



3-1 DEEP FOUNDATIONS

Deep foundations are structural components that in comparison to shallow foundations transfer load into deeper layers of earth materials. Deep foundations, generically referred to herein as piles, can be driven piles, drilled shafts, micropiles, and grouted-in-place piles. Vertical ground anchors (tie-downs) are also classified as deep foundations. Piles can be used in a group with a cap footing, or as a single pile/shaft supporting a column.

Structure Designer (SD) is responsible for calculating the pile factored design loads and for providing structural details. Geotechnical Services (GS) of Materials Engineering and Testing Services and Geotechnical Services (METS-GS) is responsible for providing foundation recommendations that include site seismicity, factored downdrag loads, pile tip elevations (based on the factored design loads provided by SD), construction recommendations (pile acceptance criteria, testing requirements, etc.), and the Log of Test Borings. SD and Geotechnical Designer (GD) will determine pile type, size and special construction requirements, if any. SD is responsible for ensuring that the intent of the geotechnical and structural design is preserved in the contract plans and specifications. At the submittal of Plans and Quantities (P&Q), any information absent from the Foundation Recommendations should be included in the project engineer's Memo to Specifications Engineer. If necessary, a meeting with the specifications engineer, the GD and SD should occur to discuss the foundation related specifications. When draft specifications are available, review of plans and specifications by GS completes the Plans, Specs and Estimates (PS&E) process, allowing GS to verify concurrence between the plans and Foundation Recommendations.

Caltrans' practice is to design abutments and bents/piers in accordance with the Load and Resistance Factor Design (LRFD) as specified in the current *AASHTO LRFD Bridge Design Specifications* (LRFD BDS), with California Amendments. The SD needs to provide information and controlling factored loads for each limit state, so that the GD can provide a design to meet or exceed the load demands. The GD will determine the required nominal resistance and resistance factors for the applicable limit states. This information is used to calculate nominal resistance that will be shown in the Pile Data Table on the contract plans.

Standard Plan Piles

The Standard Plans, Sheets B2-3 (16" AND 24" CAST-IN-DRILLED-HOLE CONCRETE PILE), B2-5 (PILE DETAILS CLASS 90 AND CLASS 140), and B2-8 (PILE DETAILS CLASS 200) provide the upper limit of nominal axial structural resistance in tension and compression.

When Standard Plan Class piles are specified, and unless otherwise specified in the Standard Plans or the contract Special Provisions, the contractor has the option of using any of the alternatives for that class of pile. Should any of the alternative standard piles be undesirable, that alternative must be disallowed in the contract Special Provisions or on the Pile Data Table.

Special Consideration for Alternative ‘X’ Piles

The 12-inch square precast prestressed Class 90 and Class 140 concrete piles, Alternative ‘X’, do not have the lateral capacity necessary for the pile spacing design charts in Section 6 of the *Bridge Design Details* (BDD) manual for either Strutted Abutments or Cantilever Abutments. If these design charts are used, the foundation report and Special Provisions must stipulate that Alternative ‘X’ piles have a dimension ‘T’ not less than 14 inches for the specific locations involved. This information should be included in the Memo to Specifications Engineer.

Permissible (Allowable) Horizontal Load

The permissible horizontal load for deep foundations at abutments is the horizontal load that results in a horizontal displacement of 0.25 in. at the top or cut-off elevation of the pile/shaft. The pile/shaft horizontal force under LRFD Service-I Limit State load combination must be less than the permissible horizontal load. Where standard plan piles are used, the pile-to-cap connection is intended to be a pin connection. In the case of battered piles, the horizontal component of a battered pile’s axial load may be subtracted from the total lateral load to determine the applied horizontal or lateral loads on pile foundations.

Driven Piles

Driven piles may be precast prestressed concrete, cast-in-steel-shell (CISS) concrete, rolled HP sections, steel pipe or timber. Piles with a solid cross section that displace the soil around the pile during driving are classified as displacement piles. Open cross sections, such as steel HP piles, and open ended pipe piles, will either displace the soil or cut through the soil (non-displacement) depending on the diameter of the pile, properties of the soil and depth of pile penetration. Typically, steel HP piles and open-ended pipe piles 24 inches and greater in diameter are non-displacement piles. Such piles are useful for penetration where hard driving conditions are expected.

Site specific issues including noise, vibration, ground heave, settlement, limited headroom, constructability, and drivability must be considered when selecting driven piles. Liquefaction, scour potential, or other conditions may control the calculated specified tip elevation, and therefore the nominal driving resistance may exceed controlling nominal resistance. In that case, GS may perform a drivability analysis to verify that the piles can be driven to the specified tip elevation with acceptable driving stresses and blow counts.

Steel HP Piles

Steel HP sections are usually specified where displacement piles cannot penetrate foundation materials such as rock, cobbles, gravel, and dense sand. Steel HP sections are also preferable for longer piles because they can be spliced more easily than precast prestressed concrete piles. Steel HP piles may not be feasible where highly corrosive soils and/or waters are encountered.



If steel HP piles are allowed as an alternative to a Standard Plan Class pile, the SD shall provide allowable HP sizes to the Specification Engineer. The HP 14x89 steel pile is usually specified for nominal axial structural resistance (in compression) of 400 kip, HP 10x57 for 280 kip and HP 10x42 for 180 kip. The SD should note in the Memo to Specification Engineer when other steel sections are acceptable for substitution, and verify with the Cost Estimating Branch that a recommended nonstandard HP section is available. Larger pile sections may be required if increased lateral load resistance is needed, or hard driving is anticipated. Pile anchors must be designed for the applied load and detailed on the plans. Anchor bars shall be epoxy-coated.

Cast-in-Steel-Shell (CISS) Concrete Piles and Steel Pipe Piles

Cast-in-steel-shell concrete piles are driven pipe piles that are filled with cast-in-place reinforced concrete no deeper than the shell tip elevation. CISS piles provide excellent structural resistance against horizontal loads and are a good option under the following conditions: 1) where poor soil conditions exist, such as soft bay mud deposits or loose sands; 2) if liquefaction or scour potential exist that will cause long unsupported pile lengths; or 3) if large lateral soil movements are anticipated.

If composite action is required for flexural capacity, the design engineer must assure that a reliable shear transfer mechanism exists. Welded studs or shear rings may be required, especially for large diameter piles.

CISS piles and steel pipe piles can be open ended or closed ended. Caution should be exercised when requiring closed end pipe piles to penetrate very dense granular soils, very hard cohesive soils or rock. Generally, open-ended pipe piles up to 16 inches in diameter tend to plug during driving while diameters 24 inches and greater tend not to plug. Once plugged, an open-ended pipe behaves like a displacement pile and driving becomes more difficult. When excessive blow counts or high driving stresses are anticipated or encountered, GD may recommend center relief drilling through open-ended pipe piles to penetrate to the specified tip elevation. When appropriate, GS will perform a drivability analysis and recommend a pile wall thickness suitable for the expected driving stresses.

A soil plug is left intact at the bottom of the open-ended CISS piles so that the pile is not undermined during the cleaning process. A plug two diameters in length can usually maintain water control, but a seal course may be required for some combinations of high hydraulic head and permeable soils. For geotechnical purposes it is desirable to leave as much of the soil plug in place as possible. The GD may recommend a minimum soil plug thickness.

Timber Piles

Timber piles can be specified where conditions are suitable, usually for temporary construction (e.g. railroad shoofly trestles). For timber piles to be used in permanent construction the pile cut-off must be below the lowest possible ground water level and there must be no exposure to marine borers. Because of their flexibility, low ductility, and difficult cap connections, timber piles are not permitted where seismic considerations are critical.

The nominal resistance for timber piles is 180 kips. Pile information for timber piles should be detailed on the contract plans, similar to other types of driven piles.

Drilled Shafts

Drilled shafts also known in the industry as cast-in-drilled-hole (CIDH) piles or caissons, are a possible solution when driven piles are not suitable, large vertical or lateral resistance is required, or to address constructability issues. A CIDH pile is more advantageous than a driven pile when installation noise and/or vibration are concerns, but disposal of hazardous drill spoils may be costly. When battered piles are required, CIDH piles should not be used because of the increased risk of caving and the difficulty of placing concrete and reinforcement in a sloping hole.

To conform to the specifications, the plans must call out “steel casings” when being used for constructability even though contributing to structural capacity of the pile, and call out “driven steel shell” when providing extra geotechnical axial resistance. Casings can be specified to be left in place permanently or to be removed during concrete placement. A casing may be driven, drilled, vibrated, or oscillated into place, whereas a shell must be driven into place. Considerations for driving stresses and center relief drilling for shells are the same as for CISS and steel pipe piles.

Both tip and side resistances for CIDH piles are developed in response to pile vertical displacement. The maximum or peak resistance values are seldom cumulative because they are not likely to occur at the same displacement. Since side resistance is usually mobilized at small displacement, CIDH piles rely on this component of resistance for most of their capacity, in particular under Service Limit State load. Displacement compatibility must be considered when adding tip resistance and side resistance. The mobilization of tip resistance for CIDH piles is uncertain. The situation is worse in wet method construction conditions, where soft, compressible drill spoils and questionable concrete quality are both possible at the pile tip. GD recommendations may discard or include only a fraction of the tip resistance, especially in wet method construction conditions.

When ground water is anticipated, CIDH piles must be at least 24 inches in diameter and designed to accommodate the construction techniques associated with drilled shafts in wet holes. PVC inspection pipes are installed to permit Gamma-Gamma Logging (GGL) and Cross-hole Sonic Logging (CSL) tests of these CIDH piles. See Attachment 2 for reinforcing steel clearance requirements in conjunction with inspection pipes.



When considering CIDH piles in wet conditions, caution should be exercised in the following cases: 1) lack of redundancy in single column bents; 2) soft cohesive soils, loose sands, or boulders at the support location (constructability); and 3) presence of high ground water pressure that will make it difficult to establish a differential water pressure head for slurry construction. Driven piles should be considered for these situations. If driven piles cannot be used, the designer should anticipate the possibility of defective, non-repairable piles. In such a case, the Pile Mitigation Plan (Bridge Construction Memo 130-12.0) will require replacement or supplemental piles, the location of which should be anticipated in the design phase (see MTD 3-7).

For 5-foot diameter and larger Type-II shafts (*Caltrans Seismic Design Criteria, 7.7.3.5*) a construction joint is required below the embedded column rebar cage. The construction joint will allow the contractor to place the CIDH concrete without supporting the column rebar cage and to cast the rest of the Type-II shaft and column concrete in a dry condition. It is important that District and GD be notified since the construction joint will require the placement of a permanent casing in the hole to allow workers to prepare the joint. The GD may want to exclude this zone from the geotechnical capacity calculations and the District Project Engineer will need to obtain a site classification from the Cal-OSHA Mining and Tunneling Unit, as described in the *Highway Design Manual* Topic 110. The plans must show the location of the construction joint, and the Special Provisions must describe the required joint preparation. The Contractor may elect to eliminate the construction joint and associated permanent casing, provided that the concrete is not placed under slurry. A note must be added to the CIDH plans indicating this exception.

When Standard Plan CIDH piles are specified for prestressed concrete bridges that utilize a diaphragm type abutment, the detail shown in *Bridge Design Aids* (BDA), page 1-4, “DIAPHRAGM ABUTMENT WITH FOOTING” should be used. Displacement due to superstructure prestress shortening creates undesirable stresses in the stiff CIDH piles. (See MTD 5-2 for more information.)

To ensure constructability and quality, the length of CIDH piles should be limited to 30 times the pile diameter. When caving conditions exist, the use of a permanent casing should be considered and discussed with the GD. A permanent casing might also be required for CIDH piles very near to utilities or traffic (especially in medians) where caving would threaten existing facilities. To prevent binding of the drilling tool, the casing diameter should be at least 8 inches greater than the CIDH pile or rock socket diameter. When permanent steel casing with internal shear connectors is used, the casing diameter should be increased accordingly.

The standard specification that allows the contractor to revise specified pile tip elevations is intended for driven piles. Tip elevation revision is generally not allowed for drilled shafts. An engineered length for skin friction resistance primarily controls the specified CIDH pile depth and the drilling process has no inherent measurement (analogous to a blow count) to verify this resistance. Constructing to the specified tip elevation (when controlled by lateral loads) is particularly important for single column bents, where a change in the pile’s lateral stiffness could affect the dynamic response of the entire structure.

The contract items for CIDH piles are as shown in BDA Chapter 11. Standard sizes for CIDH augers and steel shells/casings are shown in Attachment 3. These sizes are preferred for CIDH piles with or without a steel shell or permanent casing. Because the contractor owns and reuses temporary casings, the indicated sizes are required whenever a temporary casing is expected.

Rock Sockets

Rock sockets are drilled shafts that require drilling and excavation into rock. Rock sockets are generally utilized to transfer structural loads into rock overlain by soil and/or overburden materials. Advances in drilling technologies and equipment have allowed contractors new methods of advancing holes through rock without the need for blasting and mining methods, as used to construct pier columns. In cases where rock drilling is anticipated and conventional drilling (e.g. soil augers) may not be effective in advancing the hole, the rock socket zone should be identified and the pay limits of rock socket should be shown on the plans. Additionally, pile cut-off and tip elevations should be shown in the Pile Data Table. Rock sockets can develop large geotechnical nominal resistances in relatively short socket lengths, therefore field inspection during construction by the GD may be necessary to verify that the desired length and rock conditions are as anticipated.

Pier Columns

Pier columns are utilized when site conditions indicate that excavation by hand, blasting, and mechanical/chemical splitting of the rock is needed.

Pier column excavation in rock is usually more expensive than drilling methods, and the pay limits must be clearly defined. The pier column cut-off elevation and tip elevation (upper and lower limits of the hard material) should be shown in the Pile Data Table. The pay limits for Structure Excavation (Pier Column) and Structure Concrete (Pier Column) shall be shown on the plans. (See BDD page 7-20 and BDA Chapter 11 for details.)

Pile Extensions

The standard plan piles (Class 90, 140 and 200) are not to be used for pile extensions. When pile extensions are preferable, the pile and the extension shall be designed as a column. Upon request, the GD will provide soil profile data for the designer's analysis of the lateral response of the pile extension. The contract plans should give the option to furnish and drive full-length precast prestressed piles or steel pipe piles. An extended pipe pile should be filled with reinforced concrete from at least one foot below finished grade up to the bent cap. Special seismic detailing may be required to control plastic hinge locations in these pipe pile extensions.



Vertical Ground Anchors (Tie-downs)

Vertical ground anchors (also known as tie-downs) are often used where piles are not economical or feasible due to site conditions. For example, where rock exists close to the ground surface, piles driven to refusal may be too short to develop sufficient geotechnical tensile capacity to resist uplift forces. Vertical ground anchors are very effective when combined with spread footings founded on rock, or when used as part of a seismic retrofit system to resist uplift forces.

Vertical ground anchors require no entries in the Pile Data Table. The SD is responsible for calculating the factored design load, (FDL), for each applicable limit state. The GD is responsible for specifying the minimum unbonded length of prestressing strands or bars and the pullout resistance factor, ϕ , for each applicable limit state. The Contractor is responsible for determining the bonded length of prestressing strands or bars and the vertical ground anchors installation method and for complying with the acceptance criteria specified in the Standard Specifications, Section 46.

Performance testing of a number of vertical ground anchors (to be determined by the GD) and proof testing of all the other anchors are required to 100% of the factored test load, (FTL), where $FTL = FDL/\phi$. For the extreme event limit state where a vertical ground anchor is a failure-critical member, that is, the failure of an anchor may result in the collapse of the bridge, the factored design load should be increased by 5%. For each vertical ground anchor, the factored test load and the lock-off load shall be indicated on the contract plans. Vertical ground anchors can either be active or passive depending on the lock-off load in the prestressing strands or bars. Passive anchors are usually prestressed to a lock-off load equal to 10% of FTL, whereas active anchors are prestressed to a higher lock-off load. Active vertical ground anchors can be locked-off to a maximum load of 100% of the unfactored design load, at which they are called fully active. The SD should evaluate the bridge stability for the case when the footing lifts off due to the elongation of the unbonded length of the vertical ground anchors.

Design Considerations

Settlement

In general, the total permissible settlement under the Service-I Limit State should be limited to one inch for multi-span structures with continuous spans or multi-column bents, one inch for single span structures with diaphragm abutments, and two inches for single span structures with seat abutments. Different permissible settlement under service loads may be allowed if a structural analysis verifies that required level of serviceability is met. The SD will provide both total and permanent Service-I Limit State support loads to the GD. When evaluating settlement, the GD should consider group effects.

Nominal Resistance in Uplift or Tension

The details for the standard Class 90, Class 140, Class 200 piles, and 16" and 24" cast-in-drilled-hole (CIDH) Standard Plan piles have been designed for a nominal axial structural resistance in tension equal to 50 percent of the nominal axial structural resistance in compression. The Standard Plans, Sheets B2-3, B2-5, and B2-8 show the nominal axial structural resistance for both tension and compression. The demand for uplift resistance at any pile must be limited to the structural resistance of the pile and the pile's connection to the footing.

The SD and GD must verify that the required nominal axial resistance in tension can be obtained geotechnically. End bearing piles or piles with large end bearing contributions may have limited tensile capacity. When liquefaction or scour is anticipated, the skin resistance in the susceptible soil must be ignored.

Static analysis or load tests may be used to evaluate pile nominal resistance in tension. Group effects should be considered when evaluating nominal resistance in tension. The factored design load for Strength or Extreme Event Limit States shall not exceed the factored nominal resistance in tension for the respective limit state. The resistance factors for tension under Strength and Extreme Event Limit States are provided in California Amendments to the AASHTO LRFD BDS.

Horizontal Resistance

A lateral load test or a soil-structure interaction analysis, such as using p-y (Load-Deflection) curves, is required to determine the pile horizontal resistance. Group effects should be considered when evaluating the horizontal resistance of pile groups. P-multipliers used to incorporate group effects in the p-y method of analysis are provided in Section 10.7.2.4 of the LRFD BDS. Lateral resistance provided by the embedded pile cap may be considered in the evaluation of the horizontal resistance of pile groups.

In general, the SD will perform the lateral load analysis. The SD will determine when such an analysis is necessary and will request the GD to provide input soil parameters or perform lateral load analysis.

To increase horizontal capacity, driven piles may be battered at abutments and retaining walls. The horizontal component of the axial load can be added to the horizontal resistance of battered piles. In general, battered piles should not be used at bents and piers.

Pile Length for Horizontal Load

To obtain stability against horizontal load, the pile length should not be less than the critical length multiplied by a factor of safety. A lateral stability analysis in which the governing design lateral load is applied at the top of the pile and the pile length is varied to develop a pile length versus pile top deflection plot is necessary to determine the critical length. The critical length is that length of pile for which greater lengths do not result in a significant reduction in the deflection at the pile top.



Typically, the foundation recommendations do not consider pile penetration depths for lateral loading unless the SD makes a request to the GD. The SD must verify that adequate pile penetration is provided for structural stability against scour, liquefaction, or lateral soil flows induced by seismic events.

Nominal Horizontal Resistance (Strength and Extreme Event)

The horizontal resistance is governed by the structural capacity if the pile is longer than the critical length. The nominal horizontal resistance under Strength or Extreme Event Limit States is taken as equal to the applied horizontal load at which the structural capacity of the pile under the combined effects of bending and axial load or under shear (whichever smaller) is reached. The factored lateral load for Strength or Extreme Event Limit States shall not exceed the factored nominal horizontal resistance for the respective limit state.

Piles do not normally add significant lateral stiffness to pile caps that are embedded in a competent soil (refer to Caltrans Seismic Design Criteria for definition of competent soil). However, a pile must be designed to conform to the expected relative displacement between the ground and footing in the event the relative displacement exceeds the pile's elastic displacement capacity. At a minimum, piles must have enough lateral capacity to force plastic hinges into the columns, while still maintaining sufficient axial capacity.

The Class piles detailed in the Standard Plans are designed to pin at the pile cap without transferring any appreciable moment to the structure. Fixed head piles designed to transfer moment to the pile cap require a case-by-case design that considers the effects of shear, moment, axial load, and stability. The foundation design shall take into consideration lateral pile demands, pile stiffness, and the soil capacity.

Test Piles and Field Verification of Axial Nominal Resistance

In some situations GS may recommend field verification of axial nominal resistance of test piles and production piles during installation, by using pile load tests or dynamic tests. Such situations include, but are not limited to the following conditions:

- When subsurface conditions are highly variable.
- To confirm geotechnical design parameters.
- To determine whether the specified tip elevation could be revised, including evaluation of pile set-up or relaxation.
- When the Wave Equation is used for bearing analysis.
- Low redundancy or risk of failure.
- For driven piles greater than 36-inches in diameter.

Dynamic testing of driven piles correlates the axial nominal resistance determined by static load testing with the axial nominal resistance calculated by simpler field verification or construction control methods, such as the Modified Gates Formula and Wave Equation analysis. (Refer to Standard Plans B2-9, B2-10 and B2-11 for details and pay limits of Caltrans Standard Plan piles when a pile load test is recommended.) A five-pile load test pile group is required when both a tension and compression test is required. A three-pile load test pile group is adequate if only a tension test is required.

The Structure Plans should show pile load test locations, control areas, connection details, and the layout of both anchor and load test piles. If possible, all pile load test piles should be incorporated into the permanent structure.

For Standard Plan driven piles, the formula in Standard Specification 49-2.01 is sufficient for the field verification of pile nominal resistance. This formula should not be used when the driven pile diameter is greater than or equal to 18 inches or the required nominal driving resistance exceeds 600 kips. In such cases, GS will recommend the appropriate test method(s) for the verification of the axial nominal resistance during installation and the pile acceptance criteria. These recommendations will need to be incorporated in the Special Provisions in order to supersede the Standard Specification 49-2.01. The acceptance criteria, when a pile load test is performed, is in accordance with the provisions set forth in Sections 10.7.3.8 and 10.7.3.10 of the LRFD BDS for compression and tension, respectively.

Non-Destructive Testing of Welds for Steel Piles, Shells, and Casings

Steel pipe piles, casings, and shells can be classified as Redundant (R) or Non-redundant (N). Examples of class N piles are: unfilled pipe piles or CISS piles installed in a pile group of less than 5 piles and designed for non-elastic performance, permanent steel casings shown on the plans that are required for strength of Type-I shafts, and permanent steel casings used in Type-II shafts supporting single column bents, only if casing contribution to the strength of the shaft has been considered in design. Examples of redundant piles are pile groups with at least 5 piles, and steel casings in Type-II shafts used to support a column in a multi-column bent.

For non-redundant piling, where the tensile or bending strength of the steel piles is checked in the design, the special provisions should indicate the zone of the pile that will require non-destructive testing (NDT) of welded splices. Longitudinal and spiral seams in steel pipes are visually inspected at fabrication, but the SD may choose to require full NDT on the seam, especially near a welded splice. Because the eventual pile tip elevation is uncertain, the specification of “no-splice zones” in steel piles should be avoided. For redundant piling, NDT required in AWS D1.1 is usually adequate. The Project Engineer should include the NDT requirements in the Memo to Specifications Engineer.



Corrosion

A site is considered corrosive when one or more of the following conditions exist:

- The pH is 5.5 or less.
- The soil contains a chloride concentration of 500 ppm or greater.
- The soil has a minimal resistivity of 1000 ohm-centimeters or less.
- The soil contains sulfate concentration of 2000 ppm or greater.

The Foundation Report will indicate whether the site is corrosive or not. For additional assistance regarding corrosion protection of deep foundations, contact the Corrosion and Field Investigation Branch (CFIB) of METS-GS.

Steel Piling

Steel piling may be used in corrosive soil and water environments provided that adequate corrosion mitigation measures are specified. Caltrans typically includes a corrosion allowance (sacrificial metal loss) for steel pile foundations. Other corrosion mitigation measures may include coatings and/or cathodic protection.

Caltrans currently uses the following corrosion rates for steel piling exposed to corrosive soil and water:

Soil Embedded Zone:	0.001 inch per year
Immersed Zone (salt water):	0.004 inch per year
Scour Zone (salt water):	0.005 inch per year

Steel piles should not be used in the splash zone, unless alternative mitigation measures such as protective barrier coatings and/or cathodic protection are considered. For steel piling driven into undisturbed soil, the region of greatest concern for corrosion is the portion of the pile from the bottom of the pile cap or footing down to 3 feet below the water table. This region of the soil typically has a source of oxygen to sustain corrosion. The 14-inch and 16-inch Alternative 'W' class pipe piles shown in Standard Plans B2-5 and B2-8 respectively, have been predesigned for corrosion rate of 0.001 inch per year. The SD shall evaluate structural performance of such piles when exposed to higher corrosion rates.

The corrosion loss should be doubled for steel H-piling since there are two surfaces on either side of the web and flanges that are exposed to the corrosive soil and/or water. For pipe piles, shells, and casings, the corrosion allowance is only needed for the exterior surface of the pile. The interior surface of the pile (soil plug side) will not be exposed to sufficient oxygen to support significant corrosion.



Concrete Piling

Reinforced concrete piles should be designed in accordance with LRFD BDS Article 5.12.3, “Concrete Cover”. This article includes specific information regarding concrete cover, use of supplementary cementitious materials, use of a reduced water-to-cement ratio concrete mix, and epoxy coated reinforcing steel for corrosion mitigation against exposure to corrosive soil and/or water. MTD 10-05 and 10-06 also provide additional background for the protection of reinforced concrete against corrosion due to chlorides, acids and sulfates, and the use of prefabricated epoxy coated reinforcement for marine environments.

Furthermore, to facilitate construction of the CIDH piles and drilled shafts, the minimum concrete cover to reinforcement (including epoxy coated rebar) will be according to Table 3-1*. For shaft capacity calculations, only 3 inches of cover is assumed effective and shall be used in calculations.

Table 3-1. Minimum Concrete Cover for Drilled Shafts

“D”=Diameter of the Drilled Shaft	Concrete Cover
16" and 24" Standard Piles	See Standard Plan B2-3
$24" \leq D \leq 36"$	3"
$42" \leq D \leq 54"$	4"
$60" \leq D < 96"$	5"
96" and larger	6"

* For shaft capacity calculations, only 3 inches of cover is assumed effective and shall be used in calculations.

Ground Anchors

Ground anchors are typically specified with corrosion protection provisions. Ground anchors are sheathed full-length with corrugated plastic, and pre-grouted. In addition, the steel in the unbonded length is sheathed with smooth plastic. Both corrugated and smooth plastic can be either polyvinyl chloride or high density polyethylene (HDPE). These anchor systems may also require the use of corrosion inhibiting grease in the unbonded length within the smooth sheathing.

Original signed by Barton J. Newton

Barton J. Newton
State Bridge Engineer
Deputy Division Chief, Structure Policy & Innovation
Division of Engineering Services