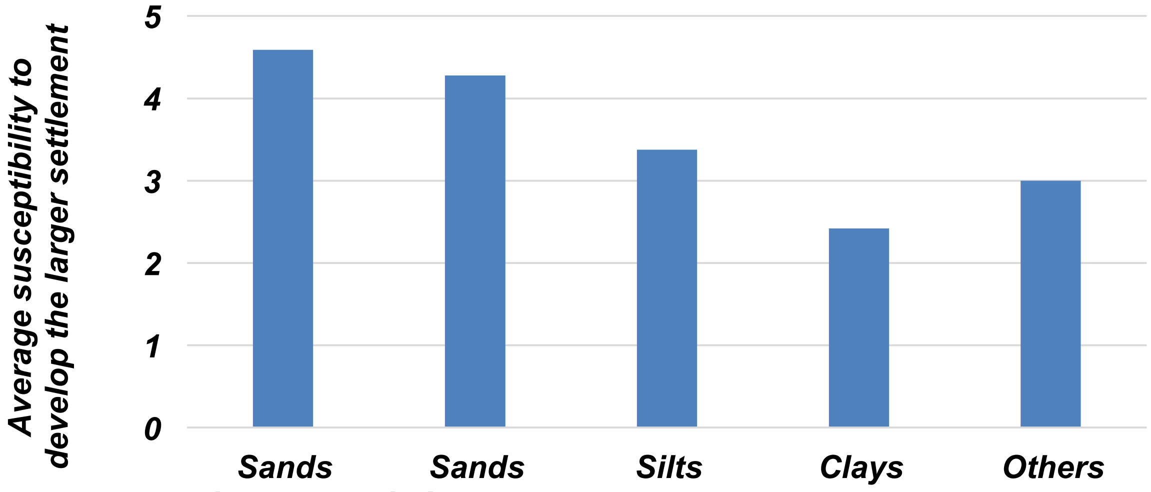


What was the approximate level of ground settlements experienced in the project?

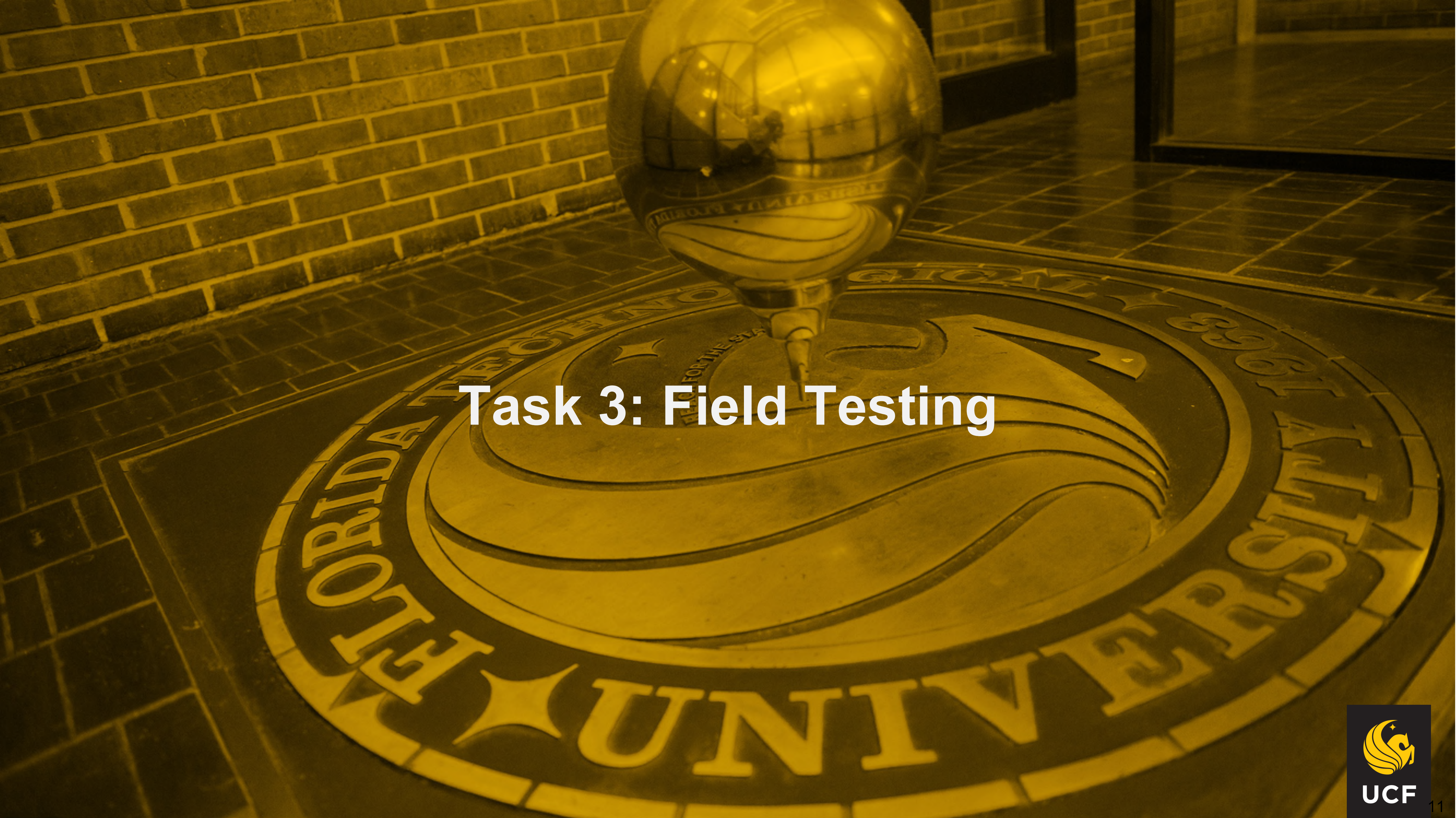


From your experience, what is the maximum distance from the pile driving source at which infrastructure (e.g., buildings, public utilities, bridges, etc.) is not affected by pile driving?





Number of questions: 20
Respondent population: 44



Task 3: Field Testing

Instrumentation plan

Procedure:

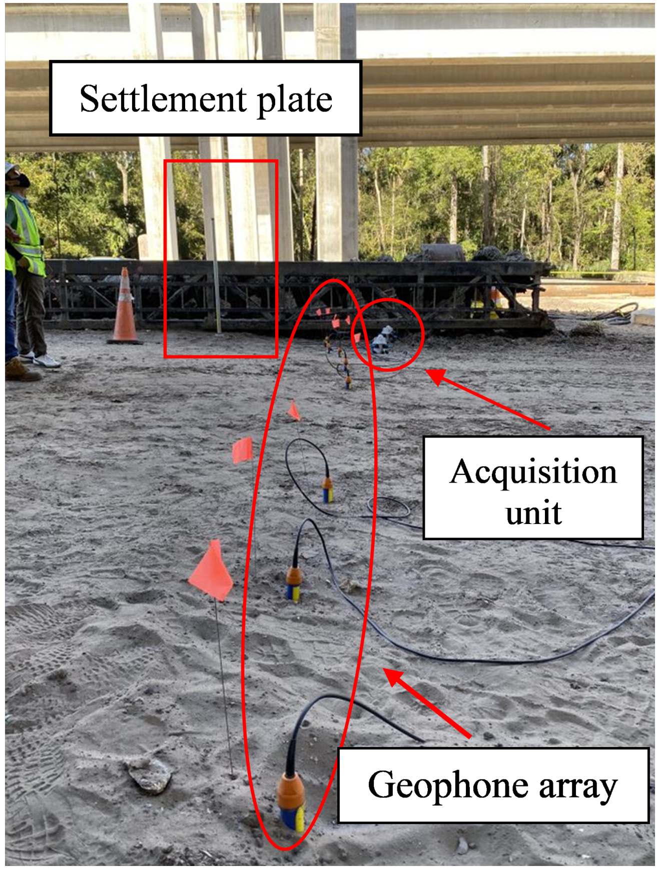
Site selection → Instrumentation → Data collection → Analysis

Field testing measurements:

Vertical Particle Velocity (to compute PPV)

Ground Deformations (settlement / heave)

- Eighteen 5 Hz geophones (Sercel)
- Acquisition units (Sercel Unite)
- Survey equipment and survey nails
- Settlement plate



Site locations

Project site locations:

- Four sites in Lake County.
- Five sites in Orange County.
- One site in Volusia County and Osceola County each.

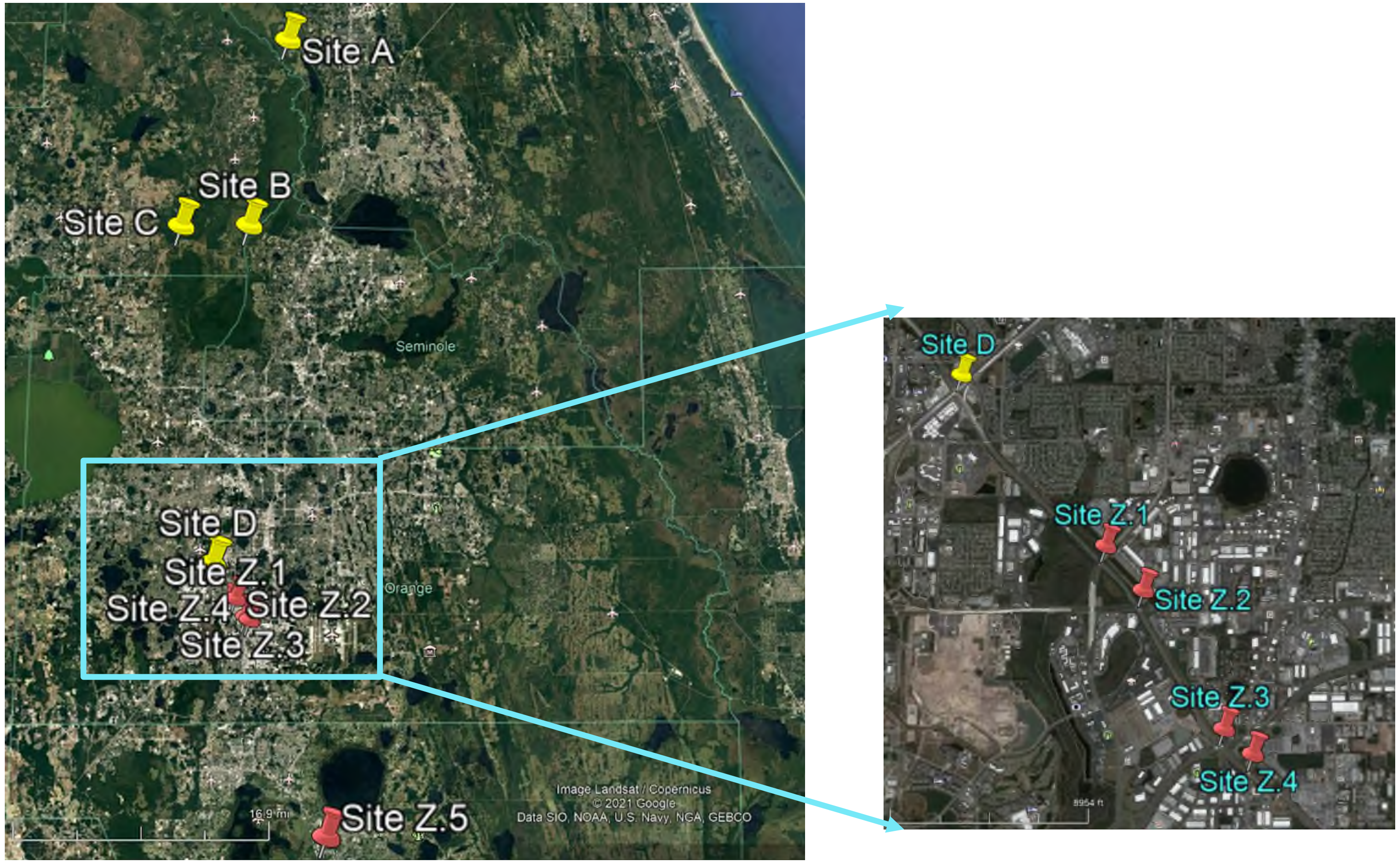


Image Landsat / Copernicus
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Data SIO, NOAA, U.S. Navy, NGA, GEBCO

Site A:

- Subdivided in 3 “sites” referring to different piers.
- Multiple piles driven per pier.
- Splicing occurred in some piles.
- Site used to validate the numerical model as well.

Site B:

- 1 test pile installed.
- Cofferdam installed around the pier prior to test pile.

Site C:

- Installation of 7 consecutive piles.
- Used to analyze ground response under multiple driving cycles.

Site D:

- PDA measurements for 1 test pile.
- Used to replicate forces in GRLWEAP and pile penetration in PLAXIS 2D.

Sites Z.1 through Z.5:

- PPV measurements by Bayraktar et al. (2013).
- Used to validate computed PPVs.

Description of selected sites

Sites A.1 and A.2

- 2 piers
- Two 24-in piles driven per pier
- Lengths: 125 to 135 ft
- APE D50-52
- EDC data obtained

Site A.3

- 1 pier
- One 24-in pile
- Length:160 ft
- Pile spliced at 80 ft
- APE D70-52
- EDC data obtained

Site B

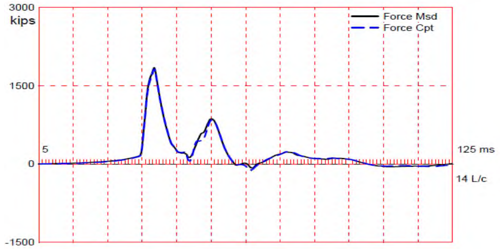
- One 24-in test pile
- Length: 65 ft
- Cofferdam installed around pier
- APE D70-52

Site C

- 7 production 24-in piles
- Spacing: 8.7 ft
- Penetration: 90 ft
- APE D70-52
- PDA data obtained

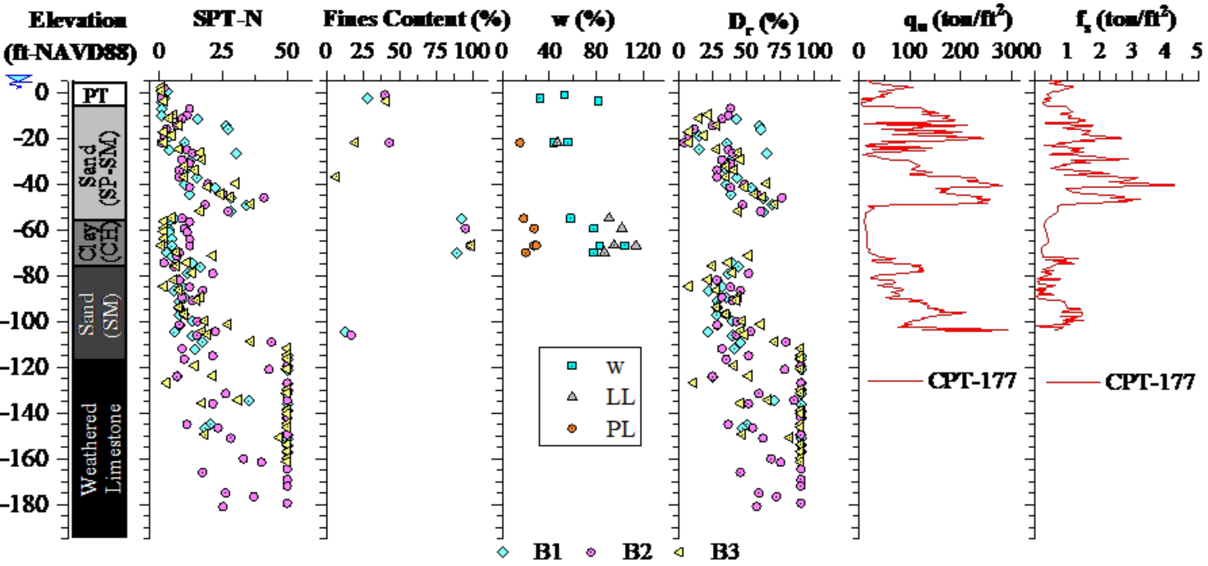
Site D

- One 24-in PCP up to 90 ft of penetration
- APE D70-52
- PDA records obtained from the contractor

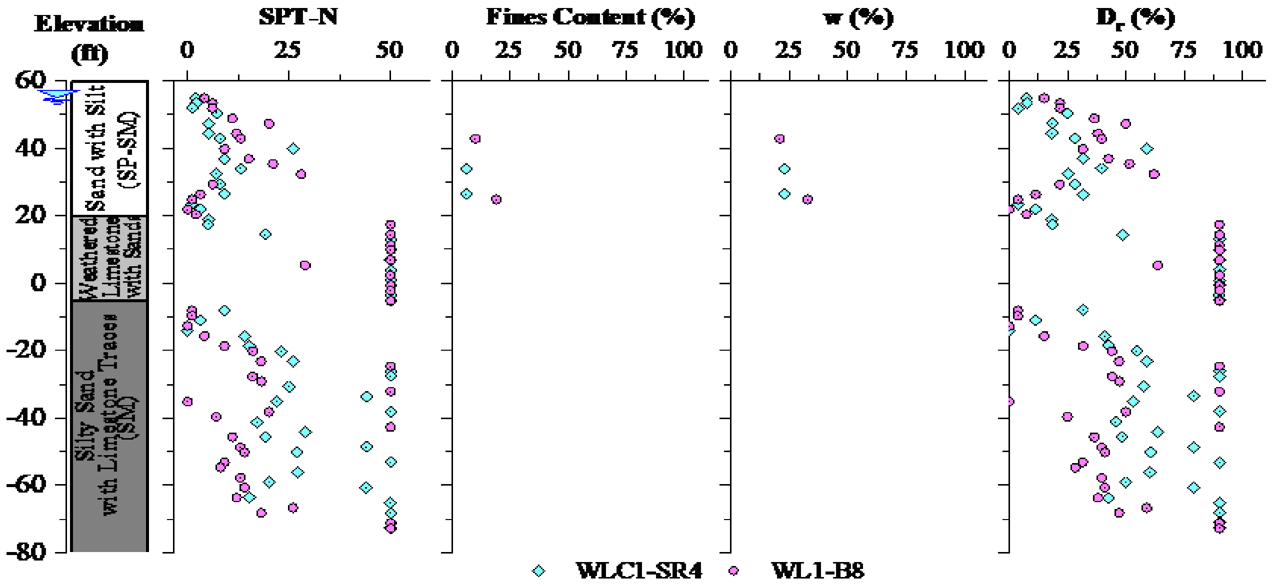


Summarized subsurface conditions

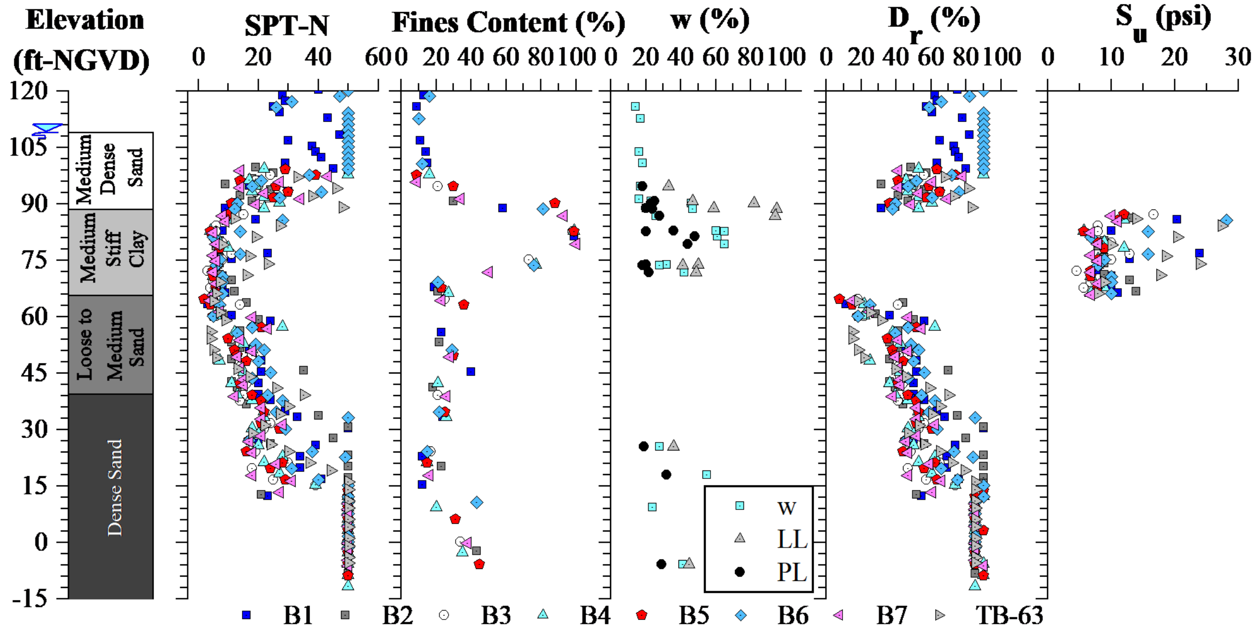
Site A



Site B



Site D

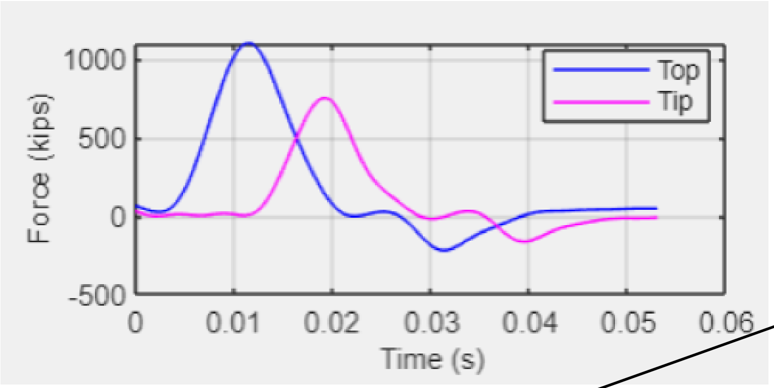


In summary:

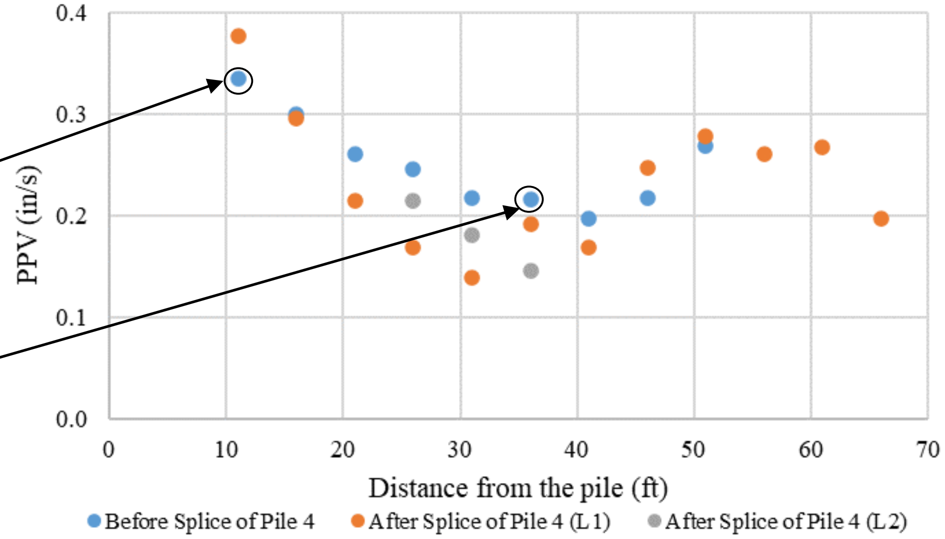
- ➢ Soil profiles defined using SPTs, CPTs, and index properties. Relative density (D_r) and undrained shear strength (S_u) defined using correlations with SPT blow count by Kulhawy and Mayne (1990).
- ➢ Soil conditions: mainly poorly graded sands and silty sands (i.e., SP and SM). Relative densities in the loose to medium-dense range.
- ➢ Shallow groundwater table encountered at all sites. Minor presence of interbedded fat clay layers (CH).

Typical results

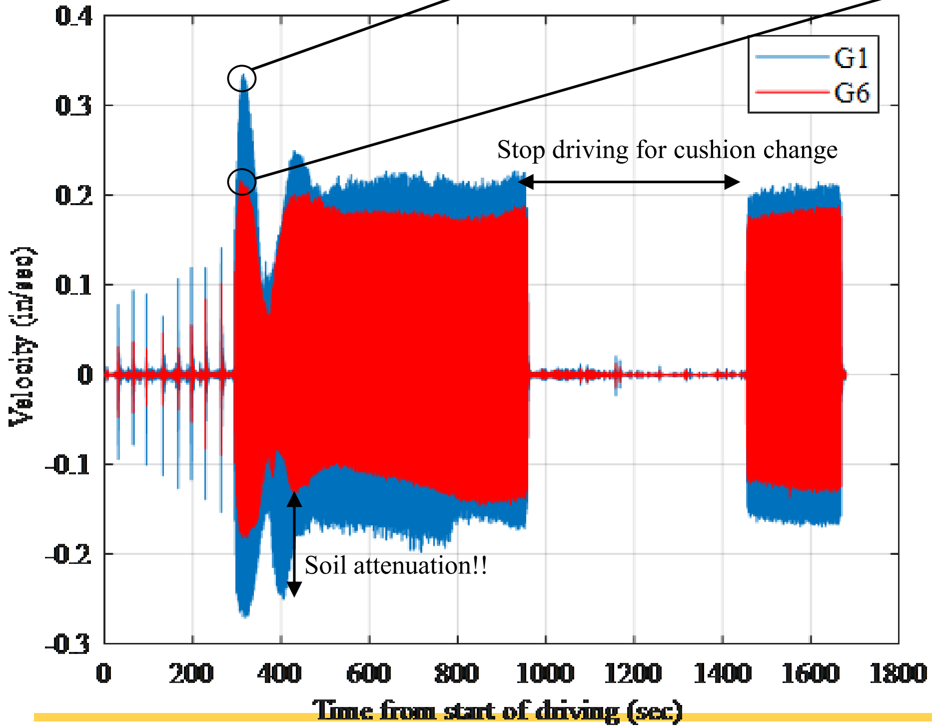
Forces computed at the top of the pile via EDC or PDA (Figure shows <u>EDC</u> for Site A)



PPV attenuation with distance (Site A3)



Velocity time history during entire pile driving (Site A3)

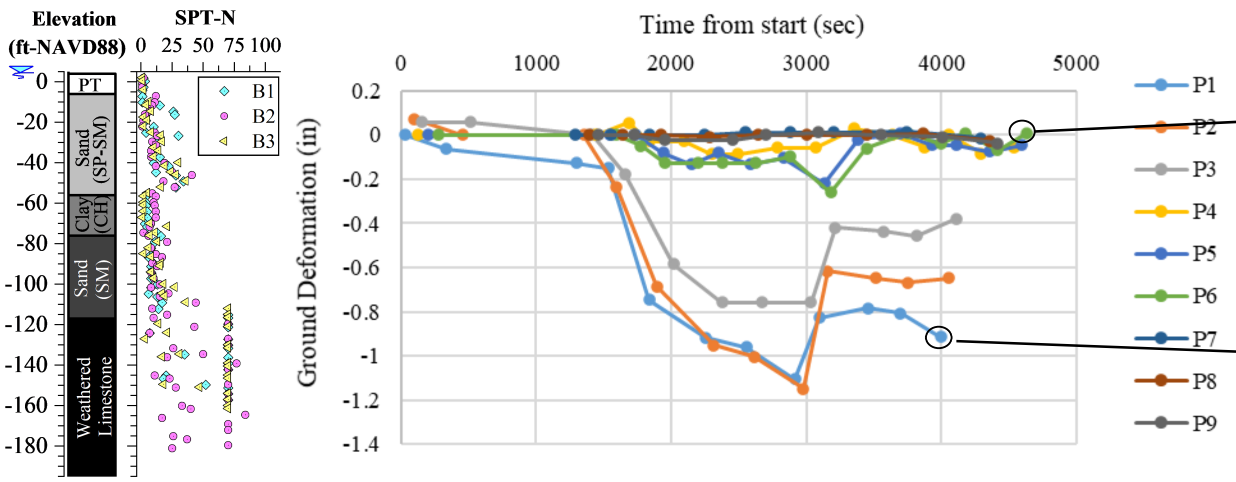


Lessons learned in terms of ground vibrations:

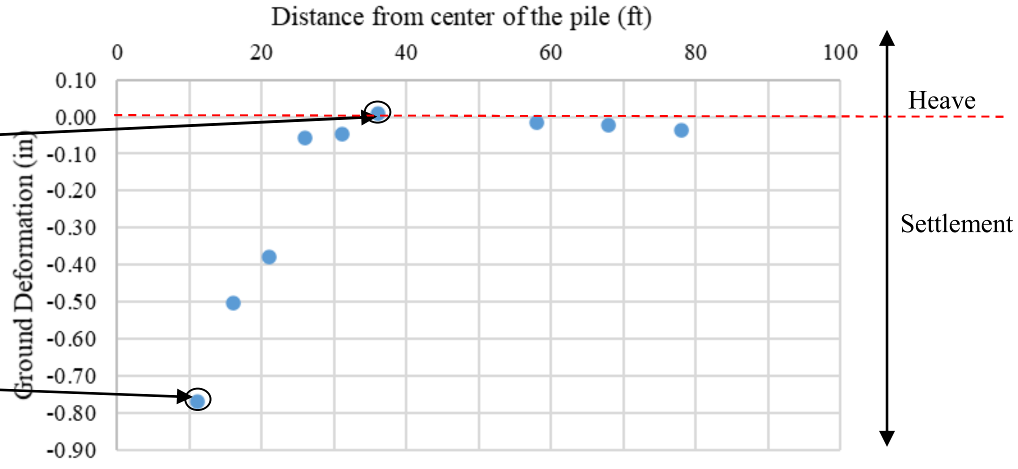
- PPVs normally occurred at initial stages of driving. Location of the tip of the pile relative to geophone locations matter.
- Larger PPVs occurred during installation of first pile in a pier, indicating possible changes in soil density as piles are installed.
- PPVs next to sheet pile walls or cofferdams were significantly reduced.
- Changes in fuel settings and pile cushions changed the magnitude of PPVs. Transfer energy from hammer to pile changed.

Typical results

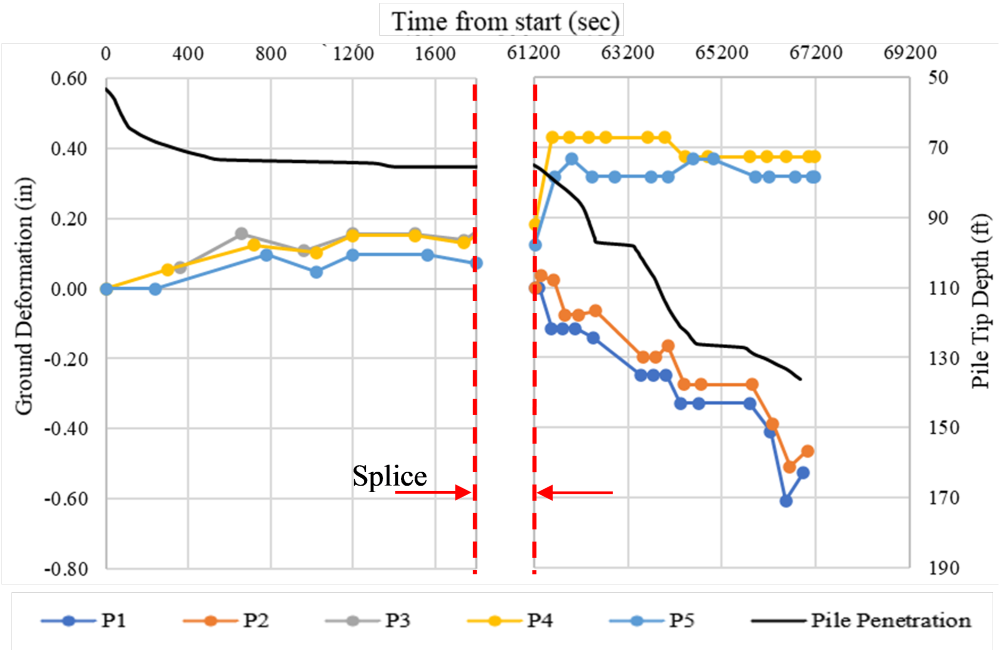
Ground deformation time history at each nail (Site A1)



Residual ground deformation due to pile driving (Site A1)



Ground deformation time history at each nail (Site A3)



Lessons learned in terms of ground deformations:

- ➢ Settlement was typically measured during driving first pile in a pier. Heave observed for second pile in most cases. Indication of relative density changes as piles are installed.
- ➢ Most ground deformations occurred at the beginning of driving. Location of the pile tip relative to survey nails matter.
- ➢ As expected, the larger the input energy, the larger the ground deformations. Presence of sheet piles or cofferdams reduced ground deformations.
- ➢ Predrilling as a variable affecting ground deformations was not quantified in the field since all installed piles were predrilled. It must have influenced the resulting ground deformations.

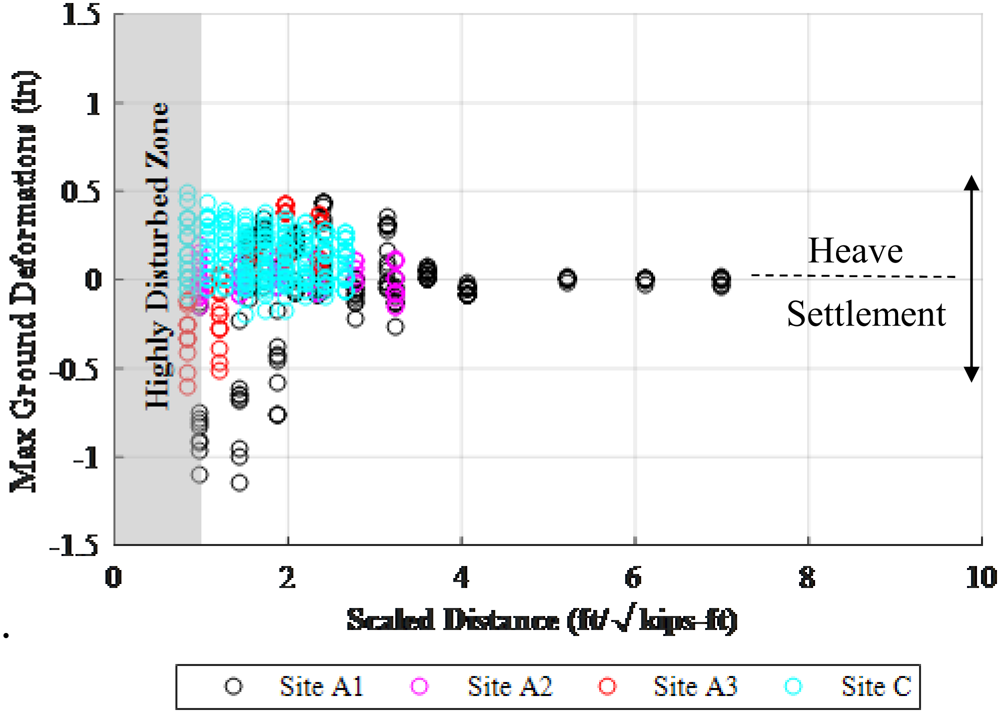
Summary of results

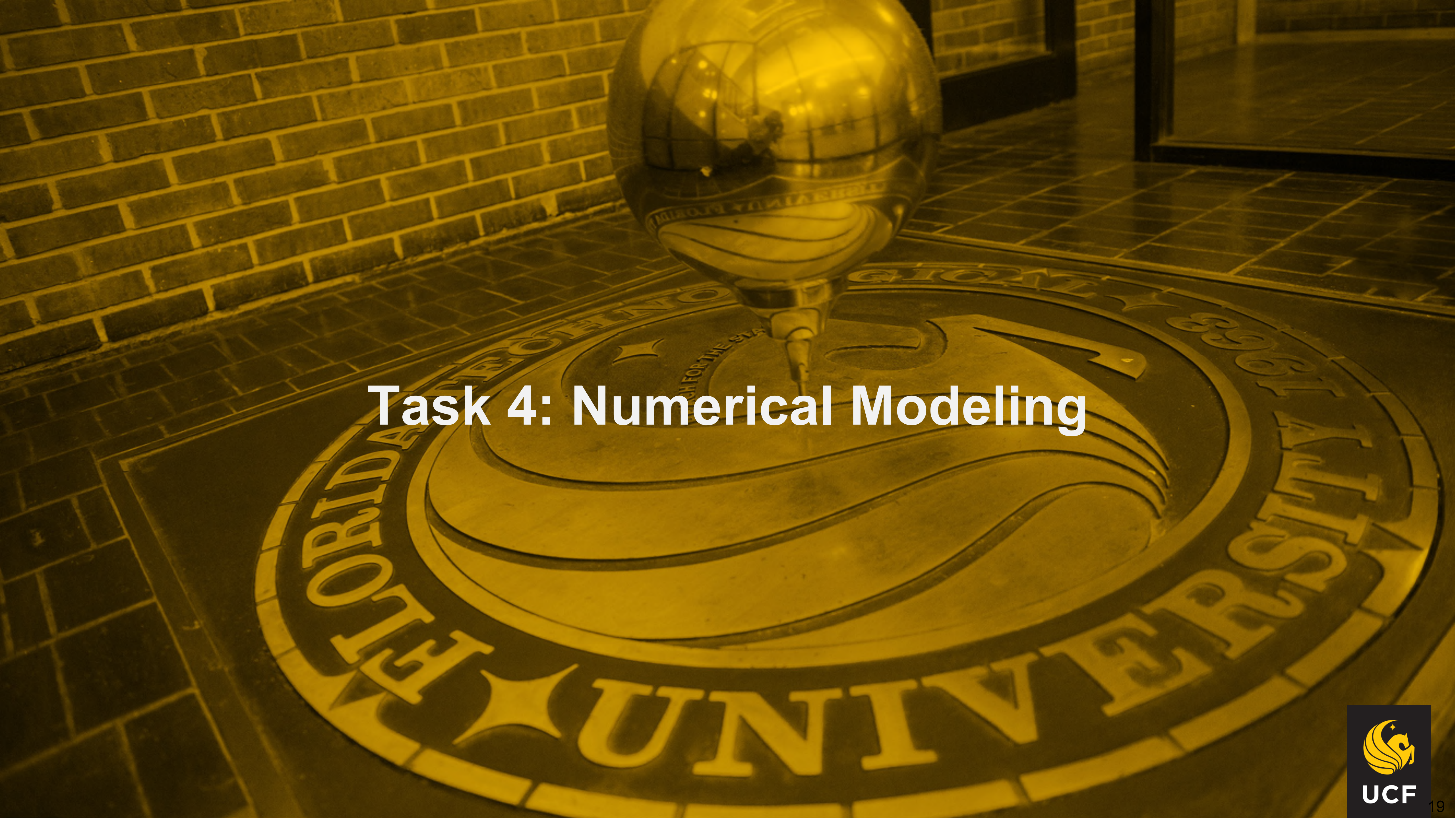
Site	Pile	Length (ft)	Pre-drilling (ft)	Hammer Type	Max PPV (in/s)	Max Residual Sett. (in)	Max Residual Heave (in)
A1	Pile 13	125	22	APE D50-52	>0.3	0.80	0.01
A1	Pile 10	125	35	APE D50-52	>0.3	0.10	0.45
A2	Pile 8	135	28	APE D50-52	0.7	0.15	0.15
A2	Pile 15	135	24	APE D50-52	>0.3	0.08	0.05
A3	Segment 1	80	28	APE D70-52	0.4	0.00	0.20
A3	Segment 2	80	-	APE D70-52	0.4	0.60	0.20
B	Pile 12	65	N/A	APE D70-52	0.2	0.00	0.00
C	Piles 2-6	110	N/A	APE D70-52	>0.3	0.10	0.40

Possible higher PPVs occurred. Geophone gains malfunctioned during the test.

- Max. settlement exceeded 1.0 in at site A1. Heave never exceeded 0.5 in.
- Settlement became negligible approx. beyond 4 $ft/\sqrt{kips - ft}$.
- Similar influence zone found for heave.
- Large ground deformations can occur independently from FDOT PPV limit.

Measured ground deformations vs scaled distance





Task 4: Numerical Modeling

Numerical modeling: Progress flowchart

Testing Site Selection

6 construction sites

Summarize Soil Profile

From soil borings and FDOT soil boring viewer

Analyze CAPWAP/ PDA/EDC Results

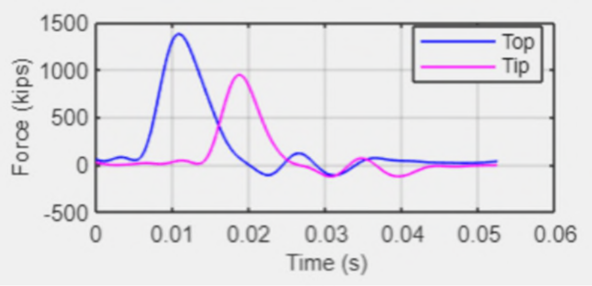
From geotechnical reports

GRLWEAP "Calibration" from PDA/EDC Results

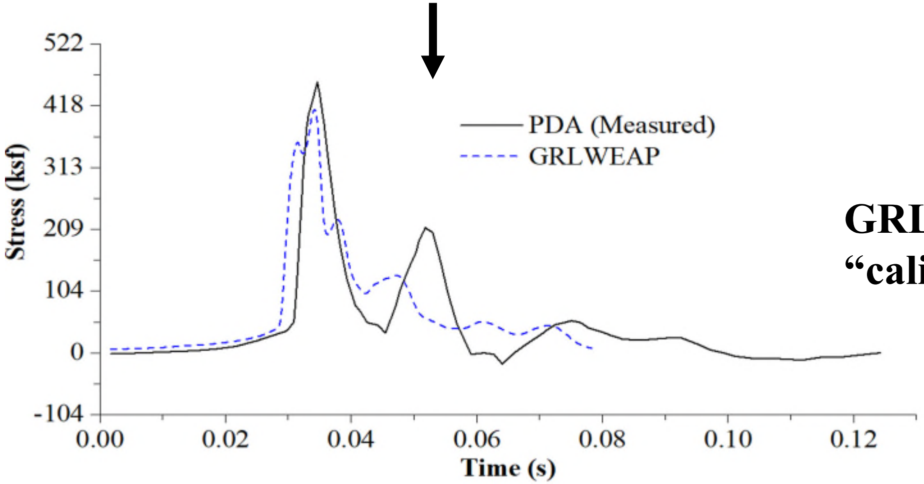
To define force time history and penetration per blow

PLAXIS

Apply force time history to analyze response in soil continuum

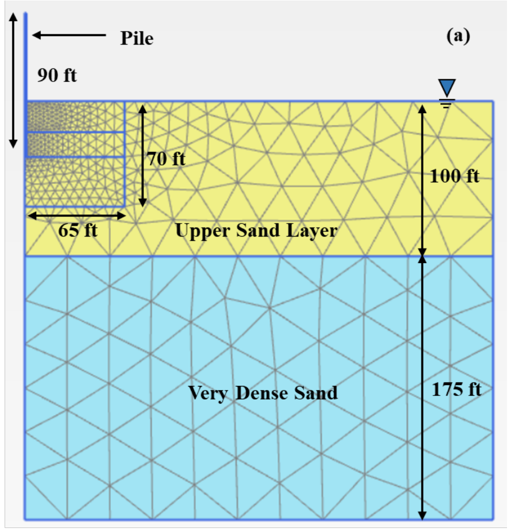


Typical EDC measurement. (CAPWAP/PDA provides similar)



GRLWEAP "calibration"

Typical stress function at the top of the pile



Numerical simulations

1. Definition: material properties (HS-small, UBC3D, and Hypo models) and drainage conditions. Type of analysis: dynamic with consolidation
2. Definition of model geometry (pile, soil clusters)
3. Definition of mesh
4. Initialization of soil stress field
5. Activation stage: pile cluster
6. Dynamic analysis: application of hammer blow time history.
7. Continuous pile driving approach.
8. Updated mesh for large deformation analysis.

Analysis of variables involved

Variables involved in the analysis:

- Relative density of the soils → To study dynamic response of the soil
- Peak particle velocity → To quantify vibration effects
- Distance from the pile → To determine attenuation characteristics
- Type of hammer and rated energy → To define input energy

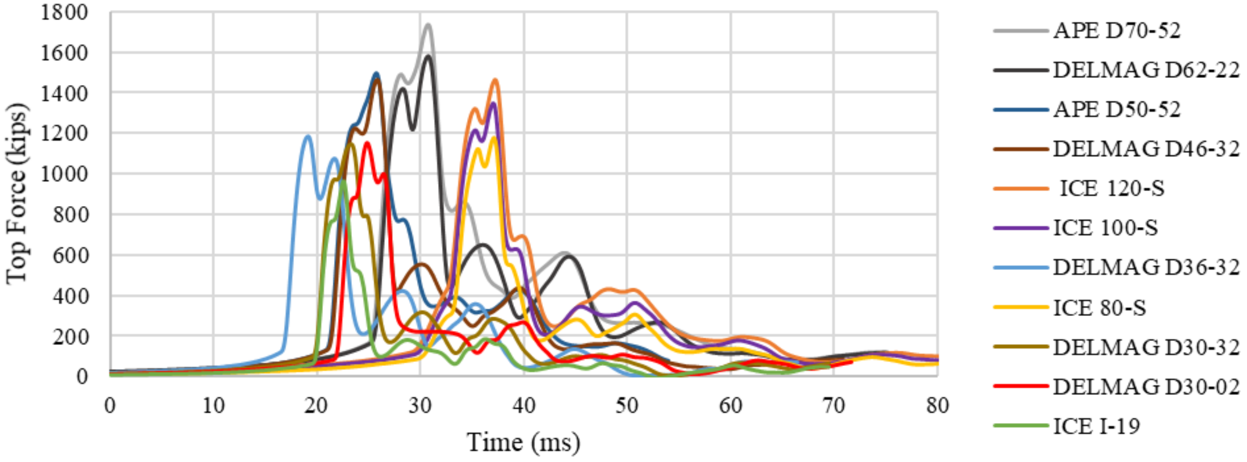
Survey of common hammers used in Florida:

- **Heung et al. (2007)** presented a total of 25 pile driving projects along Florida’s Turnpike.

Hammer Type	Rated Energy (kips-ft)
APE D70-52	173.6
DELMAG D62-22	164.6
APE D50-52	124.0
DELMAG D46-32	122.2
ICE 120-S	120.0
ICE100-S	100.0
DELMAG D36-32	90.6
ICE80-S	80.0
DELMAG D30-32	75.4
DELMAG D30-02	66.2
ICE I-19	43.2

Hammers used in this project field visits

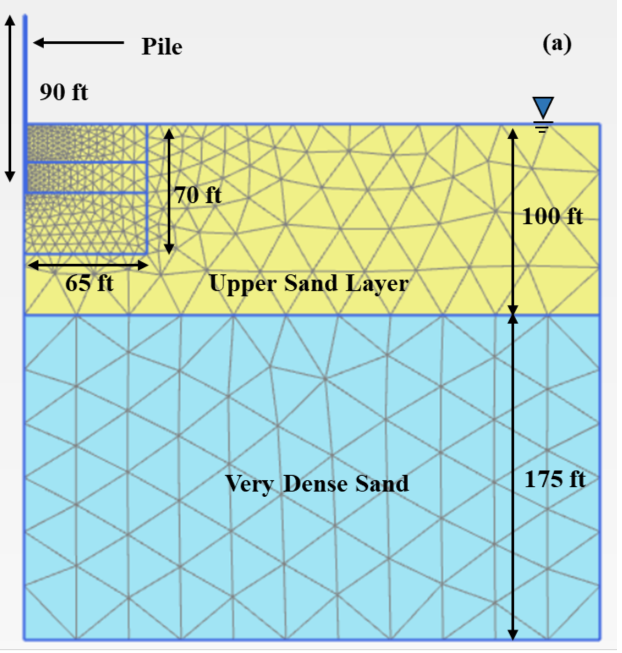
Selected hammer types for the parametric studies



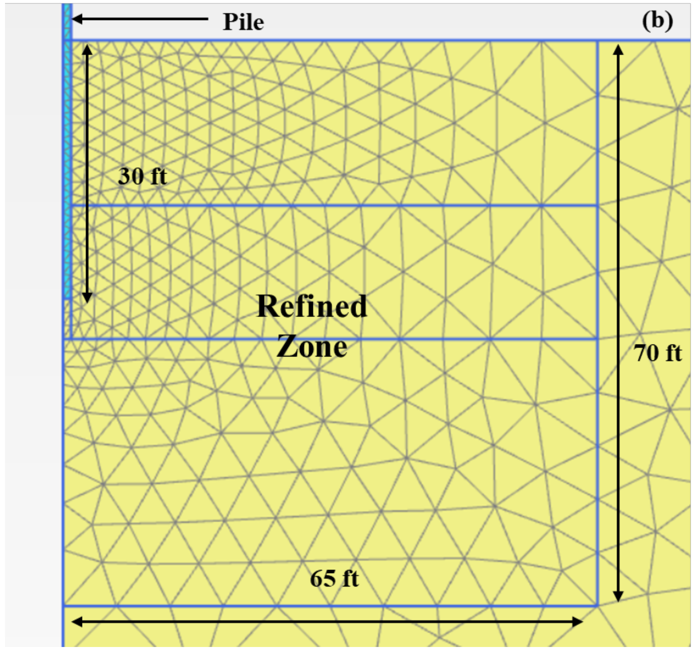
Analysis of force time histories applied at the top of the pile for commonly used hammers in Florida

Analysis of variables involved

Numerical model mesh



Close up view



Upper sand layer:

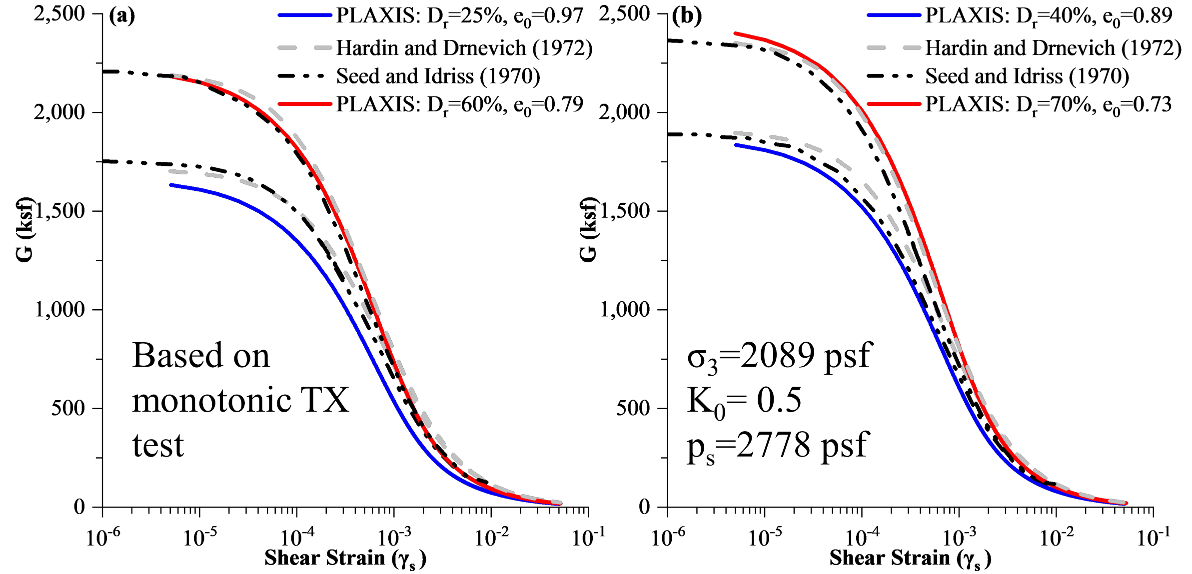
- D_r: 25%, 40%, 55%, 60%, 70%, and 75%
- Hypoplasticity model for sands enhanced with intergranular strain concept

Lower sand layer:

- D_r: 90%
- HS small model

e_{d0}=1.10 Maximum void ratio
e_{c0}=0.58 Minimum void ratio

From Lade et al. (1998) and Zapata-Medina et al. (2019):
Poorly Graded Sands tested for similar relative densities

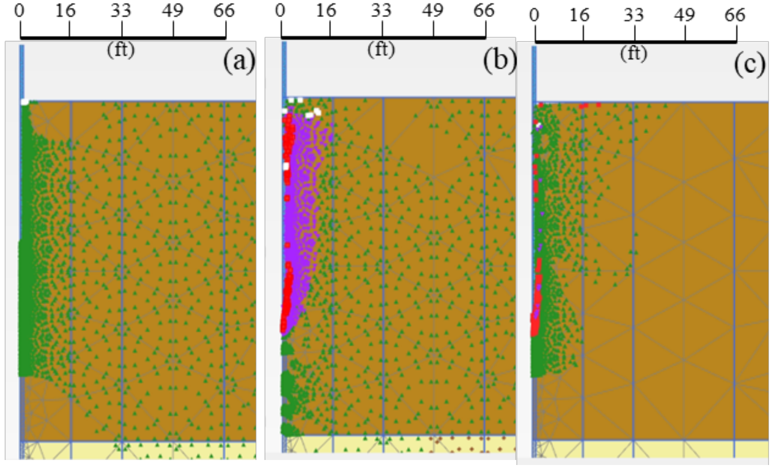


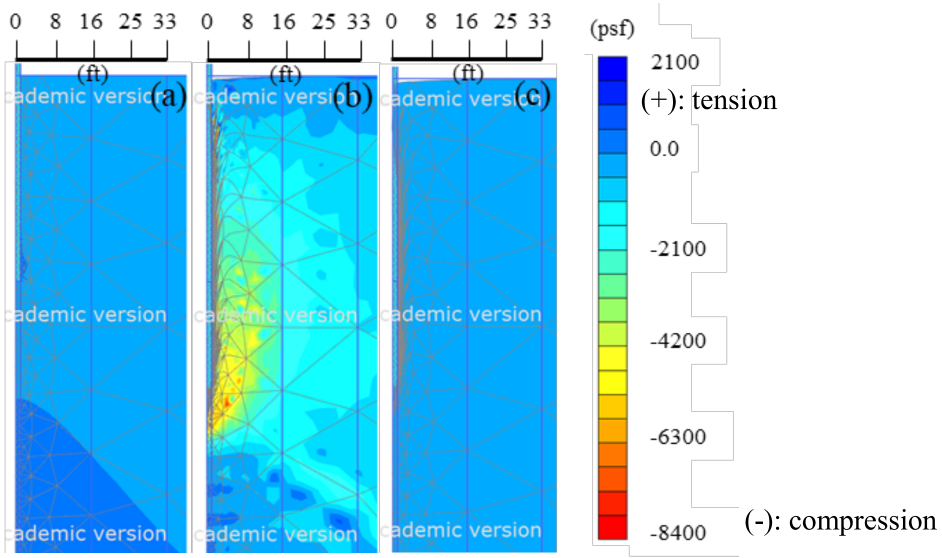
No.	Parameter	Description	Value	Unit
1	ϕ_c	Critical state friction angle	31	°
2	pt	Shift of the mean stress due to cohesion	0	psf
3	h_s	Granular hardness	25062	ksf
4	n	Exponent for pressure sensitive of a grain skeleton	0.37	-
5	e_{d0}	Minimum void ratio at zero pressure (ps = 0)	0.58	-
6	e_{c0}	Critical void ratio at zero pressure (ps = 0)	1.096	-
7	e_{i0}	Maximum void ratio at zero pressure (ps = 0)	1.315	-
8	α	Exponent for transition between peak and critical stresses	0.05	-
9	β	Exponent for stiffness dependency on pressure and density	1.4	-
10	m_R	Stiffness increase for 180° strain reversal	5	-
11	m_T	Stiffness increase for 90° strain reversal	2	-
12	R_{max}	Size of elastic range	5.00×10^{-5}	-
13	β_r	Material constant representing stiffness degradation	0.1	-
14	χ	Material constant for evolution of intergranular strains	1.0	-

Soil response in very close proximity to the pile

Plastic points

Liquefaction: Purple; Hardening: Green; Failure: Red





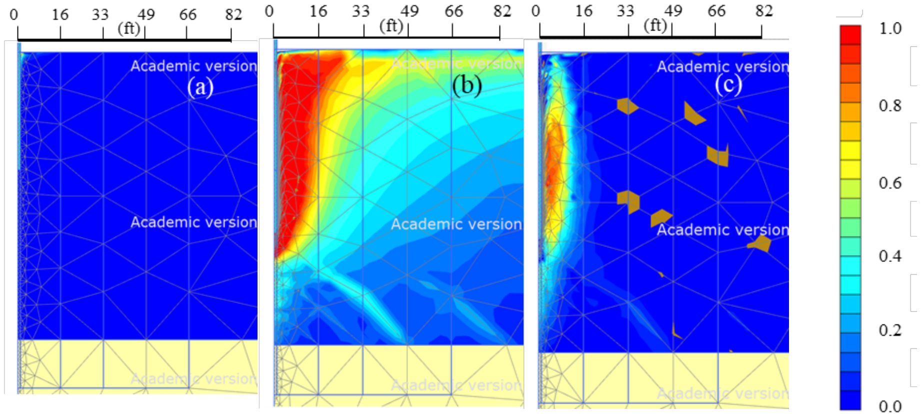
Contours showing excess porewater pressure buildup for the first 33 ft away from the pile.

- ➢ Excess pwp up to a distance of 15 ft away from the pile.

Modeling features:

- ➢ Soil liquefaction is possible for the highly disturbed zone next to the pile.
- ➢ UBC3D-PLM: model used to study soil liquefaction caused by impact pile-driving induced buildup of excess pore water pressures.
- ➢ APE D50-52 used in the analysis.
- ➢ Sand layer D_r= 40%. Parameters defined using correlations given by Beaty and Byrne (2011).

Stages: **a)** Initial stage,
b) 100 hammer blows,
c) Consolidation stage.



State variable: excess pore water pressure ratio (r_u)

$$r_u = 1 - \frac{\sigma'_v}{\sigma'_{v0}}$$

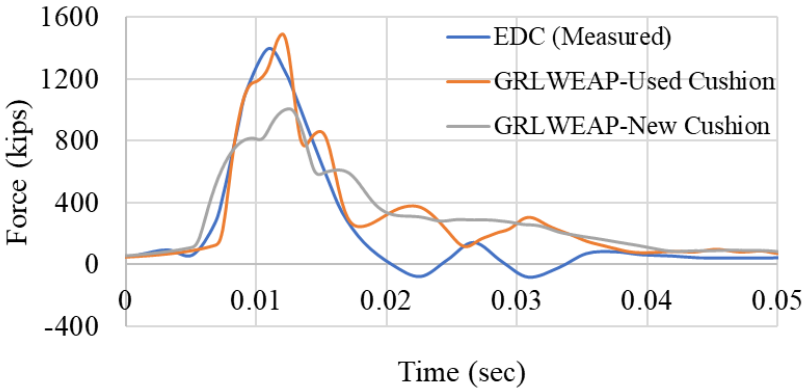
σ'_v ⟶ Vertical effective stress at the end of calculation stage

σ'_{v0} ⟶ Vertical effective stress at the initial condition

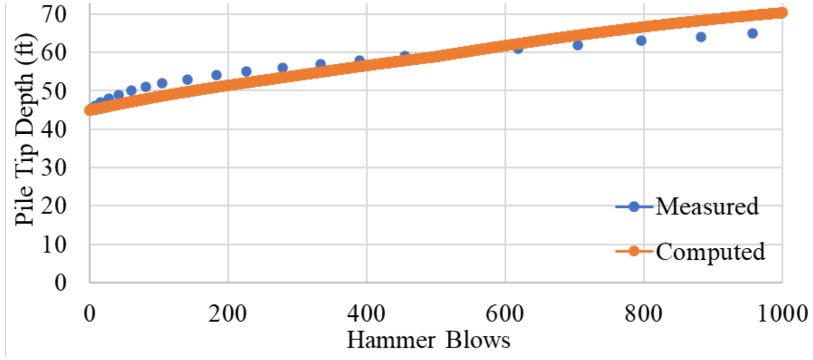
Comment: From these analyses, a highly disturbed zone was defined herein as the zone where very large deformations and PPVs occurred. Mainly within 1.0 to 1.5 $ft/\sqrt{kips - ft}$ away from the pile.

Numerical model validation with measurements of pile 13/site A1

- Pile cushion modeled both as “used” and “new” in GRLWEAP and force time history adjusted to match the pile penetration observed in the field.
- Site specific soil profile at the site with upper sand layer: D_r=40%, Hypoplasticity model for sands. Pile penetration process matched.
- PPV values matched well for scaled distances beyond 3.0 $ft/\sqrt{kips-ft}$.
- Maximum settlements matched well at a scaled distance of 1.0-2.0 $ft/\sqrt{kips-ft}$. Computed values were slightly conservative from 2.0 to 5.0. Maximum settlements became approximately negligible beyond 5.0.
- Even when computed PPVs were below the 0.5 in/s limit, maximum settlements were approximately 1.2 in !

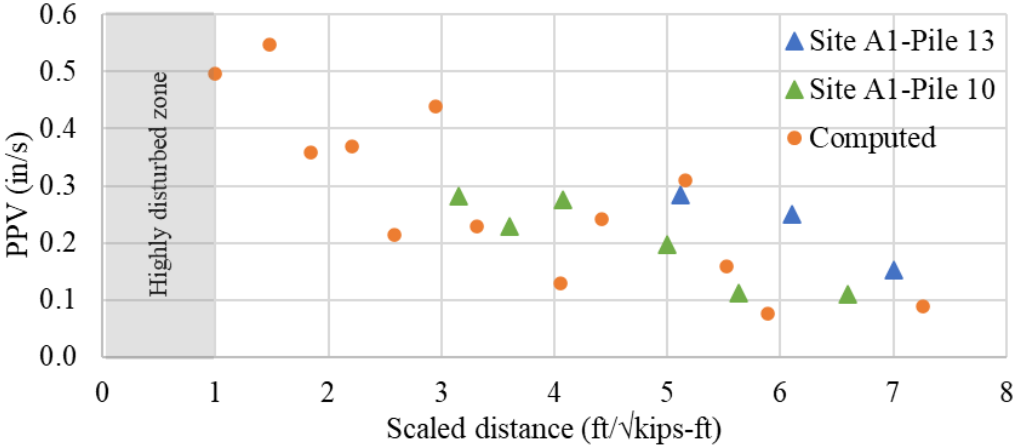


Force time history for single hammer blow at the top of the pile

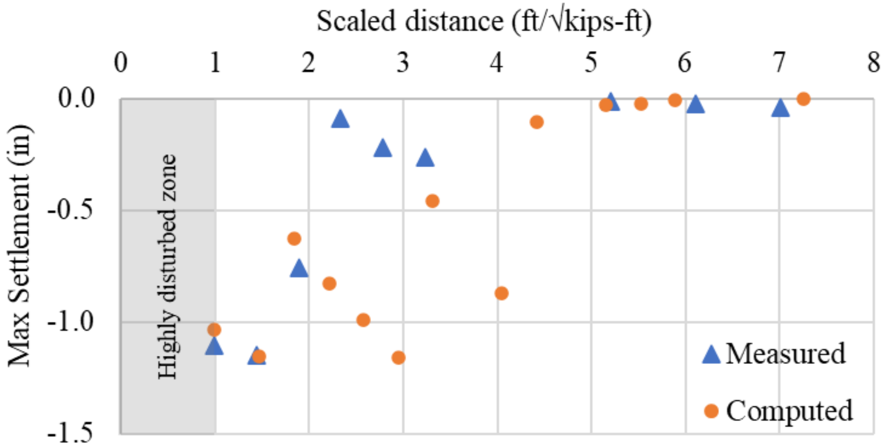


Comparison of measured and computed pile tip depth versus hammer blows

Comparison of measured and computed PPVs



Comparison of measured and computed max. settlements



Summary of numerical analyses performed

MODEL GEOMETRY				ANALYSIS IDENTIFICATION NUMBER						
				Relative Density (%)						
Model	**Pile Length (ft)**	**Pre-Drilling Depth (ft)**	**Hammer Type**	**25**	**40**	**50**	**55**	**60**	**70**	**75**
Baseline	90	30	APE D70-52	1	2	3	4	5	6	7
			ICE 120-S	8	9	10	11	12	13	14
			DELMAG D36-32	15	16	17	18	19	20	21
			DELMAG D62-22	22	23	24	25	26	27	28
			APE D50-52	29	30	31	32	33	34	35
			DELMAG D46-32	36	37	38	39	40	41	42
			ICE 100-S	43	44	45	46	47	48	49
			DELMAG D30-32	50	51	52	53	54	55	56
			ICE 80-S	57	58	59	60	61	62	63
			DELMAG D30-02	64	65	66	67	68	69	70
			ICE I-19	71	72	73	74	75	76	77
M1	90	23	APE D70-52	78	79	80	81	82	83	84
			ICE 120-S	85	86	87	88	89	90	91
			DELMAG D36-32	92	93	94	95	96	97	98
M2	130	40	APE D70-52	99	100	101	102	103	104	105
			ICE 120-S	106	107	108	109	110	111	112
			DELMAG D36-32	113	114	115	116	117	118	119
M3	130	46	APE D70-52	120	121	122	123	124	125	126
			ICE 120-S	127	128	129	130	131	132	133
			DELMAG D36-32	134	135	136	137	138	139	140

- ➢ Effects of:

  i) soil density,
  ii) hammer force time history and rated energy,
  iii) pre-drilling depth.

  on pile driving induced ground response were investigated.

- ➢ Analyses conducted in terms of:
  i) vertical pile penetration,
  ii) ground vibrations (PPV),
  iii) maximum ground deformations.

- ➢ 140 numerical simulations.

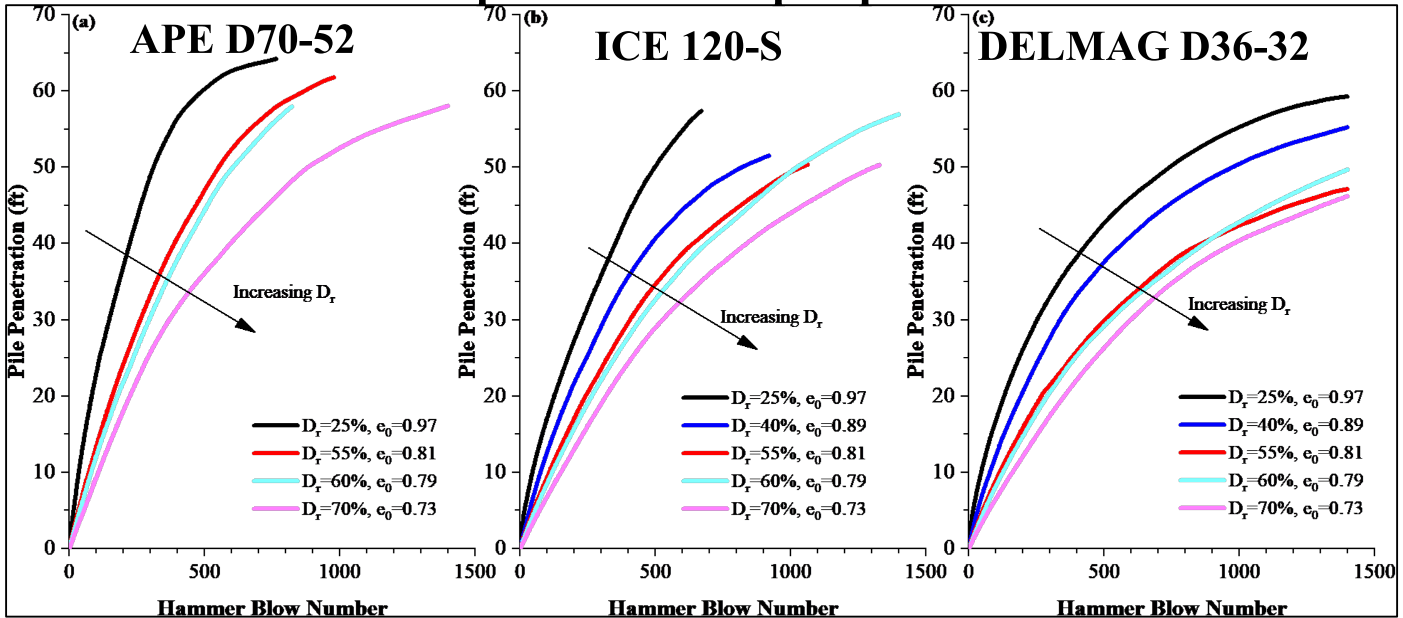
- ➢ 3,500 hours of computational effort.

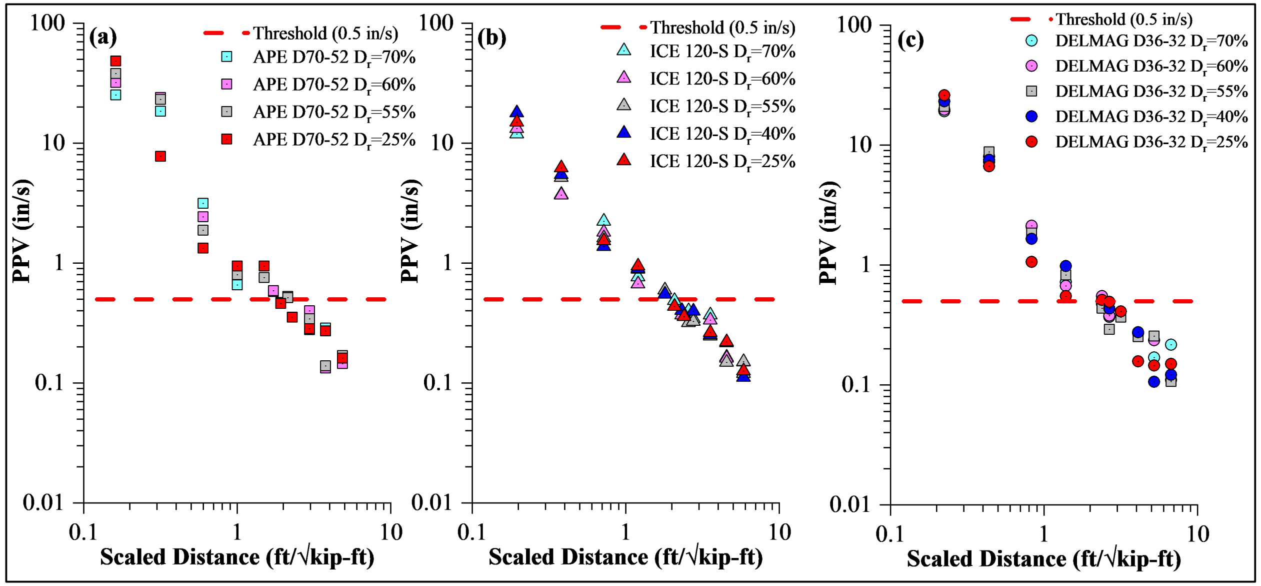
Effect of soil relative density

Lessons learned:

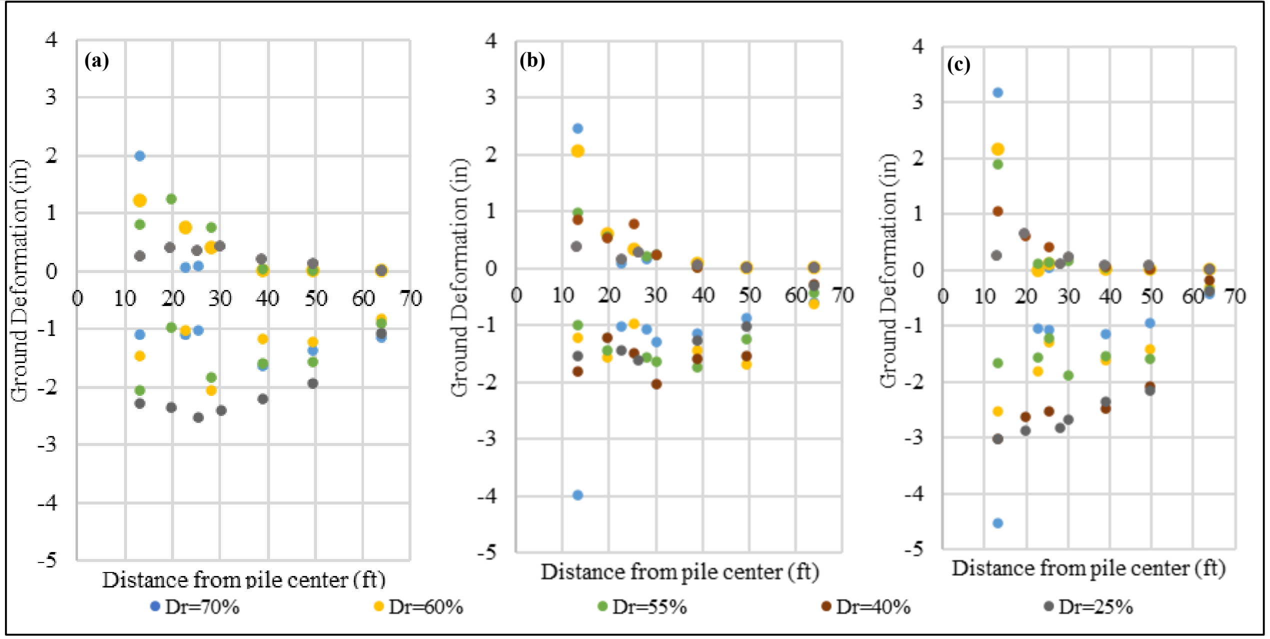
- ➢ The driving "effort" increases as D_r increases
- ➢ The looser the material, the higher the computed PPV, particularly in the highly disturbed zone
- ➢ The scatter in the results reduces away from the pile.
- ➢ At approx. 2 $ft/\sqrt{kip-ft}$, FDOT PPV value of 0.5 in/s was achieved, regardless of D_r.
- ➢ In general, the lower the D_r, the larger the computed settlements. The higher the D_r the larger the computed heave.

Computed vertical pile penetration





Computed PPV attenuation



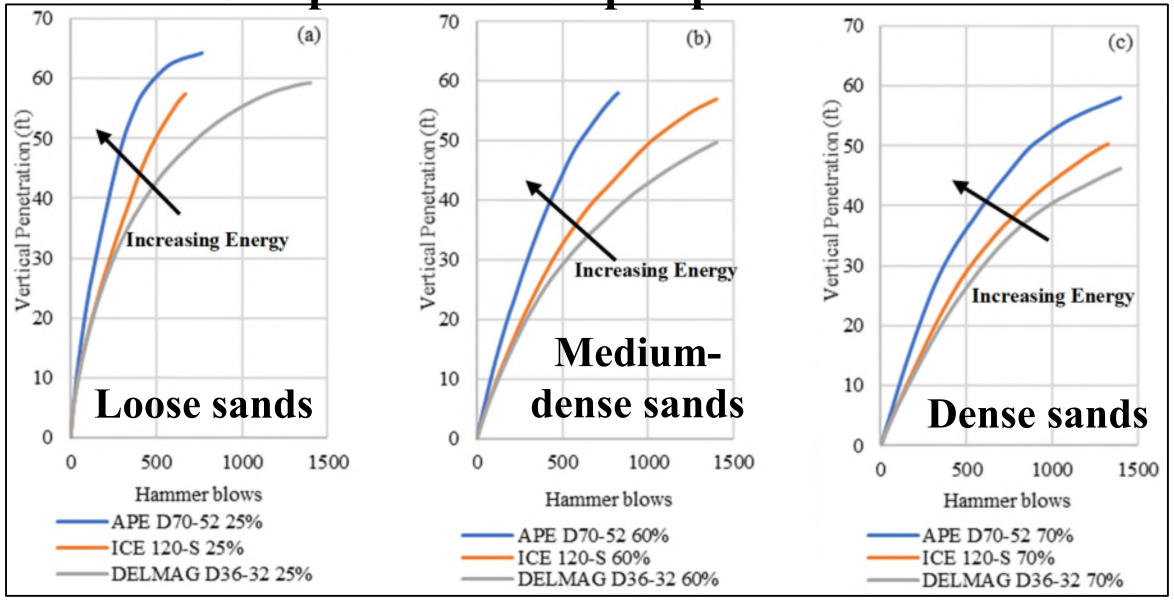
Computed max. ground deformations

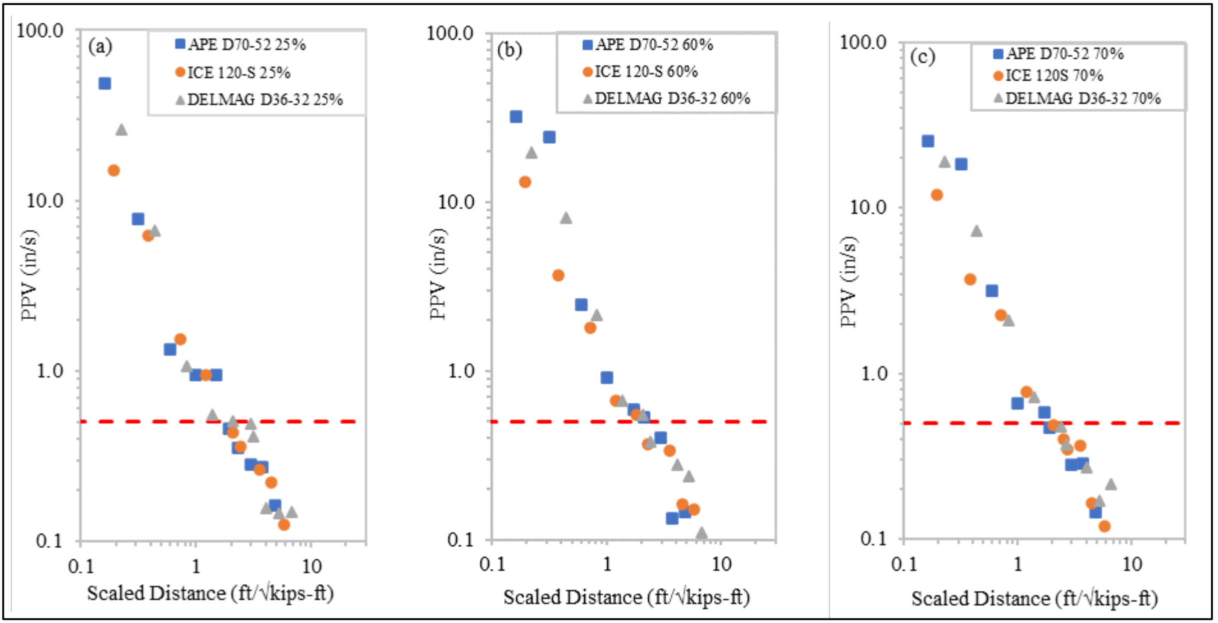
Effect of input energy

Lessons learned:

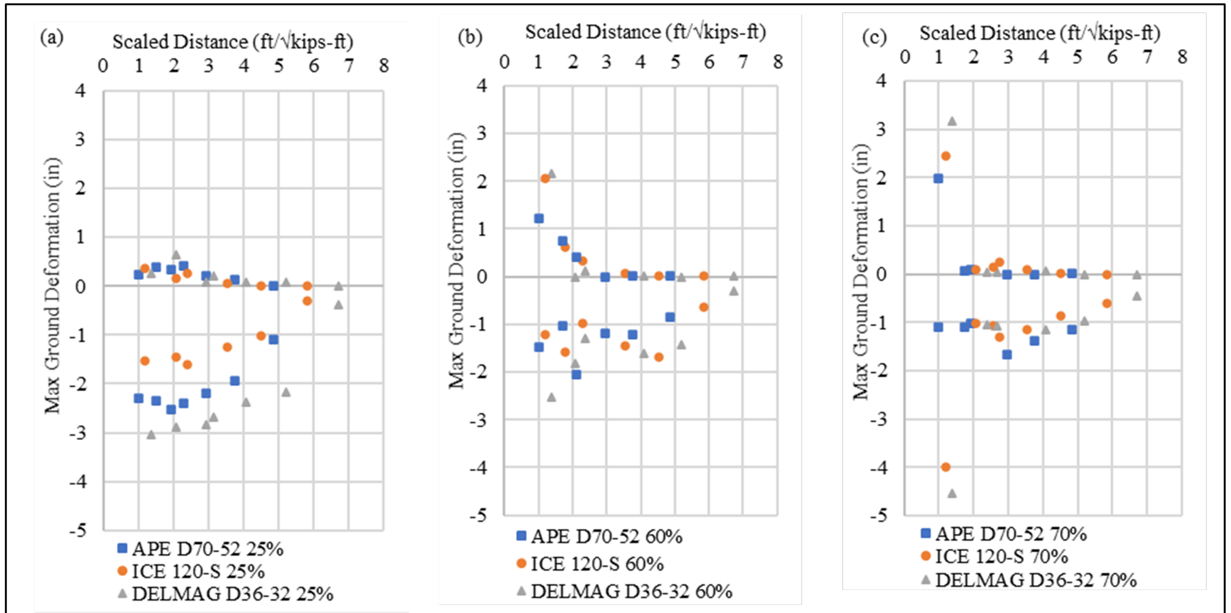
- The "effort" required to install each pile is highly dependent on the input energy. Rated energies used in the analyses:
  APE D70-52 > ICE 120-S > DELMAG D36-32.
- PPV values are the highest with APE D70-52 at $0.3 ft/\sqrt{kip-ft}$. Scatter in attenuation curves reduces beyond that point.
- In general, the largest input energy the largest settlements in dense sands.

Computed vertical pile penetration





Computed PPV attenuation



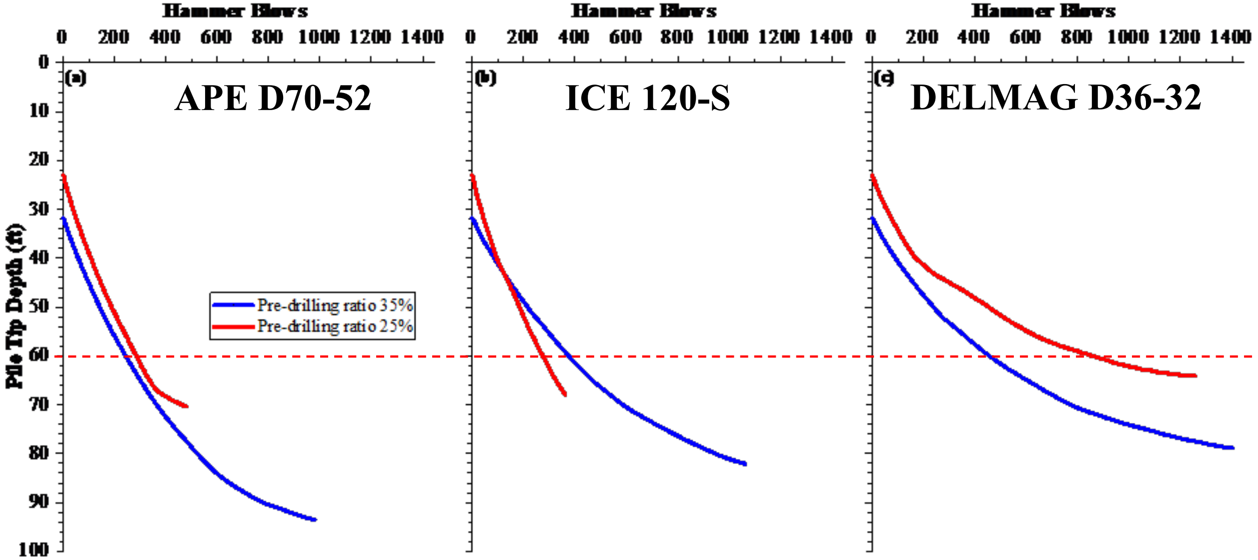
Computed max. ground deformations

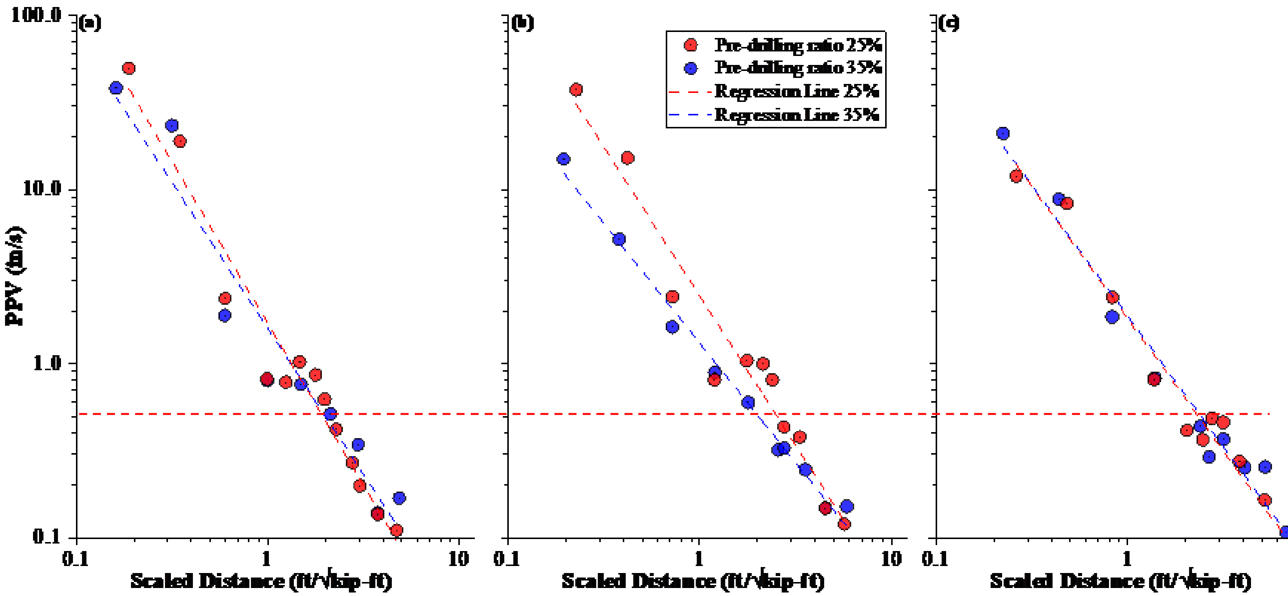
Effect of pre-drilling depth

Lessons learned:

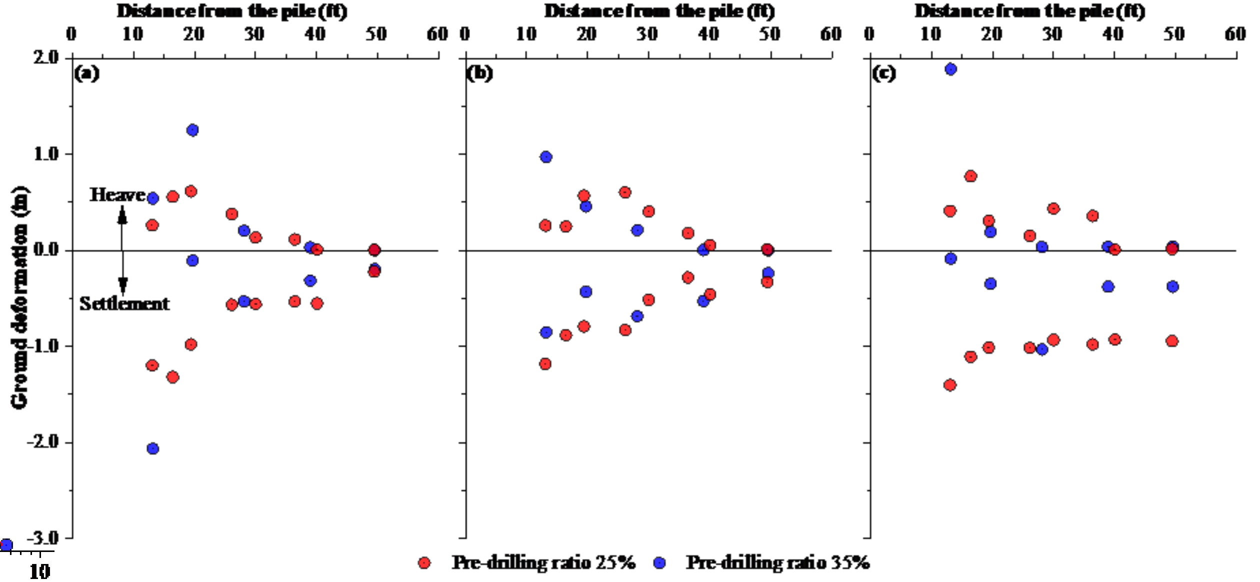
- Selected pre-drilling depths: 23ft and 30ft (predrilling-to-pile length ratios of 25% and 35%).
- PPV increases as pre-drilling depth decreases. Effect is not that noticeable for scaled distances corresponding to PPV values below 0.5 in/s.
- Ground deformations (either settlement or heave) are affected by the amount of pre-drilling.

Computed vertical pile penetration





Computed PPV attenuation

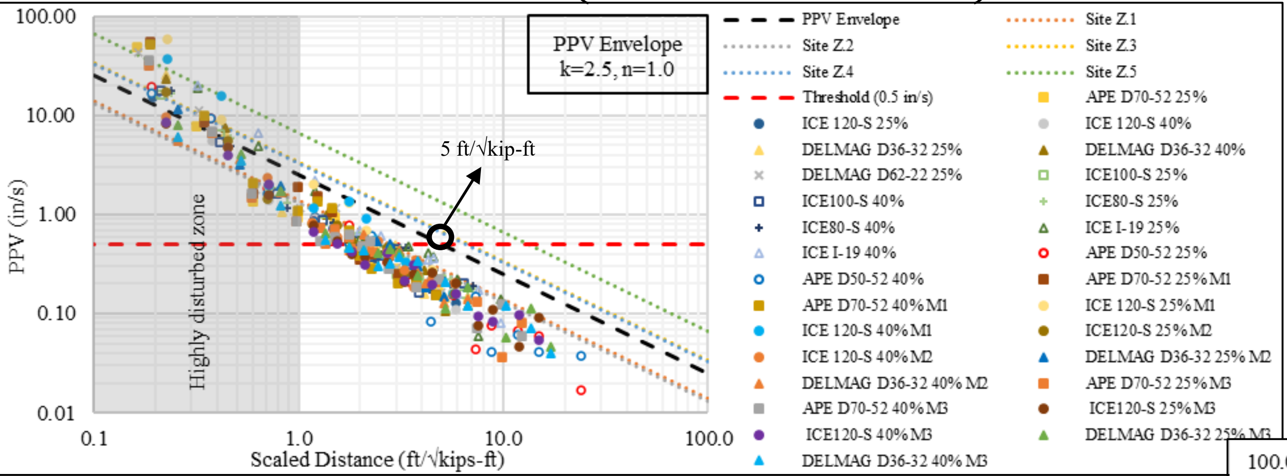


Computed max. ground deformations

Task 5: Proposed Charts, Equations, Correlations

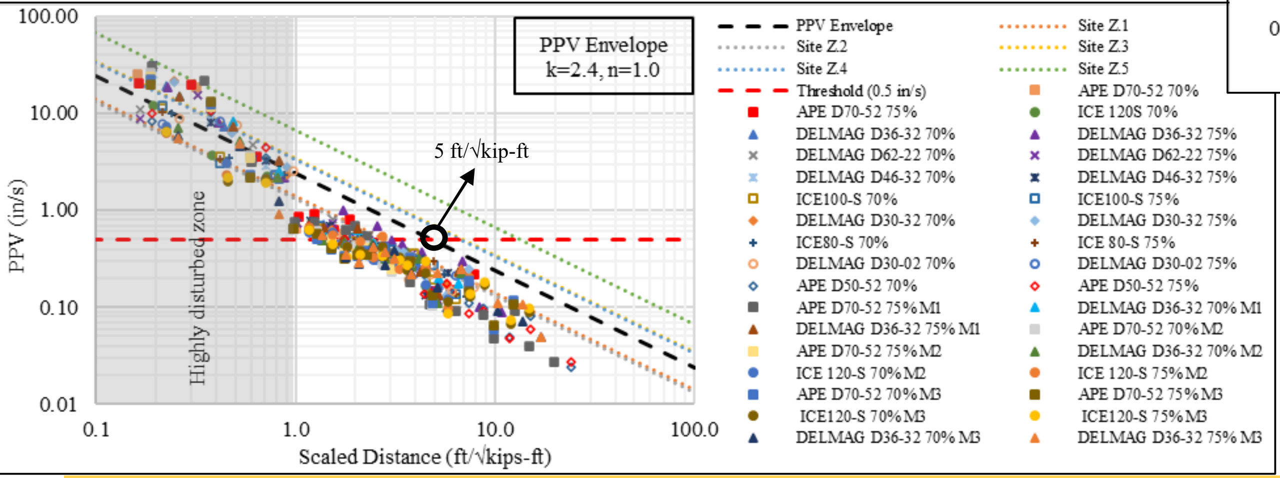
Proposed charts: PPV attenuation curves

For loose sands (25% < Dr ≤ 40%):



- ➢ The scale distance to satisfy the FDOT PPV requirement is approx. 5 $ft/\sqrt{kips-ft}$ (i.e., smaller than recommended by FDOT standard specs. (2021) of 16.0 $ft/\sqrt{kip-ft}$).

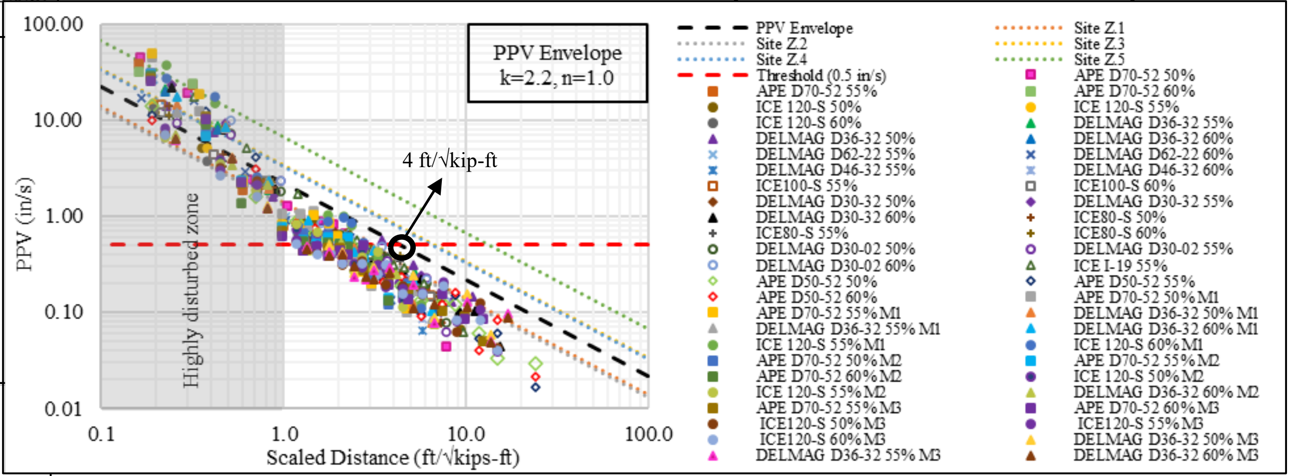
For dense sands (60% < Dr ≤ 75%):



Comments on PPV envelopes versus scaled distance:

- ➢ Developed for precast concrete piles,
- ➢ FDOT limit shown: Horizontal red dashed line (0.5 in/s),
- ➢ Results compared vs. other case histories from past FDOT-sponsored research projects. Results matched well PPV limits by Bayraktar et al. (2013).

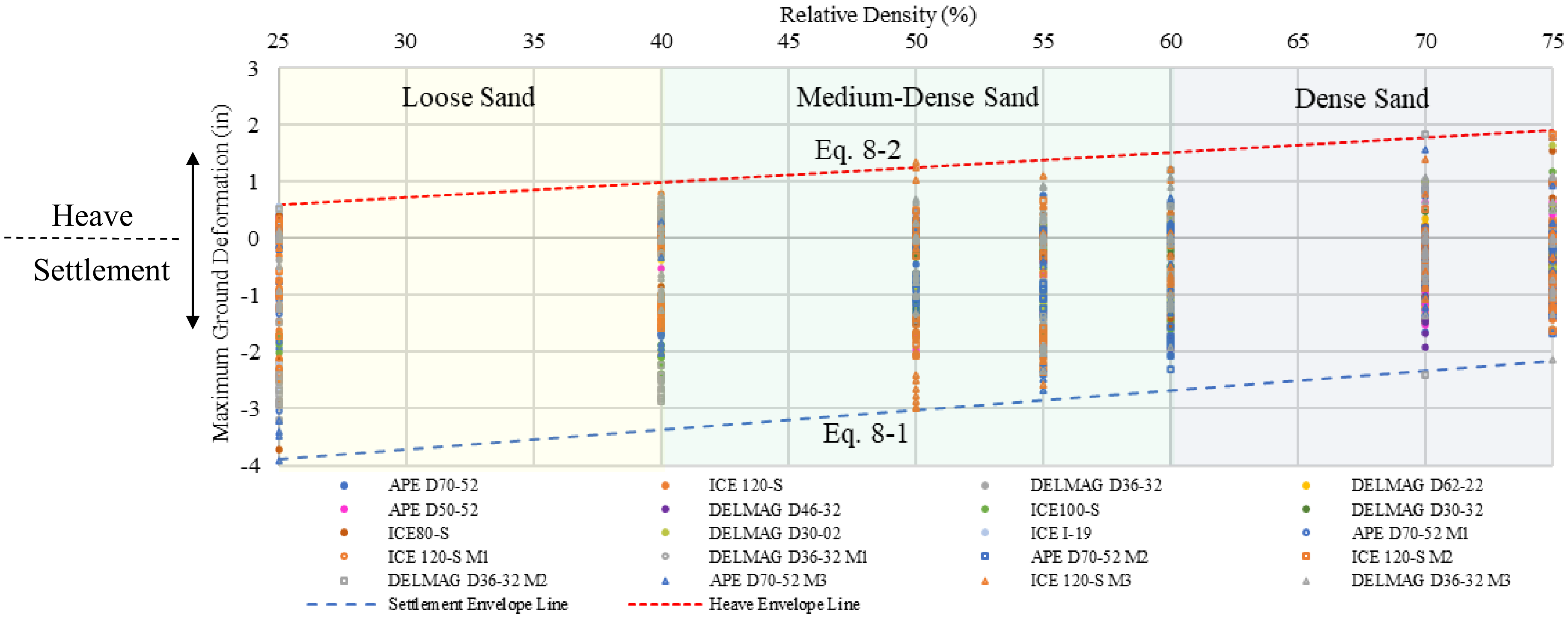
For medium-dense sands (40% < Dr ≤ 60%):



- ➢ Rated energy is used for calculation of scaled distance.

Proposed charts and equations: Max. ground deformations

Max. computed ground deformations for various Dr (%) and rated energies.



Settlement: $$S\ (in) = \frac{3.5}{100}\ D_r(\%) - 4.8 \qquad \textbf{(8-1)}$$

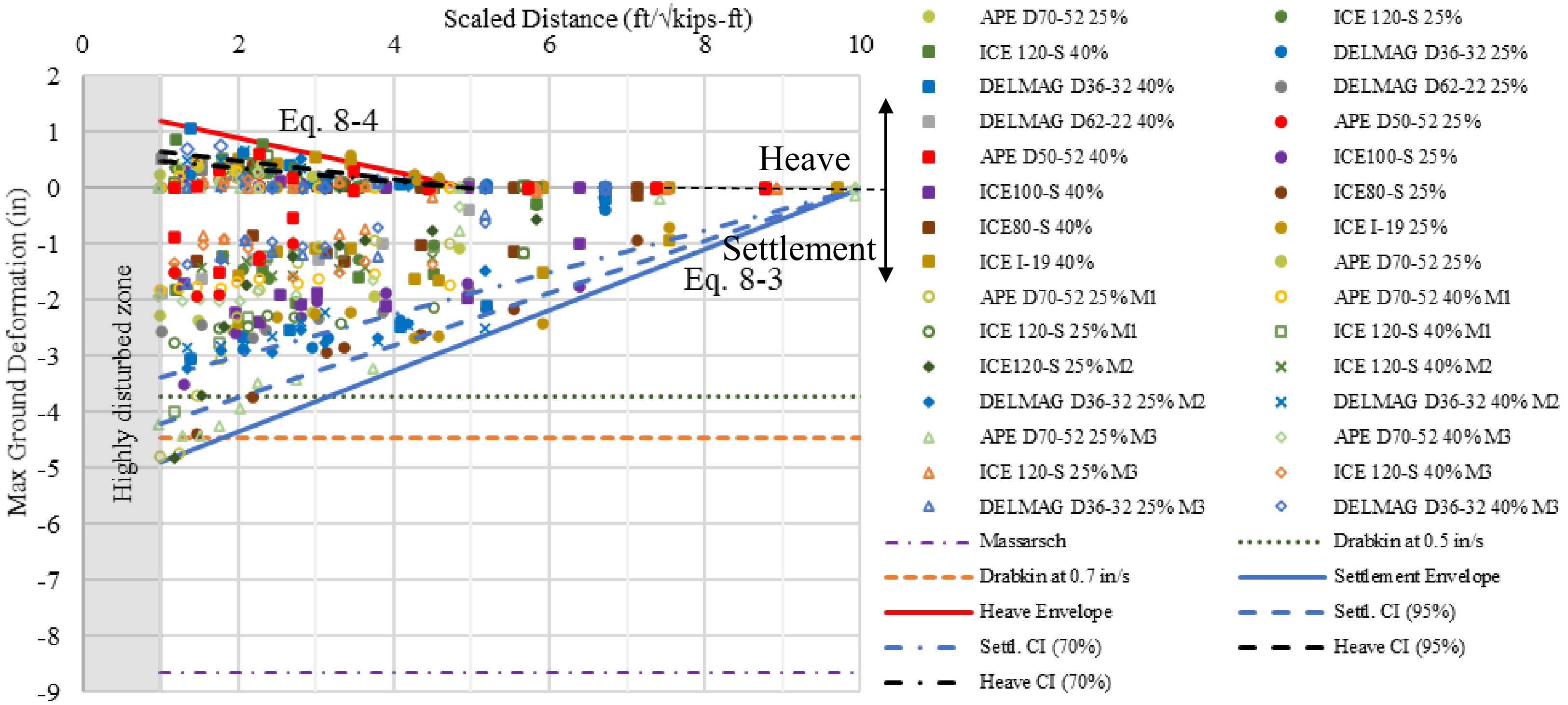
Heave: $$H\ (in) = \frac{2.6}{100}\ D_r(\%) \qquad \textbf{(8-2)}$$

Comments:

- Ground deformations were obtained from multiple time histories along the soil continuum at the ground surface for all the considered input energies.
- Ground deformations correspond to those computed after the condition of PPV= 0.5 in/s stipulated by FDOT was satisfied.
- Total of 884 data points.
- Larger settlement was computed for loose sands, while larger heave occurred on dense sands.

Proposed charts and equations: Max. ground deformations vs. scaled distance

Ground deformation "attenuation" curves for <u>loose sands</u> ($25\% < Dr \leq 40\%$)



Comments:

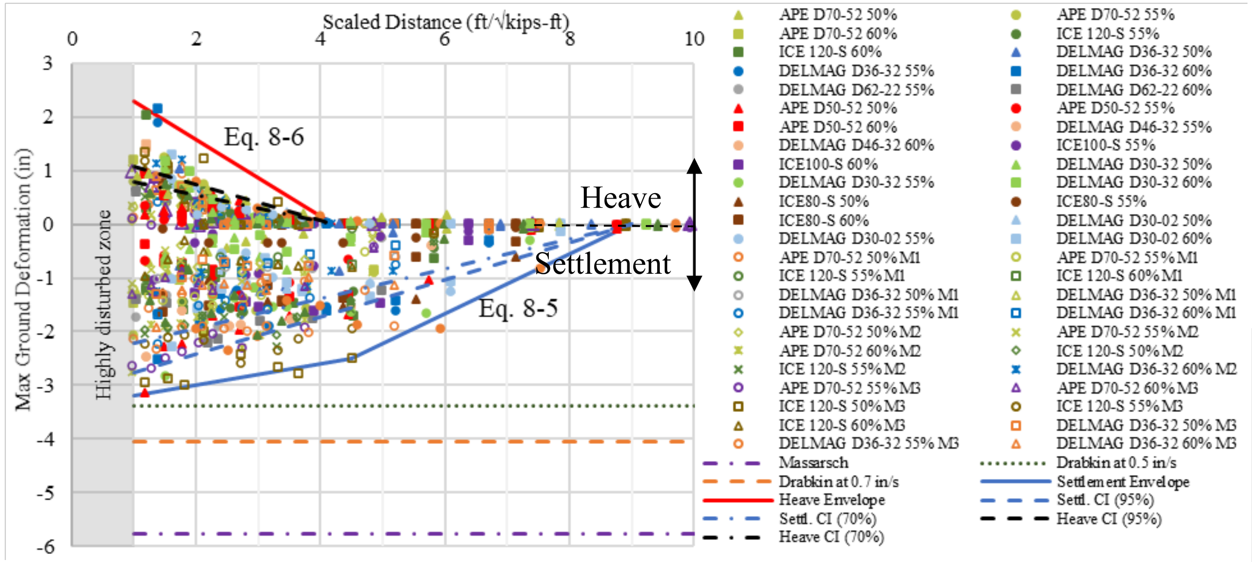
- ➢ Data points in the figure do not necessarily comply with the FDOT limit of 0.5 in/s.
- ➢ Data shown only beyond the highly disturbed zone surrounding the pile (i.e., scaled distance beyond $1.0\ ft/\sqrt{kips-ft}$).
- ➢ Calculated settlements with other methods: Massarsch (2004) and Drabkin et al. (1996) are also shown.
- ➢ Proposed envelopes matched well those by Drabkin et al. (1996). Massarsch's method is independent of the scaled distance and was overly conservative.

Settlement: $S\ (in) = 0.54\ D/\sqrt{E}\ - 5.44 \qquad for\ 1.00 < D/\sqrt{E} \leq 10.00$ **(8-3)**

Heave: $H\ (in) = -0.30\ D/\sqrt{E}\ + 1.50 \qquad \text{for } 1.00 < D/\sqrt{E} \leq 5.00$ **(8-4)**

$D/\sqrt{E}$ in (ft/√kips−ft)

Proposed charts and equations: Max. ground deformations vs. scaled distance



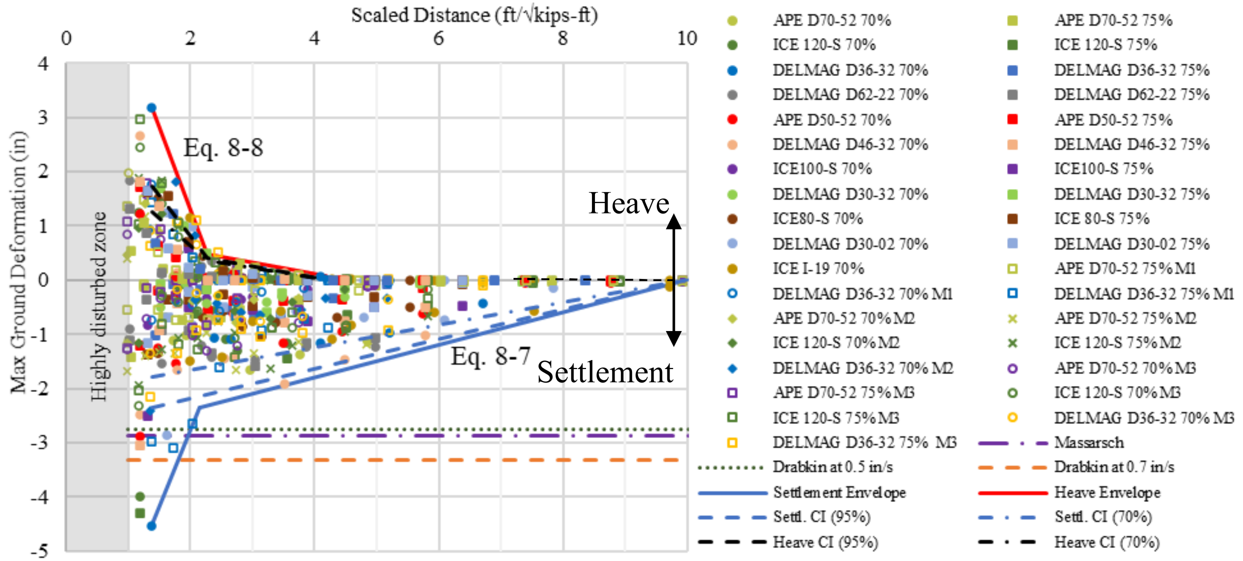
Medium-dense sands (40% < Dr ≤ 60%)

Settlement: $$S\ (in) = \begin{cases} 0.20\ D/\sqrt{E}\ - 3.40 & for\ 1.00 < D/\sqrt{E} \le 4.50 \\ 0.56\ D/\sqrt{E} - 5.00 & for\ 4.50 < D/\sqrt{E} \le 9.00 \end{cases} \quad \textbf{(8-5)}$$

Heave: $$H\ (in) = -0.72\ D/\sqrt{E} + 3.01 \quad \text{for } 1.00 < D/\sqrt{E} \le 4.20 \quad \textbf{(8-6)}$$

Additional Comments:

- Maximum settlement envelopes in medium to dense sands matched well Drabkin et al. (1996). Massarsch (2004) overpredicted settlements in medium-dense sands but matched well in dense sands.
- Heave was observed and computed in medium to dense sands. Volumetric expansion of the soil due to soil dilation shearing mechanisms at shaft and pile tip were captured.



Dense sands (60% < Dr ≤ 75%)

Settlement: $$S\ (in) = \begin{cases} 2.80 D/\sqrt{E}\ - 8.37 & for\ 1.00 < D/\sqrt{E} \le 2.15 \\ 0.30 D/\sqrt{E}\ - 2.99 & for\ 2.15 < D/\sqrt{E} \le 10.00 \end{cases} \quad \textbf{(8-7)}$$

Heave: $$H\ (in) = \begin{cases} -3.01 D/\sqrt{E}\ + 7.33 & \text{for } 1.00 < D/\sqrt{E} \le 2.30 \\ -0.22 D/\sqrt{E}\ + 0.99 & \text{for } 2.30 < D/\sqrt{E} \le 4.20 \end{cases} \quad \textbf{(8-8)}$$

$D/\sqrt{E}$ in (ft/√kips−ft)

Conclusions, Recommendations, and Implementation

Conclusions and recommendations

1. Accurate definition of hammer, driving accessories, and pile properties is important to predict ground deformations caused by pile driving.

2. Field data: most of ground deformations occurred during the initial stages of pile driving. Spherical waves emanating from the pile tip had less influence at the ground surface as the pile penetration increased.

3. Geo-structures provide a protection barrier against pile driving-induced vibration and ground deformations.

4. When multiple piles were driven in a pier, installation of the first pile caused soil densification in loose to medium-dense sands due to their contractive response.

5. From the parametric study: i) the larger the rated hammer energies the larger the ground deformations, ii) settlement was observed in loose sands and heave was observed in dense sands, and iii) pre-drilling reduces ground deformations and vibrations.

6. Negligible ground deformations were computed beyond a scaled distance of 10.0 ft/√(kip-ft) regardless of the soil or input energy.

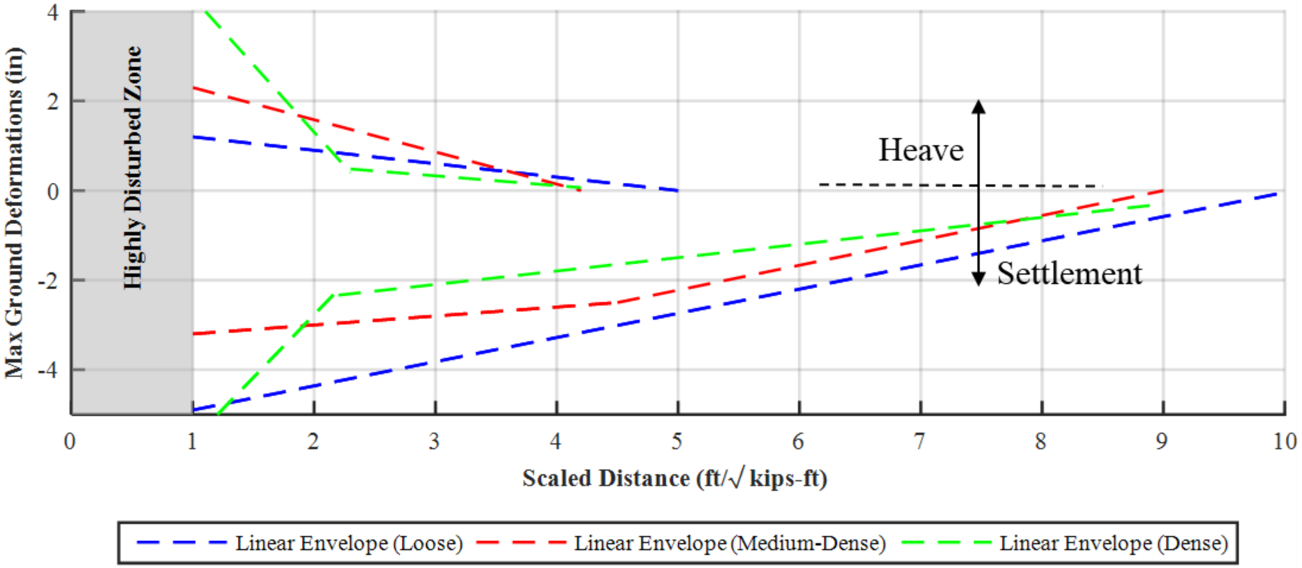
7. Even if the PPV limit of 0.5 in/s is not exceeded, large ground deformations can still occur due to soil densification in loose sands (i.e., settlement) or soil volumetric expansion in dense sands (i.e., heave). Proposed eqns.:

Settlement: $S\ (in) = \frac{3.5}{100} D_r(\%) - 4.8$ **(8-1)**

Heave: $H\ (in) = \frac{2.6}{100} D_r(\%)$ **(8-2)**

Conclusions and recommendations

8. Equations were proposed to provide maximum expected ground deformations (i.e., envelopes) due to impact pile driving. Those are not intended to be used as settlement troughs (i.e., shapes of settlement profiles for computation of differential settlements or angular distortions).



Loose sands ($25\% < \mathrm{Dr} \leq 40\%$)

Settlement: $S\ (in) = 0.54\ D/\sqrt{E}\ - 5.44 \quad for\ 1.00 < D/\sqrt{E} \leq 10.00$

Heave: $H\ (in) = -0.30\ D/\sqrt{E} + 1.50 \quad \text{for } 1.00 < D/\sqrt{E} \leq 5.00$

Medium-dense sands ($40\% < \mathrm{Dr} \leq 60\%$)

Settlement:
$$S\ (in) = \begin{cases} 0.20\ D/\sqrt{E}\ - 3.40 & for\ 1.00 < D/\sqrt{E} \leq 4.50 \\ 0.56\ D/\sqrt{E} - 5.00 & for\ 4.50 < D/\sqrt{E} \leq 9.00 \end{cases}$$

Heave: $H\ (in) = -0.72\ D/\sqrt{E} + 3.01 \quad \text{for } 1.00 < D/\sqrt{E} \leq 4.20$

Dense sands ($60\% < \mathrm{Dr} \leq 75\%$)

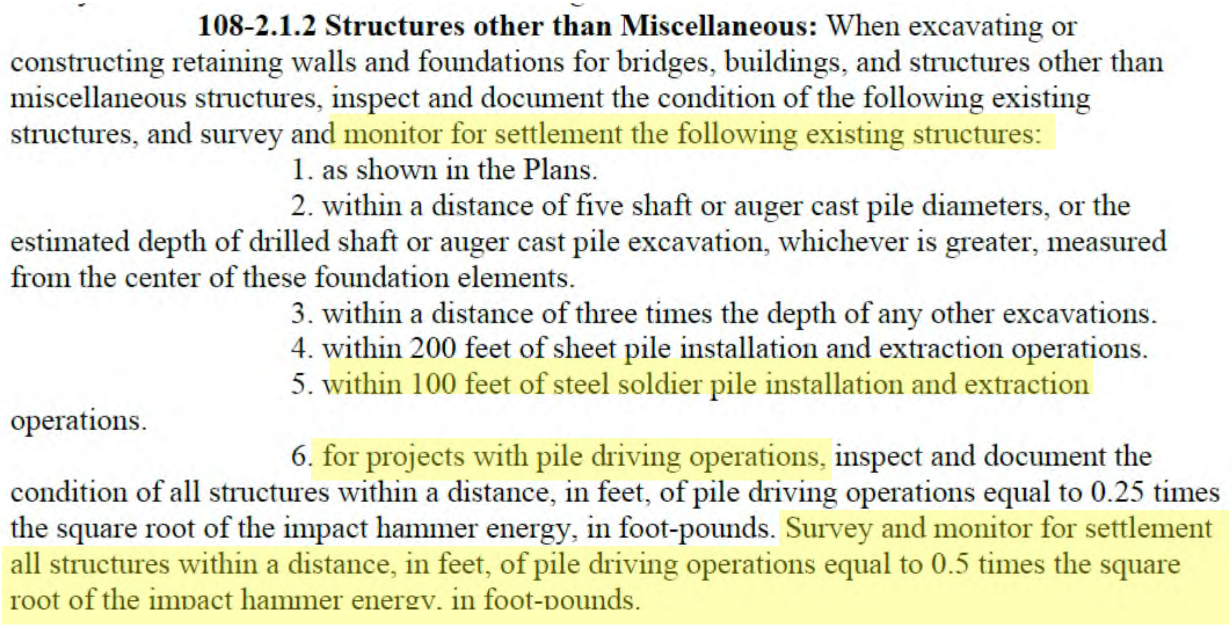
Settlement:
$$S\ (in) = \begin{cases} 2.80D/\sqrt{E}\ - 8.37 & for\ 1.00 < D/\sqrt{E} \leq 2.15 \\ 0.30D/\sqrt{E}\ - 2.99 & for\ 2.15 < D/\sqrt{E} \leq 10.00 \end{cases}$$

Heave:
$$H\ (in) = \begin{cases} -3.01D/\sqrt{E}\ + 7.33 & \text{for } 1.00 < D/\sqrt{E} \leq 2.27 \\ -0.22D/\sqrt{E}\ + 0.99 & \text{for } 2.27 < D/\sqrt{E} \leq 4.20 \end{cases}$$

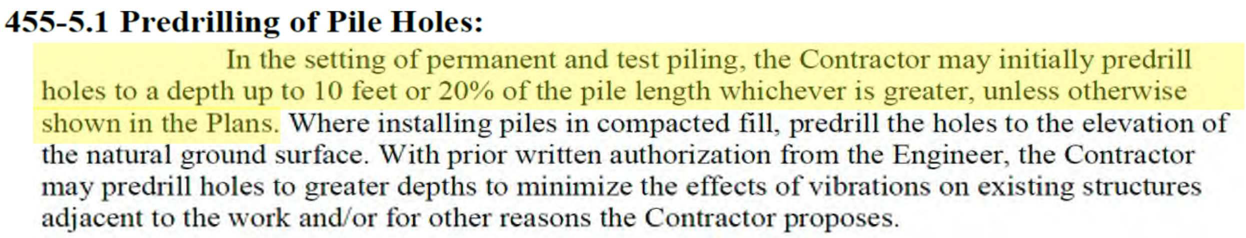
$D/\sqrt{E}$ in (ft/√kips−ft)

Implementation and suggested revisions

Comments on FDOT Standard Specifications for Road and Bridge Construction


108-2.1.2 Structures other than Miscellaneous: When excavating or constructing retaining walls and foundations for bridges, buildings, and structures other than miscellaneous structures, inspect and document the condition of the following existing structures, and survey and monitor for settlement the following existing structures:

1. as shown in the Plans.
2. within a distance of five shaft or auger cast pile diameters, or the estimated depth of drilled shaft or auger cast pile excavation, whichever is greater, measured from the center of these foundation elements.
3. within a distance of three times the depth of any other excavations.
4. within 200 feet of sheet pile installation and extraction operations.
5. within 100 feet of steel soldier pile installation and extraction operations.
6. for projects with pile driving operations, inspect and document the condition of all structures within a distance, in feet, of pile driving operations equal to 0.25 times the square root of the impact hammer energy, in foot-pounds. Survey and monitor for settlement all structures within a distance, in feet, of pile driving operations equal to 0.5 times the square root of the impact hammer energy, in foot-pounds.


455-5.1 Predrilling of Pile Holes:

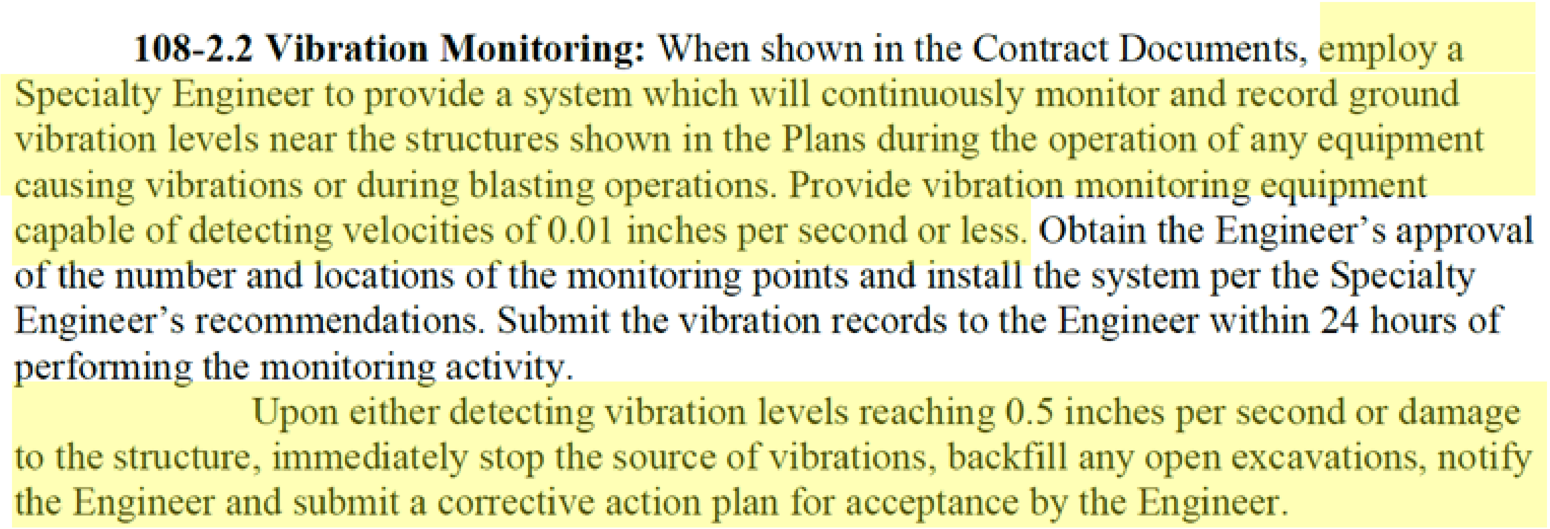
In the setting of permanent and test piling, the Contractor may initially predrill holes to a depth up to 10 feet or 20% of the pile length whichever is greater, unless otherwise shown in the Plans. Where installing piles in compacted fill, predrill the holes to the elevation of the natural ground surface. With prior written authorization from the Engineer, the Contractor may predrill holes to greater depths to minimize the effects of vibrations on existing structures adjacent to the work and/or for other reasons the Contractor proposes.

- In the standard specification: 0.5 $ft/\sqrt{lb-ft}$ is equal to 16.0 $ft/\sqrt{kip-ft}$).
- Our research found negligible ground deformations beyond a scaled distance value of 10.0 $ft/\sqrt{kip-ft}$.
- Our research found based on PPV attenuation curves that at scaled distances larger than approx. 5.0 $ft/\sqrt{kips-ft}$, the PPV limit of 0.5 in/s is not exceeded.

- Our research found that pre-drilling operations reduce ground deformations and vibrations due to an increase in the radial distance between the ground surface and pile tip.
- Our research highlights the importance of pre-drilling to limit ground deformations due to pile driving.

Implementation and suggested revisions

Comments on FDOT Standard Specifications for Road and Bridge Construction



108-2.2 Vibration Monitoring: When shown in the Contract Documents, employ a Specialty Engineer to provide a system which will continuously monitor and record ground vibration levels near the structures shown in the Plans during the operation of any equipment causing vibrations or during blasting operations. Provide vibration monitoring equipment capable of detecting velocities of 0.01 inches per second or less. Obtain the Engineer's approval of the number and locations of the monitoring points and install the system per the Specialty Engineer's recommendations. Submit the vibration records to the Engineer within 24 hours of performing the monitoring activity.

Upon either detecting vibration levels reaching 0.5 inches per second or damage to the structure, immediately stop the source of vibrations, backfill any open excavations, notify the Engineer and submit a corrective action plan for acceptance by the Engineer.

- Our project found that even if the PPV limit of 0.5 in/s is not exceeded, large ground deformations can still occur in sands in the form of either settlement or heave mainly depending on the soil relative density.
- The vibration monitoring in the specifications does not address ground deformations in sandy soils induced by pile driving. Only "vibration levels" need to be monitored and recorded.
- Consider proposed equations and charts to predict and evaluate maximum ground deformations due to pile driving operations.

Quantitative and Qualitative Benefits

Quantitative:

- We produced pile driving induced deformation charts, correlations, and equations relating PPV, D_r , distance from source, and input energy to be used in FDOT projects.
- We measured ground deformation time histories and ground vibration measurements from 11 bridge construction sites in Florida.
- We identified the effects of different variables such as pre-drilling, hammer rated energy, and D_r on the pile driving induced ground deformations.
- We provided a large database with 76 case histories reported around the world.
- We developed and calibrated a robust numerical model of dynamic ground deformations due to impact pile driving.
- We compared two pile driving numerical modeling approaches and provided recommendations on their accuracy.

Qualitative:

- This research provided a better understanding of the geotechnical mechanisms that cause pile driving induced ground deformations which can help preventing future infrastructure damage as a result of excessive ground deformations and vibrations that are generated in deep foundation installations using impact pile driving methods.

Acknowledgments

To the project manager for his guidance and support: Larry Jones (FDOT)

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- Tharwat Hannadawod (District 5)
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- Chris Briggs and Jose Medina at Jacobs (Site A)
- Arnaldo Larrazabal at RS&H (Site B and C)
- Roger Gobin at WSP (Site D)

To FDOT in general! For the research opportunity and support providing education and research training for 4 MS/Ph.D. at UCF!

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