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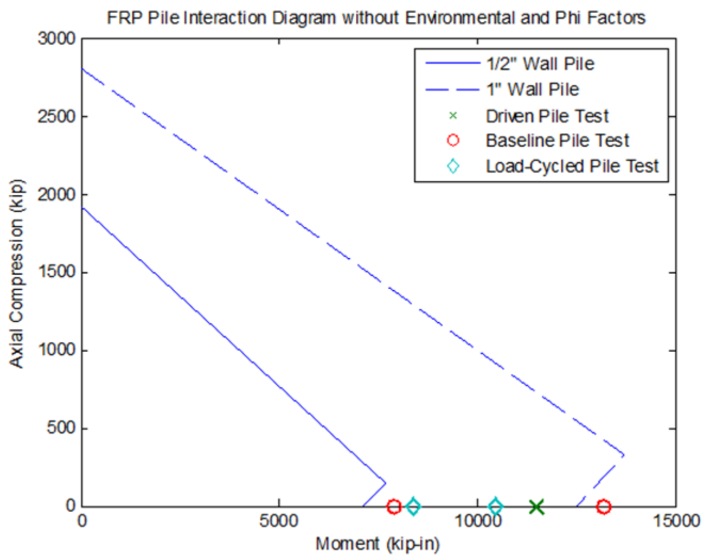

Technical Report 17-1

Experimental Evaluation and Design of Unfilled and Concrete-Filled FRP Composite Piles

Task 4A – Design Specifications

Final Report – Task 4A, January 2017

Technical Report Documentation Page

1. Report No. ME 17-1	2.	3. Recipient's Accession No.
4. Title and Subtitle Experimental Evaluation and Design of Unfilled and Concrete-Filled FRP Composite Piles Task 4A – Design Specifications		5. Report Date August 2015 6.
7. Author(s) Dale Lawrence Roberto Lopez-Anido Thomas Sandford		8. Performing Organization Report No. AEWC Report Number 1199.4
9. Performing Organization Name and Address University of Maine – Advanced Structures and Composites Center		10. Project/Task/Work Unit No. 11. Contract © or Grant (G) No. Contract # 20130731*535
12. Sponsoring Organization Name and Address Maine Department of Transportation		13. Type of Report and Period Covered 14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract (Limit 200 words)

The overall goal of this project is the experimental evaluation and design of unfilled and concrete-filled FRP composite piles for load-bearing in bridges. This report covers Task 4A, Design Specifications.

Structural design specifications are based on the AASHTO LRFD Bridge Design Specifications Article 5.13.4 – Concrete Piles (Reference 1). This technical report has been prepared to reflect the results of field and laboratory testing presented by Lawrence (Reference 2).

Geotechnical design specifications are proposed modifications to the AASHTO LRFD Bridge Design Specifications Articles 10.5 - Limit States and Resistance Factors and 10.7 – Driven Piles (Reference 1).

17. Document Analysis/Descriptors Bridge piles, fiber reinforced polymer composites, design specifications		18. Availability Statement	
19. Security Class (this report)	20. Security Class (this page)	21. No. of Pages 27	22. Price

Technical Report

Draft LRFD Design Specifications for Fiber Reinforced Polymer (FRP) Concrete Piles

Submitted by:

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Project Task 1199.4

Project: Experimental Evaluation and Design of Unfilled and Concrete-Filled FRP Composite Piles

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Summary

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1. AASHTO. LRFD Bridge Design Specifications, Seventh Edition, U.S. Customary Units, 2014.

2. Lawrence, D. "Experimental Evaluation of Fiber Reinforced Polymer Piles in Load Bearing Applications, M.S. Thesis, University of Maine, 2015.

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Section 1 Structural Design Specifications

1.1 General

All loads resisted by the footing and the weight of the footing itself shall be assumed to be transmitted to the piles. Piles installed by driving shall be designed to resist driving and handling forces. For transportation and erection, a precast pile should be designed for not less than 1.5 times its self-weight.

Any portion of a pile where lateral support adequate to prevent buckling may not exist at all times shall be designed as a column.

The points or zones of fixity for resistance to lateral loads and moments shall be determined by an analysis of the soil properties, as specified in Article 10.7.3.13.4. FRP piles shall be embedded into footings or pile caps, as specified in Article 10.7.1.1. Anchorage reinforcement shall consist of either an extension of the pile reinforcement or the use of dowels. Uplift forces or stresses induced by flexure shall be resisted by the reinforcement. The steel ratio for anchorage reinforcement shall not be less than 0.005, and the number of bars shall not be less than four. The reinforcement shall be developed sufficiently to resist a force of $1.25F_yA_s$.

Changed “concrete piles” to “FRP piles.”

In addition to the requirements specified in Sections 1.1 through 1.5, piles used in the seismic zones shall conform to the requirements specified in Article 5.13.4.6.

1.2 Splices

Splicing of FRP piles shall not be allowed, unless approved by the Engineer of Record.

1.3 Precast FRP Piles

1.4 Pile Dimensions

Precast FRP piles shall be of uniform section.

Removed tapered piles.

1.4.1 Reinforcing

Reinforcing shall be designed to withstand all tensile driving forces. Any potential longitudinal reinforcement provided by the FRP shell shall be neglected in tension during driving.

Driving concrete filled FRP piles without additional longitudinal reinforcement has been shown to cause cracking in the concrete. This led to a loss of composite action during flexure.

1.5 Cast-in-Place FRP Piles

Piles cast in FRP shells lowered into pre-drilled holes may be used only where soil conditions permit.

Shells for cast-in-place piles shall be of sufficient thickness and strength to hold their form and to show no harmful distortion during driving or after adjacent shells have been driven. The contract documents shall stipulate that alternative designs of the shell need to be approved by the Engineer before any driving is done.

Cast-in-place FRP piles include piles cast in driven FRP shells that remain in place or FRP shells placed in drilled holes.

1.5.1 Pile Dimensions

Cast-in-place FRP piles shall have a uniform section.

1.5.2 Reinforcing

Cast-in-place FRP piles may be constructed without reinforcing bars, provided that they do not receive any hammer blows from pile driving, including restrikes or additional driving to mitigate heaving. Piles subjected to tensile forces may also require reinforcing bars to prevent cracking in the concrete.

For cast-in-place concrete piling, clear distance between parallel longitudinal, and parallel transverse reinforcing bars shall not be less than five times the maximum aggregate size or 5.0 in., except as noted in Article 5.13.4.6 for seismic requirements.

1.6 Seismic Requirements

See Article 5.13.4.6 for seismic requirements.

Seismic requirements for concrete piles shall apply to FRP piles.

1.7 Flexure

The moment capacity of a concrete filled FRP tube shall be calculated according to the AASHTO LRFD Guide Specifications for Design of Concrete-Filled FRP Tubes for Flexural and Axial Members Article 2.9 (Reference 3).

A material bias factor shall be applied to the ultimate strain and strength of the FRP laminate. The bias factor shall be applied concurrently with the environmental reduction factor, C_E, as in Article 2.6.1.2 (Reference 3). This factor is used to account for the decrease in material properties seen between coupon level tests and full scale pile tests. The material bias factors shall be:

Material bias factors are taken as the ratio of strain at failure for full-scale pile tests to the strain at failure during coupon level tests conducted by Lawrence (Reference 2). Hollow piles tested in flexure failed in compression between 19.4% and 25.7% of the coupon strain. Concrete-filled piles tested in flexure failed in tension between 38.2% and 66.2% of the coupon strain.

- For compressive properties

$\gamma_c = 0.2$

- For tensile properties

$\gamma_t = 0.4$

The material bias factors may be increased if the behavior of the FRP piles is verified using full scale testing.

Example pile calculations are presented in Appendix A.

1.8 Axial Compression

The axial compressive resistance of the FRP pile shall be calculated according to the AASHTO LRFD Guide Specifications for Design of Concrete-Filled FRP Tubes for Flexural and Axial Members (Reference 3) Article 2.10.

The material bias factors, γ_t and γ_c, found during flexural testing shall also be applied to material properties for the axial compressive resistance.

Example pile calculations are presented in Appendix A.

1.9 Shear

The shear resistance of the FRP pile shall be calculated according to the AASHTO LRFD Guide Specifications for Design of Concrete-Filled FRP Tubes for Flexural and Axial Members (Reference 3) Article 2.12.

1.10 Combined Axial Compression and Flexure

Piles designed for combined axial compression and flexure shall be designed according to the AASHTO LRFD Guide Specifications for the Design of Concrete-Filled FRP Tubes for Flexural and Axial Members (Reference 3) Article 2.11.

Interaction diagrams are presented in Figures 1 and 2 for FRP piles tested by Lawrence (Reference 2). These piles are nominally 24 in in diameter with 1 in and 0.5 in thick FRP shells.

Example pile calculations are presented in Appendix A.

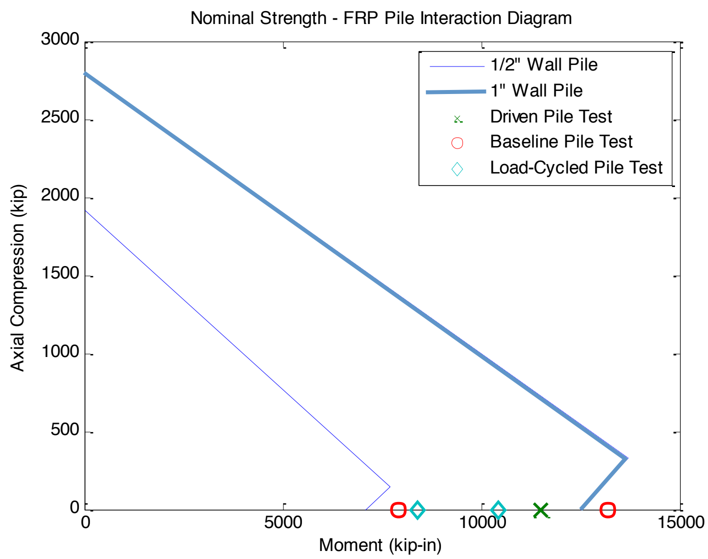

Figure 1: FRP Pile Interaction Diagram – Nominal Strength

The FRP pile nominal strengths in Figure 1 do not include environmental reduction factors, material bias factors, or LRFD phi factors. Flexural tests results by Lawrence (Reference 2) are included on the x axis for comparison with theoretical nominal strengths. All test data is for ½" wall, concrete-filled FRP piles.

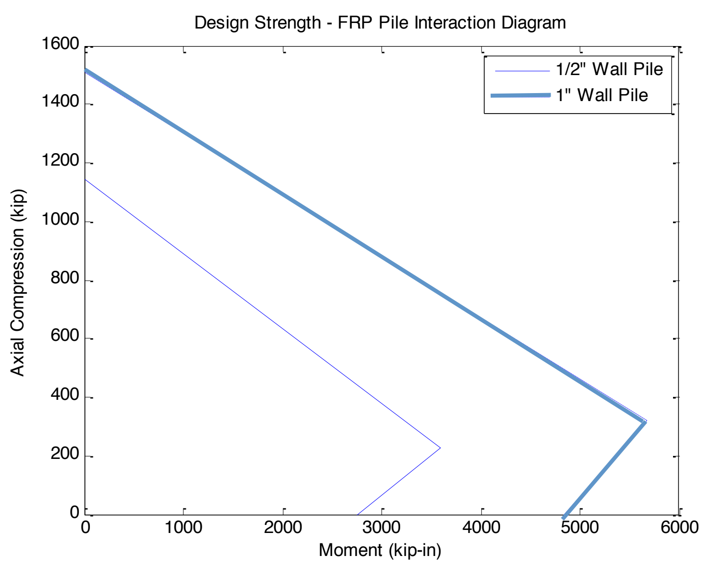

Figure 2: FRP Pile Interaction Diagram – Design Strengths

The FRP pile design strengths in Figure 2 include the environmental reduction factors and LRFD phi factors proposed in Reference 3, and the material bias factors proposed in this specification.

Section 2 Geotechnical Design Specifications

Amend the following Articles of the AASHTO LRFD Bridge Design Specifications (Reference 1).

10.7.3.8.6b – α-Method

The alpha method is recommended for the side capacity of FRP piles in cohesive material.

10.7.3.8.6e – Tipe Resistance in Cohesive Soils

The tip capacity shall be taken as 9*S_u as described in Article 10.7.3.8.6e.

10.7.3.8.6f - Meyerhof Method

Remove Article 10.7.3.8.6f – Nordlund/Thurman Method in Cohesionless Soils and replace with 10.7.3.8.6f – Meyerhof Method

The effective stress method proposed by Meyerhof and presented in NavFAC (Reference 4) shall be used to calculate the side resistance of FRP piles in cohesionless soils. The Meyerhof method uses the following relationship:

$$Q_s = K_h \sigma'_v \tan(\delta)$$

where:

Q_s = side resistance of the pile

K_h = earth pressure coefficient

δ = interface friction angle

Earth pressure coefficient values can be found in Reference 4.

Limiting values of effective overburden stress associated with the Meyerhof Method (Reference 5) shall be taken as the effective stress at z_c for:

$R_D \leq 30\%$ $\quad z_c = 10D$

$R_D \geq 70\%$ $\quad z_c = 20D$

where:

R_D = relative density of the soil

D = pile diameter

z_c = depth at limiting effective overburden stress

The Meyerhof Method includes a limiting overburden stress. Methods for calculating the limiting overburden stress are outlined in NavFAC (Reference 4).

Peak interface friction angles were found to range from 28.2 to 32.6 degrees and constant volume interface friction angles were found to range from 26.3 to 32.1 degrees for piles tested by Lawrence (Reference 2).

NavFAC (Reference 4) uses a limiting stress at z_c =20D for all relative densities.

The Meyerhof Method (Reference 6) is recommended for the end capacity in cohesionless soils using the following relationship:

$$Q_p = N_q^* \sigma_v' \leq N_q^* \sigma_l'$$

where:

Q_p = end resistance of the pile

N_q^* = bearing capacity factor for driven piles

σ_l' = limiting stress at pile tip defined as:

$$\sigma_l' = \tan(\varphi) \quad \text{(ksf)}$$

Values for N_q^* are calculated as (Reference 7 from Reference 6):

φ	N_q^*
30	57
31	68
32	81
33	96
34	115
35	143
36	168
37	194
38	231
39	276
40	346
41	420
42	525
43	650
44	780
45	930

Note that the N_q^* for the Meyerhof Method is different from N_q^* presented by AASHTO for the Nordlund Method.

10.7.3.13.5 – FRP Piles

Added Article 10.7.3.13.5

The nominal axial compression resistance for FRP piles shall be as specified in Section 1.

The Intact Rock Method (IRM) by Armitage and Rowe (Reference 8) shall be used to calculate end resistance bearing pressure on bedrock.

$$q_p = 2.5 q_u$$

where:

q_p = end bearing pressure capacity of the bedrock

q_u = unconfined compressive strength of the bedrock.

For principal Maine bedrocks, the following unconfined compressive strengths are typical (Reference 9).

Rock Type	**Unconfined Compressive Strength, q_u (ksi)**
Igneous	5.4
Metamorphic	5.2
Sedimentary	4.4

For open-ended piles, multiply the end bearing pressure capacity obtained by IRM by the cross-sectional area of the shoe to obtain the end capacity. For filled pipe piles, the cross-sectional area to be used for the end capacity is the total end cross-sectional area including filling area divided by 9.3 to account for discontinuous contact between the pile and the bedrock surface.

Preliminary estimates of the effective length of laterally unsupported piles should be determined based on the provisions in Article 10.7.3.13.4 using Davison and Robinson. Detailed evaluations of effective length, depth to fixity and lateral pile resistance shall be performed considering the effects of soil-structure interaction in accordance with LRFD Article 10.7.3.12.

10.7.5 – Corrosion and Deterioration

Corrosion and deterioration shall be accounted for using the environmental reduction factor, C_E, in accordance with the AASHTO LRFD Guide Specifications for Design of Concrete-Filled FRP Tubes for Flexural and Axial Members (Reference 3). This factor is implemented in the structural design of FRP piles outlined in Section 1.

10.7.8 – Drivability Analysis

Added the following section to Article 10.7.8

Cast-in-Place FRP Piles

- In compression:

$$\sigma_{dr} = \varphi_{da}(0.10 f_{fcu}) \quad (10.7.8\text{-}8)$$

where:

f_{fcu} = compressive strength of the FRP shell (ksi)

Precast FRP Piles

FRP piles are treated as a reinforced concrete pile.

- In tension:

$$\sigma_{dr} = 0.7\varphi_{da} f_y \quad (10.7.8\text{-}9)$$

where:

f_y = yield strength of the steel reinforcement (ksi)

- In compression:

$$\sigma_{dr} = \varphi_{da} 0.85 f'_{cc} \quad (10.7.8\text{-}9)$$

FRP piles ar treated as a reinforced concrete pile with confined concrete compressive strength.

where:

f'_{cc} = confined compressive strength of the concrete according to the AASHTO LRFD Guide Specifications for Design of Concrete-Filled FRP Tubes for Flexural and Axial Members (Reference 3) (ksi)

10.5.5.2.3 - Resistance Factors for Driven Piles

Remove Table 10.5.5.2.3-1 Resistance Factors for Driven Piles and replace with the following table.

Condition/Resistance Determination Method		Resistance Factor
Nominal Bearing Resistance of Single Pile – Dynamic Analysis and Static Load Test Methods, φ_{dyn}	Driving criteria established by successful static load test of at least one pile per site condition and dynamic testing* of at least two piles per site condition, but no less than 2% of the production piles	0.80
	Driving criteria established by successful static load test of at least one pile per site condition without dynamic testing	0.75
	Driving criteria established by dynamic testing* conducted on 100% of production piles	0.75
	Driving criteria established by dynamic testing*, quality control by dynamic testing* of at least two piles per site condition, but no less than 2% of the production piles	0.65
	Wave equation analysis without pile dynamic measurements or load test but with field confirmation of hammer performance	0.50
	FHWA-modified Gates dynamic pile formula (End of Drive condition only)	0.40
	Engineering News (as defined in Article 10.7.3.8.5) dynamic pile formula (End of Drive condition only)	0.10
Nominal Bearing Resistance of Single Pile - Static Analysis Methods, φ_{stat}	Side Resistance and End Bearing: Clay and Mixed Soils:	
	α-method (Tomlinson, 1987; Skempton, 1951)	0.35
	β-method (Esrig & Kirby, 1979; Skempton, 1951)	0.25
	λ-method (Vijayvergiya & Focht, 1972; Skempton, 1951)	0.40
	Side Resistance and End Bearing: Sand	
	Meyerhof method	0.45
	SPT-method (Meyerhof)	0.30
	CPT-method (Schmertmann)	0.50
	End bearing in rock (Intact Rock Method; Armitage and Rowe, 1987)	0.45
Block Failure, φ_{b1}	Clay	0.60
Uplift Resistance of Single Piles, φ_{up}	Meyerhof method	0.35
	α-method	0.25
	β-method	0.20
	λ-method	0.30
	SPT-method	0.25
	CPT-method	0.40
	Static load test	0.60
	Dynamic test with signal matching	0.50
Group Uplift Resistance, φ_{ug}	All soils	0.50
Lateral Geotechnical Resistance of Single Pile or Pile Group	All soils and rock	1.0
Structural Limit State	Steel piles	See the provisions of Article 6.5.4.2
	Concrete piles	See the provisions of Article 5.5.4.2.1
	Timber piles	See the provisions of Article 8.5.2.2 and 8.5.2.3
	FRP piles	See the provisions of Section 1
Pile Drivability Analysis, φ_{da}	Steel piles	See the provisions of Article 6.5.4.2
	Concrete piles	See the provisions of Article 5.5.4.2.1
	Timber piles	See the provisions of Article 8.5.2.2
	FRP piles	φ_{da} shall be taken as 1.0 for FRP piles

*Dynamic testing requires signal matching, and best estimates of nominal resistance are made from a restrike. Dynamic tests are calibrated to the static load test, when available.

Appendix A Example Structural Pile Calculation

Material Properties

$C_E := 0.65$	Environmental reduction factor
$\gamma_c := 0.2$	Material bias factor - compression
$\gamma_t := 0.4$	Material bias factor - tension
$f_{fcu} := 72.0 \cdot C_E \cdot \gamma_c = 9.36$	FRP Compressive Strength (ksi)
$\varepsilon_{fcu} := 0.0217 \cdot C_E \cdot \gamma_c = 0.003$	FRP Compressive Strain at Failure
$E_{fcu} := \frac{f_{fcu}}{\varepsilon_{fcu}} = 3318$	FRP Compressive Modulus (ksi)
$f_{ful} := 76.0 \cdot C_E \cdot \gamma_t = 19.76$	FRP Tensile Strength (ksi)
$\varepsilon_{ful} := 0.0256 \cdot C_E \cdot \gamma_t = 0.007$	FRP Tensile Strain at Failure
$E_{ful} := \frac{f_{ful}}{\varepsilon_{ful}} = 2969$	FRP Tensile Modulus (ksi)
$f'c := 4.35$	Unconfined Concrete Compressive Strength (ksi)
$D_i := 22.5$	Inner FRP Shell Diameter (in)
$D_o := 23.5$	Outer FRP Shell Diameter (in)
$D := \frac{\left(D_i + D_o\right)}{2} = 23$	Avereage FRP Shell Diameter (in)
$t := \frac{\left(D_o - D_i\right)}{2} = 0.5$	FRP Shell Thickness (in)
$E_{fl} := E_{ful} = 2969$	Longitudinal Design Modulus of FRP Shell (ksi)
$f_{fuh} := 25.3 \cdot C_E \cdot \gamma_t = 6.578$	Hoop Tensile Strength (ksi)
$\varepsilon_{fuh} := 0.0227 \cdot C_E \cdot \gamma_t = 0.006$	Hoop Tensile Strain at Failure
$E_{fh} := \frac{f_{fuh}}{\varepsilon_{fuh}} = 1115$	Hoop Tensile Modulus (ksi)

*** Stress-Strain was bi-linear for hoop tension

Flexural Resistance Factor

$$E_c := 1265 \cdot \sqrt{f'c} + 1000 = 3.638 \cdot 10^3$$

$$n := 0.8 + \frac{f'c}{2.5} = 2.54$$

$$\varepsilon'_c := \frac{f'c}{E_c} \cdot \frac{n}{(n-1)} = 0.00197$$

$$\beta := 0.633 \cdot \left(\frac{\varepsilon_{fcu}}{\varepsilon'_c}\right)^{0.247} \cdot f'c^{0.222} = 0.958$$

$$\alpha := 7.325 \cdot \left(\frac{\varepsilon_{fcu}}{\varepsilon'_c}\right)^{-0.917} \cdot f'c^{-1.086} = 1.069$$

$$\theta := \operatorname{acos}\left(1 - 2 \cdot \beta \cdot \left(\frac{f_{fcu}}{f_{ful} + f_{fcu}}\right)\right) = 1.177$$

$$t_b := \frac{\alpha \cdot f'c \cdot D}{4 \cdot \pi \cdot (f_{ful} - f_{fcu})} (2 \cdot \theta - \sin(2\ \theta)) = 1.345$$

$$\rho_b := \frac{4 \cdot t_b}{D} = 0.234$$

$$A_{FRP} := \left(D_o^{\ 2} - D_i^{\ 2}\right) \cdot \frac{\pi}{4} = 36.128$$

$$A_{tot} := 23.5^2 \cdot \frac{\pi}{4} = 433.736$$

$$\rho := \frac{A_{FRP}}{A_{tot}} = 0.083$$

$$\frac{\rho}{\rho_b} = 0.356$$

$$\varphi_{flex1} := \begin{array}{l} \text{if } \rho \le \rho_b \\ \quad \varphi_{flex1} \leftarrow 0.55 \\ \text{else if } \rho \ge \rho_b \\ \quad \varphi_{flex1} \leftarrow 0.65 \\ \text{else} \\ \quad \varphi_{flex1} \leftarrow 0.3 + 0.25 \cdot \dfrac{\rho}{\rho_b} \end{array} = 0.55$$

$$\varphi_{flex1} = 0.55$$

Pure Flexure

$$M_n := 0.0045 \cdot {D_o}^3 \cdot f'c \cdot \left(100 \cdot \frac{(4 \cdot t)}{D_o} \cdot \frac{f_{ful}}{f'c}\right)^{0.815} = 4.995 \cdot 10^3 \quad \text{kip-in}$$

$$M_r := \varphi_{flex1} \cdot M_n = 2.747 \cdot 10^3$$

Pure Axial Compression

$$E_c := 1265 \cdot \sqrt{f'c} + 1000 = 3.638 \cdot 10^3$$

$$n := 0.8 + \frac{f'c}{2.5} = 2.54$$

$$\varepsilon'_c := \frac{f'c}{E_c} \cdot \frac{n}{(n-1)} = 0.002$$

$$\varepsilon_{fe} := \varepsilon_{fuh} \cdot 0.55 = 0.003$$

$$f_{l1} := \frac{(2 \cdot E_{fh} \cdot t \cdot \varepsilon_{fe})}{D} = 0.157$$

$$f_{l2} := \left(\frac{\left(\frac{0.01}{\varepsilon'_c} - 1.5 \right)}{\frac{12}{f'c} \cdot \left(\frac{\varepsilon_{fe}}{\varepsilon'_c} \right)^{0.45}} \right) = 1.034$$

$$f_l := min(f_{l1}, f_{l2}) = 0.157$$

$$\varepsilon_{ccu} := \varepsilon'_c \cdot \left(1.5 + 12 \cdot \frac{f_l}{f'c} \cdot \left(\frac{\varepsilon_{fe}}{\varepsilon'_c} \right)^{0.45} \right) = 0.004$$

$$\varepsilon_{ccu'} := \varepsilon_{ccu}$$

$$\varepsilon_{ccu} := \begin{Vmatrix} \text{if } \varepsilon_{ccu} \ge 0.01 \\ \quad \| \varepsilon_{ccu} \leftarrow 0.01 \\ \text{else} \\ \quad \| \varepsilon_{ccu} \leftarrow \varepsilon_{ccu'} \end{Vmatrix} = 0.004$$

$$\Psi_f := 0.95$$

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$$f'_{cc} := \begin{array}{l} \text{if } f_l = f_{l1} \\ \quad f'_{cc} \leftarrow f'c + 3.3 \cdot \Psi_f \cdot f_l \\ \text{else if } f_l = f_{l2} \\ \quad f''_{cc} \leftarrow f'c + 3.3\ \Psi_f \cdot f_{l1} \\ \quad E_2 \leftarrow \dfrac{(f''_{cc} - f'c)}{\varepsilon_{ccu}} \\ \quad f'_{cc} \leftarrow f''_{cc} + 0.01 \cdot E_2 \end{array} = 4.843$$

$$f'_{cc} = 4.843$$

$$P'_n := \frac{(\pi \cdot D_i^{\ 2})}{4} \cdot 0.85\ f'_{cc} + \pi \cdot D \cdot t \cdot E_{fl} \cdot \varepsilon_{ccu} = 2.069 \cdot 10^3$$

$$P_n := 0.85 \cdot P'_n = 1.759 \cdot 10^3 \quad \text{kip}$$

$$\varphi_{axial} := .65$$

$$P_r := \varphi_{axial} \cdot P_n = 1.143 \cdot 10^3$$

Combined Flexure and Axial Compression

$$\alpha_1 := 0.83$$

$$\beta_1 := 0.88$$

$$c := \left(D_o - \frac{t}{2}\right) \cdot \frac{\varepsilon_{ccu}}{\varepsilon_{ccu} + \varepsilon_{ful}} = 8.767$$

$$\theta_a := \operatorname{acos}\left(1 - \frac{(2 \cdot \beta_1 \cdot c)}{D_i}\right) = 1.251$$

$$M_{nb} := \frac{(\pi \cdot D^3 \cdot t)}{8 \cdot c} \cdot E_{fl} \cdot \varepsilon_{ccu} + \frac{D_i^{\,3}}{12} \cdot \alpha_1 \cdot f'_{cc} \cdot (\sin(\theta_a))^3 = 6.524 \cdot 10^3$$

$$M_{rb} := \varphi_{flex1} \cdot M_{nb} = 3.588 \cdot 10^3$$

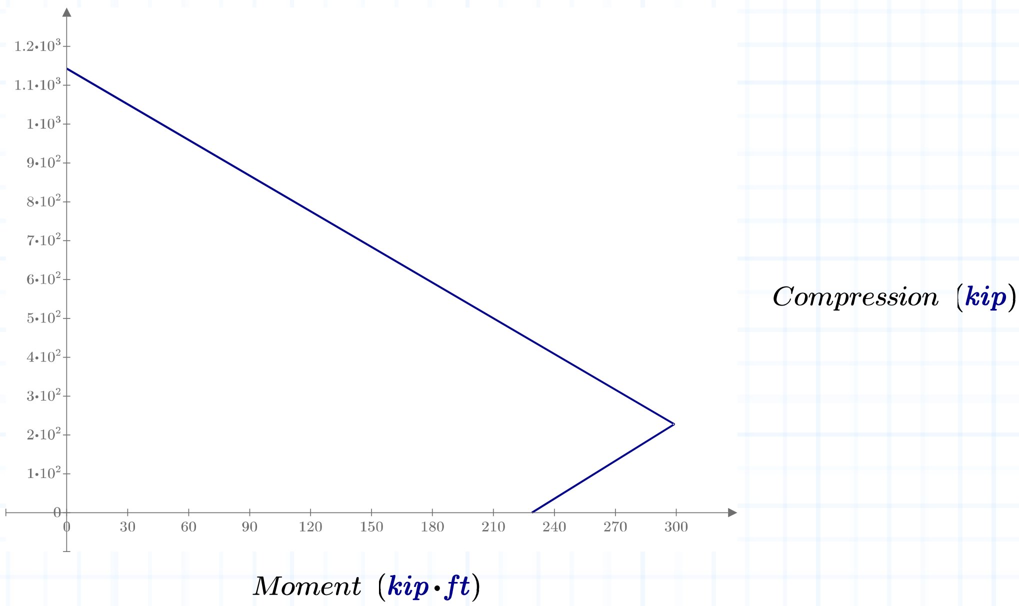
$$P_{nb} := \frac{(\pi \cdot D \cdot t)}{2 \cdot c} \cdot E_{fl} \cdot \varepsilon_{ccu} \cdot (2 \cdot c - D) + \frac{D_i^{\,2}}{8} \cdot \alpha_1 \cdot f'_{cc} \cdot (2 \cdot \theta_a - \sin(2 \cdot \theta_a)) = 350.0$$

$$P_{rb} := \varphi_{axial} \cdot P_{nb} = 227.502$$

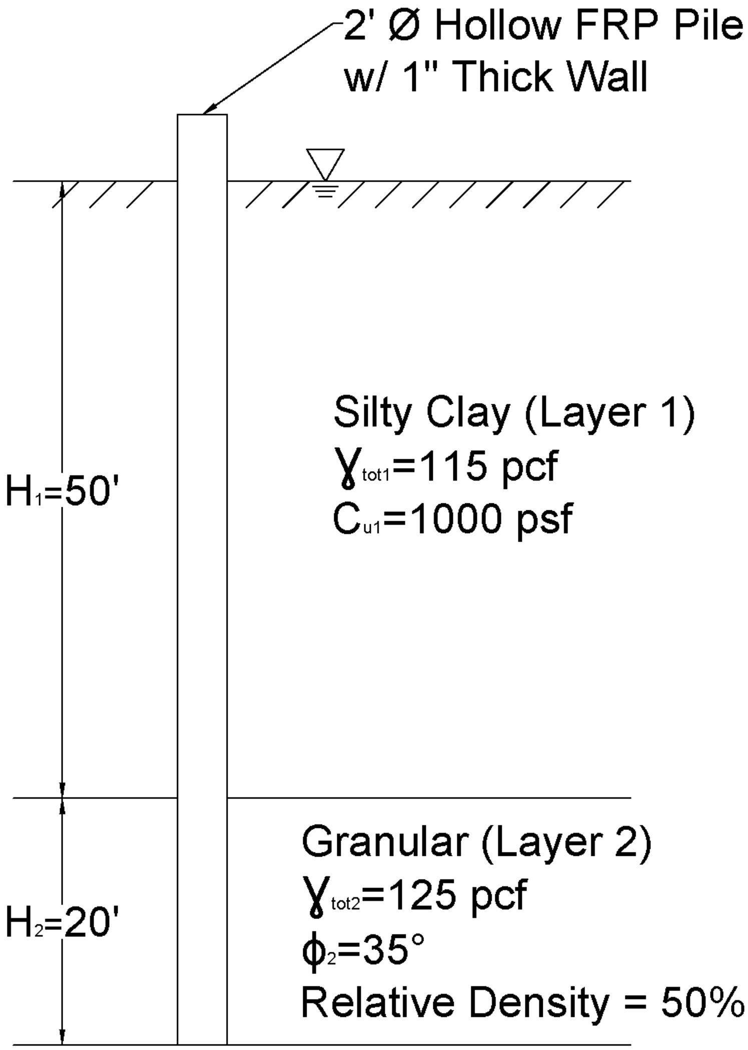
Interaction Diagram

$$\text{Compression} := \begin{bmatrix} P_r \\ P_{rb} \\ 0 \end{bmatrix} \cdot \boldsymbol{kip} = \begin{bmatrix} 1.143 \cdot 10^3 \\ 227.502 \\ 0 \end{bmatrix} \boldsymbol{kip}$$

$$\text{Moment} := \begin{bmatrix} 0 \\ M_{rb} \\ M_r \end{bmatrix} \cdot \boldsymbol{kip} \cdot \boldsymbol{in} = \begin{bmatrix} 0 \\ 299.015 \\ 228.93 \end{bmatrix} \boldsymbol{kip} \cdot \boldsymbol{ft}$$



Appendix B Example Geotechnical Capacity Pile Calculation



A. Granular
γ_{totA}=130 pcf
ϕ_A=38°

B. Clay
γ_{totB}=120 pcf
C_{uB}=2000 psf

C. Metamorphic Rock
q_{uC}=5.2 ksi
(Intact Rock Method)

Pile Properties

$D := 2\ ft$ Pile diameter

$t := 1\ in$ Pile wall thickness

$\delta := 29\ deg$ Interface friction angle (FRP to granular soil)

$A_{toe} := 2 \cdot t \cdot \pi \cdot D = 1.05\ ft^2$ Area of steel driving shoe (Assume 2X pile area)

Silty Clay Layer Properties (Layer 1)

$\gamma_{tot1} := 115\ pcf$ Total unit weight of silty clay

$C_{u1} := 1000\ psf$ Undrained shear strength

$H_1 := 50\ ft$ Height of clay layer

$\gamma_w := 62.4\ pcf$ Unit weight of water

Granular Layer Properties (Layer 2)

$\gamma_{tot2} := 125\ pcf$ Total unit weight of granular soil

$\phi_2 := 35\ deg$ Angle of internal friction

$H_2 := 20\ ft$ Height of granular layer

End Bearing Layer Properties - Case A (Granular)

$\phi_A := 38\ deg$ Angle of internal friction

$\gamma_{totA} := 130\ pcf$ Total unit weight of granular soil at tip of pile

End Bearing Layer Properties - Case B (Clay)

$\gamma_{totB} := 120\ pcf$ Total unit weight of clay at tip of pile

$C_{uB} := 2000\ psf$ Undrained shear strength of clay at tip of pile

End Bearing Layer Properties - Case C (Metamorphic Rock)

$q_{uC} := 5.2\ ksi = 748.8\ ksf$ Unconfined compressive strength of Metamorphic Rock (from Intact Rock Method)

Side Shear - Silty Clay Layer (Layer 1)

$A_{s1} := \pi \cdot D \cdot H_1 = 314.2\ ft^2$ Area along side of pile in silty clay layer

$\alpha_1 := 0.95$ from FHWA for D = 25*B

$Q_{s1} := A_{s1} \cdot C_{u1} \cdot \alpha_1 = 298.5\ kip$ Side shear resistance in silty clay layer

Side Shear - Granular Layer

$\sigma'_{v50} := (\gamma_{tot1} - \gamma_w) \cdot H_1 = 2630\ psf$ Effective stress at 50' depth

$\sigma'_{v70} := \sigma'_{v50} + (\gamma_{tot2} - \gamma_w) \cdot H_2 = 3882\ psf$ Effective stress at 70' depth

$H_{limit} := 15 \cdot D = 30\ ft$ Limiting depth (Meyerhof Method)

$\sigma'_{limit} := H_{limit} \cdot (\gamma_{tot2} - \gamma_w) = 1878\ psf$ Constant over granular layer

$K_0 := 1 - \sin(\phi_2) = 0.43$

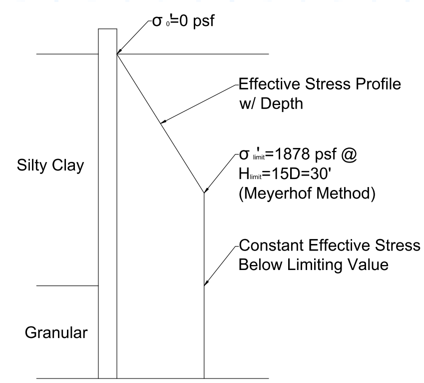
$\tau := \sigma'_{limit} \cdot K_0 \cdot \tan(\delta) = 443.9\ psf$

$A_{s2} := H_2 \cdot \pi \cdot D = 125.7\ ft^2$ Area along side of pile in granular layer

$Q_{s2} := A_{s2} \cdot \tau = 55.8\ kip$ Inside shear neglected

Total Side Shear Resistance

$Q_s := Q_{s1} + Q_{s2} = 354.2\ kip$



End Resistance - Case A (Granular)

$N_q := 231$ Nq* (Meyerhof Method) for phi = 38 degrees

$Q_{pA} := A_{toe} \cdot \sigma'_{v70} \cdot N_q = 939.1$ ***kip***

$Q_{pA;limit} := N_q \cdot \tan\left(\phi_A\right) \cdot 1$ ***ksf*** $\cdot A_{toe} = 189$ ***kip***

$$Q_{pA} := \text{if } Q_{pA} < Q_{pA;limit} \quad = 189 \; \textbf{kip}$$
$$\left\| Q_{pA} \leftarrow Q_{pA} \right.$$
$$\text{else}$$
$$\left\| Q_{pA} \leftarrow Q_{pA;limit} \right.$$

End Resistance - Case B (Clay)

$Q_{pB} := A_{toe} \cdot 9 \cdot C_{uB} = 18.8$ ***kip***

End Resistance - Case C (Metamorphic Rock)

$Q_{pC} := A_{toe} \cdot q_{uC} \cdot 2.5 = 1960.4$ ***kip***

Ultimate Capacity (Unfactored) - Case A (Granular at Tip)

$Q_{uA} := Q_s + Q_{pA} = 543.2$ ***kip***

Ultimate Capacity (Unfactored) - Case B (Clay at Tip)

$Q_{uB} := Q_s + Q_{pB} = 373.1$ ***kip***

Ultimate Capacity (Unfactored) - Case C (Metamorphic Rock at Tip)

$Q_{uC} := Q_s + Q_{pC} = 2315$ ***kip***

Factored Resistance Factors

$\phi_{\alpha} := 0.35$ phi for alpha method

$\phi_{M} := 0.45$ phi for Meyerhof Method

$\phi_{IRM} := 0.45$ phi for Intact Rock Method

Factored Resistance - Case A (Granular at Tip)

$$Q_{RA} := Q_{s1} \cdot \phi_{\alpha} + Q_{s2} \cdot \phi_{M} + Q_{pA} \cdot \phi_{M} = 214.6 \ \textbf{kip}$$

Factored Resistance - Case B (Clay at Tip)

$$Q_{RB} := Q_{s1} \cdot \phi_{\alpha} + Q_{s2} \cdot \phi_{M} + Q_{pB} \cdot \phi_{\alpha} = 136.2 \ \textbf{kip}$$

Factored Resistance - Case C (Metamorphic Rock at Tip)

$$Q_{RC} := Q_{s1} \cdot \phi_{\alpha} + Q_{s2} \cdot \phi_{M} + Q_{pC} \cdot \phi_{IRM} = 1011.7 \ \textbf{kip}$$