3-1 DEEP FOUNDATIONS

Deep foundations are structural assemblies that transfer load into deeper earth materials. Deep foundations, generically referred to herein as piles, can be driven piles, drilled shafts, or alternatively micropiles and grouted-in-place piles. Caltrans deep foundations consist of a single pile or a group of piles with a cap footing. Structure Design (SD) is responsible for calculating the pile load demands and for providing structure details. Geotechnical Services (GS) is responsible for providing foundation recommendations that include site seismicity, factored downdrag loads, pile tip elevations (based on the load demands provided by SD), construction recommendations (pile acceptance criteria, testing requirements, etc.), the Log of Test Borings, and Information Handouts. SD and GS will reach a consensus on 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 P&Q, any information absent from the Foundation Recommendations should be included in the project engineer's Memo to Specifications Engineer. When draft specifications are available, PS&E review by GS completes the process, allowing GS commentary on the plans and specifications. If necessary, a meeting with the specifications engineer, the geotechnical designer, and the SD's project engineer should occur to discuss the foundation related specifications.

Current Caltrans' practice is to design abutments in accordance with the Working Stress Design (WSD) methodology and bents/piers in accordance with the Load and Resistant Factor Design (LRFD) as specified in the current LRFD BDS (AASHTO LRFD Bridge Design Specifications (BDS) with interims, and Caltrans Amendments). Loads from the LRFD Service-I Limit State shall be used as design loads for WSD of the abutments. The structure designer needs to give the geotechnical designer the foundation design information and loads or demands for the applicable limit states, so that the geotechnical capacity of the pile selected will meet or exceed these demands. GS will provide factored 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 PILES), B2-5 (PILE DETAILS CLASS 90 AND CLASS 140), and B2-8 (PILE DETAILS CLASS 200) provide the upper limit of structural pile design capacities in tension and compression.

When a Standard Plan pile is 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 infeasible to construct, 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 pile, 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, Cantilever Abutments, Type 1 Retaining Walls, or Counterfort Retaining Walls. If these design charts are used, the Special Provisions shall stipulate that Alternative 'X' piles must 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.

Lateral Resistance

The allowable horizontal resistance of a single Standard Plan pile fully embedded in soil with a corrected standard penetration resistance value, ($N1_{60}$), of 10 or greater and a 1/4 inch maximum horizontal deflection at the pile top under Service Limit State Load is given in Article 4.5.6.5.1 of the Caltrans Bridge Dessign Specification LFD version of April 2000 (Based on AASHTO Bridge Design Specifications, AASHTO 16th Edition, 1996). The lateral pile resistance in LFD BDS is based on soil failure and can be increased provided a geotechnical analysis supports the increase. In all cases where the Standard Plan piles are used, the pile-to-pile cap connection is intended to be a pinned connection. When the soil in the upper zone of the embedded piles has a corrected standard penetration resistance value less than 10, the lateral resistance values are not applicable and a special pile design will be required. In the case of battered piles, the horizontal component of the axial load can be added to the lateral resistance.

Driven Piles

Driven piles can be precast prestressed concrete, cast-in-steel-shell (CISS) concrete, steel HP, 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 diameter are non-displacement piles. Such piles are useful for penetration where boulders or hard strata are expected.

Site specific issues including noise, vibration, ground heave, settlement, headroom, constructibility, and driveability must be considered when selecting driven piles. Liquefaction, scour potential, or other conditions may require deeper pile penetration and therfore overcoming significantly higher driving resistance compared to the required geotechnical nominal resistance. In that case, driveability must be evaluated to verify that the piles can penetrate to the specified tip elevation with acceptable driving stresses and blow counts.

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 piling.

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 sections are also preferable for longer piles because they can be spliced easier 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 Structure Designer shall provide allowable HP sizes to the Specification Engineer. The HP 14x89 steel pile is usually specified for a nominal resistance of 400 kip, HP 10x57 for 280 kip and HP 10x42 for 180 kip. The design engineer 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 nonstandard HP section is available. Larger pile sections may be required if increased lateral load resistance is needed or hard driving is anticipated. Refer to LFD BDS (2000) 4.5.6.5.1 for the assumed lateral pile resistance values for standard plan piles under Service Limit State loading.

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 exists 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 faced with excessive blow counts or high driving stresses, GS 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.

The 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.

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

For non-redundant piling, where the tensile or bending strength of the steel piles is critical in the design, the special provisions should indicate the zone of the pile that will require non-destructive testing (NDT) on welded splices. Longitudinal and spiral seams in steel pipes are visually inspected at fabrication, but the design engineer 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 other than what is required in AWS D1.1 is usually not required. The Project Engineer should include the NDT requirements in the "Memo to Specification Engineer".

Drilled Shafts

Drilled shafts, also known in the industry as cast-in-drilled-hole (CIDH) or caissons—are a possible solution when driven piles are not suitable, large vertical or lateral resistance is required, or to address the constructibility issues. A CIDH pile is more advantageous than a driven pile in terms of noise and vibration, but disposal of hazardous drill spoils may be costly. CIDH piles are commonly constructed by drilling techniques.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 follow this naming convention: although contributing to structural capacity of the pile steel casings are used for constructability, while driven steel shells are used for extra geotechnical axial resistance. Casings can be specified to be left in place permanently or to be removed during concrete placement. Precise nomenclature is required to direct the contractor to certain bearing specifications. In addition, to verify its geotechnical compressive and tensile capacity, a shell must be driven into place. A casing may be driven,drilled, vibrated, or oscillated into place.

Both tip and side resistances for CIDH piles develop 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 skin resistance or adhesion 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 conditions, where soft, compressible drill spoils and questionable concrete quality are both possible at the pile tip. GS recommendations may discard or include some fraction of the tip resistance, especially in wet 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 piles in wet holes. PVC inspection pipes are installed to permit gamma-gamma and Cross-hole Sonic Logging (CSL) tests of these CIDH piles. See Attachment 2 for reinforcing steel clearance requirements in conjunction with inspection tubes.

When contemplating CIDH piles in wet condition, caution should be exercised in the following cases: 1) single column bent pile extensions lacking redundancy; 2) soft cohesive soils, loose sands, or boulders at the support location (constructibility); 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 substituted, 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.

For Type-II shafts (Caltrans Seismic Design Criteria, 7.7.3.5) an optional construction joint should be allowed below the embedded column rebar cage. Allowing the construction joint will give the contractor the option of placing the CIDH concrete without supporting the column rebar cage. This construction joint will also allow the contractor to cast the rest of the type-II shaft and column concrete in dry condition. It is important that District and Geotechnical Services be aware of this option since the construction joint may require the placement of permanent casing in the hole to allow workers to prepare the joint. The geotechnical designer may want to exclude this zone from the geotechnical capacity calculations and the District Project Engineer will need to obtain a classification from the Cal-OSHA Mining and Tunneling Unit, as described in the

Highway Design Manual Topic 110. If the optional construction joint is allowed, the plans should show its location and the Special Provisions should describe the required joint preparation. The Memo to Specification Engineer should highlight the presence of an optional construction joint when ground water is anticipated.

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 Memo to Designers (MTD) 5-2 for more information.

To ensure constructibility 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 GS. 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 piles. 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 the 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 dynamics of the entire structure.

The contract items for CIDH piles are as shown in Bridge Design Aids 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. Advancements 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, then the rock socket zone should be identified and the pay limits of rock socket should be shown on the plans. Additionally, pile cut-off elevations and tip elevations should be shown in the Pile Data Table. Typically, rock sockets embedded into zones of strong rock can develp large geotechnical nominal resistance in relatively short socket lengths, therefore, field inspection during construction by the geotechnical designer may be necessary to verify that the design criteria was met.

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 may be 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 Bridge Design Details page 7-20 and Bridge Design Aids Chapter 11 for details.

Pile Extensions

The standard pile types (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, GS will provide soil profile data for 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 1 foot below finished grade up to the bent cap. Special seismic detailing my be required to control plastic hinge locations in these pipe pile extensions.

Tiedown Anchors

Tiedown anchors can be used where site conditions prevent traditional piles from achieving the necessary tensile capacity. For example, where rock exists close to the ground surface (or scour elevation), piles driven to refusal may be too short to develop the side shear that resists uplift. Tiedowns are also effective when combined with spread footings sitting directly on rock, or as part of a seismic retrofit strategy to add uplift capacity to a footing.

Tiedown anchor details typically include prestressing strands or rods grouted into a drilled hole. The final stiffness of the element depends on the unbonded length of tendon, and should be accounted for in the design. The prestressing force, if any, increases the stiffness of the tiedowns by engaging the soil before foundation loads are applied. The lockoff load for fully active systems is 100% of the design load, while passive anchors receive a nominal lockoff, usually 10% of the design load.

Tiedown anchors require no entries in the Pile Data Table. In current practice, the design engineer specifies the unbonded length of tendon, while the contractor calculates the bonded length. Field testing to 125% of the design load verifies the resistance of each tiedown anchor. The design load, "T", must be shown on the plans.

Alternative Piles

The Alternative Pile option is an attempt to take advantage of new pile types that can be used, where appropriate, as alternatives to a State-designed pile. A number of proprietary systems have been approved (see SSP 49-470), including variations on micropiles and grout injection piles. To be approved, each vendor's pile system must go through an extensive review process, including both analysis and full-scale load testing to geotechnical failure.

The design engineer should consult with GS when a site appears favorable for an Alternative Pile. Alternative Pile designs have been developed in response to site constraints such as low overhead clearance (6 feet minimum), vibration restrictions, and hard-driving soils containing large cobbles. High-capacity micropiles can be successfully installed through an existing pile cap to seismically retrofit a foundation without increasing its size.

When an Alternative Pile is listed in the specifications, the contractor has the option to select an Alternative Pile vendor. The contractor is responsible for preparing pile working drawings and for designing the pile to satisfy the demands shown in the Pile Data Table. The pile vendor is required to verify the pile's geotechnical design with a performance test prior to production installation. Proof testing of the production piles is also required.

Design Considerations

Settlement

In general, the total permissible settlement under the Service-I Limit State should be limited to one inch for multi-span sturctures with continuous spans for 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. Structure Design will provide both total support loads and per pile loads to Geotechnical Services. When evaluating support settlement, Geotechnical Services 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) piles have been designed for a structural nominal axial strength in tension equal to 50 percent of the structural nominal axial compressive strength. The Standard Plans, Sheets B2-3, B2-5, and B2-8 show the structural nominal axial strength for both tension and compression. The demand for uplift resistance at any pile must be limited to the structural capacity of the pile and the pile’s connection to the footing.

SD and GS must concur 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 all compromised strata is unreliable and should be ignored.

Static analysis or load test 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 State shall not exceed the factored nominal resistance in tension for the respective limit state. The recommended resistance factor for tension for Strength and Extreme Event Limit States are provided in California Amendments to the AASHTO LRFD BDS.

Horizontal Resistance

Lateral load test or a soil-structure interaction analysis, such as using p-y (Load-Deflection) curves, is required to determine pile horizontal resistance. Group effects should be considered when evaluating 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, SD will perform lateral load analysis. SD will determine when such analysis is necessary and request GS to provide input soil parameters or perform lateral load analysis.

To increase horizontal capacity, driven piles may be battered at abutments and retaining wall. 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, pile length should not be less than the critical length. 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 dertermine 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 recommendation does not consider pile penetration depths for lateral loading unless the structure designer makes a request to GS. The structure designer must verify that adequate pile penetration is provided for structural stability against scour, liquefaction, or lateral soil flows induced by seismic events. See Bridge Design Aids Chapter 12 for guidelines on checking the lateral stability of pile extensions.

Allowable Horizontal Resistance

Allowable horizontal resistance of piles should be evaluated at 1/4 inch pile top or cap displacement. Horizontal load or demand on the pile from the Service I Limit State load combination shall be less than the allowable horizontal resistance.

Nominal Horizontal Resistance

The horizontal load is limited to structural capacity for piles longer than the critical length. The nominal horizontal resistance under Strength or Extreme Event Limit State 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 is smaller) is reached. The applied factored lateral load for Strength or Extreme Event Limit State shall not exceed the factored nominal horizontal resistance for the respective limit state. The resistance factor (φ) is 1.0 for Strength and Extreme Event 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. Fixed head piles designed to transfer moment to the pile cap require a case-by-case design that considers the effects on shear, moment, axial load, and stability. The design shall take into consideration lateral pile demands, pile stiffness, and the soil capacity.

Test Piles and Field Verification of Axial Nominal Resistance

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

- At locations where the geotechnical investigation is limited or indicates variable, discontinuous stratigraphy.
- For cast-in-drilled-hole piles located in unproven soil formations.
- To determine whether the specified tip elevation could be revised, including evaluation of pile set-up or relaxation.
- In conjunction with the use of the Wave Equation for bearing analysis.
- Low redundancy or tolerance for 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 verfication 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 49-1.08 is sufficient for the field verification of pile nominal resistance. This formula should not be used when the driven pile diameter is greater than 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 49-1.08. 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.

Corrosion

A site is considered corrosive when pH is 5.5 or less, or when the soil has a minimum resistivity of less than 1000 ohm-centimeters, and either contains a chloride concentration of 500 ppm (or greater), or a 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 Corrosion Technology Branch of Materials Engineering and Testing Services.

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 protections 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 replenishible source of oxygen needed to sustain corrosion.

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 mineral admixtures, 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.

Memo to Designers 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.

Tieback and Tiedown Anchors

Both tieback and tiedown anchors are typically specified with corrosion protection. Both types of anchors are sheathed full-length with corrugated plastic and pre-grouted. In addition, the steel in the unbonded area 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 Kevin Thompson)

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