Cast-In-Drilled-Hole (CIDH) Pile Foundations

This document presents the design methods and communication steps between Bridge Design (BD) and Geotechnical Services (GS) for the load and resistance factor design (LRFD) of Cast-In-Drilled-Hole (CIDH) pile foundations used for support of bridges, retaining walls, non-standard walls, signs, and other structures. Also included are the procedures for documenting design information that might be needed to evaluate the impact of an anomaly (i.e., construction defect), and related analytical procedures. The Appendices include three design calculation examples.

Standards relating to CIDH pile foundation investigations, design, and reporting are:

- Caltrans Seismic Design Criteria (SDC)
- AASHTO LRFD Bridge Design Specifications with CA Amendments (AASHTO)
- Drilled Shafts: Construction Procedures and Design Methods (O'Neill & Reese, 1999)
- Caltrans Standard Specifications, Standard Plans, Bridge Standard Detail Sheets (XS Sheets)
- Bridge Memos to Designers (MTD) 3-1, *Deep Foundations*
- Bridge Design Aids
- Bridge Construction Records and Procedures Manual, Volume II
- Caltrans Geotechnical Manual
    - *Foundation Reports for Bridges*
    - *Geotechnical Investigations*

Geotechnical Service's role in CIDH pile foundation design is to provide the Bridge Designer with a Foundation Report addressing the following:

- Design Tip Elevations for piles for Service, Strength, and Extreme Event Limit State.
- The Controlling Design Tip Elevation.
- The Steel Casing Specified Tip Elevation (if applicable).
- Recommendations relating to specifications and construction.

The Bridge Designer's role in CIDH pile design includes:

- Providing the project schedule including due dates for reports.
- Providing the foundation design data and factored design load information.
- Providing the latest plan sheets pertinent to foundation design (e.g., General Plan, Foundation Plan, Foundation Detail Sheets, etc.).

Terminology

1. Cast-In-Drilled Hole (CIDH) Concrete Piles: CIDH concrete piles, also known as drilled shafts, can be used as smaller-diameter piles that are connected to a pile cap supporting a column or as a larger pile (typically 5 feet or larger) that directly supports a column and is either a Type I or Type II shaft (as determined by the Bridge Designer). Standard Plan CIDH concrete piles are either 16 or 24 inches in diameter, whereas special design CIDH concrete piles range from 30 inches and greater. Piles placed in wet conditions must be at least 24 inches in diameter to accommodate inspection pipes for acceptance testing. CIDH concrete pile lengths should be limited to 30 times the pile diameter to help ensure constructability and quality.
    i. Type I Shaft: The reinforcement consists of one continuous cage that extends from the pile tip to the bent cap.
    ii. Type II Shaft: The reinforcement consists of one cage that extends from the pile tip to the pile cut-off elevation. The column cage is a smaller-diameter cage that extends into the CIDH concrete pile reinforcement cage to form a lap splice. For a 5-foot diameter or larger Type II shaft, a construction joint is mandatory at the bottom of the column rebar cage elevation. The construction joint requires the placement of a permanent steel casing/shell in the hole to allow workers to clean and prepare the joint.
2. Rock Socket: A pay item for the length of a CIDH concrete pile that is constructed in rock that requires a core barrel, cluster hammer, or other hard rock tool for excavation. The rock is usually stronger than concrete, and typically the side resistance is controlled by the compressive strength of concrete, not the rock strength.
3. Driven Steel Shell: A smooth-walled steel pipe. The steel shell is used for geotechnical resistance and structural capacity. The shell must be installed with an impact hammer.
4. Permanent Casing: A corrugated metal pipe (CMP), however the contractor may choose to use a substitute (e.g., smooth-walled steel pipe). Used for constructability. The contractor will determine the permanent casing diameter and thickness.
5. Permanent Steel Casing: A smooth-walled steel pipe. Used for structural capacity. The casing thickness will be specified by Bridge Design and shown on plans.

Investigations

The goal of the geotechnical investigation for a CIDH pile foundation is to determine the properties and behaviors of the soil and/or rock, and the groundwater condition that can affect foundation design and construction. All subsurface conditions that might influence the foundation design and performance should be investigated.

Perform a literature search (see *Geotechnical Investigations* module) to gather all relevant information related to site geology, strength of soil and rock, and geologic hazards. Then, develop a prudent exploration plan considering site constraints, geologic variability, and available resources. Borings should be located as close as possible to the proposed foundation.

The exploration plan should include:

- An appropriate number of exploratory borings and/or cone penetration tests (CPT) to develop the design soil profile (AASHTO Table 10.4.2-1).
- An appropriate depth of exploration for the borings or CPT. The depth of exploration should generally extend below the anticipated pile tip elevation a minimum of 20 feet, or a minimum of two times the maximum pile group dimension, whichever is deeper (AASHTO Table 10.4.2-1).
- Standard penetration tests (SPT). When SPTs are to be performed, sampling intervals should be limited to no more than 5 feet.
- Groundwater measurements.
- Soil and water samples for corrosion testing in accordance with current *Caltrans Corrosion Guidelines*.
- Collecting samples for laboratory testing: (e.g., classification tests, consolidation test, soil strength parameters required for design).
    - Classification tests to help determine if soil is cohesive or cohesionless if field identification is inconclusive.
    - Consolidation test if fills are to be constructed in vicinity of piles (e.g., at abutments).

Design

The following provides design methodologies used to calculate settlement (Service-I Limit State) and pile resistance (Strength and Extreme Event Limit States) in accordance with AASHTO 10.8.1. For appropriate resistance factors refer to AASHTO Table 10.5.5.2.4-1.

CIDH concrete pile design considers three material types: soil, intermediate geomaterial (IGM), and rock, defined as follows:

CIDH Concrete Pile Design Material Types

Material Type	General Description	Properties
Cohesive soil	Clay, Plastic Silt	$S_u \leq 5.0$ ksf, $q_u \leq 10.0$ ksf
Cohesionless soil	Sand, Gravel, Non-plastic Silt	$N_{60} \leq 50$
Cohesive IGM	Shale, Mudstone, Over-consolidated Clay	5.0 ksf < S_u < 50.0 ksf
Cohesionless IGM	Poorly Indurated Sandstone, Siltstone, Tuff, Granular Residual Soil, Granular Till	$50 < N_{60} \leq 100$
Rock	All rock not defined as IGM	$S_u \geq 50$ ksf, UCS $\geq$ 100 ksf

Soil and IGM properties used for design should come from: (1) SPT correlations (see *Soil Correlations* module) and/or (2) results of laboratory tests under similar field conditions.

Rock properties used for design should come from laboratory unconfined compression tests.

The design must also account for geologic hazards such as:

- Liquefaction (see *Liquefaction Evaluation* module)
- Lateral spreading (see *Lateral Spreading* module)
- Scour: Foundations that are constructed in a watercourse must meet AASHTO guidelines regarding scour depths (AASHTO C2.6.4.4.2). The top of the pile cap must be below the degradation plus contraction scour depth. The bottom of the pile cap must be below the degradation plus contraction plus local pier scour depth.

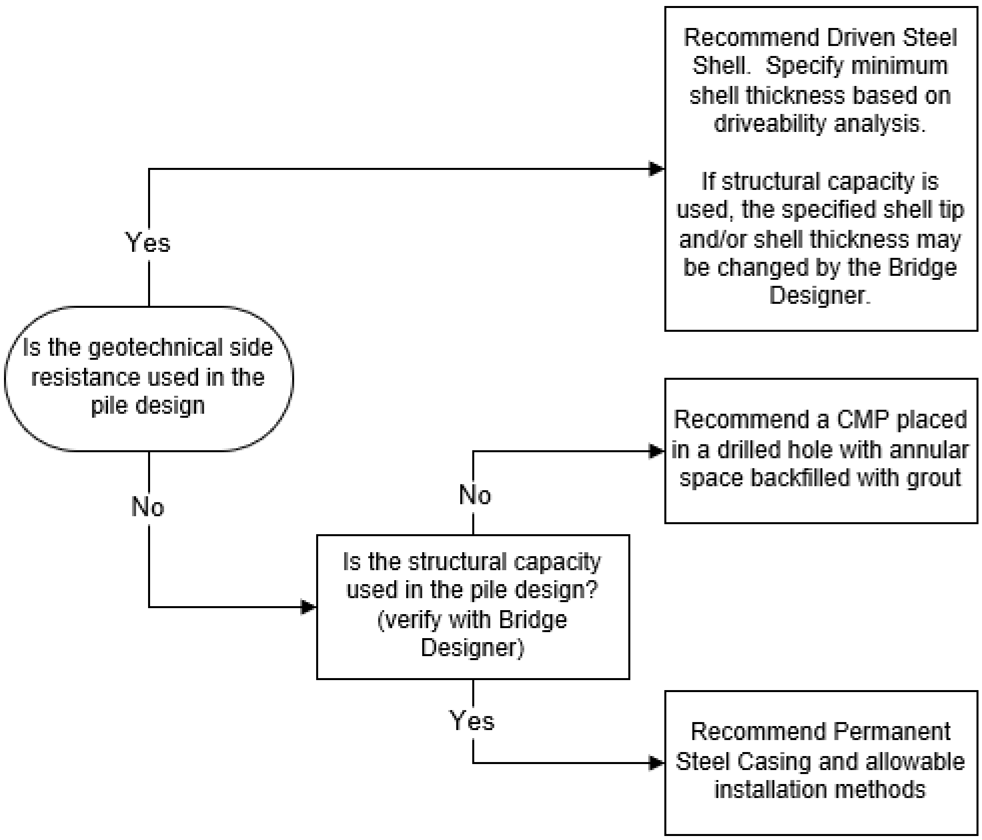
CIDH Concrete Pile Design and Construction Considerations

The following table presents options for selection of a permanent steel casing, permanent casing, or driven steel shell.

Permanent Steel Casing, Permanent Casing, or Driven Steel Shell in CIDH Concrete Pile Design and Construction

Types	Used for Constructability?	Used for Structural Capacity?	Used for Geotechnical Resistance?	Installation Method
Permanent Steel Casing (smooth-wall steel pipe)	Yes	Yes	No	Drilled, vibrated, oscillated/rotated into place, or placed in drilled hole and annular space backfilled with grout
Permanent Casing (CMP)	Yes	No	No	Placed in a drilled hole and annular space backfilled with grout
Driven Steel Shell (smooth-wall steel pipe)	Yes	Yes	Yes	Impact hammer

The flow chart shows the most cost-effective options for use of a permanent steel casing, permanent casing, or driven steel shell.



Tip Resistance Considerations

Tip resistance may be used according to the following table for pile diameters of 36 inches and larger. Tip resistance is disallowed for pile diameters less than 36 inches.

Material at Pile Tip	Dry Method[1]	Slurry Displacement Method
Cohesive Soil	No	No
Cohesionless Soil	No	No
Intermediate Geomaterial	Allowed[2]	Allowed[2]
Rock	Allowed[2]	Allowed[2]

1. Per Standard Specification 49-3.02A(2) dry hole and dewatered hole definitions
2. Shaft Inspection Device testing required per SSP 49-3.02A(4)(d)(iv)

CIDH Concrete Pile and Rock Socket Measurement Considerations

Determination of CIDH Concrete Pile and Rock Socket should comply with the following table and be used to determine the Top of Rock Socket Elevation reported in the Foundation Design Recommendations table and the Pile Data Table.

Fracture Density	Rock Hardness						
	Very Soft	Soft	Mod. Soft	Mod. Hard[1]	Hard[1]	Very Hard[1]	Ext. Hard[1]
Very Intensely Fractured	CIDH Pile	CIDH Pile	CIDH Pile	CIDH Pile	CIDH Pile	CIDH Pile	CIDH Pile
Intensely Fractured	CIDH Pile	CIDH Pile	CIDH Pile	CIDH Pile	CIDH Pile (Rock Socket)	CIDH Pile (Rock Socket)	CIDH Pile (Rock Socket)
Moderately Fractured	CIDH Pile	CIDH Pile	CIDH Pile	CIDH Pile (Rock Socket)	CIDH Pile (Rock Socket)	CIDH Pile (Rock Socket)	CIDH Pile (Rock Socket)
Slightly Fractured	CIDH Pile	CIDH Pile	CIDH Pile	CIDH Pile (Rock Socket)	CIDH Pile (Rock Socket)	CIDH Pile (Rock Socket)	CIDH Pile (Rock Socket)
Very Slightly Fractured	CIDH Pile	CIDH Pile	CIDH Pile	CIDH Pile (Rock Socket)	CIDH Pile (Rock Socket)	CIDH Pile (Rock Socket)	CIDH Pile (Rock Socket)
Unfractured	CIDH Pile	CIDH Pile	CIDH Pile	CIDH Pile (Rock Socket)	CIDH Pile (Rock Socket)	CIDH Pile (Rock Socket)	CIDH Pile (Rock Socket)

1. Uniaxial Compressive Strength > 3500 psi (ASTM D 7012)

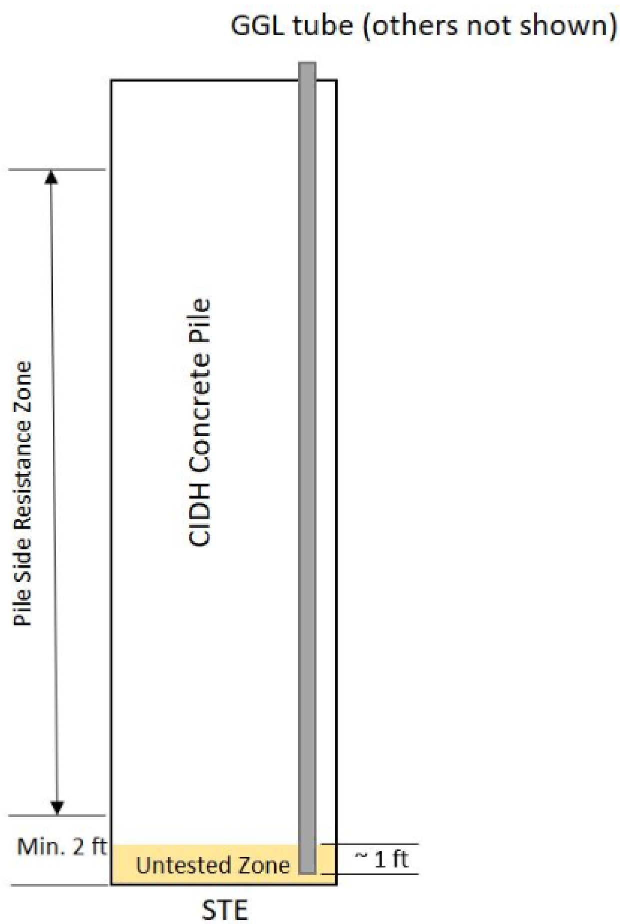
CIDH Concrete Pile with Tension Demands

When determining geotechnical side resistance for tension, a reduction factor must be applied for uplift. Reduction factor for various subsurface materials are as follows: 0.75 for cohesionless soil, 1.0 for cohesive soil or cohesive IGM, 0.7 for cohesionless IGM and varies from 0.7 for extremely fractured rock, and 1.0 for unfractured rock.

Quality Assurance Considerations

If it is anticipated that the slurry displacement method will be used for concrete placement, CIDH pile acceptance testing in accordance with California Test (CT) 233, "*Method of Ascertaining the Homogeneity of Concrete in CIDH Piles Using the Gamma-Gamma Test Method*" will be required. Due to the limitations of the gamma-gamma logging (GGL) equipment, there is a zone of untested concrete at the bottom of the pile. To account for this zone of untested concrete, the specified tip elevation must be lowered a minimum of two feet below the bottom of the side resistance zone (Figure 1).

Figure 1: Untested Concrete Zone at Bottom of CIDH Pile



Design Information and Communication (Preliminary Foundation Report)

After the field investigation and testing has been completed, review the design information provided by the Bridge Designer which should include:

- General Plan
- Preliminary Foundation Design Data Sheet (MTD 3-1, Attachment 1)

Table X: Preliminary Foundation Design Data Sheet

Support Location	Foundation Type(s) Considered	Estimate of Maximum Factored Compression Loads (Strength Limit State) (kips)
Abutment 1		
Pier 2		
Abutment 3		

Design Process (Preliminary Foundation Report)

Complete the CIDH pile foundation design process by following the steps below:

Step 1: Evaluation of Support Location and Foundation Type

- Verify that the foundation location and type is acceptable considering the known subsurface information and geological hazards (e.g., scour, lateral spreading, liquefaction).

Step 2: Calculate the Preliminary Tip Elevations

- Calculate the preliminary tip elevations meeting the controlling compression and tension requirements for the Strength Limit State at each support location.

Step 3: Complete Preliminary Foundation Design Recommendations table.

- Present the tip elevations for compression in the Preliminary Foundation Design Recommendations table under the Preliminary Tip Elevation column in the Preliminary Foundation Report.

Step 4: Reporting

- Complete the Preliminary Foundation Report according to the *Foundation Reports for Bridges* module.

Design Information and Communication (Foundation Report)

Review the design information provided by the Bridge Designer, which should include:

- General Plan
- Foundation Plan
- Scour Data Table (MTD 3-1, Attachment 1) or Hydraulics Report (if scour potential exists)
- Foundation Design Data Sheet (MTD 3-1, Attachment 1)
- Foundation Factored Design Loads information (MTD 3-1, Attachment 1)

Table X: Foundation Design Data Sheet (MTD 3-1, Attachment 1)

Support No.	Pile Type	Finished Grade Elevation (feet)	Cut-off Elevation (feet)	Pile Cap Size (feet)		Permissible Settlement under Service Load (inches)	Number of Piles per Support
				B	L		
Abut 1	30-in diam. CIDH						
Pier 2	96-in diam. CIDH			N/A	N/A		
Abut 3	30-in diam. CIDH						

Table X: Foundation Factored Design Loads (MTD 3-1, Attachment 1)

Support No.	Service-I Limit State (kips)		Strength/Construction Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)			
	Total Load per Support	Permanent Load per Support	Compression		Tension		Compression		Tension	
			Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1							N/A	N/A	N/A	N/A
Pier 2										
Abut 3							N/A	N/A	N/A	N/A

Design Process (Foundation Report)

Complete the CIDH pile foundation design process by following the steps below:

Step 1: Evaluation of Support Location and Foundation Type

- Verify that the foundation location and type is acceptable considering the known subsurface information and geological hazards (e.g., scour, lateral spreading, liquefaction).

Step 2: Determine the Specified Tip Elevation for the Steel Casing (if applicable).

(Commentary: Depending on the method of excavation, the diameter of the rock socket may need to be sized at least 8 inches smaller than the nominal casing size to permit seating of casing and insertion of rock drilling equipment (AASHTO 2017, C10.8.1.3).

Step 3: Calculate the Design Tip Elevations for the Piles

- Calculate the design tip elevations meeting the controlling compression and tension requirements for the Strength Limit State and the Extreme Event Limit State at each support location.
- Using the lowest design tip elevation from Strength and Extreme Limit State, calculate the settlement under the Service-I Limit State load and verify that it is less than the permissible settlement.

  If the calculated settlement is less than the permissible settlement, then a note should be placed at the bottom of the pile data table stating that the settlement requirement is met, and a settlement tip (c) is not included in the table.

  For piles embedded adequately into dense granular soils such that the equivalent footing is located on or within the dense granular soil, and furthermore are not subjected to downdrag loads, a detailed assessment of the pile group settlement may be waived. If the design tip for service limit state is waived, then a note should be placed at the bottom of the pile data table.

- If the permissible settlement is exceeded, calculate the design tip elevation for Service-I Limit State.

Step 4: Complete the Tables

- Present the tip elevations for compression, tension, and settlement in the Foundation Design Recommendations and Pile Data Tables.

Step 5: Prepare and Send Draft Foundation Report

- Complete the Draft Foundation Report according to the *Foundation Reports for Bridges* module.

Step 6: Determine the Specified Tip Elevation (Lateral Tip Considerations)

- Obtain the lateral tip elevation from the Bridge Designer
  - If the lateral tip is higher than or equal to the specified tip elevation, then there is no action required by the GP.
  - If the lateral tip is lower, the GP must verify that the pile can be installed to the lateral tip elevation and that all other recommendations in the report are correct (e.g., pile tip is now below groundwater, pile tip is now in rock).

Step 7: Reporting

- Complete the Foundation Report per the *Foundation Reports for Bridges* module.

Design Data Documentation

If the contractor uses either the slurry displacement method to place concrete or temporary casing to aid in dewatering the drilled hole, the CIDH pile will be inspected in accordance with California Test (CT) 233, "*Method of Ascertaining the Homogeneity of Concrete in CIDH Piles Using the Gamma-Gamma Test Method*". If anomalies are detected in the pile, the Foundation Testing Branch will issue a *Pile Acceptance Report* stating that the pile be rejected. Geotechnical Services will then participate in a process, initiated by Structure Construction, to evaluate the impact of the anomaly on the pile's capacity and determine a path forward.

To meet the time requirements in the Standard Specifications for evaluating the effect of anomalies on the geotechnical capacity of CIDH piles, pertinent geotechnical design information must be readily accessible to the GP and BC. The following CIDH pile information must be retained in the Geotechnical Design Office's *Electronic Project File Storage System* (EPFSS).

- Foundation Report
- Log of Test Borings
- Geological profile used for each CIDH pile design
- Soil and or rock strength parameters used in the design of each support/pile
- Calculations and/or computer/spreadsheet outputs used to determine each pile SPTE

Evaluation of Anomalous CIDH Concrete Piles

When a pile is rejected, the State has limited time per Standard Specification section 49-3.02A(4)(d)(iv) *Rejected Piles*, to determine which of the following options is available to the contractor:

1. The pile must be supplemented or replaced.
2. The pile must be repaired.
3. The pile is adequate with the anomaly left in place.

The *Pile Design Data Form* (PDDF) is used by Structure Construction with input by the Foundation Testing Branch, Bridge Design, GS, and the METS Corrosion Branch to determine acceptable options for a rejected pile. The FTB will complete Part 1 of the PDDF, which will identify the location and extent of the anomaly and attach the PDDF to the *Pile Acceptance Report*. A copy of the *Pile Acceptance Report* is sent to the Geoprofessional and the Chair of the CIDH Pile Mitigation Committee. SC will request that BD, GS, and METS Corrosion complete their respective sections and return the PDDF to SC. The information completed by the FTB for Part 1 is used by the Geoprofessional to complete Part 2, BD to complete Part 3, and METS Corrosion to complete Part 4 of the PDDF. See MTD 3-7, *Design Data Documentation and Evaluation of Anomalous Concrete Shafts,* for details.

The FTB produces a plot on the PDDF (Part 1) that identifies the top and the bottom of the anomaly and the percentage of the pile affected. Using the original design calculations and assumptions, calculate (Appendix C):

- the area of the anomaly on the outside of the pile and the skin friction contribution of that area.

  <*Note: The geotechnical evaluation assumes that the anomaly does not affect the ability of the pile to transmit load to the pile below the anomaly*.>

- the area of the anomaly at the pile base and the end bearing contribution of that area.

If the required nominal resistance exceeds the anomaly-reduced nominal resistance, the pile is unacceptable. If the pile is determined to be adequate with the anomaly in place, then the contractor may either repair the pile and receive full payment or leave the anomaly in place and incur an administrative deduction as specified in the contract.

Complete Part 2 of the PDDF using the results of the analysis and return the form to SC.

If the capacity of the pile is determined to be inadequate by GS, BD, or METS Corrosion, then the anomaly mitigation process will initiate and the FTB, SC, GS, and BD will collectively determine if the pile should be repaired, supplemented, or replaced. The standard repair techniques are excavation, and removal and replacement for anomalous concrete near the top of the pile, or grouting repair to lower potions of the pile. If the standard repair methods are not feasible, SC will hold a *CIDH Pile Non-Standard Mitigation Meeting*, per *Bridge Construction Memo BCM 130-21* to determine an acceptable mitigation strategy.

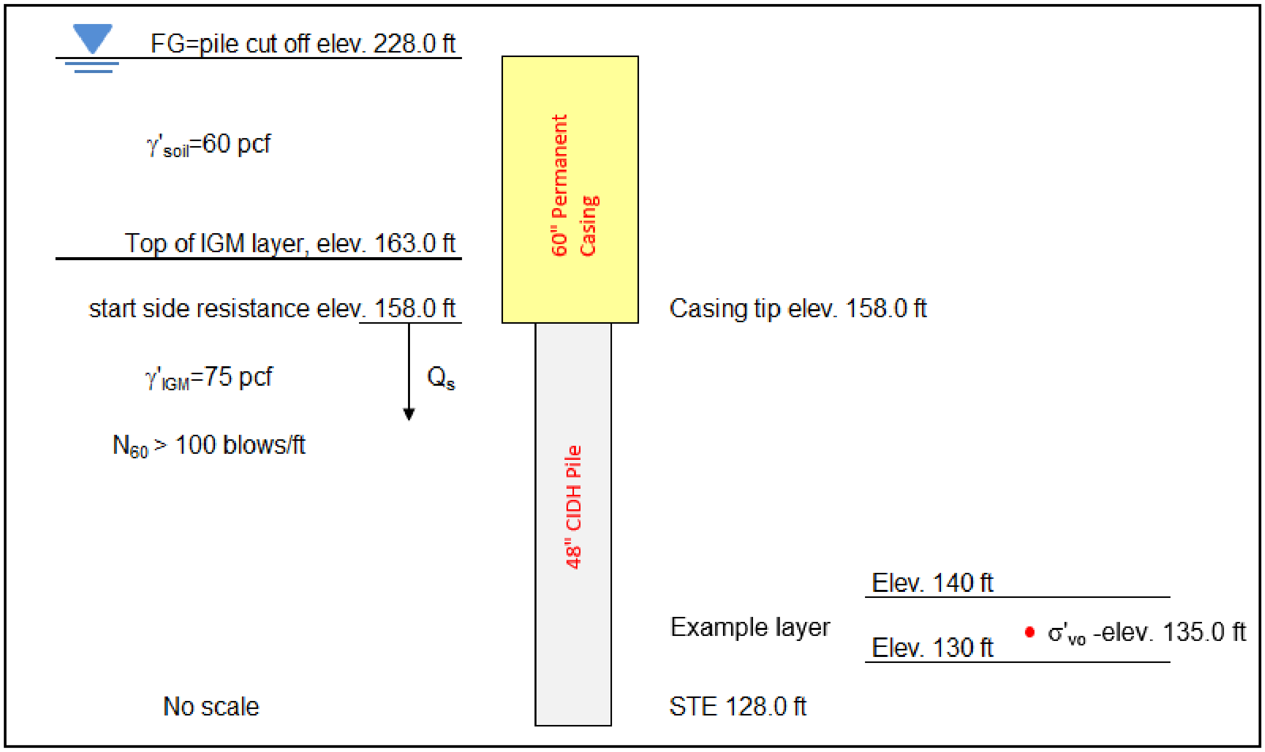
Attachments

- Appendix A: Example Design Calculations for Cohesionless IGM
- Appendix B: Example Design Calculations for Rock
- Appendix C: Example Calculations for Anomaly Evaluation Process

Appendix A: CIDH Pile Design in Cohesionless Intermediate Geomaterial (IGM)

The following presents the design calculations for a 10-foot layer (Elevation 140 feet to 130 feet) located at an abutment. Figure A1 shows the CIDH pile and soil/rock layers.

Figure A1: Simplified Soil/Rock Profile for Abutment 1 Design



Information provided by Bridge Design

- 60-inch Permanent Steel casing is required.
- 48-inch CIDH pile (below casing tip).
- Controlling Factored Design Load = 1146 kips (Strength Limit State).
- The pile cutoff elevation is 228.0 ft.

Geotechnical Design Considerations

- The side resistance of the permanent casing is not used in the design.
- Due to the anticipation that concrete placement for the CIDH piles will require slurry displacement methods, the calculated geotechnical capacity of the piles is based on side resistance of 48-inch diameter CIDH piles and no tip resistance was considered. The design pile side resistance starts at elevation 158.0 ft.
- The required nominal resistance is factored design load divided by the resistance factor of side resistance or 1146 kips / 0.7 = 1637 kips, round up to 1640 Kips.

- Cohesionless IGM design method (for granular geomaterials with SPT N_{60} value greater than 50) will be used to determine pile side resistance.
- Based on the LOTB, representative N_{60} value of 100 was selected for cohesionless IGM design method. N_{60} values should be limited to 100 or less (O'Neill et al., 1996, p.32).
- Assume groundwater surface is at elevation 228.0 ft.

Step 1: Determine the Preconsolidation Pressure (σ'_p):

$$\sigma'_p = 0.2\ N_{60}\ \sigma_p$$

Where:

N_{60} = representative SPT blow count corrected for hammer efficiency effect

σ_p = atmospheric pressure taken as 2120 psf

$$\boldsymbol{\sigma'_p} = 0.2 * 100 * 2120\ \text{psf} = \mathbf{42400\ psf} \quad (N_{60} \leq 100)$$

Step 2: Determine the Overconsolidation Pressure (OCR):

$$\text{OCR} = \frac{\sigma'_p}{\sigma'_{vo}}$$

Where:

σ'_{vo} = effective overburden pressure at mid layer

σ'_{vo} = (60 pcf)(elev. 228 ft -163 ft) + (75 pcf)(elev. 163 ft- 158 ft) + (75 pcf)(elev. 158 ft – 140 ft) + (75 pcf)(10 ft/2) = 6000 psf

$$\mathbf{OCR} = \frac{42400\ \text{psf}}{6000\ \text{psf}} = \mathbf{7.07}$$

Step 3: Determine the Effective Friction Angle (ϕ'):

$$\phi' = \arctan\left\{\left[\frac{N_{60}}{12.2 + 20.3\left(\frac{\sigma'_{vo}}{\sigma_p}\right)}\right]^{0.34}\right\}$$

$$\boldsymbol{\phi'} = \arctan\left\{\left[\frac{100}{12.2+20.3\left(\frac{6000\ \text{psf}}{2120\ \text{psf}}\right)}\right]^{0.34}\right\} = \mathbf{48.5^o}$$

Step 4: Determine the Coefficient of Horizontal Earth Pressure (K_o):

$$K_o = (1 - \sin\phi')\, OCR^{\sin\phi'}$$

$$\mathbf{K_o} = (1 - \sin(48.5°))7.07^{\sin(48.5°)} = \mathbf{1.086}$$

Step 5: Determine the Unit Side Resistance (q_s):

$$q_s = K_o \tan\phi' \, \sigma'_{vo}$$

As the "wet method" (slurry displacement method) is anticipated for the concrete placement, a reduction factor of 0.75 is applied to the effective internal friction angle, ϕ' (O'Neill et al., 1996, p.102).

$\mathbf{q_s} = (1.086)\tan(0.75 * 48.5°)\, 6000\text{ psf} = \mathbf{4800\ psf}$ ("wet method")

Step 6: Determine the Side Resistance (R_s) *of the layer:*

$R_s = q_s\, \pi\, D\, L$

Where:

D = pile diameter

L = layer thickness

R_s = (4800 psf)(3.14)(4 ft)(10 ft) = 602880 lbs = **603 kips**

Table A1 presents side resistance values for the other IGM layers.

Table A1: Side Resistance

Bottom Elevation (feet)	Layer Thickness (feet)	γ' (pcf)	$\sigma'_{v(mid\ layer)}$ (psf)	N_{60}	σ'_p (psf)	OCR	Φ' (degrees)	K_o	q_s (psf)	R_s (kips)
158	70	0	4275[1]	0	0	0	0	0	0	0
150	8	75	4575	100	42400	9.27	50.6	1.27	4530	455
140	10	75	5250	100	42400	8.08	49.6	1.17	4663	586
130	10	75	6000	100	42400	7.07	48.5	1.09	4800	603

1: total overburden

Total = 1645 kips

Design Summary

Additional pile length should be added to the specified tip to account for the untested zone at bottom of pile. For this example, additional 2 ft was added to the required pile length.

Calculated Tip Elevation =	130 feet	(from calculations)
	- 2 feet	(additional length for untested zone)
Design/Specified Tip Elevation =	128 feet	

Foundation Reports for Bridges Section 3.14.3, "Notes for Construction (CIDH Piles)" requires reporting of "*how the geotechnical resistance is derived*." Table A2 presents what would be reported for this example.

Table A2: CIDH Concrete Pile Side Resistance Zone Elevations

Support Location	Side Resistance Start Elevation (feet)	Side Resistance End Elevation (feet)	Specified Tip Elevation (feet)
Abutment 1	158.0	130.0	128.0

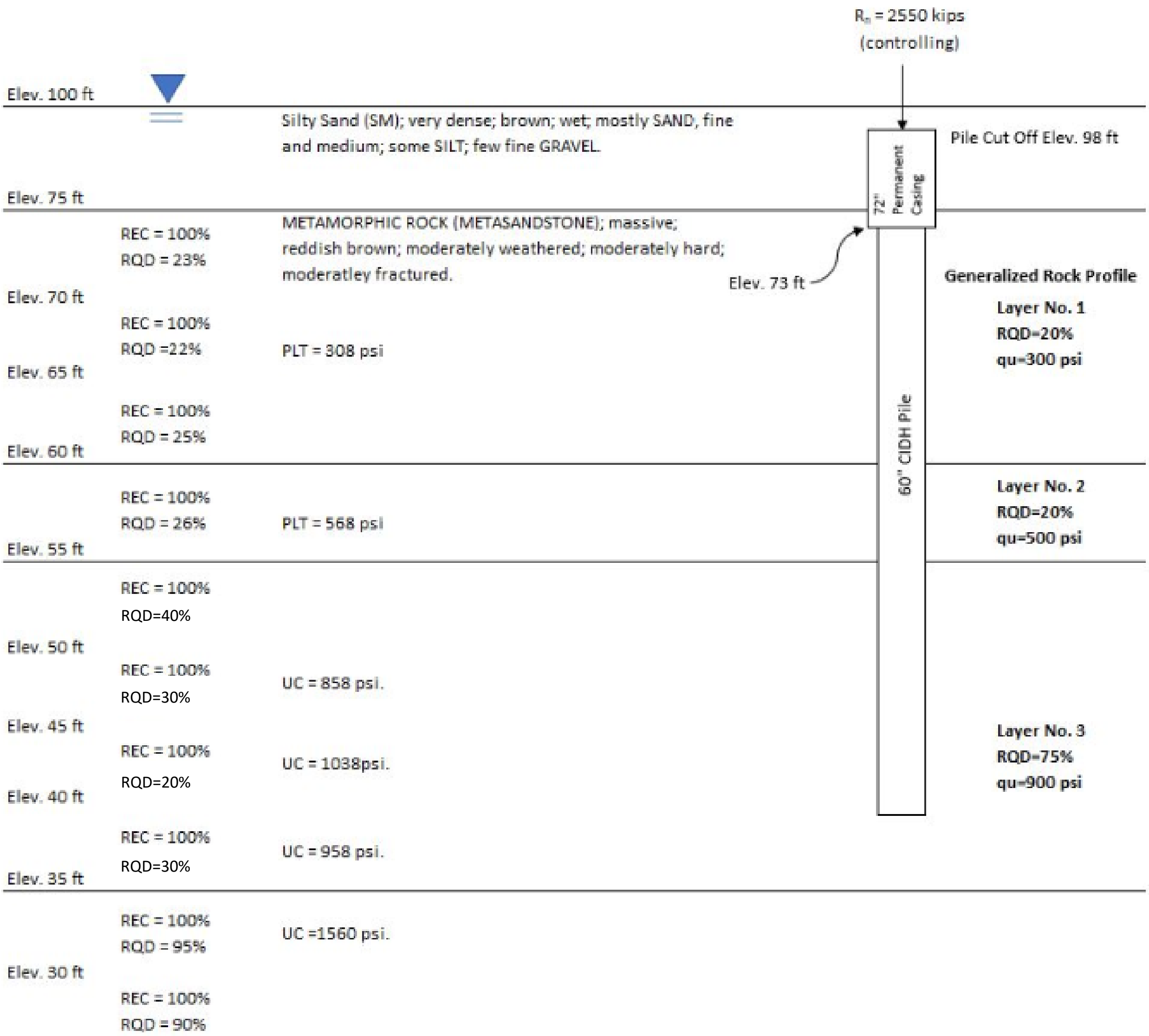
References

O'Neill M., Townsend F., Hassan, K., Buller, A, and Chan P., "Load Transfer for Drilled Shafts in Intermediate Geomaterials," FHWA-RD-95-172, Federal Highway Administration, McLean, VA, 1996

Appendix B: CIDH Pile Design in Rock

Appendix B presents the design calculations for a 60-inch diameter CIDH pile in rock located at Bent 2. Figure B1 shows the CIDH pile and soil/rock layers.

Figure B1: Simplified Soil/Rock Profile Design



<u>Information provided by Bridge Design</u>

- 72-inch Permanent Steel casing is required.
- 60-inch CIDH pile (below casing tip).
- At Bent 2, the Controlling Factored Design Load = 1783 kips (Strength Limit State)
- The pile cutoff elevation is 98.0 ft.

Geotechnical Design Considerations

- The side resistance of 72-inch diameter permanent casing is not used in the design.
- Due to the anticipation that concrete placement for the CIDH piles will require slurry displacement methods, the calculated geotechnical capacity is based on side resistance of 60-inch diameter CIDH piles and no tip resistance was considered in this example.
- The controlling design load is 1783 kips (Strength Limit State). The required nominal resistance is 1783 kips /0.7 = 2547 kips, round up to 2550 Kips.
- The rock joints are closed.
- The fracture spacing contributing to the RQD is predominately 4-6 inches and the fracture orientation is steep. Therefore, AASHTO equation 10.8.3.5.4b-2 is used.

Step 1: Determine the Factored Nominal Resistance (R_R)

$$R_R = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(AASHTO 2017, 10.8.3.5-1)}$$

Where:

R_p = nominal tip resistance

R_s = nominal side resistance

φ_{qp} = resistance factor for tip resistance

φ_{qs} = resistance factor for side resistance

Tip resistance is not used in this design example, therefore:

$$R_R = \varphi_{qs} R_s$$

Step 2: Determine the Unit Side Resistance (q_s) *of CIDH pile in rock*

Due to the low RQD values, assume temporary casing may be used during construction.

$$\frac{q_s}{P_a} = 0.65 \alpha_E \sqrt{\frac{q_u}{P_a}} \qquad \text{(AASHTO 2017, 10.8.3.5.4b-2)}$$

Where:

P_a = atmospheric pressure taken as 2.12 ksf

α_E = joint modification factor (AASHTO 2017, Table 10.8.3.5.4b-1)

q_u = uniaxial compressive strength of rock in ksf

Equation 10.8.3.5.4b-2 becomes:

$q_s = 0.946\alpha_E\sqrt{q_u}$ (q_s, q_u are in ksf)

$q_s = 2.5\alpha_E\sqrt{q_u}$ (q_s, q_u are in psi)

$q_s = 30\alpha_E\sqrt{q_u}$ (q_s, q_u are in psf)

Figure B2: AASHTO Table 10.8.3.5.4b-1

Table 10.8.3.5.4b-1—Estimation of α_E (O'Neill and Reese, 1999)

RDQ (%)	Joint Modification Factor, α_E	
	Closed Joints	Open or Gouge-Filled Joints
100	1.00	0.85
70	0.85	0.55
50	0.60	0.55
30	0.50	0.50
20	0.45	0.45

For Layer No. 1:

Representative rock properties from elev. 73 ft to elev. 60 ft:

RQD = 20 %, use α_E = 0.45 for closed joints.

q_u = 300 psi

Pile nominal side resistance, R_s, calculation:

$$q_s = 2.5\alpha_E\sqrt{q_u} = 2.5(0.45)\sqrt{300\text{psi}} = 19.5 \text{ psi}$$

$$R_s = 19.5 \text{ psi } (3.14)(60 \text{ in})(\text{elev. } 73 \text{ ft} - \text{elev. } 60 \text{ ft})\left(\frac{12 \text{ in}}{1 \text{ ft}}\right)\left(\frac{1 \text{ kip}}{1000 \text{ lb}}\right) = 573 \text{ kips}$$

If the RQD values are below 20% for most of the pile length, an alternative design method, such as the cohesionless Intermediate Geo Material method, should be used. Do not combine side resistances from IGM and rock design methods.

For Layer No. 2:

Representative rock properties from elev. 60 ft to elev. 55 ft:

RQD = 20 %, use α_E = 0.45 for closed joints

q_u = 500 psi

Pile nominal side resistance, R_s, calculation:

$$q_s = 2.5\alpha_E\sqrt{q_u} = 2.5(0.45)\sqrt{500\text{psi}} = 25.2\text{ psi}$$

$$R_s = 25.2\text{ psi }(3.14)(60\text{ in})(\text{elev. }60\text{ ft} - \text{elev. }55\text{ ft})\left(\frac{12\text{ in}}{1\text{ ft}}\right)\left(\frac{1\text{ kip}}{1000\text{ lb}}\right) = 285\text{ kips}$$

For Layer No. 3:

Representative rock properties from elev. 55 ft to elev. 35 ft:

RQD = 30 %, use α_E =0.50 for closed joints

q_u = 900 psi

Pile nominal side resistance, R_s, calculation:

$$q_s = 2.5\alpha_E\sqrt{q_u} = 2.5(0.50)\sqrt{900\text{psi}} = 37.5\text{ psi}$$

Required Nominal Resistance at top of pile is 2550 kips. At top of layer no. 3, elev. 55 ft, the total pile side resistance is 573 kips + 285 kips = 858 kips. The pile design still needs 1692 kips (= 2550 kips – 858 kips) side resistance. Check for the minimum pile length (L_{min}) required in layer no. 3 to meet the 1692 kips in side resistance.

$$L_{min} = \frac{1692\text{ kips}}{(37.5\text{ psi})(3.14)(60\text{ in})\left(\frac{12\text{ in}}{\text{ft}}\right)\left(\frac{1\text{ kip}}{1000\text{ lb}}\right)} = 19.9\text{ ft, say }20\text{ ft}$$

Design pile tip elevation is Elev. 55 ft - 20 ft = 35 ft

Design Summary

Additional pile length should be added to the specified tip to account for the untested zone at bottom of pile. For this example, additional 2 ft was added to the required pile length.

Calculated Tip Elevation =	35 feet	(from calculations)
	- 2 feet	(additional length for untested zone)
Design/Specified Tip Elevation =	33 feet	

Foundation Reports for Bridges Section 3.14.3, "Notes for Construction (CIDH Piles)" requires reporting of "*how the geotechnical resistance is derived*." Table B1 presents what would be reported for this example.

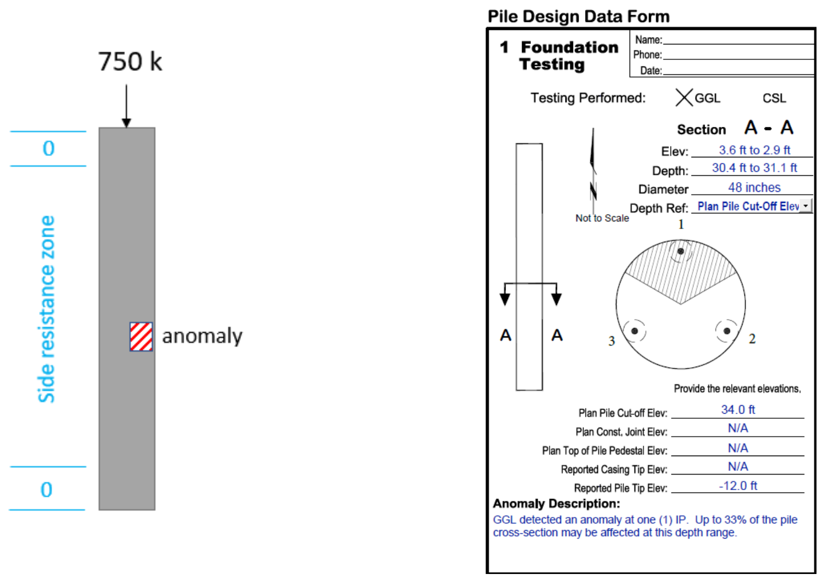
Table B1: CIDH Concrete Pile Side Resistance Zone Elevations

Support Location	Side Resistance Start Elevation (feet)	Side Resistance End Elevation (feet)	Specified Tip Elevation (feet)
Bent 2	73.0	35.0	33.0

Appendix C: Example Calculations for Anomaly Evaluation Process

Example 1: In this example the GP has received the Pile Design Data Form (PDDF) and it indicates that the pile has an anomaly at a depth of 30.4 feet that affects 33% of the pile's cross section. The anomaly is within the pile's Design Side Resistance Zone (Figure C1).

Figure C1: Anomaly Description



Pile Design Data Form

1 Foundation Testing

Name:
Phone:
Date:

Testing Performed: X GGL CSL

Section A - A

Elev: 3.6 ft to 2.9 ft
Depth: 30.4 ft to 31.1 ft
Diameter: 48 inches
Depth Ref: Plan Pile Cut-Off Elev

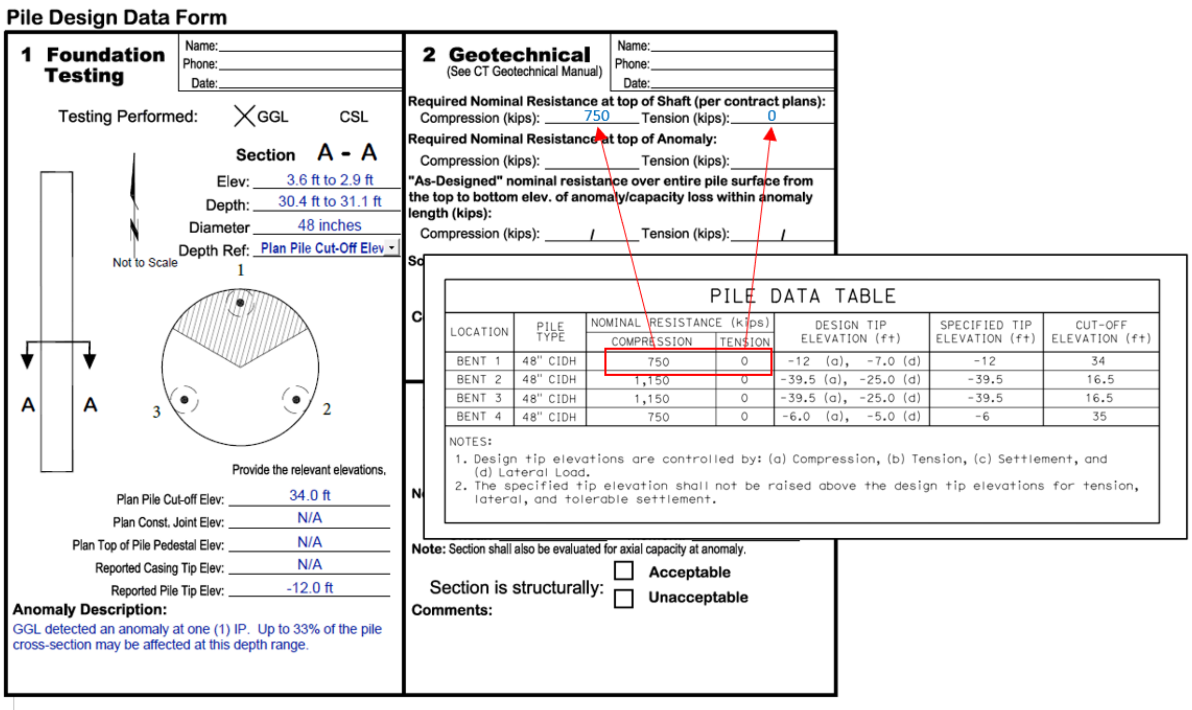
Provide the relevant elevations.

Plan Pile Cut-off Elev: 34.0 ft
Plan Const. Joint Elev: N/A
Plan Top of Pile Pedestal Elev: N/A
Reported Casing Tip Elev: N/A
Reported Pile Tip Elev: -12.0 ft

Anomaly Description:
GGL detected an anomaly at one (1) IP. Up to 33% of the pile cross-section may be affected at this depth range.

Step 1: Determine the required nominal resistance at the top of pile: Use the Pile Data Table or Foundation Report to determine the required nominal resistance (compression and tension) at top of pile (Figure C2).

Figure C2: Required Nominal Resistance at Top of Pile



Pile Design Data Form

1 Foundation Testing

Name:
Phone:
Date:

Testing Performed: X GGL CSL

Section A - A

Elev: 3.6 ft to 2.9 ft
Depth: 30.4 ft to 31.1 ft
Diameter 48 inches
Depth Ref: Plan Pile Cut-Off Elev

Not to Scale

Provide the relevant elevations.

Plan Pile Cut-off Elev: 34.0 ft
Plan Const. Joint Elev: N/A
Plan Top of Pile Pedestal Elev: N/A
Reported Casing Tip Elev: N/A
Reported Pile Tip Elev: -12.0 ft

Anomaly Description:
GGL detected an anomaly at one (1) IP. Up to 33% of the pile cross-section may be affected at this depth range.

2 Geotechnical (See CT Geotechnical Manual)

Name:
Phone:
Date:

Required Nominal Resistance at top of Shaft (per contract plans):
Compression (kips): 750 Tension (kips): 0

Required Nominal Resistance at top of Anomaly:
Compression (kips): Tension (kips):

"As-Designed" nominal resistance over entire pile surface from the top to bottom elev. of anomaly/capacity loss within anomaly length (kips):
Compression (kips): / Tension (kips): /

Sc

C

N

PILE DATA TABLE

LOCATION	PILE TYPE	NOMINAL RESISTANCE (kips) COMPRESSION	NOMINAL RESISTANCE (kips) TENSION	DESIGN TIP ELEVATION (ft)	SPECIFIED TIP ELEVATION (ft)	CUT-OFF ELEVATION (ft)
BENT 1	48" CIDH	750	0	-12 (a), -7.0 (d)	-12	34
BENT 2	48" CIDH	1,150	0	-39.5 (a), -25.0 (d)	-39.5	16.5
BENT 3	48" CIDH	1,150	0	-39.5 (a), -25.0 (d)	-39.5	16.5
BENT 4	48" CIDH	750	0	-6.0 (a), -5.0 (d)	-6	35

NOTES:
1. Design tip elevations are controlled by: (a) Compression, (b) Tension, (c) Settlement, and (d) Lateral Load.
2. The specified tip elevation shall not be raised above the design tip elevations for tension, lateral, and tolerable settlement.

Note: Section shall also be evaluated for axial capacity at anomaly.

Section is structurally: ☐ **Acceptable** ☐ **Unacceptable**

Comments:

Step 2: Determine the required nominal resistance at top of anomaly (Figures C3 and C4).

Figure C3: Nominal Resistance at Top of Anomaly

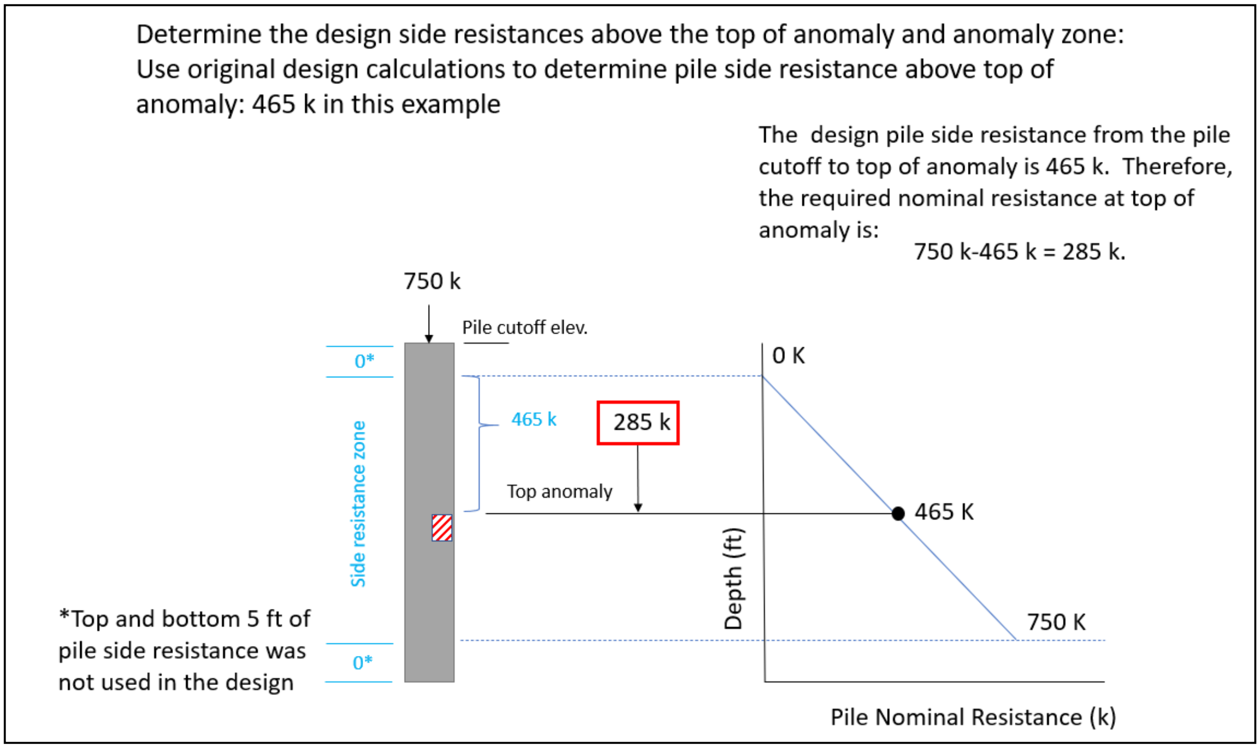
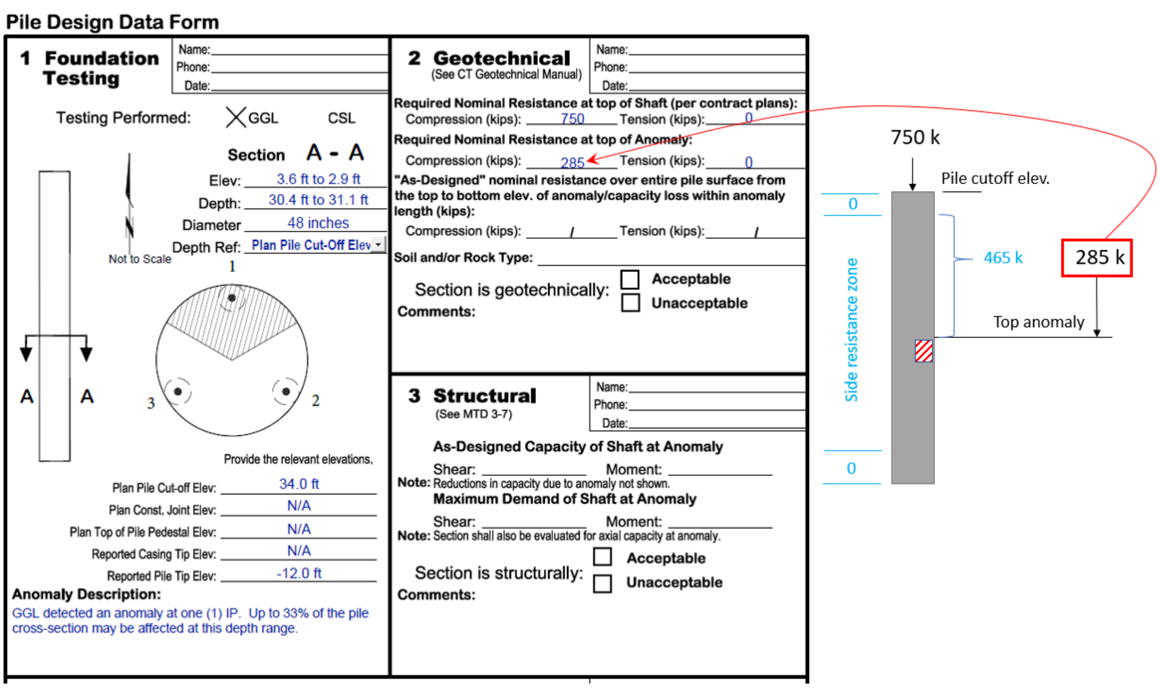


Figure C4: Enter the Required Nominal Resistances on the PDDF



Step 3: Determine the as-designed nominal resistance from top to bottom of anomaly (Figures C5 and C6).

Figure C5: As-built Nominal Resistance from Top to Bottom of Anomaly

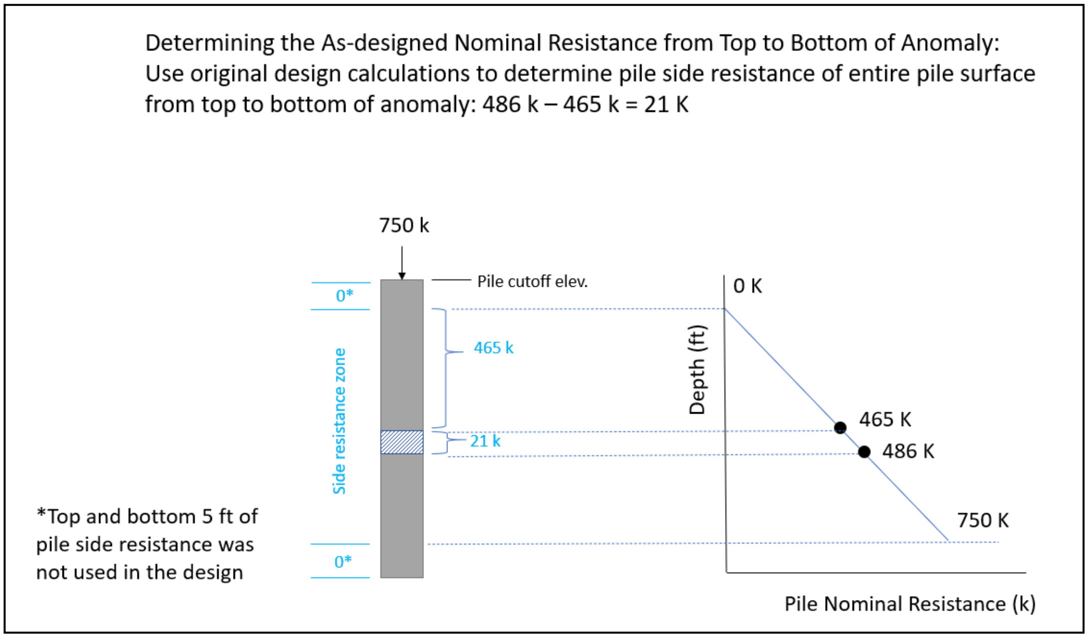
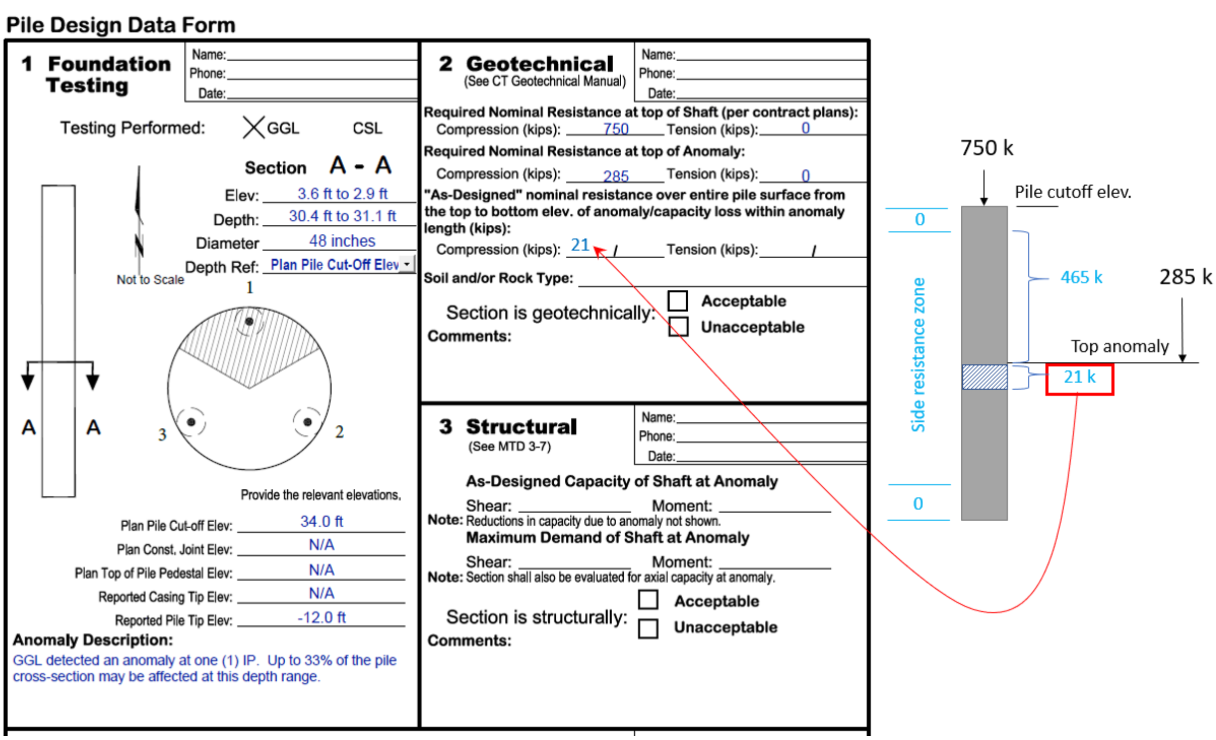


Figure C6: Enter the As-designed loads on the PDDF



Pile Design Data Form

1 Foundation Testing Name: Phone: Date:

Testing Performed: X GGL CSL

Section A - A

Elev: 3.6 ft to 2.9 ft
Depth: 30.4 ft to 31.1 ft
Diameter: 48 inches
Depth Ref: Plan Pile Cut-Off Elev

Not to Scale

Provide the relevant elevations.

Plan Pile Cut-off Elev: 34.0 ft
Plan Const. Joint Elev: N/A
Plan Top of Pile Pedestal Elev: N/A
Reported Casing Tip Elev: N/A
Reported Pile Tip Elev: -12.0 ft

Anomaly Description:
GGL detected an anomaly at one (1) IP. Up to 33% of the pile cross-section may be affected at this depth range.

2 Geotechnical (See CT Geotechnical Manual) Name: Phone: Date:

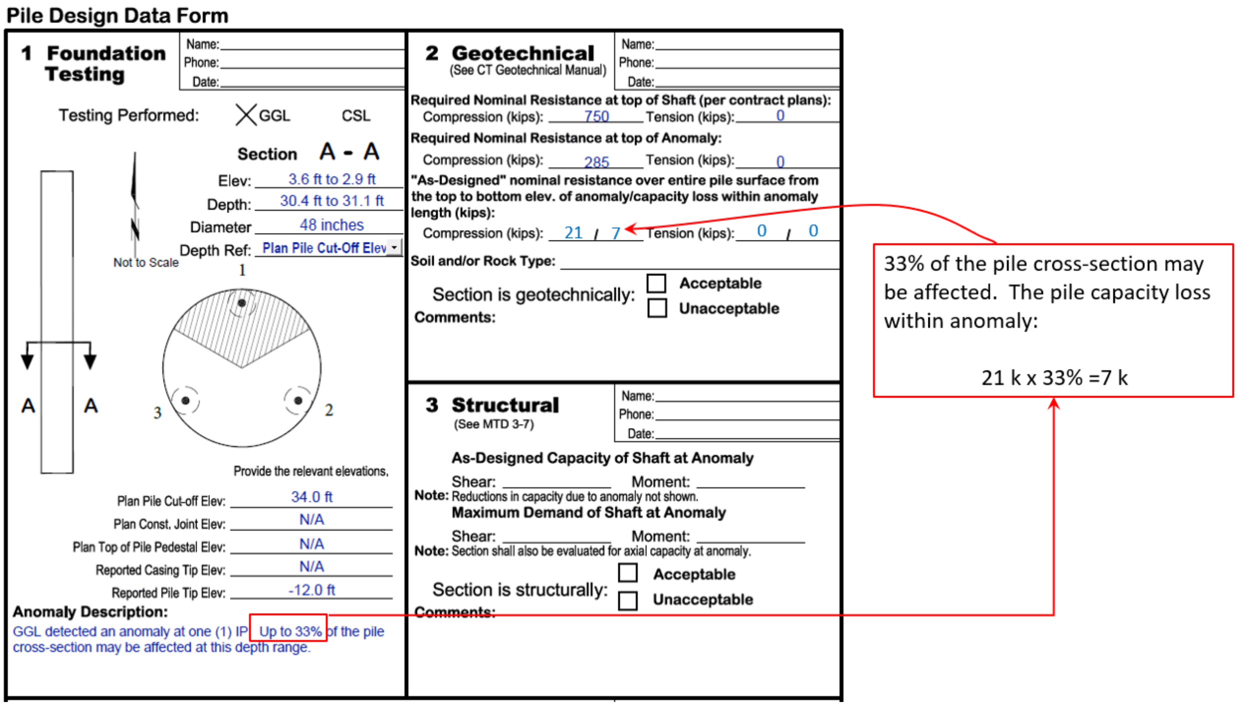
Required Nominal Resistance at top of Shaft (per contract plans):
Compression (kips): 750 Tension (kips): 0
Required Nominal Resistance at top of Anomaly:
Compression (kips): 285 Tension (kips): 0
"As-Designed" nominal resistance over entire pile surface from the top to bottom elev. of anomaly/capacity loss within anomaly length (kips):
Compression (kips): 21 / Tension (kips): /
Soil and/or Rock Type:
Section is geotechnically: ☐ **Acceptable** ☐ **Unacceptable**
Comments:

3 Structural (See MTD 3-7) Name: Phone: Date:

As-Designed Capacity of Shaft at Anomaly
Shear: Moment:
Note: Reductions in capacity due to anomaly not shown.
Maximum Demand of Shaft at Anomaly
Shear: Moment:
Note: Section shall also be evaluated for axial capacity at anomaly.
Section is structurally: ☐ **Acceptable** ☐ **Unacceptable**
Comments:

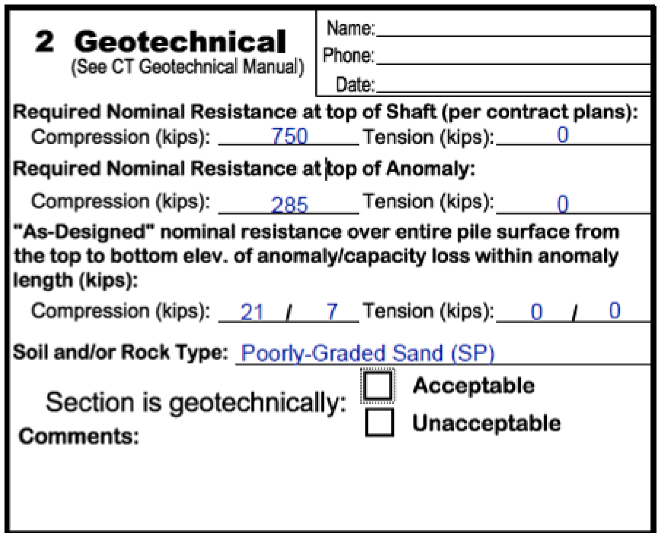
Step 4: Determine the capacity loss within the anomaly length (Figure C7).

Figure C7: Capacity Loss within the Anomaly



Step 5: Use the information on the LOTB sheet to describe the soil and/or rock type within the anomalous zone (Figure C8).

Figure C8: Soil and/or Rock Type Information on PDDF



2 Geotechnical
(See CT Geotechnical Manual)

Name:
Phone:
Date:

Required Nominal Resistance at top of Shaft (per contract plans):
Compression (kips): 750 Tension (kips): 0

Required Nominal Resistance at top of Anomaly:
Compression (kips): 285 Tension (kips): 0

"As-Designed" nominal resistance over entire pile surface from the top to bottom elev. of anomaly/capacity loss within anomaly length (kips):
Compression (kips): 21 / 7 Tension (kips): 0 / 0

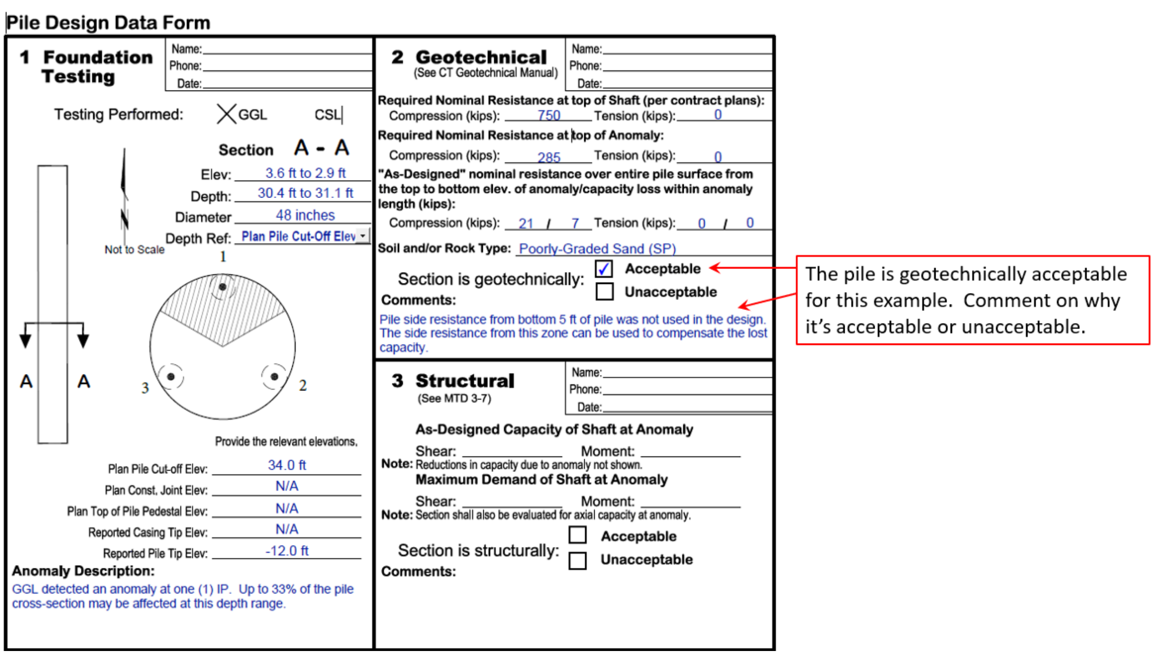
Soil and/or Rock Type: Poorly-Graded Sand (SP)

Section is geotechnically: ☐ **Acceptable** ☐ **Unacceptable**

Comments:

Step 6: Determine if the pile is geotechnically acceptable. Since side resistance from bottom 5 feet of pile was not used in the design, it can be used to compensate for the lost capacity (Figure C9).

Figure C9: The Pile is Geotechnically Acceptable



Pile Design Data Form

1 Foundation Testing

Name:
Phone:
Date:

Testing Performed: ☒ GGL CSL

Section A - A
Elev: 3.6 ft to 2.9 ft
Depth: 30.4 ft to 31.1 ft
Diameter 48 inches
Depth Ref: Plan Pile Cut-Off Elev

Not to Scale

Provide the relevant elevations.
Plan Pile Cut-off Elev: 34.0 ft
Plan Const. Joint Elev: N/A
Plan Top of Pile Pedestal Elev: N/A
Reported Casing Tip Elev: N/A
Reported Pile Tip Elev: -12.0 ft

Anomaly Description:
GGL detected an anomaly at one (1) IP. Up to 33% of the pile cross-section may be affected at this depth range.

2 Geotechnical
(See CT Geotechnical Manual)

Name:
Phone:
Date:

Required Nominal Resistance at top of Shaft (per contract plans):
Compression (kips): 750 Tension (kips): 0

Required Nominal Resistance at top of Anomaly:
Compression (kips): 285 Tension (kips): 0

"As-Designed" nominal resistance over entire pile surface from the top to bottom elev. of anomaly/capacity loss within anomaly length (kips):
Compression (kips): 21 / 7 Tension (kips): 0 / 0

Soil and/or Rock Type: Poorly-Graded Sand (SP)

Section is geotechnically: ☑ **Acceptable** ☐ **Unacceptable**

Comments:
Pile side resistance from bottom 5 ft of pile was not used in the design. The side resistance from this zone can be used to compensate the lost capacity.

3 Structural
(See MTD 3-7)

Name:
Phone:
Date:

As-Designed Capacity of Shaft at Anomaly
Shear: Moment:
Note: Reductions in capacity due to anomaly not shown.
Maximum Demand of Shaft at Anomaly
Shear: Moment:
Note: Section shall also be evaluated for axial capacity at anomaly.

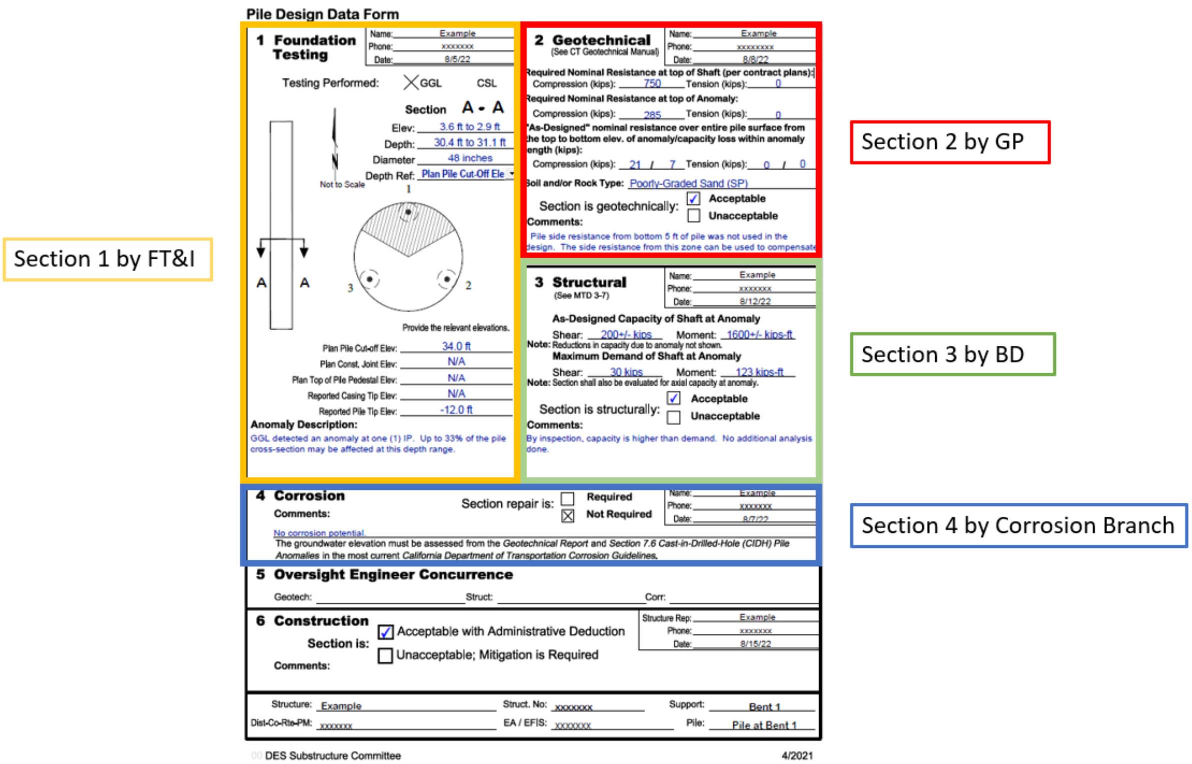
Section is structurally: ☐ **Acceptable** ☐ **Unacceptable**

Comments:

The pile is geotechnically acceptable for this example. Comment on why it's acceptable or unacceptable.

Step 7: Return the Completed PDDF to Structure Construction within one or two working days. Figure C9 presents an example of the completed PDDF.

Figure C10: Example of Completed PDDF (Sections 1-4)



Pile Design Data Form

1 Foundation Testing — Name: Example; Phone: xxxxxxx; Date: 8/5/22

Testing Performed: ☒ GGL CSL

Section A - A
Elev: 3.6 ft to 2.9 ft
Depth: 30.4 ft to 31.1 ft
Diameter: 48 inches
Depth Ref: Plan Pile Cut-Off Ele

Not to Scale

Provide the relevant elevations.
Plan Pile Cut-off Elev: 34.0 ft
Plan Const. Joint Elev: N/A
Plan Top of Pile Pedestal Elev: N/A
Reported Casing Tip Elev: N/A
Reported Pile Tip Elev: -12.0 ft

Anomaly Description:
GGL detected an anomaly at one (1) IP. Up to 33% of the pile cross-section may be affected at this depth range.

2 Geotechnical (See CT Geotechnical Manual) — Name: Example; Phone: xxxxxxxx; Date: 8/8/22

Required Nominal Resistance at top of Shaft (per contract plans):
Compression (kips): 750 Tension (kips): 0
Required Nominal Resistance at top of Anomaly:
Compression (kips): 285 Tension (kips): 0
"As-Designed" nominal resistance over entire pile surface from the top to bottom elev. of anomaly/capacity loss within anomaly length (kips):
Compression (kips): 21 / 7 Tension (kips): 0 / 0
Soil and/or Rock Type: Poorly-Graded Sand (SP)

Section is geotechnically: ☑ Acceptable ☐ Unacceptable
Comments:
Pile side resistance from bottom 5 ft of pile was not used in the design. The side resistance from this zone can be used to compensate

3 Structural (See MTD 3-7) — Name: Example; Phone: xxxxxxx; Date: 8/12/22

As-Designed Capacity of Shaft at Anomaly
Shear: 200+/- kips Moment: 1600+/- kips-ft
Note: Reductions in capacity due to anomaly not shown.
Maximum Demand of Shaft at Anomaly
Shear: 30 kips Moment: 123 kips-ft
Note: Section shall also be evaluated for axial capacity at anomaly.

Section is structurally: ☑ Acceptable ☐ Unacceptable
Comments:
By inspection, capacity is higher than demand. No additional analysis done.

4 Corrosion — Name: Example; Phone: xxxxxxx; Date: 8/7/22

Section repair is: ☐ Required ☒ Not Required
Comments:
No corrosion potential.
The groundwater elevation must be assessed from the *Geotechnical Report* and *Section 7.6 Cast-in-Drilled-Hole (CIDH) Pile Anomalies* in the most current *California Department of Transportation Corrosion Guidelines.*

5 Oversight Engineer Concurrence
Geotech: Struct: Corr:

6 Construction — Structure Rep: Example; Phone: xxxxxxx; Date: 8/15/22

Section is: ☑ Acceptable with Administrative Deduction ☐ Unacceptable; Mitigation is Required
Comments:

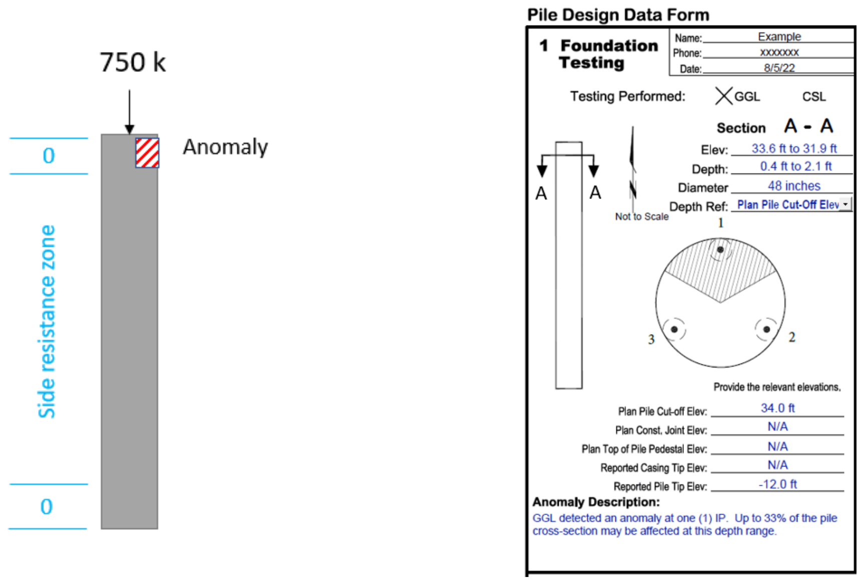
Structure: Example Struct. No: xxxxxxx Support: Bent 1
Dist-Co-Rte-PM: xxxxxxx EA / EFIS: xxxxxxx Pile: Pile at Bent 1

DES Substructure Committee 4/2021

Structures Construction will determine if mitigation is required based on the information provided in Sections 2, 3 and 4. Mitigation is not required when pile is checked acceptable in Sections 2 and 3, and repair for corrosion is not required in Section 4.

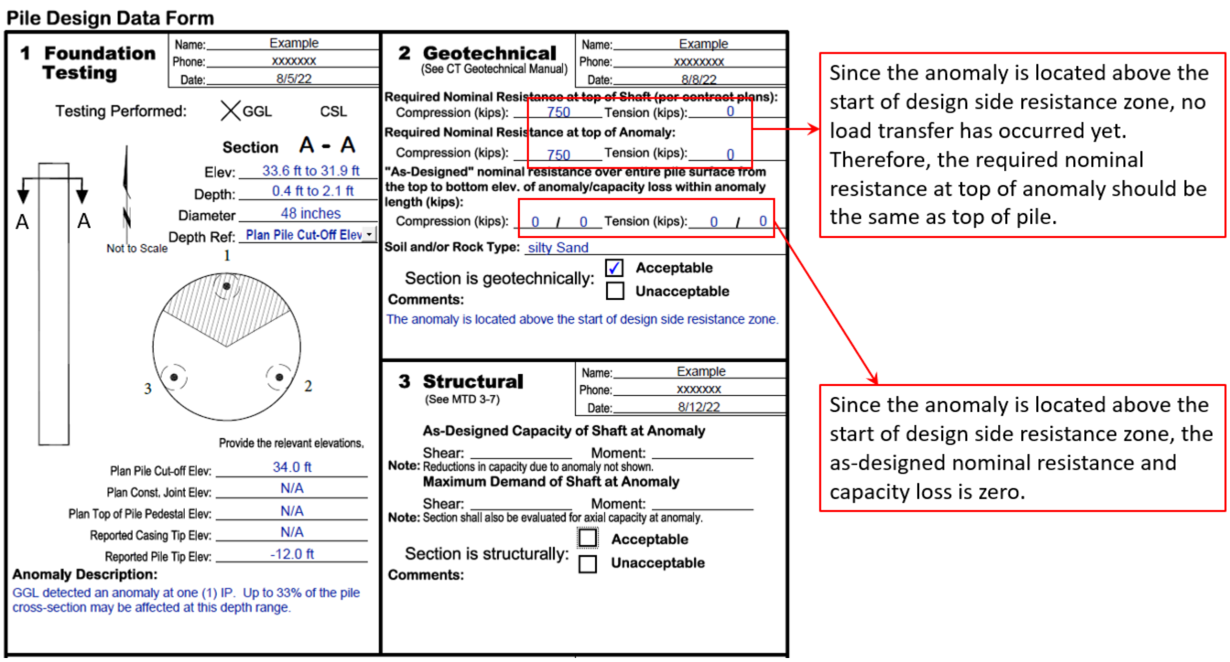
Example 2: In this example the GP has received the PDDF and it indicates that the pile has an anomaly at a depth of 0.4 feet that affects 33% of the pile's cross section. The anomaly is above the pile's Design Side Resistance Zone (Figure C11).

Figure C11: Anomaly Description



Since the anomaly is located above the design side resistance zone, no side resistance load transfer has occurred yet. Therefore, the required nominal resistance at top of anomaly is the same as at top of pile. The as-designed nominal resistance and capacity loss is zero (Figure C12).

Figure C12: PDDF for Anomaly Above Design Side Resistance Zone



Pile Design Data Form

1 Foundation Testing
Name: Example
Phone: xxxxxxx
Date: 8/5/22

Testing Performed: X GGL CSL

Section A - A
Elev: 33.6 ft to 31.9 ft
Depth: 0.4 ft to 2.1 ft
Diameter 48 inches
Depth Ref: Plan Pile Cut-Off Elev

A A
Not to Scale
1 2 3

Provide the relevant elevations.
Plan Pile Cut-off Elev: 34.0 ft
Plan Const. Joint Elev: N/A
Plan Top of Pile Pedestal Elev: N/A
Reported Casing Tip Elev: N/A
Reported Pile Tip Elev: -12.0 ft

Anomaly Description:
GGL detected an anomaly at one (1) IP. Up to 33% of the pile cross-section may be affected at this depth range.

2 Geotechnical (See CT Geotechnical Manual)
Name: Example
Phone: xxxxxxxx
Date: 8/8/22

Required Nominal Resistance at top of Shaft (per contract plans):
Compression (kips): 750 Tension (kips): 0
Required Nominal Resistance at top of Anomaly:
Compression (kips): 750 Tension (kips): 0
"As-Designed" nominal resistance over entire pile surface from the top to bottom elev. of anomaly/capacity loss within anomaly length (kips):
Compression (kips): 0 / 0 Tension (kips): 0 / 0
Soil and/or Rock Type: silty Sand

Section is geotechnically: ☑ **Acceptable** ☐ **Unacceptable**
Comments:
The anomaly is located above the start of design side resistance zone.

3 Structural (See MTD 3-7)
Name: Example
Phone: xxxxxxx
Date: 8/12/22

As-Designed Capacity of Shaft at Anomaly
Shear: Moment:
Note: Reductions in capacity due to anomaly not shown.
Maximum Demand of Shaft at Anomaly
Shear: Moment:
Note: Section shall also be evaluated for axial capacity at anomaly.

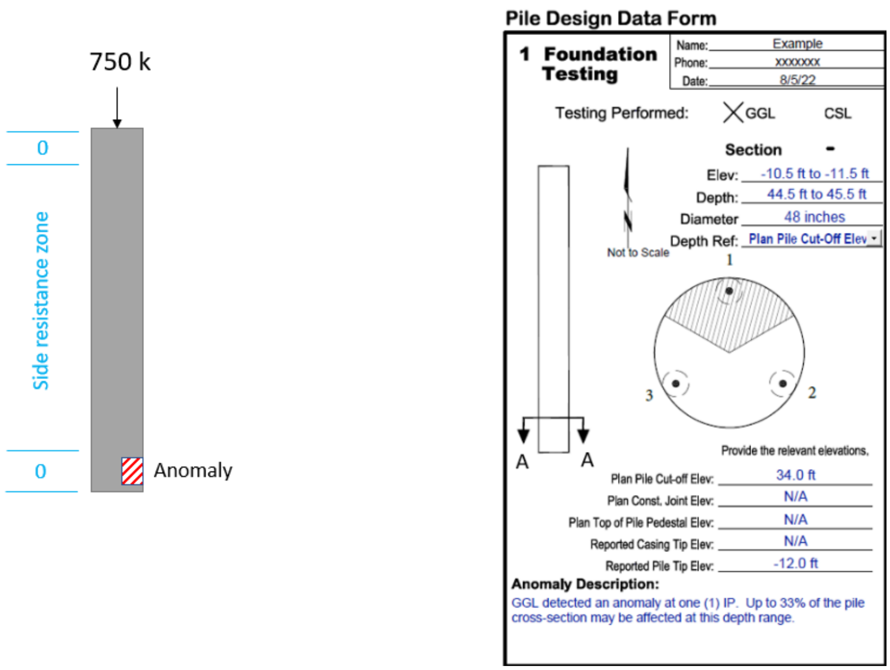
Section is structurally: ☐ **Acceptable** ☐ **Unacceptable**
Comments:

Since the anomaly is located above the start of design side resistance zone, no load transfer has occurred yet. Therefore, the required nominal resistance at top of anomaly should be the same as top of pile.

Since the anomaly is located above the start of design side resistance zone, the as-designed nominal resistance and capacity loss is zero.

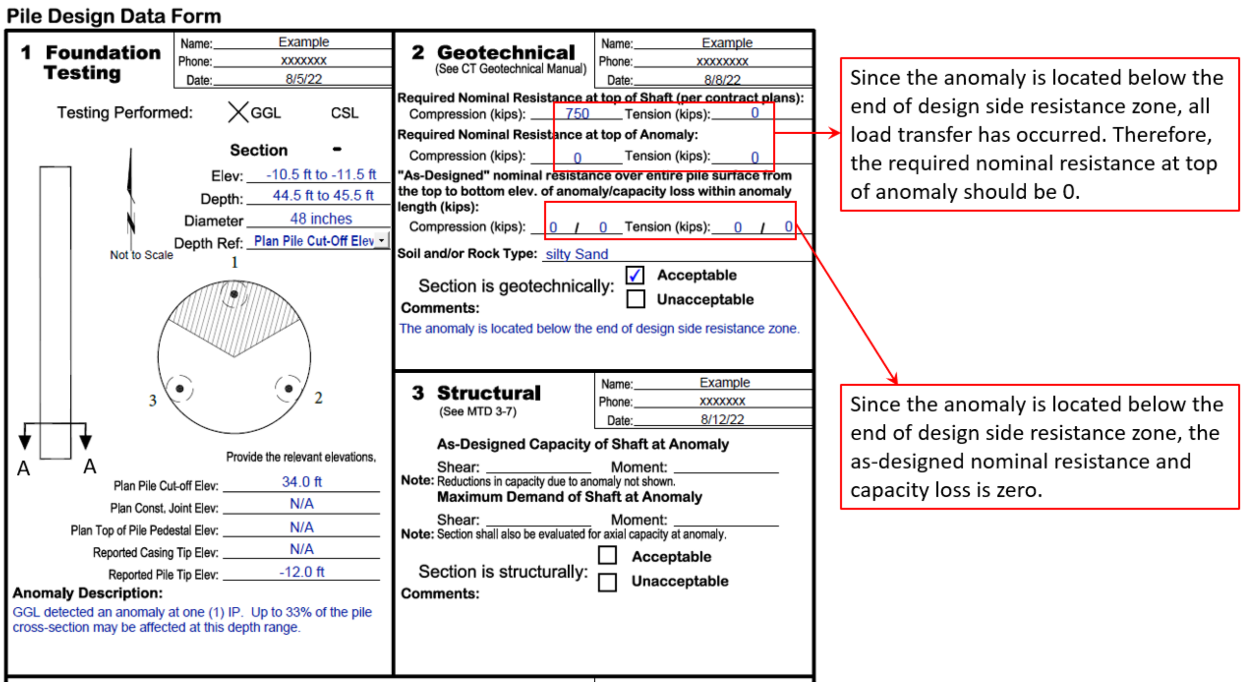
Example 3: In this example the GP has received the PDDF and it indicates that the pile has an anomaly at a depth of 44.5 feet that affects 33% of the pile's cross section. The anomaly is below the pile's Design Side Resistance Zone (Figure C13).

Figure C13: Anomaly Description



Since the anomaly is located below the design side resistance zone, all side resistance load transfer has occurred. Therefore, the required nominal resistance at top of anomaly should be 0. The as-designed nominal resistance and capacity loss is zero (Figure C14).

Figure C14: PDDF for Anomaly Below Design Side Resistance Zone


Pile Design Data Form

1 Foundation Testing
Name: Example
Phone: xxxxxxx
Date: 8/5/22

Testing Performed: X GGL CSL

Section -
Elev: -10.5 ft to -11.5 ft
Depth: 44.5 ft to 45.5 ft
Diameter 48 inches
Depth Ref: Plan Pile Cut-Off Elev
Not to Scale

Provide the relevant elevations.
Plan Pile Cut-off Elev: 34.0 ft
Plan Const. Joint Elev: N/A
Plan Top of Pile Pedestal Elev: N/A
Reported Casing Tip Elev: N/A
Reported Pile Tip Elev: -12.0 ft

Anomaly Description:
GGL detected an anomaly at one (1) IP. Up to 33% of the pile cross-section may be affected at this depth range.

2 Geotechnical
(See CT Geotechnical Manual)
Name: Example
Phone: xxxxxxxx
Date: 8/8/22

Required Nominal Resistance at top of Shaft (per contract plans):
Compression (kips): 750 Tension (kips): 0
Required Nominal Resistance at top of Anomaly:
Compression (kips): 0 Tension (kips): 0
"As-Designed" nominal resistance over entire pile surface from the top to bottom elev. of anomaly/capacity loss within anomaly length (kips):
Compression (kips): 0 / 0 Tension (kips): 0 / 0
Soil and/or Rock Type: silty Sand
Section is geotechnically: ☑ Acceptable ☐ Unacceptable
Comments:
The anomaly is located below the end of design side resistance zone.

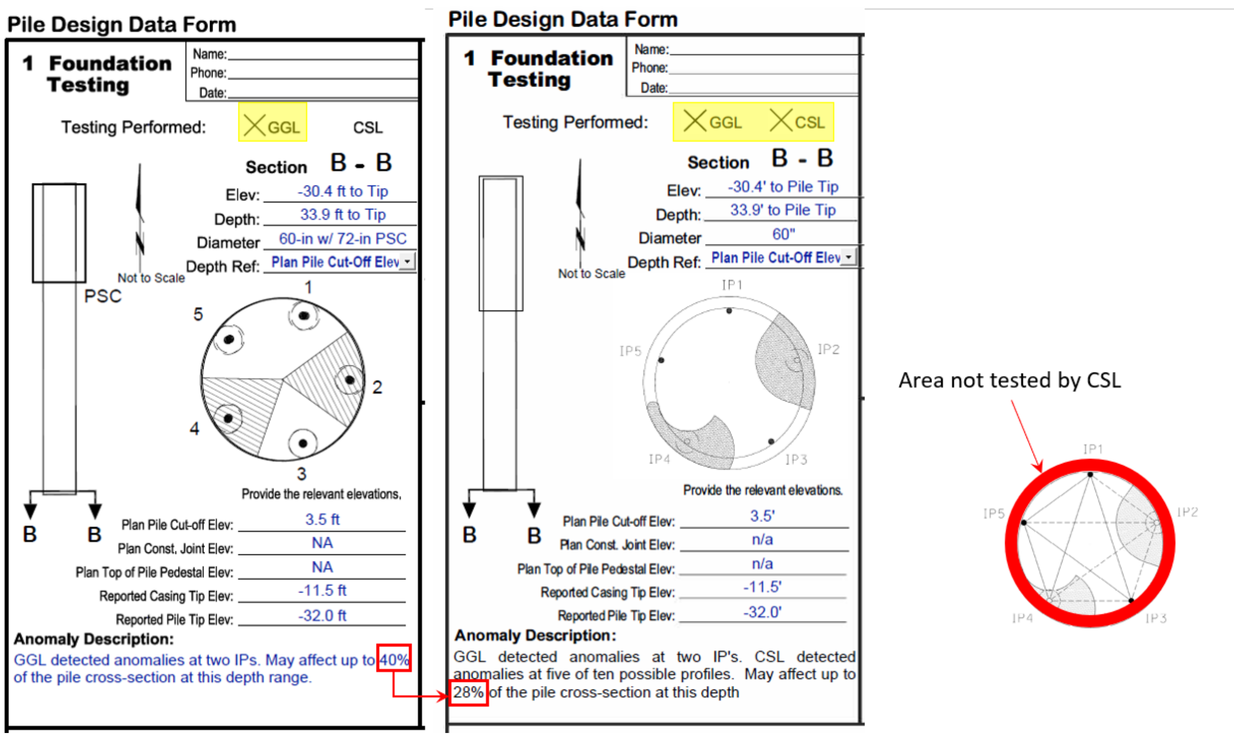
3 Structural
(See MTD 3-7)
Name: Example
Phone: xxxxxxx
Date: 8/12/22

As-Designed Capacity of Shaft at Anomaly
Shear: ______ Moment: ______
Note: Reductions in capacity due to anomaly not shown.
Maximum Demand of Shaft at Anomaly
Shear: ______ Moment: ______
Note: Section shall also be evaluated for axial capacity at anomaly.
Section is structurally: ☐ Acceptable ☐ Unacceptable
Comments:

Example 4: Combined GGL-CSL Test Results

FT&I will perform GGL first and may follow up with a CSL to better define the anomalous area inside the rebar cage. CSL cannot test the annulus space between borehole walls and rebar cage. A comparison of GGL and GGL-CSL test results is shown in Figure C15.

Figure C15: Comparison of GGL and GGL-CSL Test Results

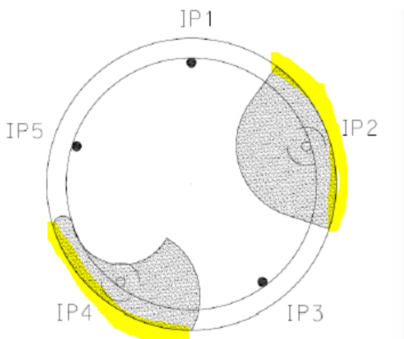


In this example, for determining the side resistance loss, assume 40% of the pile surface area is anomalous. The pile surface area is not tested by CSL, therefore, the CSL test result will not decrease the pile surface anomalous area detected by GGL. For determining the tip resistance loss, assume 28% of pile cross section is anomalous (Figure C16).

Figure C16: Side Resistance Loss and Tip Resistance Loss of GGL-CSL Testing

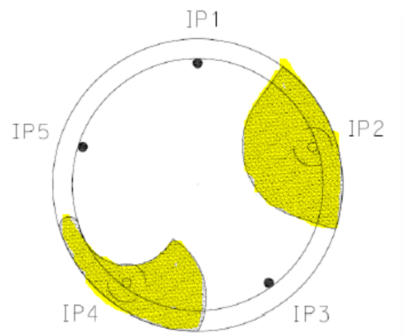
Side Resistance

For determining the side resistance loss, assume 40% of pile surface area is anomalous.



Tip Resistance

For determining the tip resistance loss, assume 28% of pile cross section is anomalous.



References

1. MTD 3-7 – Design Data Documentation and Evaluation of Anomalous Concrete Shafts
2. BCM 130-10 – Testing of CIDH Piling
3. BCM 130-20 – Cast-In-Drilled-Hole (CIDH) Pile Preconstruction Meeting
4. BCM 130-21 – CIDH Pile Non-Standard Mitigation Meeting
5. California Test (CT) 233 – Method of Ascertaining the Homogeneity of Concrete in Cast-In-Drilled-Hole (CIDH) Piles Using the Gamma-Gamma Test Method