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Change Letter – Revision No. 2
October 30, 2015


Revisions

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Background
The State of California, Department of Transportation, Division of Engineering Services, Foundation Manual, November 2008, Revision 1, December 2010 has been updated. The revisions include:

- Updated references to the current 2010 Standard Specifications (SS).
- Terms were updated to reflect the 2010 Standard Specifications. For example, approve is revised to authorize; tiebacks and tiedowns changed to sub-horizontal and vertical ground anchors; and hammer weight was changed to ram weight.
- Revised the numbers and names of the forms referenced in the manual to be consistent with recent revisions. Added descriptions and titles to the tables.
- Updated the format of the manual to meet the standards in the Style Guide for Structure Construction Technical Manuals.
- Chapter 2, Type Selection - Added information on micropiles, alternative piles, ground anchors and soil nails.
- Chapter 4, Footing Foundations - Added technical update for LRFD and Cal-Osha.
- Chapter 5, Pile Foundation-General - Added description of differences between 2006 SS and 2010 SS for CIDH piling.
- Chapter 6, Cast-in-Place Piles - Includes additional information on reverse circulation, CIDH pile preconstruction meeting, updated specification revisions, and added reference to Appendix K-2 for construction checklist.
- Chapter 7, Driven Piles - Updated the hydraulic hammer information, Drivability Study information, Hammer Acceptance Study information, corrected the description of the acceptance criteria calculation for battered piles, changed references for hammer weight to ram weight, and added references to Appendix K-1 for the construction checklist.
• Chapter 9, *Slurry Displacement Piles* - Updated API test procedures and Internet links, added definitions for *dry hole*, *dewatered hole* and *wet hole*, and added information for the CIDH non-standard pile mitigation meeting.

• Chapter 10, *Pier Columns* - Added the definition of the pier column and added blasting information.

• Chapter 11, *Ground Anchors & Soil Nails* - Replaced references for *tiebacks* and *tiedowns* with *sub horizontal* and *vertical ground anchors*, updated specification references for design and testing of ground anchors, added reference to Appendix K-6 for the construction checklist, and added information to soil nail verification test.

• Chapter 12, *Cofferdams and Seal Courses* - Added reference to Appendix K-3 for the construction checklist.

• Chapter 13, *Micropiles* - Renamed for Micropiles only. All other information was moved to the new Chapter 14. Revised and updated micropile information and added reference to Appendix K-5 for the construction checklist.

• Chapter 14, *Specialty Piles and Special Considerations for Pile Foundations* - Added as a new chapter. Chapter 14 discusses specialty piles and special considerations. This includes information on alternative piles, continuous flight auger piling and other specialty piling. Also includes information on overhead sign structure pile formations, tip grouting, type II shafts, and soldier piling.

• Appendix B, *Contract Administration* - Removed references to CPD 00-05 and 01-12 since these CPD’s are now incorporated into the *Construction Manual*.

• Appendix E, *Driven Piles* - Changed the reference from *hammer weight* to *ram weight*.


• Appendix H, *Ground Anchors & Soil Nails* - Replaced references to *tiebacks* and *tiedowns* with *ground anchors*, removed all construction checklists, and added case studies.

• Appendix J, *Micropiles* - Added Spanish Creek Case Study for micropiles for bridge foundations.

• Appendix K, *Foundation Construction Checklists* - Replaced references to *tiebacks* and *tiedowns* with *ground anchors* and revised the micropile checklist to match updated specifications.

STEVE ALTMAN
Deputy Division Chief
Structure Construction
Division of Engineering Services
Acknowledgements

The 2015 edition of the Foundation Manual was updated by a group of dedicated Senior Bridge Engineers from Structure Construction (SC).

Thanks to David Keim, P.E., SC Substructure Technical Team Chairman, for his contributions and leadership. Thanks to Basem Alsamman, P.E., Vice Chairman. Thanks to the SC Substructure Technical Team members for their valuable contributions and teamwork. Members include: Daniel Dait, P.E., Mark Darnell, P.E., Hanna Dergham, P.E., Rich Foley P.E., Victor Francis P.E., Jeff Kress, P.E., Arvind Patel, P.E., and Mark Woods, P.E. Thanks to Jerry De Santos P.E., Supervising Bridge Engineer, SC Substructure Technical Team Sponsor, for his contributions and sound guidance.

Special thanks to the Caltrans engineers who drafted the original 1984 Foundation Manual and to the Caltrans engineers who drafted the 1996 and 2008 revisions. Their vision, dedication, and research, produced a manual that has been used throughout Caltrans and beyond.

Signed,

STEVE ALTMAN
Deputy Division Chief
Structure Construction
Division of Engineering Services
Preface

The Foundation Manual is intended to provide the field engineer with information that will assist in solving foundation problems and in making engineering decisions.

Although the field engineer is required to make engineering decisions throughout the life of a construction project, none is probably more important than the engineer’s decision regarding the suitability or unsuitability of the foundation material supporting a spread footing foundation. The engineer must decide if the foundation material encountered at the planned bottom of footing elevation is, in fact, representative of the material shown on the Log of Test Borings sheet and therefore suitable for the imposed loads. If not representative, the engineer must decide what action to take.

This is not to minimize the importance of pile supported foundations, which have their own unique problems that require decisions based on sound engineering judgment. Information in this manual will aid in answering a multitude of questions such as: What action does the engineer take when pile bearing capacity is not obtained at specified tip or reaches “refusal” 15 feet above tip elevation?

All types of foundations are discussed in the manual, along with related problems and possible solutions. There is no one solution that will always solve a particular problem. Each situation must be reviewed and a decision made based on the available data and engineering principals.

There is no substitute for utilizing sound engineering judgment in solving engineering problems. If all problems are solved in this manner, the engineer can be confident that an appropriate solution was used to solve the problem.
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13. Arthur Miller. Figure used for Figure 7-1. State of California, Department of Transportation, District 02 Reprographics.


CHAPTER 1

1-1 Introduction

The ultimate strength and longevity of any structure depends on the adequacy of the foundation. Engineers administering projects for Structure Construction (SC) are responsible for ensuring that the foundation work of every structure can sustain the design loadings throughout its design life.

Personnel working for SC must learn the provisions of the contract documents (Standard Specifications, Standard Plans, contract plans, Special Provisions) and all relevant documents related to each structure. Thorough understanding of all project documents enables effective contract administration and resolution of any foundation issues. Structure Representatives must be resourceful and know whom to contact for advice to resolve project problems.

“It is the responsibility of the Structure Representative to clear up any problem areas prior to the start of construction, or as soon thereafter as possible.”

This chapter provides an overview of the foundation investigation process, and describes how the Log of Test Borings (LOTB) and the Foundation Report for a Caltrans structure project are developed. It guides the reader in interpreting and effectively using the LOTB and the Foundation Report during administration of structure projects.

1-2 Who Performs Foundation Investigations?

Foundation investigations for the various structures designed and constructed by the Division of Engineering Services are performed and coordinated by one of Geotechnical Services’ four Geotechnical Design Sections:

- Geotechnical Design Section – North – Districts 1, 2, 3, 5, 6, 9 & 10.
- Geotechnical Design Section – West – District 4 & Toll Bridge Program.
- Geotechnical Design Section – South 1 – Districts 7 & 12.
- Geotechnical Design Section – South 2 – Districts 8 & 11.

1 Bridge Construction Memo (BCM) 2-2.0, Pre-Job Discussion with Design, Architecture and Geology.
Geotechnical Design Section personnel or “Geoprofessionals” are available to support SC employees throughout a construction project. These engineers specialize in geotechnical engineering and engineering geology. SC engineers should schedule preconstruction meetings with appropriate Geotechnical Design Section personnel to forge a working relationship with the Geoprofessionals who performed the foundation investigation, wrote the Foundation Report, and developed the LOTB. These meetings should address potential foundation problems or other areas that may need extra attention.

Once construction projects are under way, Geotechnical Design Section personnel lend their expertise as needed and when problems or challenges occur during foundation work. They advise by phone, and often visit project sites to evaluate difficult foundation installations and recommend solutions. The Engineer should inform the Geotechnical Design Section as early as possible of any problems, changes, or differences with structure foundations. Early notification provides the best chance of resolving difficult or problem foundations with the most economical solution.

Sometimes consultant engineers design structure projects. Consultant geotechnical companies produce foundation investigations for these projects with Caltrans’ oversight. Issues related to project foundations that are designed by consultants should be discussed with the Caltrans oversight Engineer assigned to the project.

1-3 Foundation Investigation Overview

At the beginning of a design project for a new structure, widening, strengthening, or seismic retrofit, the Designer submits a Foundation Investigation Request to the appropriate Geotechnical Design Section. From this request, a Geoprofessional is assigned to perform the foundation investigation.

The assigned Geoprofessional performs a foundation investigation by first collecting as much information as possible about the proposed site. This includes reviewing preliminary structure plans, previously written foundation reports, as-built plans, the area’s historical seismicity information, and historical subsurface conditions. This planning phase of the investigation gives the Geoprofessional an idea of what to watch for during fieldwork.

A drilling plan is important to make efficient use of the available drilling equipment and personnel. Once all preliminary subsurface information from the site is collected, the drilling plan is generated to outline locations for drilling relative to the structure’s proposed foundation. The Geoprofessional directs the foundation drilling crew during the

---

2 BCM 2-2.0, Pre-Job Discussion with Design, Architecture and Geology.
subsurface drilling operation (described later in this chapter) as they collect soil samples and perform *in situ* testing at the site.

Information necessary to develop the project’s LOTB includes: soil samples collected during the subsurface drilling operation, results of *in situ* tests, manual field tests, and various recorded observations. Once the LOTB is completed, it is transmitted to the Designer and is included in the structure plans.

The Geoprofessionals in Geotechnical Services analyze information compiled in the LOTB, along with the loads provided by Structure Design, and make foundation recommendations. The recommended foundation type and other important information is included in the Foundation Report, and transmitted to the Designer. These recommendations are used to complete the structure design. The Foundation Report is included in the RE Pending File and the Supplemental Project Information handout for the contractor at time of bid.

### 1-4 Subsurface Drilling Operation

Results obtained from the subsurface drilling operation are the most important aspects of a foundation investigation. Foundation drilling crews conduct one or more drilling operations at the structure’s proposed location. The subsurface investigation helps determine the depth of rock, rock type and quality, soil types, soil strengths, and groundwater levels. Determining these parameters enables development of a soil/rock profile, which is a visual representation of the subsurface conditions interpreted during subsurface investigations and laboratory testing. The soil/rock profile can be determined by interpolating between comparable lenses of material in individual borings within the LOTB.

The appearance and feel of the cuttings, difficulties or changes in the rate of drilling tool advancement, and other observations help estimate the mechanical properties or strengths of the soil or rock lenses. These observations are noted within field logs. Any groundwater encountered during the drilling operation also is noted, and special care is taken to accurately determine its elevation and whether the groundwater encountered is static or under pressure (“perched” or in an “artesian” condition). These observations, along with the results from field and laboratory testing, are used to develop the soil/rock profile.

Two important facets of the subsurface drilling operation are the recovery of soil samples retrieved during drilling operations and the *in situ* soil tests. Soil samples are divided into two categories: disturbed and undisturbed. Disturbed soil samples are those that have experienced large structural disturbances during the sampling operation and may be used for identification and classification tests. Undisturbed samples are those in which
structural disturbance is kept to a minimum during the sampling process. Undisturbed samples are used for consolidation and strength tests. Examples of these strength tests are direct shear, triaxial shear, and unconfined compression tests. The strength tests provide shear strength values, which are then used as design parameters in static analysis for pile foundations. Consolidation tests provide information needed to estimate settlements of spread footings or pile groups, and are performed on cohesive soils.

Several types of soil samplers are used to retrieve undisturbed samples during subsurface investigations. Types include the California Sampler (which is the primary tool used by Geotechnical Services), the Shelby Tube, the Piston Sampler, and the Hydraulic Piston Sampler. Undisturbed soil samples provide the best opportunity to evaluate the soil in its natural, undisturbed state. Destructive testing of these samples provides the most accurate soil data; however, undisturbed samples from non-cohesive, or cohesionless soils are difficult to obtain, trim, and test in the laboratory. Soft, saturated clays; saturated sands; and intermixed deposits of soil and gravel are difficult to sample and test in the laboratory. To overcome these difficulties, in situ test methods are used to measure soil parameters.

When standard drilling and sampling methods cannot be used to obtain high-quality, undisturbed samples, in situ tests are used to provide information about the material characteristics. The most common of these tests is the Standard Penetration Test (SPT). This test identifies a penetration resistance value, “N,” which can be used to obtain estimates for the angle of internal friction of a cohesionless soil, the unconfined compressive strength of a cohesive soil, and the material’s unit weight (refer to Appendix C). The SPT is performed using a split-spoon sampler and provides a disturbed sample for visual inspection and classification. Other in situ tests include the static cone test, pressure meter test, vane shear test, and the borehole shear test. They provide soil strength values such as cohesion, angle of internal friction, and shear strength.

Design parameters obtained from field and laboratory testing are used for static analytical design procedures, pile and footing foundations. They may also provide valuable information to the Engineer during the course of administering a construction project.

1-5 Log of Test Borings

After the subsurface investigation and laboratory testing is complete, the project’s LOTB is developed. The LOTB includes a plan view showing the location of each boring retrieved during the subsurface drilling operation. It provides a graphic description of the various layers of geological formations, soils, and the location of the groundwater table (if encountered). Various soil and rock properties also are described. Each LOTB includes a standard legend on the left side of the sheet that describes the different symbols and notations used within the LOTB. Examples of LOTB are included in the Caltrans Soil and
1-6 Foundation Report

The Foundation Report contains all information retrieved during the foundation investigation, and provides the Designer with a description and evaluation of the geological formations and soils at the proposed project site. It also describes and evaluates any seismic hazards that may be present, such as the anticipated amount of ground shaking and probability of liquefaction at the site. The report gives recommendations for:

- The type of foundation to be used to support the proposed structure.
- Seismic design criteria, such as peak horizontal bedrock acceleration to be used in the seismic analysis.
- Recommendations for bottom of footing elevations, pile type, and size and tip elevations.

Most reports include special comments about anticipated foundation-related concerns during construction such as caving, soil compaction problems, expected variations in pile driving, and potential problems due to groundwater. This section of the report may even suggest that job-specific specifications be included in the contract’s special provisions. The Structure Representative should pay particular attention to these comments as advance knowledge of potential foundation problems allows for more effective problem-solving and mitigation. The Foundation Report normally is included in the RE Pending File and is included in the Supplemental Project Information handout available to the Contractor at time of bid. If the Structure Representative did not receive a copy of the Foundation Report for a specific project, he or she should contact the SC Headquarters Office to request one.

The Structure Representative should review the project plans to verify that the footing elevation, pile tip elevations, and type of piling recommended in the Foundation Report are shown on the contract plans. In addition, the Structure Representative should confirm that any suggested specifications or design features mentioned within the special comments section of the Foundation Report are included in the contract plans and specifications. Consult with the Designer and Geoprofessional if there are any discrepancies. Contract change orders most likely will be required to address these discrepancies.

http://onramp.dot.ca.gov/hq/oscnet/sc_manuals
Constructability issues discussed in the *Foundation Report* should be discussed with the Contractor as early as possible. Once the Contractor begins work, the Structure Representative should observe and note how the Contractor makes preparations to deal with the constructability issues discussed in the *Foundation Report*. Detailed and thorough documentation of all conversations with the Contractor about these issues will be of important if any claims are submitted by the Contractor.

### 1-7 Applicability of the Log of Test Borings (LOTB) and Foundation Report to the Contract

It is very important for Structure Representatives and all SC field staff to be aware of how the contract specifications describe the applicability of the **LOTB**, the *Foundation Report*, or any record of subsurface investigation produced by Caltrans. The contract specifications describe the Contractor’s responsibilities for reviewing these documents prior to performing work for Caltrans.

In the past, the **LOTB** and other information provided to the Contractor at the time of bid were not considered part of the contract and were provided for information only. The *2006 Standard Specifications* states that the Contractor is required to investigate the site, and includes other available information. Information provided by Caltrans will be used by the Contractor to develop a competitive bid. Therefore, the accuracy of this information is essential for a claim-free contract. Even though Caltrans takes responsibility for the information that is provided, the Contractor still is required to carefully examine the site and the Caltrans-provided information, and is responsible for conclusions they draw from that investigation. The *2010 Standard Specifications* simplifies this by requiring the Contractor to “Examine the job site and Bid Documents.” The **LOTB** and the *Foundation Report* are included within the Bid Documents as the *Supplemental Project Information* handout.

### 1-8 Basic Soil Properties

Geotechnical Services recently published the *Caltrans Soil and Rock Logging, Classification, and Presentation Manual* (Appendix A). It contains information about the field and laboratory procedures used in soil classification and descriptions. It will help the Engineer clearly understand and interpret information presented in the **LOTB** and the *Foundation Report*, and facilitate communication with Geotechnical Services.

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4 2010 SS, Section 2-1.30, *Job Site and Document Examination* or 2006 SS, Section 2-1.03, *Examination of Plans, Specifications, Contract, and Site of Work*.
The information presented in Chapter 2 of the Caltrans Soil and Rock Logging, Classification, and Presentation Manual (Appendix A) is particularly important as it outlines procedure and methodology to identify and classify rock and soil samples. Information presented in the logs and descriptions is based on the ASTM D 2488-06 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) and the Engineering Geology Field Manual published by the Bureau of Reclamation.

Table 1-1 lists soil particle size definitions used by Geotechnical Services:

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<th>CLASSIFICATION</th>
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<tr>
<td>Boulders</td>
<td>Particles of rock that will not pass a 12-inch square opening.</td>
</tr>
<tr>
<td>Cobbles</td>
<td>Particles of rock that will pass a 12-inch square opening but will be retained on a 3-inch sieve.</td>
</tr>
<tr>
<td>Course Gravel</td>
<td>Particles of rock that will pass a 3-inch sieve but will be retained on a 3/4-inch sieve.</td>
</tr>
<tr>
<td>Fine Gravel</td>
<td>Particles of rock that will pass a 3/4-inch sieve but will be retained on a No. 4 sieve.</td>
</tr>
<tr>
<td>Course Sand</td>
<td>Particles of rock that will pass a No. 4 sieve but will be retained on a No. 10 sieve.</td>
</tr>
<tr>
<td>Medium Sand</td>
<td>Particles of rock that will pass a No. 10 sieve but will be retained on a No. 40 sieve.</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>Particles of rock that will pass a No. 40 sieve but will be retained on a No. 200 sieve.</td>
</tr>
<tr>
<td>Silt</td>
<td>Soil passing a No. 200 sieve that is non-plastic or very slightly plastic and exhibits little or no strength when air-dried. Silts that exhibit some plastic properties are qualified as elastic silts.</td>
</tr>
<tr>
<td>Clay</td>
<td>Soil passing a No. 200 sieve that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and that exhibits considerable strength when air-dried. Clay is qualified as fat or lean depending on the amount of plasticity.</td>
</tr>
</tbody>
</table>

Geoprofessionals provide descriptive components and describe soils by name and group symbol to complete the identification. Some descriptive components such as consistency, apparent density, and percent or proportion of soils are mandatory while others, such as particle shape, are not. Refer to Figure 2-5 Identification and Description Sequence of the Caltrans Soil and Rock Logging, Classification, and Presentation Manual for a complete list of descriptive components (Appendix A). An example of a complete descriptive sequence for a sample is:

Well-graded SAND with GRAVEL (SW), medium dense, brown to light gray, wet, about 20% coarse subrounded to rounded flat and elongated GRAVEL, about 75% coarse to fine rounded SAND, about 5% fines, weak cementation.

Visual inspection generally is sufficient to differentiate between the coarse-grained soils. However, distinctions between soil particles such as silts and clays can be difficult.
Several simple field exercises using settling measures, plasticity, dry strength, and permeability characteristics of the soil result in more accurate soil classifications. Soil samples also can be taken to the laboratory and tested to determine plasticity, unit weight, unconfined compressive strengths, and other mechanical properties to refine field classification. Alternatively, the following tests can be used to help classify soils in the field:

- **Water dispersion:** Once a soil is dispersed in water, sand grains settle rapidly—usually in less than one minute. Silt settles more slowly, usually from 10 to 60 minutes. Clay will remain in suspension for several hours.

- **Plastic thread formation:** Sand, having little to no plasticity, will not form a plastic thread by rolling it on a smooth surface. Silt will form a thread when rolled, but it is weak and crumbles as it dries. Clay forms a plastic thread of high strength, which dries slowly and usually becomes stiff and tough as it dries.

- **Dry strength:** Sand has no unconfined dry strength. Silt has very little dry strength and easily powders when rubbed. Clay has a high dry strength and will not powder easily.

- **Shaking or patting:** A rough indication of the plasticity (clay content) of a soil can be determined by observing a sample’s reaction to shaking or patting. For example, when a sample of silt is subjected to this type of movement, water appears on the surface. However, when shaking or patting a sample of clay soil, this reaction occurs slowly or not at all due to the level of the sample’s plasticity.

### 1-9 Geotechnical Drilling and Sampling Equipment

Many different tools are used by foundation drilling crews and Geoprofessionals to obtain samples and evaluate subsurface conditions.

The SC employees must have a good working knowledge of the equipment used during the subsurface drilling operation for their projects. Different tools used to perform a drilling operation have different levels of reliability. Reliability of the tool used during a subsurface investigation is an important factor in determining the accuracy of information provided in the *Foundation Report*.

Table 1-2 describes equipment used by Geotechnical Services and consultant geotechnical companies.
### Table 1-2. Equipment for Subsurface Drilling Operations

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>2¼-Inch Cone Penetrometer</strong></td>
<td>The 2¼-Inch Cone Penetrometer is an in-situ testing apparatus that drilling crews use during subsurface drilling operations. The test is conducted using an air compressor to drive the testing apparatus through the soil. The Geoprofessional records the drilling rate in seconds-per foot of penetration. Test results are shown graphically to indicate the soil’s varying densities as the cone penetrates the different soil layers.</td>
</tr>
<tr>
<td><strong>Sample Boring</strong></td>
<td>Sample Boring is a manual boring technique where a 1-inch sample tube is driven using a 28-pound hand hammer, with a 12-inch free fall. Blows per foot are recorded by the Geoprofessional in a manner similar to the Cone Penetrometer test. This technique is used only for soft soil sites, and in areas where it is difficult to get a drilling rig on-site.</td>
</tr>
<tr>
<td><strong>Rotary Boring</strong></td>
<td>Rotary Boring is a rapid drilling method used for penetrating soil and rock. Borings up to 200 feet and more in depth can be taken using this method. The hole is advanced by rapid rotation of the drilling bit, and water or drilling mud is used to flush out the drill cuttings and to lubricate the cutting tool.</td>
</tr>
<tr>
<td><strong>Auger Borings</strong></td>
<td>An Auger Boring can be advanced without water or drilling mud and provides a dry hole. It gives a good indication of material that is likely to cave in during an excavation or drilling operation. It also gives an accurate reading of the groundwater elevation. Most equipment can drill to depths of 100 to 200 feet.</td>
</tr>
<tr>
<td><strong>Diamond Core Boring</strong></td>
<td>A Diamond Core Boring is used when rock is encountered during a drilling operation. It allows the drilling crew to recover continuous sections of rock cores. The Geoprofessional can inspect the cores to determine the rock’s competency.</td>
</tr>
<tr>
<td><strong>Electronic Cone Penetrometer</strong></td>
<td>The Electronic Cone Penetrometer is an apparatus that drives a cone into soil similar to the 2¼-Inch Cone Penetrometer, but it is capable of providing other soil parameters such as soil type, shear strengths, stiffness, bearing capacities, porewater pressures, relative densities, and shear wave velocities.</td>
</tr>
<tr>
<td><strong>Bucket Auger</strong></td>
<td>The Bucket Auger is a drilling tool that is used to excavate a larger diameter hole (24 to 36 inches). It is considered to be the best indicator for the presence of cobbles and boulders, and is a good indicator of material that is likely to cave in during an excavation.</td>
</tr>
</tbody>
</table>
CHAPTER 2

Type Selection

2-1 Introduction

All structure foundations have one fundamental characteristic in common: they transmit service and ultimate loads from the structure into the supporting geologic medium. Appropriateness of the different types of structure foundations is determined by:

- Loading requirements.
- Site-specific geologic conditions.
- Site accessibility.
- Overhead clearance.
- Existing utilities.
- Proximity of existing facilities to buildings and railroads.
- Vertical clearances.
- Noise restrictions.

The Foundation Report is the primary source for information about a project’s structure foundations. It is prepared by Geoprofessionals within Geotechnical Services. The Designer selects the appropriate foundation type based on data and recommendations contained in the report. The Foundation Report also may include recommendations and engineering data for several foundation types – in which case, field conditions and/or economics generally determine the foundation type.

2-2 Types

Structure foundations can generally be classified in the following categories:

1. Footing foundations (frequently referred to as spread footings).
2. Driven pile-supported foundations.
3. Non-driven pile supported foundations.
4. Special case foundation types, which include:
   a. Micropiles.
   b. Alternative piles.
   c. Pier columns.
   d. Ground anchors.
   e. Soil nails.
Seal courses may be specified as a foundation aid when groundwater and soil heave is anticipated. Seal course concrete is placed under water, the general purpose being to seal the bottom of a tight cofferdam or cast-in-steel-shell (CISS) pile against hydrostatic pressure. After the concrete cures, the water is pumped out of the cofferdam or steel shell, and construction of the footing can occur in dry conditions.

Generally, footing foundations are more economical than pile-supported foundations. Cast-in-drilled-hole (CIDH) concrete piles that are constructed in dry conditions tend to be the most economical. Pile-supported foundations with large diameter steel pipe piles generally are the most expensive.

Various geologic and non-geologic features affecting foundation type selection are presented in the following table; most are presented in more detail elsewhere in this manual.

Table 2-1. Features Affecting Foundation Type Selection.

<table>
<thead>
<tr>
<th>TYPE SELECTION</th>
<th>CIRCUMSTANCES FOR USE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Footing Foundations</td>
<td>Footing use is virtually unlimited. Geologic considerations include the soil profile, the location of the water table and any potential fluctuation, and the potential for scour or undermining. Non-geologic considerations include the size and shape of the footing, adjacent structures, and existing utilities.</td>
</tr>
<tr>
<td>2. Driven Piles</td>
<td>Driven piles are used where the underlying geologic formation will not support a footing foundation or discourages the use of CIDH concrete piles. Pile types are precast concrete, structural steel sections, steel pipe, and timber. Geologic considerations include the soil profile, driving difficulties, and corrosive soils. Non-geologic considerations include adjacent structures, existing utilities, required pile length, restricted overhead clearances, accessibility, and noise restrictions.</td>
</tr>
<tr>
<td>3. Non-Driven Piles</td>
<td>Non-driven piles consist of cast-in-drilled-hole (CIDH) concrete piles and alternative footing design piles. CIDH piles are used extensively where piles are required and foundation conditions permit their use. The slurry displacement method of construction of CIDH piles is used where driven piles are impractical and ground conditions necessitate its use. Alternative footing design piles are used on an experimental basis when conditions warrant their use. Geologic considerations include location of the water table and potential fluctuation, potential for caving and the soil profile. Non-geologic considerations include adjacent structures, existing utilities, restricted overhead clearances, and accessibility.</td>
</tr>
<tr>
<td>4. Special Case Foundations:</td>
<td>The following special applications have limited use: A. Micropiles, B. Alternative Piles, C. Pier Columns, D. Ground Anchors, and E. Soil Nails:</td>
</tr>
<tr>
<td>A. Micropiles</td>
<td>These are small-diameter piles (less than 12-inches) that are drilled and filled with reinforcement and grout.</td>
</tr>
<tr>
<td>B. Alternative Piles</td>
<td>These are newer pile types, such as Auger-cast piles, auger displacement piles, and soil mixing piles. They are generally used for soil improvement and for</td>
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<td></td>
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<tr>
<td>---</td>
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</tr>
<tr>
<td><strong>C. Pier Columns</strong></td>
<td>special applications where environmental constraints preclude use of standard foundations. Pier columns are an extension of the pier to a planned elevation into rock. They are generally used for hillside structures, eliminating the extensive excavation that would be required for large-spread footings. The location and type of existing structures may restrict excavation limits.</td>
</tr>
<tr>
<td><strong>D. Ground Anchors</strong></td>
<td>Ground anchors are used for earth-retaining structures where it is not feasible to excavate and construct a footing foundation or pile cap for a conventional retaining wall. They can also be used to address uplift concerns in seismic zones and for seismic retrofitting of existing footing foundations where uplift and overturning must be prevented. Geologic considerations include the soil profile and corrosive soil problems. Non-geologic considerations include adjacent structures, accessibility, and existing utilities.</td>
</tr>
<tr>
<td><strong>E. Soil Nails</strong></td>
<td>Soil nails are used for retaining walls in a similar manner as ground anchors, except soil nails are not pretensioned. They are not used to prevent uplift.</td>
</tr>
</tbody>
</table>
CHAPTER 3

Contract Administration

3-1 Introduction

Contract Administration is defined as the sum total of all the actions required by the Engineer to ensure that the contemplated work is constructed and completed by the Contractor in accordance with all terms of the contract.

These actions include, but are not limited to:
- Interpretation and enforcement of the plans and specifications.
- Compliance with applicable Caltrans policies and procedures.
- Objective and subjective decision-making (i.e., engineering judgment).
- Sampling, testing, and inspection of the work.
- Problem-solving that may result in contract changes to meet design intent.
- Proper documentation to defend Caltrans’ position regarding the accuracy of the information provided at the time of bid.

A well-administered contract is not always free from challenges and difficulty, but it will make possible a foundation that is best for the intended structure. Foundation operations are “high-risk” activities for all parties involved, as they have the potential to impact construction budgets and schedules. Although the Contractor’s contractual obligation is to construct and complete the project in accordance with the contract documents, changes to the contract are sometimes necessary to meet the intent of the Designer. Therefore, the best results are obtained when Caltrans and the Contractor work together. This enables both to identify issues as early as possible and work together for a resolution. Caltrans promotes “partnering” relationships with the Contractor to effectively complete the contract to the benefit of both parties; maintain cooperative communication, and resolve conflicts or challenges at the lowest possible level. This process is particularly important in foundation work where risks to the project are high and contract change orders may be required to effectively administer the contract.

The Engineer must thoroughly understand the contemplated work to determine if the Contractor successfully completed the contract. To achieve this, the Engineer must conduct a thorough study of the contract documents. This includes the Standard Specifications, Standard Plans, contract plans, Special Provisions, the Log of Test Borings, and the Foundation Report. The Engineer must become completely familiar
with the contract plans and their requirements as well as the Contractor’s construction schedule. In addition, the Engineer should:

- Check footing elevations.
- Ensure that there is adequate cover.
- Verify design bearing pressures.
- Look for special treatment of foundation provisions.
- Check the proximity of utilities, existing structures, highways and railroads, etc.
- Thoroughly understand the construction sequence.

A field investigation should be made of the proposed project site and, to the extent possible, the location of all utilities and obstructions should be verified prior to the start of construction. Conflicts or potential problems must be communicated to the appropriate parties so that a path to resolution may begin.

The table below outlines the documents to be reviewed by the Engineer in addition to the information described above.

**Table 3-1. Documents to be Reviewed.**

<table>
<thead>
<tr>
<th>DOCUMENT</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Log of Test Borings</td>
<td>Prepared by Geotechnical Services, this log provides the results of the geotechnical investigation. It provides a description of the soil or rock sampled in the field, test results for laboratory-tested samples, and groundwater elevations. It can be used to obtain soil profiles.</td>
</tr>
<tr>
<td>RE Pending File</td>
<td>This file is a collection of all correspondence relative to a particular project and, therefore, provides a historical outline of its development, and information relative to existing or proposed utilities, potential problems, and any other special considerations.</td>
</tr>
<tr>
<td>Preliminary Report</td>
<td>Prepared by the Preliminary Investigations Branch of Photogrammetry and Preliminary Investigations, this report is based on information furnished by the District and by data obtained during a field investigation of the proposed site. The report furnishes the Designer with the required roadway geometrics, clearances, proposed and existing utilities and/or obstructions, and will present any potential problems or other special considerations.</td>
</tr>
<tr>
<td>Foundation Report</td>
<td>Prepared by Geotechnical Services, this report is very informative and must be thoroughly reviewed. It provides detailed information about the foundation investigation conducted for the structure or project. It is part of the RE Pending File and included in the Supplemental Project Information handout to contractors. This report contains a description of the area geology, a Log of Test Borings for selected locations, and recommendations for foundation types and construction considerations.</td>
</tr>
<tr>
<td>As-Built Drawings</td>
<td>Prepared by Structure Construction after successful completion of a contract, these documents can be useful when widening or constructing new structures near or adjacent to existing structures.</td>
</tr>
</tbody>
</table>

The contract plans and specifications, the documents outlined above in Table 3-1, and a field investigation of the site must all be reviewed for compatibility. It is important that
all ambiguities, discrepancies, and omissions be resolved expeditiously to avoid unnecessary work delays.

In the past, the Log of Test Borings (LOTB) and other information provided to the Contractor at the time of bid were not considered part of the contract and were provided for information only. They now are included within the Bid Documents as the Supplemental Project Information handout. The 2006 Standard Specifications state that the Bidder is required to investigate the site and other available information, and it is understood that the information provided by Caltrans will be used by the Contractor to develop a competitive bid. Caltrans takes responsibility for the information provided. The Bidder is required to carefully examine the site and the information provided and is responsible for the conclusions that are drawn from these materials. The 2010 Standard Specifications simplifies this by requiring the Contractor to “Examine the job site and Bid Documents.” The LOTB and Foundation Report are included within the Bid Documents as Supplemental Project Information.

It is imperative for the Engineer to meet with the Designer and the Geoprofessional to discuss substructure considerations and foundation details. If an on-site meeting is impractical, the meeting should be held by telephone or teleconference. Any questions or inconsistencies must be clarified to establish an understanding of the foundation material as well as the potential risks or challenges anticipated in constructing the foundation. This also would be the appropriate moment to discuss the project with the Bridge Construction Engineer, preferably at the job site.

Once the contract documents have been reviewed and meetings held, the Engineer should have a firm grasp of the project’s technical and contractual requirements, as well as subsurface conditions that are expected to be encountered at the job site’s various foundation locations. Special attention should be given to those locations requiring extreme care in performing the work and resolving any remaining issues concerning utility relocations. These challenges and concerns should be presented at the preconstruction conference(s) to be held with the Contractor and other interested parties.

Preconstruction conferences usually are held when the Contractor begins mobilizing to the site, but well before work actually starts on the project. Five general subjects normally covered are:

1. Safety.
2. Labor compliance and affirmative action.
3. Utilities.
4. Environmental considerations.
5. Performance issues related to the work.

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1 2006 SS, Section 2-1.03, Examination of Plans, Specifications, Contract, and Site of Work.
2 2010 SS, Section 2-1.30, Job Site and Document Examination.
Depending on a particular District’s policies and the project’s complexity, more than one meeting may be appropriate in order to limit the scope and number of individuals present. Meetings should result in a common understanding of the proposed work, the risks, challenges, and potential solutions that may be expected throughout the contract.

The preconstruction conference presents an excellent opportunity to focus on inherent risks in foundation work, specific project challenges, and specifications that could have significant impacts on the Contractor’s operations. Since contracts vary and many specifications govern foundation work, it is impossible to list all of the items that might be considered. Table 3-2 lists the specification items that must be covered and understood for effective contract administration:

Table 3-2. Specifications to be Reviewed at the Preconstruction Conference.

<table>
<thead>
<tr>
<th>ITEM</th>
<th>REFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Boring Information</td>
<td>2010 SS, Section 2-1.30, Job Site and Document Examination, or 2006 SS, Section 2-1.03, Examination of Plans, Specifications, Contract, and Site of Work.</td>
</tr>
<tr>
<td>Excavation Safety Plans; Trench Safety</td>
<td>2010 SS, Section 7-1.02K(6)(b), Excavation Safety, or 2006 SS, Sections 5-1.02A, Excavation Safety Plans, &amp; 7-1.01E, Trench Safety.</td>
</tr>
<tr>
<td>Differing Site Condition</td>
<td>2010 SS, Section 4-1.06, Differing Site Conditions, or 2006 SS, Section 5-1.116, Differing Site Conditions.</td>
</tr>
<tr>
<td>Source of Materials</td>
<td>2010 SS, Section 6-2, Material Source, or 2006 SS, Section 6-1.01, Source of Supply and Quality of Materials.</td>
</tr>
<tr>
<td>Water Pollution</td>
<td>2010 SS, Section13, Water Pollution Control, or 2006 SS, Section7-1.01G, Water Pollution.</td>
</tr>
<tr>
<td>Sound Control Requirements</td>
<td>2010 SS, Section 14-8, Noise and Vibration, or 2006 SS, Section7-1.011, Sound Control Requirements.</td>
</tr>
<tr>
<td>Public Safety</td>
<td>2010 SS, Section 7-1.04, Public Safety, or 2006 SS, Section 7-1.04 Public Safety.</td>
</tr>
<tr>
<td>Preservation of Property</td>
<td>2010 SS, Section 5-1.36, Property and Facility Preservation, or 2006 SS, Sections 7-1.11, Preservation of Property, &amp; 19-1.02, Preservation of Property</td>
</tr>
<tr>
<td>Protection of Utilities</td>
<td>2010 SS, 5-1.36D, Non-Highway Facilities, or 2006 SS, Section 8-1.10, Utility and Non-Highway Facilities.</td>
</tr>
<tr>
<td>Cofferdams</td>
<td>2010 SS, Section 19-3.03C, Cofferdams, or 2006 SS, Section 19-3.03, Cofferdams.</td>
</tr>
<tr>
<td>Water Control &amp; Foundation Treatment</td>
<td>2010 SS, Section 19-3.03D, Water Control and Foundation Treatment, or 2006 SS, Section 19-3.04, Water Control and Foundation Treatment.</td>
</tr>
<tr>
<td>Foundation Inspection</td>
<td>2010 SS, Section 19-3.03B(1), Structure Excavation, General, or 2006 SS, Section 19-3.05, Inspection.</td>
</tr>
<tr>
<td>ITEM</td>
<td>REFERENCE</td>
</tr>
<tr>
<td>--------------------------</td>
<td>---------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Seal Course</td>
<td>2010 SS, Sections 51-1.03D(3), <em>Concrete Placed Under Water</em>, &amp; 51-1.04, <em>Payment</em>, or 2006 SS, Section 51-1.10, <em>Concrete Deposited Under Water</em>.</td>
</tr>
<tr>
<td>Special Concrete Mix Designs</td>
<td>Special Provisions</td>
</tr>
<tr>
<td>Applicable Caltrans Policies</td>
<td>Various Manuals</td>
</tr>
</tbody>
</table>

On some projects, the scope and complexity of foundation work may require scheduling a separate preconstruction meeting just to address foundation work. For projects with cast-in-drilled-hole (CIDH) piles, the *Special Provisions* require a separate CIDH pile preconstruction meeting to establish contacts and communication protocols for the Contractor, Engineer, and their representatives involved in CIDH pile design and construction.

### 3-2 Utilities

All utility locations shown on the contract plans should be verified with the utility representative. Utilities constructed by local municipalities and Caltrans are not verified by the Utilities Service Alliance (USA) and Caltrans and each individual municipality will be required to identify and locate them. The Engineer should request as-built plans from the local municipality and conduct field meetings to verify the locations of these existing facilities prior to excavation.

The Contractor is required to notify the proper agencies to have the existing underground utilities located in the field prior to commencing excavation operations. The status of utilities not yet relocated and field evidence of additional existing utilities also must be discussed. Problems in this area could result in serious delays. If not solved at the preconstruction conference, these utility issues should be resolved at the earliest possible opportunity.

The Contractor’s proposed methods of performing foundation work adjacent to utilities also should be discussed at the preconstruction conference. All parties should be advised of any proposed change orders that potentially may affect their work or property.

All preconstruction conferences should be well documented. When appropriate, minutes of the meeting should be distributed to all attendees, to confirm positions taken and agreements made at the meeting.
Proposed foundation changes, whether the result of geologic or non-geologic conditions, should be discussed with the Bridge Construction Engineer. Depending on the extent of the proposed change, it may be advisable to consult with Structure Design and Geotechnical Services.

### 3-3 Change Orders

Certain revisions in excavation limits, footing elevations and sizes, and changes to or elimination of seal course concrete are presented in the contract documents. This gives the Engineer the authority to give written direction to the Contractor to implement various changes in the field. As most items are final pay items, a contract change order ultimately will be needed in order to allow the quantity change for the items affected by this revision. Once it is determined that a change is necessary, the Contractor is issued a contract change order describing the work to be done, the basis of compensation, and the extent of any time extension.

To eliminate any possible misunderstanding about field revisions of foundations, a letter should be sent to the Contractor prior to commencing foundation operations. Table 3-3 below lists the items that the letter should advise on. An example of this letter is provided in Appendix C, *Footing Foundations*.

**Table 3-3. Items to Consider for Letter to Contractor prior to Construction of Foundations.**

<table>
<thead>
<tr>
<th>ITEM</th>
<th>REMINDER/STATEMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A reminder that the contract specifications gives the Engineer the right to revise, as may be necessary to secure a satisfactory foundation, the footing size and bottom of footing elevations shown on the contract plans.</td>
</tr>
<tr>
<td>2</td>
<td>On projects involving seal courses, a reminder that the contract specifications allows the Engineer to revise or eliminate the seal course shown on the contract plans.</td>
</tr>
<tr>
<td>3</td>
<td>A statement to the effect that final footing elevations and/or the need for seal courses will be determined by the Engineer at the earliest possible time consistent with the progress of the work, and that the Contractor will be notified, in writing, of the Engineer’s decision.</td>
</tr>
<tr>
<td>4</td>
<td>Caution the Contractor that work done or materials ordered prior to receiving the Engineer’s decision regarding foundations is done at their risk, and that they assume the responsibility for the cost of alterations to such work or materials in the event revisions are required.</td>
</tr>
</tbody>
</table>

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5 Bridge Construction Memo 2-9.0, *Footing and Seal Course Revisions*.
4 Bridge Construction Memo 2-9.0, *Footing and Seal Course Revisions*.
5 2010 SS, Section 51-1.03C(1), *Construction Preparation, General* or 2006 SS, Section 51-1.03, *Depth of Footings*.
3-4 Pile Foundations

3-4.1 Driven Piles
In accordance with the contract specifications, driven piles must achieve the required nominal driving resistance and penetrate to the specified tip elevation unless otherwise permitted in writing by the Engineer. Nominal driving resistance is usually determined by the Gates Formula. Additional information regarding this formula can be found in Chapter 7, Driven Piles, and in Bridge Construction Memo 130-4.0, Pile Driving Acceptance Criteria. The nominal driving resistance for large diameter piles is determined from non-destructive testing such as the pile driving analyzer or static pile load tests. Driven piles that are to be load-tested, need to be driven to the specified tip elevation shown on the contract plans. The nominal driving resistance will be determined from the pile load test. Revisions to specified tip elevations may be required as a result of the values obtained during testing. Procedures for load testing piles are discussed in Chapters 7, Driven Piles, and 8, Static Pile Load Testing and Pile Dynamic Analysis, of this Manual.

During pile-driving operations, one of the following scenarios will occur:

- The pile will achieve the required nominal driving resistance and specified tip elevation.
- The pile will achieve the required nominal driving resistance but falls short of the specified tip elevation.
- The pile will not achieve the required nominal driving resistance at the specified tip elevation.

As a result of this variability, the Contractor may decide to furnish pilings of a longer length than those shown on the contract plans. Sometimes the Contractor will elect to continue driving the pile beyond the specified tip elevation even though the required nominal driving resistance has been achieved. This is often done to avoid the cost of cutting off the extra length of pile so that the top of the pile is at the specified cutoff elevation. In these situations, the Contractor should be notified in writing that the cost of additional driving and length of pile are at the Contractor’s expense.

The Engineer may revise the specified tip elevation as provided in the contract specifications, either to allow acceptance of piles that do not reach the specified tip elevation or to require continued driving until the required nominal penetration is achieved. When considering revisions to the specified tip elevation, particular attention must be paid to the information provided about the pile data sheets of the contract plans. These sheets contain information on the design requirements or constraints for the piles and may include design tip elevations for compression, tension, lateral, downdrag.

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7 2010 SS, Section 49-2.01A(4)(b), Pile Driving Acceptance Criteria or 2006 SS, Section 49-1.08, Pile Driving Acceptance Criteria.
8 2010 SS, Section 49-2.01C(1), Construction, General or 2006 SS, Section 49-1.08, Pile Driving Acceptance Criteria.
liquefaction, and scour potential, among others. The specified tip elevation is the deepest foundation elevation and is the one that controls the design. Revisions to tip elevations may impact the performance of the pile and need to be discussed with Structure Design and Geotechnical Services. This is particularly important when compression does not control the design.

When driven piling is measured for payment, the measurement and payment method referenced in the contract specifications must be used. Refer to Bridge Construction Memo 130-6.0, *Payment for Piling*, for additional information.

The Engineer should consider using lugs to reduce the additional pile length required when steel “H” piles exhibit a trend that indicate the piles need to penetrate beyond the specified tip elevation in order to achieve the required nominal resistance. Lugs are pieces of steel that are welded to the pile to increase the surface area and provide greater driving resistance. When the Engineer orders lugs, the cost of furnishing and welding steel lugs to piles is paid for as contract change order work, at force account or agreed price. Bridge Construction Memo 130-5.0, *Steel H Pipe Lugs*, describes this process and shows a detail of a pile lug.

### 3-4.2 Cast-In-Drilled-Hole Piles

On projects involving cast-in-drilled-hole (CIDH) concrete piles, the Engineer must notify the Contractor in writing that:

- CIDH piles must penetrate at least to the specified tip elevation shown on the contract plans or as ordered by the Engineer, and
- No additional payment will be made for piles that penetrate below the specified or ordered tip elevation.

Any change ordered by the Engineer must be in writing.

In certain instances, the Contractor has the option to submit a proposal to increase the diameter and revise the tip elevation of CIDH piles. These revisions must be made in accordance with the contract specifications. In this instance, the Contractor is paid the theoretical length of the specified pile to the specified tip elevation. The Engineer should consult with Structure Design and Geotechnical Services before agreeing to this change.

CIDH concrete piles sometimes are constructed in the presence of groundwater using the “wet method.” This operation uses drilling slurry to control groundwater and to maintain the stability of the drilled hole. Concrete is placed using a tremie, and visual inspection is not possible. Caltrans uses non-destructive testing for these pile types to verify pile integrity. Chapter 9, *Slurry Displacement Piles*, describes this process and outlines the roles and responsibilities of the Engineer to have the piles tested and to address the repair of any defective material identified by the testing.

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9 2010 SS, Section 49-3.02C(1), *Construction, General* or 2006 SS, Section 49-4.03, *Drilled Holes.*
3-5 As-Built Drawings and Pile Records

The Engineer is required to monitor the installation of piles during foundation operations that involve driven or CIDH piling and keep accurate records of these activities. Bridge Construction Memo 3-7.0, CIDH Concrete Piling, discusses and explains the various forms that are to be completed during these activities. The information recorded on the forms is valuable to Caltrans, as it may be used to help assist in the acceptance of piling that does not reach specified tip elevation/nominal resistance or to provide information for the resolution of construction claims. Geotechnical Services uses the information to refine recommendations for future projects. In addition to the forms, SC Headquarters Office keeps a database of various aspects of CIDH piling that is constructed using the “wet method”.

Bridge Construction Memo 9-1.0, As-Built Plans, incorporates As-Built plans as a part of the final records and reports. As-Built plans should provide an accurate portrayal of what was constructed. This information is important when changes are made to the structure after original construction is complete. For example, footing overpours need to be shown on the As-Built plans, as they could eventually become a problem during the construction of footing widenings and seismic retrofits. Other problems can arise when existing shoring and utilities that are moved or left in place were not shown on As-Built plans. These issues, among others, have added to the cost of projects involving improvements to existing structures.

3-6 Differing Site Conditions

The concept of a differing site condition is unique to substructure and foundation work. Differing Site Conditions (DSC) can be identified by either party and are defined in the contract specifications. Differing Site Conditions occur when the Contractor or Engineer finds:

- Physical conditions differing materially from the contract documents or a jobsite examination, such as ground conditions differing materially from those shown on the Log of Test Borings or Foundation Report or the presence of groundwater where none was indicated.

or

- Physical conditions of an unusual nature, differing materially from those ordinarily encountered and generally recognized as inherent in the work provided for in the contract, such as archaeological finds or buried hazardous materials where none were indicated.

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10 Bridge Construction Memo 130-13.0, CIDH Pile Information Submittal.
11 2010 SS, Section 4-1.06, Differing Site Conditions or 2006 SS, Section 5-1.116, Differing Site Conditions.
Timely notification about, documentation of, and response to differing site conditions are of critical importance. Consult with the Resident Engineer and the Bridge Construction Engineer immediately upon receipt of a Notice of Differing Site Condition.

After investigating conditions at the job site, the Engineer decides whether the Contractor’s Notice of Differing Site Condition has merit. Should the Engineer find merit, a change order would be negotiated and processed. Should the Engineer find no merit, the Contractor has a timeframe to submit a protest of the decision with a Notice of Potential Claim. If the Contractor opts to pursue the issue, the timelines established in the contract specifications must be followed. Section 3-404, “Differing Site Conditions” of the Construction Manual outlines the procedures to be followed, should the Engineer receive a Notice of Differing Site Condition or a Notice of Potential Claim regarding a differing site condition.

12 2010 SS, Section 5-1.43, Potential Claims and Disputer Resolution or 2006 SS, Section 9-1.04, Notice of Potential Claim.
Footing Foundations

4-1 Introduction

Footing foundations, also known as spread, combined, or mat footings, transmit design loads into the underlying soil mass through direct contact with the soil immediately beneath the footing. In contrast, pile-supported foundations transmit design loads into the adjacent soil mass through pile friction, end bearing, or both. This chapter addresses footing foundations. Pile foundations are covered in Chapter 5, *Pile Foundations-General*.

Each individual footing foundation must be sized so that the maximum soil-bearing pressure does not exceed the allowable soil bearing capacity of the underlying soil mass. As the load-bearing capacity of most soils is relatively low (2 to 5 Tons per Square Foot (TSF)), the result is footing areas that can be large in relation to the cross section of the supported member. This is particularly true when the supported member is a bridge column.

In addition to bearing capacity considerations, footing settlement also must be considered and must not exceed tolerable limits established for differential and total settlement. Each footing foundation also must be structurally capable of spreading design loads laterally over the entire footing area.

Since the foundation is supported only by the supporting soil mass, the quality of the soil is extremely important. The contract specifications allow the Engineer to revise the footing foundation elevations to ensure that they are on quality material. Refer to Chapter 3, *Contract Administration*, for information on the responsibility of the Engineer as it applies to footing foundations.

4-2 Types

Footing foundations can be classified into two general categories:

1. Footings that support a single structural member, frequently referred to as “spread footings.”

---

2. Footings that support two or more structural members, referred to as “combined footings.”

Typically, columns are located at the center of spread footings, whereas retaining walls are eccentrically located in relation to the centerline of a continuous footing. Locating a load away from the centroid (center) of the footing creates an eccentricity that changes the distribution of loads in the soil and may result in a bearing pressure that exceeds the allowable bearing capacity. These undesirable loading conditions increase the further the column is placed from the centroid or as the eccentricity increases. The worst of these cases is an edge-loaded footing where the edge of the column is placed at the edge of the footing. The major consideration for these footings is excessive settlement and/or footing rotation on the eccentrically loaded portion of the footing. The effect of column eccentricity on footing rotation and soil-bearing pressures is similar to a centrally loaded footing with a moment. This also will cause an unbalanced load transfer into the soil as shown in Figure 4-1.

![Figure 4-1. Loaded Footing with Moment.](image)

In Figure 4-1, the moment (M) may come from a loading condition that needs to be transferred into the soil mass or may be the resultant of the length of the eccentricity multiplied by the load (P). The phrase “outside the kern” refers to a situation when the eccentricity is so great that there is no compression or, worse, there is tension on one side of the footing.

Problems resulting from eccentricities can be addressed by combining two or more columns onto a single footing. This usually is accomplished by one of two methods. In the first method, a single rectangular or trapezoidal footing supports two columns (combined footing). In the other method, a narrow concrete beam structurally connects two spread footings. This type is a cantilever or strap footing.

Combined footings generally are required when loading conditions (magnitude and location of load) are such that single-column footings create undesirable loading conditions, are impractical, or uneconomical. Combined footings also may be required
when column spacing is such that the distance between footings is small or when columns are so numerous that footings cover most of the available foundation area. Generally, economics will determine whether these footings should be combined or remain as individual footings. A single footing that supports numerous columns and/or walls is referred to as a mat footing, and is commonly seen in building work.

Caltrans performed seismic retrofits of spread footings extensively throughout the 1990’s. Although this is not a separate category, it is important to understand that foundation work sometimes entails modifications of an existing structure. While the retrofit program is, for the most part, complete there still are structures that may need upgrades either for seismic concerns, scour, or bridge widening. Details of previous footing retrofit strategies are shown in Appendix C, *Footing Foundations*.

Footing foundations encountered in bridge construction almost always support a single structural member (column, pier, or wall) and invariably are referred to as spread footings. Although closely spaced columns do occur in multiple column bents, they are rarely supported on a combined footing. However, recent seismic and scour retrofit projects have incorporated designs that joined together the adjacent footings.

### 4-3 Bearing Capacity

The ultimate bearing capacity of a soil mass supporting a footing foundation is the maximum pressure that can be applied without causing shear failure or excessive settlement. Ultimate bearing capacity solutions are based primarily on the Theory of Plasticity; that is, the soil mass is assumed to be incompressible (does not deform) prior to shear failure. After failure, deformation of the soil mass occurs with no increase in shear (plastic flow).

The implication of the previous statements is that theoretical predictions can only be applied to soils that are homogeneous and incompressible. However, most soils are neither homogeneous nor incompressible. Consequently, known theoretical solutions used in bearing capacity analyses have been modified to provide for variations in soil characteristics. These modifications primarily are based on empirical data obtained through small and, more recently, large-scale testing.

The ultimate soil strength is referred to as Gross Ultimate Bearing Resistance \( q_n \) in Load Resistance Factor Design (LRFD) and Ultimate Gross Bearing Capacity \( q_{ult} \) when working with Working Stress Design (WSD). Once \( q_n \) and \( q_{ult} \) are calculated, the value is reduced by a factor of safety. The revised value is referred to as Allowable Bearing Capacity \( q_{all} \).
4-3.1 Failure Modes

The mode of failure for soils with bearing capacity overloads is shear failure of the soil mass that supports the footing foundation. It will occur in one of three modes:

1. General shear.
2. Punching shear.
3. Local shear.

The Theory of Plasticity describes the general shear failure mode. The other two failure modes: punching and local shear, have no theoretical solutions.

A general shear failure is shown in Figure 4-2 and can be described as follows: The soil wedge immediately beneath the footing (an active Rankine zone acting as part of the footing) pushes Zone II laterally. This horizontal displacement of Zone II causes Zone III (a passive Rankine zone) to move upward.

![Figure 4-2 General Shear Failure Concept.](image)

General shear failure is a brittle failure and usually is sudden and catastrophic. Although ground surface bulging may be observed on both sides of the footing after failure, the failure usually occurs on one side of the footing. Two examples of this failure are:

1. An isolated structure may tilt substantially or completely overturn.
2. A footing restrained from rotation by the structure will see increased stresses in the footing and column portions of the structure, which may lead to excessive settlement or collapse.

A punching shear failure (Figure 4-3) presents little, if any, ground surface evidence of failure, since the failure occurs primarily in soil compression immediately beneath the footing. This compression is accompanied by vertical movement of the footing and may or may not be observed, i.e., movement may be occurring in small increments. Footing stability usually is maintained throughout failure (no rotation).
Local shear failure (Figure 4-4) may exhibit both general and punching shear characteristics, soil compression beneath the footing, and possible ground surface bulging.

Refer to Figure 4-5 for photographs of actual test failures using a small steel rectangular plate (about 6 inches wide) and sand of different densities.

The failure mode of a given soil profile cannot be predicted. However, it can be said that the mode of failure depends substantially on the compressibility or incompressibility (Relative Density) of the soil mass. This is not to imply that the soil type of the underlying
material alone determines failure mode. For example, a shallow footing supported on very dense sand will usually fail in general shear, but the same footing supported on very dense sand that is underlain by a soft clay layer may fail in punching shear.

The ultimate bearing capacity of a given soil mass under spread footings usually is determined by one of the variations of the general bearing capacity equation, which was derived by Terzaghi and later modified by Mererhof. It can be used to compute the ultimate bearing capacity as follows:

\[
q_{ult} = \frac{\gamma B}{2} N_\gamma + c N_c + \gamma D_f N_q \quad \text{(Terzaghi)}
\]

Where: \( q_{ult} \) = ultimate bearing capacity
\( \gamma \) = soil unit weight
\( B \) = foundation width
\( D_f \) = depth to the bottom of the footing below final grade
\( c \) = soil cohesion, which for the un-drained condition equals:

\[
c = s = \frac{1}{2} q_u
\]

Where: \( s \) = soil shear strength
\( q_u = \) the unconfined compressive strength

In the above equation, \( N_\gamma \), \( N_c \), and \( N_q \) are dimensionless bearing capacity factors that are functions of the angle of internal friction. The term containing factor \( N_\gamma \) shows the influence of soil weight and foundation width. The term containing factor \( N_c \) shows the influence of the soil cohesion, and that of \( N_q \) shows the influence of the surcharge.

**4-3.2 Factors Affecting Bearing Capacity**

Several factors can affect the bearing capacity of a particular soil. They include soil type, relative density or consolidation, soil saturation and location of the water table, and surcharge loads. These factors can act individually or in concert with each other to increase or decrease the bearing capacity of the underlying soil.

When the supporting soil is a cohesionless material (sands), the most important soil characteristic in determining the bearing capacity is the relative density of the material. An increase in relative density is accompanied by an increase in the bearing capacity. Relative density is a function of both \( \phi \) and \( \gamma \); the angle of internal friction and unit weight, respectively. In cohesive soils (clays), the unconfined compressive strength (\( q_u \)) is the soil
characteristic that affects bearing capacity. The unconfined compressive strength \( (q_u) \) is a function of clay consistency. The bearing capacity increases with an increase in \( q_u \) values. The bearing capacity of both sands and clays are influenced by the location of the water table with respect to the bottom of the footing. When the distance to the water table from the bottom of the footing is greater than or equal to the width of the footing \( B \), (Refer to Figure 4-6), the soil unit weight is used in the general bearing capacity formula. At these depths, the bearing capacity is only marginally affected by the presence of water and can be disregarded. When the water table is at or below the base of the footing, a ratio between the unit weight of the soil above the water table and the submerged unit weight is used in the first term of the bearing capacity equation. The impact of the water table on the bearing capacity of the soil beneath the bottom of the footing is substantial as it effectively reduces the first term of the equation by approximately 50%. The submerged unit weight \( \gamma' \) or \( \gamma_{\text{sub}} \), as it is sometimes called, is determined as follows:

\[
\gamma' = \gamma_{\text{sat}} - \gamma_w
\]

Where: \( \gamma' = \) Submerged unit weight
\( \gamma_{\text{m}} = \) Saturated unit weight (Sometimes shown at \( \gamma_{\text{sat}} \))
\( \gamma_w = \) Unit weight of water

for \( z_w \geq B \) : use \( \gamma = \gamma_{\text{m}} \) (no effect)
for \( z_w < B \) : use \( \gamma = \gamma' + (z_w/B) \times (\gamma_{\text{m}} - \gamma') \)
for \( z_w \leq B \) : use \( \gamma = \gamma \)

![Figure 4-6. Influence of Groundwater Table on Bearing Capacity.](image)

It is apparent that bearing capacity of both cohesionless and cohesive soils will be reduced as the water table gets closer to the bottom of footings. This is validated by the general bearing capacity formula, as lower capacities will occur when the lighter submerged unit weight of soil is substituted for the dry unit weight. Therefore, the effects of the water table on the bearing capacity of the footing soil mass must be considered at all times during construction.
The depth of the footing below original ground or future finished grade is yet another factor that affects the bearing capacity of the soil beneath the foundation. The term $D_f$ is used in determining the overburden, or surcharge load, acting on the soil at the plane of the bottom of footing (Figure 4-7). This surcharge load has the net effect of increasing the bearing capacity of the soil by restraining the vertical movement of the soil outside the footing limits.

Lastly, the shape of the footing foundation affects the bearing capacity of the soil. Theoretical solutions for ultimate bearing capacity are limited to continuous footings (length/width $\geq 10$). Shape factors for footings (other than continuous footings) have been determined primarily through semi-empirical methods. In general, the ultimate bearing capacity of a foundation material supporting a square or rectangular footing is greater than the capacity of a continuous footing when the supporting material is cohesive (clay), and less than the bearing capacity of a continuous footing when the supporting material is cohesionless (sand).
Figure 4-9. Relationship of Bearing Capacity Factors to \( \phi \) and \( N \) (Standard Penetration Resistance) for Cohesionless Soils.

The general bearing capacity equation also can be used to give a field estimate of the ultimate bearing capacity of temporary footings, such as falsework pads. For cohesionless soils, a relationship between the standard penetration resistance, \( N \), and the bearing capacity factors, \( N_\gamma \) and \( N_q \), is shown in Figure 4-9. The relationship between \( N \) and the angle of internal friction, \( \phi \), also can be determined from Figure 4-9. When soils are known to have some cohesion, the value of \( \phi \) determined from Figure 4-9 then can be used in the chart shown in Figure 4-8 to determine the bearing capacity factors, \( N_\gamma \), \( N_c \), and \( N_q \). Values for \( \phi \), \( q_{us} \), \( N \), and \( \gamma \) can be found on the Log of Test Borings (LOTB) or can be approximated by using the tables for granular and cohesive soils shown in Appendix A, Foundation Investigations.
4-4 Settlement

Footing foundations will settle over time as the soil densifies from the additional weight it is required to support. Caltrans’s current practice is to limit total permissible settlement to:

- One inch for a shallow footing for multi-span structures with continuous spans or multi-column bents.
- One inch for single span structures with diaphragm abutments.
- Two inches for single span structures with seat abutments.

To achieve this, allowable bearing pressures generally are reduced to 25% to 33% of the ultimate bearing capacity as determined by the general bearing capacity formula. This reduction essentially places a factor of safety on the ultimate bearing capacity and is in line with the reductions discussed above to obtain allowable and nominal bearing capacities.

Cohesionless soils will densify under the pressure of the foundation as the individual soil particles are pushed together, effectively compacting it. In general, soils with low relative densities will see more settlement than well-compacted soils that have higher relative densities. Settlement is immediate in cohesionless materials. Cohesive soils, however, consolidate over time as the pressure of the overlying foundation forces water from the soil, relieving excess pore water pressures.

4-5 Ground Improvement/Soil Modification

Bridges frequently need to be constructed at locations where the in situ material is not suitable for the intended purpose. Instead of utilizing a pile foundation, Geotechnical Services specifies ground modification of the foundation area to “engineer” it for its intended use. Economics, soil type, and engineering loads drive the decision to use ground modification and avoid the additional cost of a pile foundation.

Ground modification techniques are used to increase the bearing capacity of the foundation material by increasing the relative compaction of the material either through densification or the introduction of grouts to compress and bind the soils. Ground modification techniques generally lend themselves to cohesionless materials. These techniques can include the following: settlement periods, vibro-compaction, jet grouting, stone columns, dynamic compaction, and wick drains, among others. These modification techniques improve the bearing capacity of the soil by increasing the relative density of the soil through external means, or by adding materials such as a cement or chemical grout to achieve a similar result. Modification of cohesive soils can be achieved; however, these methods often are time-consuming and limited to wick drains and settlement periods. As discussed later in this chapter, the replacement of poor quality soils by over-excavation and replacement with competent material may be appropriate.
Some modification techniques involve a settlement period where the underlying foundation is preloaded with a surcharge for a specified length of time prior to the foundation construction. The loading typically consists of an embankment constructed to specified limits. Geotechnical Services determines the need to preload the foundation area, specifies the limits of the embankment, and sets forth the duration of the settlement period in the contract Special Provisions.

When settlement periods are less than 60 days, the Engineer should install settlement hubs in the top of the bridge embankments and monitor (survey) and record changes to the original elevations. The Engineer is responsible for terminating a settlement period. Data from the hub elevation surveys are used to determine when this should take place. If settlement is still taking place at the end of the 60-day period, then the settlement period should be extended until the settlement has ceased. However, if no settlement occurred during the last week or two of the settlement period, the settlement period should be terminated at the end of the 60-day period or to shorten the length of the settlement period. The Contractor should be notified of this decision in writing.

Settlement platforms usually are required when settlement periods greater than 60 days are specified. Geotechnical Services has a Geotechnical Instrumentation Branch that provides advice for the installation of the settlement platforms (Refer to Appendix C, Footing Foundations, for California Test 112 - Method for Installation and Use of Embankment Settlement Devices). Unless this work is outlined in the Special Provisions, the Engineer needs to write a change order to compensate the Contractor for the initial installation of the settlement platforms.

4-6 Construction and Inspection

As discussed in Chapter 3, Contract Administration, the Engineer should have a complete understanding of all contract documents as early as practical in the construction process. This ensures that potential impact, with regard to the foundations, is identified early and paths to resolution are begun before actual construction begins.

The Engineer should write a letter reminding the Contractor of the provisions stated in the contract specifications (Refer to Chapter 3-3, Change Orders, of this Manual and Appendix C, Footing Foundations, for sample letter). This reminds the Contractor that footing elevations and seal courses shown on the contract plans are approximate only and foundation modifications may be required.

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2 Bridge Construction Memo 130-13.0, CIDH Pile Information Submittal.
3 2010 SS, Section 51-1.03C(1), Preparation, General, or 2006 SS, Section 51-1.03, Depth of Footing.
4 Bridge Construction Memo 2-9.0, Footing and Seal Course Revisions.
The Engineer should review and become familiar with the following documents as described in Chapter 3, *Contract Administration*. This table identifies specific sections of the *Standard Specifications* to consider for footing foundations.

### Table 4-1. Standard Specifications Sections for Footing Foundations.

<table>
<thead>
<tr>
<th>ISSUE</th>
<th>SPECIFICATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Contractor notification to Engineer when excavation is substantially complete and ready for inspection (prior to Engineer’s authorization to pour concrete).</td>
<td>2010 SS, Section 19-3.03B(1), <em>Structure Excavation, General</em>, or 2006 SS, Section 19-3.05, <em>Inspection</em>.</td>
</tr>
<tr>
<td>Relative compaction (of not less than 95% is required for embankments within 150 feet of bridge abutments or retaining wall footings not supported on piles).</td>
<td>2010 SS, Section 19-5.03B, <em>Relative Compaction (95%)</em>, or 2006 SS, Section 19-5.03, <em>Relative Compaction (95%)</em>.</td>
</tr>
<tr>
<td>Bridge footings constructed into an embankment (embankment constructed to the elevation of the grading plane, and finished slope extended to the grading plane before excavating for the footings).</td>
<td>2010 SS, Section 19-6.03A, <em>Construction, General</em>, or 2006 SS, Section 19-6.01, <em>Placing</em>.</td>
</tr>
<tr>
<td>Specifications for plan footing elevations and seal courses, and Engineer change orders to footing elevations (Bridge Construction Memo 2-9.0, <em>Foot ing and Seal Course Revisions</em>).</td>
<td>2010 SS, Section 51-1.03C(1), <em>Preparation, General</em>, or 2006 SS, Section 51-1.03, <em>Depth of Footing</em>.</td>
</tr>
<tr>
<td>Groundwater pumping from foundation enclosures that prevents removal of any concrete material.</td>
<td>2010 SS, Section 51-1.03C(1), <em>Preparation, General</em>, or 2006 SS, Section 51-1.04, <em>Pumping</em>.</td>
</tr>
<tr>
<td>Concrete placing, vibrating, and screeding procedures for footings.</td>
<td>2010 SS, Section 51-1.03D(1), <em>Placing Concrete, General</em>, or 2006 SS, Section 51-1.09, <em>Placing Concrete</em>.</td>
</tr>
</tbody>
</table>

### 4-7 Excavations

Excavation and trenching are inherent when constructing foundation elements, such as footing foundations. The *Caltrans Trenching and Shoring Manual* provides information about the complete process for administering, designing and reviewing excavation work and plans. What follows is a brief description of what to consider before excavation.
4-7.1 Open Excavations

The open excavation, or trench, is a potentially dangerous area at a construction site. Worker safety must be considered and addressed during excavation operations and/or shoring construction. The Division of Occupational Safety and Health (DOSH), better known as Cal/OSHA, requires each employee in an excavation be protected from cave-ins by an adequate protective system. The protective system can consist of metal or timber shoring, a shield system, a sloping system, or a sloping and benching system. When a sloping or sloping and benching system is substituted for shoring or other protective systems, and the excavation is less than 20-feet deep, DOSH requirements can be selected by the Contractor in accordance with the requirements of Section 1541.1(b) of the Construction Safety Orders. Section 1541.1(b)(1) allows slopes to be constructed (without first classifying the soil) in accordance with the requirements for a Type C soil (1½:1 maximum). Section 1541.1(b)(2) requires the Contractor’s “competent person” to first classify the soil as either a Type A, B, or C soil, or stable rock, before selecting the appropriate slope configuration. Section 1541.1(b)(3) allows the use of tabulated data under certain conditions; and Section 1541.1(b)(4) addresses engineered plans. The Engineer should refer to the Caltrans Trenching and Shoring Manual or go directly to the Cal/OSHA website when reviewing a Contractor’s excavation safety plan for compliance with the construction safety orders.

Surcharge loads from materials, equipment, or excavation spoils must be located a sufficient distance back from the edge of excavations to maintain slope stability. For sloped excavations, the minimum setback can be determined from Figure 4-10 on the following page.

5 http://www.dir.ca.gov/title8/1541_1.html
6 http://www.dir.ca.gov/samples/search/query.htm
Example:

\[ H = 20' \]
\[ \theta = 53 \text{ degrees (3/4:1)} \]
\[ \psi = 46 \text{ degrees} \]

\[ \frac{20'}{\tan(46)} = 19.31' \]
\[ \frac{20'}{\tan(53)} = 15.07' \]

\[ X = 19.31' - 15.07' = 4.24' \]

Figure 4-10. Slope Setback for Open Excavations/Trenches.
4-7.2 Cofferdams or Shored Excavations

Cofferdams and/or shored excavations require an engineered plan stamped by a registered Civil Engineer. The Contractor is responsible for designing these elements and the Engineer is responsible for review and authorization. The *Caltrans Trenching and Shoring Manual* provides procedures for reviewing and authorizing these plans. An important consideration in shored excavations is the minimum setback for a surcharge when on level ground. The setback usually is equal to the depth of the excavation unless specific surcharge loads are considered in the shoring design. The “Boussinesq” strip load formula is recommended for calculating the lateral pressures due to surcharge. (Figure 4-11). For example, no minimum setback of the surcharge load would be required if the earth support system is designed for the summation of lateral pressures due to the surcharge and earth pressures. However, a barrier should be provided to prevent material from entering the excavation. The *Caltrans Trenching and Shoring Manual* includes several examples of how this formula is used, and the SC website has a spreadsheet that can be used to calculate the pressures. 

![Figure 4-11. Effect of Surcharge Loads for Shored Excavations.](https://onramp.dot.ca.gov/hq/oscnet) 

To calculate lateral pressures due to surcharge, the “Boussinesq” strip load formula is recommended.

At Depth “H”,

\[
\sigma_s (\text{PSF}) = \frac{2q}{\pi} (\beta_1 - \sin \beta \cos 2\alpha)
\]

where \(\beta_1\) is in radians

At full height \(H\),

\[
\alpha + \frac{\beta}{2} \leq \psi
\]

\[
\alpha = \arctan \left( \frac{X}{H} \right) + \frac{\beta}{2}
\]

\[
\beta = \arctan \left( \frac{X + W}{H} \right) - \arctan \left( \frac{X}{H} \right)
\]

\[
W_{max} = \tan \psi H - \tan \left( \arctan \left( \frac{X}{H} \right) \right) H
\]

---

If the earth support system is not designed for lateral pressures due to surcharge, then a setback distance must be used. It can be calculated as shown in Figure 4-12. Setback information should be shown on the authorized shoring plans and clearly designated in the field. Refer to the Caltrans Trenching and Shoring Manual for information regarding shoring design and construction.

![Diagram of setback calculation](image)

Let \( X \) = setback of surcharge load

\[
X = \frac{H}{\tan \left( 45 + \frac{\phi}{2} \right)}
\]

\( \phi \) = \( \angle \) of internal friction

\( \psi \) = \( \angle \) of failure plane = 45 + \( \frac{\phi}{2} \)

For most soils, \( \psi \) is about 55°

Figure 4-12. Setback Calculation for Shored Excavations When Surcharges are not Considered in the Shoring Design.

4-7.3 Wet Excavations

The contract specifications describe methods to be utilized when water is encountered in excavations and seal courses are not shown on the contract plans. The means and methods used to control groundwater are at the option of the Contractor and need to be clearly understood, as there are environmental considerations when dealing with groundwater control. Contract specifications address the control and disposal of ground water. All SC employees have the responsibility to inspect structure work for compliance with environmental regulations; as such, these operations should be discussed with the Resident Engineer to ensure that the environmental considerations are addressed before any work begins.

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8 2010 SS, Section 19-3.03D, Water Control and Foundation Treatment, or 2006 SS, Section 19-3.04, Water Control and Foundation Treatment.

9 2010 SS, Section 13-4.03G, Dewatering, or Special Provisions for contracts using 2006 SS.
Sump pumps frequently are used to remove surface water and minor infiltrations of groundwater that enter an excavation. The sumps and any connecting interceptor ditches should be located well outside of the footing area, and below the bottom of footings so that groundwater will not disturb the foundation’s bearing surface.

In cohesionless (granular) soils, it is important to make sure that the fine particles within the soil mass are not carried away by the pumping operation. Loss of fines may impair the bearing capacity of the soil for the foundation under construction, and also may cause existing structures adjacent to the operations to settle. The amount of soil particles carried away can be determined by periodically collecting discharged water in a container and observing the amount of sediment. If there is a large flow of groundwater and/or prolonged pumping is required, the sump(s) should be lined with a filter material to prevent or minimize the loss of fines.

In some excavations, the use of sumps may not be sufficient to address the infiltration of groundwater into the excavation. When this is the case, cofferdams generally are used; however, some contractors will opt to lower the groundwater table. One commonly used method to achieve this is with the single-stage well point system (Figure 4-13).

![Figure 4-13. Single Stage Well Point System.](image)

A well point is a section of perforated pipe about 2 to 3 inches in diameter and 2 to 4 feet in length. Perforations are covered with a screen and the end of the pipe is equipped with a driving head and/or holes for jetting. Several well points are installed around the excavation perimeter, generally spaced at 2 to 5-foot centers. They are connected to 2 to 3-inch diameter riser pipes and are inserted into the ground by driving and/or jetting. The riser pipes are connected to a header pipe that is connected to a pump. A single stage well point system can lower the water table 15 to 18 feet below the elevation of the header pipe. For greater depths, a multiple stage system must be used. A single or multiple stage well point system is effective in fine to medium granular soils or soils containing seams.
of such material. In stratified clay soils, vertical sand drains (auger holes backfilled with sand) may be required to draw water down from above the well points.

Another system for lowering the water table is a deep well. Deep wells consist of either a submersible pump, turbine, or water ejector at the bottom of 6 to 24-inch diameter casings, either slotted or perforated. Units are screened, but filter material should be provided in the well to prevent clogging and loss of fines.

Deep wells can be spaced 25 to 120 feet apart and are capable of lowering a large head of water. They can be located a considerable distance from the excavation and are less expensive than the multiple stage well point system for dewatering large areas; however, they are only appropriate in certain soils.

If a soft clay strata overlying sand is encountered and dewatering is contemplated, lowering the water table by pumping from underlying layers of sand may not be a preferred option because it will cause large, progressive settlement of the clay strata in the surrounding area. By lowering the water table in the sand lens, the condition in the clay lens switches from an un-drained condition to a drained condition. This allows excess pore water pressures to be dissipated more quickly and to a greater extent than it would have been had the water table not been lowered. Essentially, there is an increase in the effective pressure acting on the saturated clay, i.e., density of clay above the lowered water table will increase from a submerged unit weight to a saturated unit weight, an increase of 62.4 Pounds per Cubic Foot (PCF) (Figure 4-14).

![Figure 4-14. Saturated vs. Submerged Unit Weight.]

4-7.4 Bottom of Excavation Stability

The control of groundwater is essential to the stability of a shoring system and the underlying soil intended to support the new foundations. In addition to controlling groundwater to facilitate construction operations, the Engineer also must consider soil heave and piping as they relate to the stability of the bottom of the excavation.

Heave is the phenomenon whereby the static or hydraulic pressures (head) of the surrounding material cause the upward movement of the material in the bottom of the excavation. This corresponds with settlement of the surrounding material. Heave
generally occurs in soft clays when the hydrostatic head, $62.4(h + z)$, is greater than the weight of the overburden at the bottom of the excavation, $\gamma z$ (Figure 4-15).

![Figure 4-15. Bottom of Excavation Stability Problems Due to Excess Hydrostatic Head Against an Impervious Layer.](image)

Piping is associated with pervious materials and can occur when an unbalanced hydrostatic head exists. This unbalanced head may cause large upward flows of water into the excavation, transporting material in the process, and may result in settlement of the surrounding area. Review the *Caltrans Trenching and Shoring Manual* if instability problems are expected at the bottom of excavations.

### 4-7.5 Foundation Inspection & Construction Considerations

A *Footing Foundation Construction Checklist* is presented in Appendix K-4 to assist field personnel in preparing documents and inspecting field work to ensure compliance with contract requirements.

Inspection should confirm the following:

- Stability of slopes and sides of excavations conform to CalOSHA requirements.
- Foundation materials conform to the information shown on the LOTB (allowance should be made for some non-uniformity such as small pockets and lenses of material having somewhat different properties).
- Condition of the foundation-bearing surface is undisturbed by excavation operations and uncontaminated by sloughing and/or entrance of water.
- Proximity of structures, highways, railroads, and other facilities that may require shoring or underpinning, (checked prior to excavation) conform to guidelines.
- Foundation element forms conform to layout, depth, dimensions, and construction grade shown on the contract plans. Forms are mortar-tight.
• Reinforcing steel is firmly and securely tied in place; shear steel is hooked to both top and bottom rebar mats and securely tied; and there is a proper concrete cover over the top rebar mat.

• Concrete has the proper mix number; adequate truck revolutions; concrete temperature and back-up alarm; wet down rebar and forms; does not drop over 8 feet; and has been reconsolidated and the top one foot of concrete finished no sooner than 15 minutes after initial screeding before it is cured.

• A sufficient bench width around the excavation to prevent sloughing or cave-in and provide for access and for work area.

The footing forms are either built out of timber or consist of prefabricated panels. The forms generally are secured at the bottom by stakes, horizontal kickers, or ties, and are externally braced, tied or strapped at the top. If the forms extend above the top of the footing elevation, a pour strip or similar device must be attached to the forms to designate the top-of-footing elevation.

Footings for shored excavations often are excavated and placed “neat,” which means that the excavation limits essentially are the footing limits. Concrete is placed against the sides of the excavation, eliminating the need for footing forms. Top-of-footing grades must be clearly delineated with stakes or flagged spikes driven into the sides of the excavation. “Neat” excavations must conform to the planned footing dimensions. If they vary, the exact, as-constructed footing dimensions must be placed on the “as-built” drawings. (Previous seismic retrofit projects and footing widenings were not “as-built” properly and costly contract change orders were required to address these undocumented overpours. Care should be taken to make sure that the footing concrete is not damaged during shoring removal operations.)

Whether footings are formed or excavated “neat,” a template should be constructed to ensure that the position of the vertical reinforcing steel is maintained during concrete placement. All reinforcing steel must be securely blocked and tied to prevent vertical and/or lateral displacement during concrete placement. Reinforcing steel must not be hung or suspended from the formwork or templates, as the weight of suspended rebar can cause settlement in the form panels and affect pour grades and displace during concrete placement. Top reinforcing steel mats that are supported must be blocked to the forms or sides of the excavation. The bottom reinforcing steel mat that supports the vertical column steel must be adequately blocked to prevent any settlement. In addition, reinforcing steel dowels must be tied in place before concrete placement and not “wet-set” during or after concrete placement.

The effective depth of reinforcing steel is critical and always must be verified. For a footing supporting a single column, pier or wall, the effective depth is the distance from the centroid of the reinforcing steel to the top of the concrete footing. The bottom mat must be located at the design depth, even for over-excavated footings, since the bottom mat supports the vertical column reinforcement and the location of the top mat is tied to
the bottom mat by the shear hooks. Lowering the bottom mat is not recommended, as it would require longer vertical steel, longer shear hooks, and may require mechanical or welded splices on the longitudinal bars. It should be noted that the additional concrete placed below the bottom steel mat in over-excavated footings does not increase the design depth of the footing but should be noted on the as-built plan sheets.

Footing inspections should occur as the work progresses so that deviations and non-compliant issues can be addressed in a timely manner. However, it is important to inspect the footing just prior to concrete placement to ensure that nothing has changed. All material that has sloughed into the excavation must be removed prior to placing concrete. To verify that settlement of the rebar cage has not occurred, the minimum clearances between the bottom of the excavation and the bottom reinforcing steel mat must be re-inspected. The foundation material should be wet down but not saturated. The ends of the concrete pour chutes should be equipped to prevent free fall of concrete in excess of 8 feet. This will prevent segregation of the concrete and may include a hopper and/or length of tremie tube.

4-8 Foundation Problems and Solutions

The excavated surface at the planned footing elevation must be inspected after the excavation is completed. The Engineer must conduct a thorough physical inspection of the foundation material to determine if the foundation is suitable, disturbed and/or contaminated, or unsuitable. Addressing contaminated material is the responsibility of the Contractor, while unsuitable material is the responsibility of Caltrans. The phrase “contaminated material” as used here should not be confused with materials contaminated with lead, hydrocarbons, heavy metals, etc. Information about environmentally contaminated materials is addressed in the contract plans and Special Provisions.

4-8.1 Disturbed and/or Contaminated Material
Disturbed and/or contaminated foundation material encountered at the planned bottom of footing elevation is unacceptable and must be corrected even if the material itself is suitable. Disturbance of the foundation-bearing surface usually is caused by the excavation means and methods, including excavating below the footing elevation or disturbing the grade with the teeth on the excavator bucket. Contamination usually is due to the presence of water (typically uncontrolled) or sloughing. All disturbed or contaminated material must be removed to expose a suitable foundation surface. The foundation must then be restored by the Contractor, at the Contractor’s expense, to a condition at least equal to the undisturbed foundation as determined by the Engineer.

The following precautionary measures can be taken during excavation and construction to avoid or minimize the disturbance and/or contamination of the foundation surface:
- Under-excavate with mechanical equipment and excavate to bottom of footings by hand or by using a cleanup bucket
- Divert surface water away from the excavation
- Minimize exposure of the foundation material to the elements by constructing footings as soon as possible after excavation.

4-8.2 Unsuitable Foundation Material
The importance of confirming suitable foundation material cannot be overstated. The Engineer is responsible for determining the foundation suitability as it relates to the design intent. That is, the foundation material has to have the minimum material properties required for the structure to behave as the Designer intended. Simple tests can be performed in the field to determine the bearing capacity and verify the suitability of the foundation material. They are discussed in the *Caltrans Soil and Rock Logging, Classification, and Presentation Manual* and include:

- Penetration tests – granular soils.
- Finger tests – cohesive soils.
- Pocket penetrometer – cohesive soils.

Note that these simple and expeditious tests only give an approximate evaluation of the soil at or immediately below the surface.

The *LOTB* should be reviewed when the Engineer determines that the undisturbed original material encountered at planned footing elevation is either unsuitable or of a questionable nature. It may be that the anticipated suitable material may well be just below the excavated surface. If the Engineer is certain that the material encountered at the planned footing elevation is unsuitable, then hand-excavating a small exploratory hole to determine the limits of the unsuitable material may be appropriate. The questionable material, related concerns, and possible resolutions should be discussed with Geotechnical Services and the Designer.

4.8-3 Modifications Due to Disturbed, Contaminated or Unsuitable Material
Corrective action is required whenever changes in the bottom of footing elevations are made to address disturbed, contaminated, or unsuitable material. The Contractor is responsible for corrective actions to address disturbed or contaminated material. The Engineer is responsible for actions addressing unsuitable material. The corrective actions are similar in either situation. They fall into two categories:

1. Replacement of the original foundation material to achieve the original bottom-of-footing-elevation.
2. Revisions to the structure to address a different bottom-of-footing elevation.

There are engineering/design considerations in either of these actions and it is important to discuss considerations and consequences with the Designer, Geotechnical Services, and the Contractor, and work toward a solution that fulfills the design intent and keeps the project moving forward.
Options for restoring the foundation material at the bottom-of-footing elevation to its specified elevation after removal of unsuitable or contaminated material are as follows:

- Excavate to a stratum that has sufficient bearing capacity, replace the removed, unsuitable material with concrete, and then construct the footing at the planned footing elevation.
- Excavate to a stratum that has sufficient bearing capacity, replace the removed unsuitable material with aggregate base or structure backfill to 95% compaction, and then construct the footing at the planned footing elevation.

Revisions to address a different bottom-of-footing elevation or a lower-than-anticipated bearing capacity should be discussed with the Designer. Revisions may require a major structural redesign. The following are possible options, however, they may not be the best alternatives in real construction situations.

- Maintain top-of-footing as planned and overform footing depth. The rebar cage will remain at the theoretical elevation shown on the contract plans; however, the depth between the bottom-of-footing and the bottom mat of the rebar cage will be increased by the amount of over-excavation. This option is similar to previously described methods. It essentially exchanges the use of larger/taller footing forms for a reduction in the number of concrete pours. This option may well be the preferred option for minor revisions to bottom-of-footing elevations.
- Excavate down to a stratum that has sufficient bearing capacity and increase the height of the column or wall. This method may not be acceptable if the increase in height necessitates redesign of the column or wall. This decision should be discussed with the Designer.
- Increase the footing size so that the bearing pressure does not exceed the allowable bearing capacity of the foundation material encountered at the planned footing elevation. Settlement also must be considered, as it cannot exceed tolerable limits. This decision should be discussed with the Designer and Geotechnical Services.

Although footing revisions are contemplated by the contract documents, footing revisions made necessary due to unsuitable material encountered at the planned footing elevation will require a change order. Impacts to the construction schedule also must be considered when making these decisions. The Resident Engineer should be kept aware of these issues. The preferred method for compensating the Contractor for the cost of the corrective work is by adjustment of contract items at contract unit prices, and is included in the contract specifications as the method of payment for the following revisions:

- Raising the bottom of a spread footing above the elevation shown on the contract plans.
- Lowering the bottom of a spread footing 2 feet or less below the elevation shown on the contract plans.

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10 2010 SS, Section 19-3.04, Payment, or 2006 SS, Section 19-3.07, Measurement.
For other revisions, agreed price or force account methods should be used when the Engineer determines that the above method is unsatisfactory or does not address changes to the character of the work as a result of the revisions.

4-9 Safety

Excavations are a potentially dangerous construction activity. CalOSHA has requirements that must be followed before beginning any excavation that is 5 feet or more in depth, into which a person is required to descend. This information is fully described in the *Caltrans Trenching and Shoring Manual*; however, a brief overview is provided below.

Prior to the start of excavation work, the Contractor is required to:

- Obtain a CalOSHA excavation permit.
- Identify a “competent person” responsible for the excavations.
- Provide an excavation plan to the Engineer for review and authorization prior to starting excavation as required by the contract specifications.\(^{11}\)
- Provide an engineered system stamped by an Engineer who is registered in the State of California for any engineered shoring system.
- Obtain a stamp for any sloping or benching system that is greater than 20 feet from an Engineer who is registered in the State of California.

Once authorized, the excavation needs to be inspected to ensure compliance with the authorized plan and CalOSHA requirements. Daily inspections (prior to the beginning of a shift and after any hazard-increasing occurrence, such as rain) of excavations or protective systems must be made by the Contractor’s “competent person” for evidence of any condition that could result in cave-ins, failure of a protective system, hazardous atmospheres, or any other hazardous condition. When any evidence of a situation is found that could result in a hazardous condition, exposed employees must be removed until the necessary precautions have been taken to ensure their safety.

Safety railings must be located around the excavation perimeter, preferably attached to the shoring that extends above the surrounding ground surface. If the shoring does not extend above the ground, then the railing must be located a sufficient distance back from the excavation lip to adequately protect the workmen in the excavation from being injured by falling objects or debris. Locating the safety rail back from the excavation lip usually provides more stable ground to anchor the rail posts. Spoil piles must be located more than two feet away from the excavation lip for excavations deeper than 5 feet unless there is an adequate retaining device in place to prevent materials from entering the excavation.

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\(^{11}\) 2010 SS, Section 7-1.02K(6)(b), *Excavation Safety*, or 2006 SS, Section 5-1.02A, *Excavation Safety Plans*. 
Although the vertical side of a non-shored excavation must be less than 5 feet in height, care must be exercised when working around the perimeter to avoid falling into the excavation because of sloughing or slip-out of the material at the excavation lip. Spoil piles must be located at least one foot away from the excavation lip for trenches that are less than 5 feet in depth.

Excavations can be considered confined spaces, as they are prone to hazardous atmospheres with limited access and egress. CalOSHA requires the Contractor to take adequate precautions to ensure that oxygen levels and atmospheric contaminants are within acceptable limits. Employees entering excavations should be trained in confined space protocols.

Whenever work is proceeding adjacent to or above the level of vertical projections of exposed rebar, workers must be protected against the hazards of impalement on the exposed ends of the rebar. The impalement hazard can be eliminated by either bending over the ends of the projecting rebar, or by use of one of the following methods:

- When work is proceeding at the same level as the exposed protruding rebar, worker protection can be provided by guarding the exposed ends of rebar with CalOSHA-approved protective covers, troughs, or caps. Approved manufactured covers, troughs, or caps will have the manufacturer’s name, model number, and the CalOSHA-approval number embossed or stenciled on the cover, trough, or cap. Any manufactured protective device not so identified is not legal.
- When work is proceeding above any surface of protruding rebar, impalement protection must be provided by the use of: (1) guardrails, (2) an authorized fall protection system, or (3) authorized protective covers or troughs. Caps are prohibited for use as impalement protection for workers working above a level of 7 1/2-feet above the protruding rebar.

Protective covers used for the protection of employees working above grade must have a minimum 4 x 4-inch square surface area or 4 1/2-inches in diameter, if round. Protective covers or troughs may be job-built, provided they are designed to CalOSHA minimum standards, that the design of the cover or trough was prepared by an Engineer currently registered in the State of California, and a copy of the authorized design is on file in the job records prior to their use.
5 Pile Foundations - General

5-1 Introduction

Pile foundations are used when the underlying soils are incapable of resisting the loads from the structure. The piling is placed in the ground through poor quality materials to bear on competent soils. The piles are either driven into the ground or holes are drilled and filled with reinforced concrete. The piles transfer the load by bearing on competent material or through the friction between the soil and the pile (skin friction.)

Pile foundations can be categorized into two general types: displacement piles and replacement piles. A displacement pile is a pile that is driven or vibrated into the ground and displaces the surrounding soil during installation. A replacement pile is a pile that is placed or constructed within a previously drilled borehole and replaces the excavated soil. Chapter 7, Driven Piles, contains information on displacement piles. Chapters 6, Cast-In-Drilled-Hole Piles; Chapter 9, Slurry Displacement Piles; and Chapter 10, Pier Columns, contain information on replacement, or cast-in-place piles. Chapter 13, Micropiles, contains information on alternative piles and micropiles, which can be a combination of displacement and replacement piles.

Driven piles are braced structural columns that are driven, pushed, or otherwise forced into the soil. Two types of pile foundations were developed through the ages to support structures on poor quality soil: piles and piers. Piles are more commonly used and are essentially small diameter piers that work in groups. Pier foundations are large in diameter and tend to work independently. They have gained favor over the last several years as they behave very well seismically. Piles/Piers can be classified as friction piles, end bearing piles, or a combination of the two. They can also provide lateral stability in foundations. Friction piles can transfer both tensile and compressive forces to the surrounding soil.

5-2 Specifications

The specifications for piling are contained in the Standard Specifications. Project specific requirements and revisions to the Standard Specifications are included in the

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1 2010 SS Section 49, Piling.
Special Provisions. The contract plans and Standard Plans are additional contract documents needed for pile work and describe what piling goes where for each structure. In general, the contract plans describe the intended pile type, specified tip elevation(s) and a minimum nominal resistance. The contract specifications provide project-specific requirements on how to perform the work. These documents also include specific requirements for activities such as embankment pre-drilling, load testing, and other items specific to a project. For example, if difficult driving is anticipated, the Designer may provide the option of using either steel “H” piling or precast concrete piles. When this option is written into the contract, the Contractor is allowed to choose the most economical option. If specifications allowing options are not included in the contract, then changes from one pile type to another cannot be made without a contract change order and concurrence from the Designer.

Details for the different classes of typical piles are found in the Standard Plans while details for atypical or nonstandard piles are shown on the contract plans. The Standard Plans also provide options and alternative details for the different classes of piles. Note that different pile classes are not interchangeable. For example, when Class 140 piles are specified, the Contractor can select either of the alternatives shown in the Standard Plans for Class 140 piles but cannot select an option from a different class of piles such as Class 90 or 200. Occasionally, the Designer may decide to exclude some of the alternatives for a given class of pile. In this situation, the excluded alternatives will be noted in the special provisions or contract plans.

The Standard Specifications contain the general information for pile work. This includes specifics for submittals, type of materials, quality assurance, construction procedures, measurement, and payment. Remember that the Special Provisions and the contract plans have precedence over the Standard Plans and Standard Specifications. For this reason, it is imperative that all contract documents be thoroughly reviewed well in advance of the work and inconsistencies resolved prior to start of work.

5-3 Cast-in-Place Piles

The 2010 version of the Standard Specifications identifies 3 different types of cast-in-place (CIP) piles. They are as follows:
2. CIDH concrete pile rock sockets.
3. Driven steel shells filled with concrete and reinforcement.

The first type is typically known as a Cast-in-Drilled-Hole (CIDH) Pile. The second type is essentially a CIDH pile drilled in rock. Sometimes combinations of two or more types of cast-in-place piles are used to construct a single pile. This can happen when soft materials such as clays overlay rock formations. Permanent or temporary steel casings may be utilized for the first two types. The third type involves the installation of a steel
shell, removal of the soil inside the steel shell and subsequently filling with reinforced concrete. Steel shells add to structural capacity to the pile while steel casings generally have no structural value and are only used to facilitate construction.

The 2006 version of the Standard Specifications identifies four (4) different types of cast-in-place piles. They are as follows:

1. Steel shells driven permanently to the required nominal resistance and penetration and filled with concrete.
2. Steel casings installed permanently to the required penetration and filled with concrete.
3. Drilled holes filled with concrete.
4. Rock sockets filled with concrete.

The first two types involve the installation of a permanent steel casing or shell, removal of the soil inside the casing and subsequently filling with reinforced concrete. Steel shells add to structural capacity to the pile while casings are assumed to have no structural value and are only used to facilitate construction. The third type is typically known as a CIDH Pile. The last type is essentially a CIDH pile drilled in rock. Sometimes combinations of two or more types of cast-in-place piles are used to construct a single pile. This can happen when soft materials such as clays overlay rock formations. Permanent or temporary steel casings may be utilized for these two types.

Cast-In-Drilled-Hole piles are made of reinforced concrete cast into holes drilled in the ground to a specified tip elevation. Diameters generally range from 12 to 168 inches and lengths range from 10 feet to well over 200 feet. They are satisfactory in suitable material and are generally more economical than most other types of piling. They are especially advantageous where vibration from pile driving operations might damage adjacent infrastructure, such as pipelines or buildings. The geological ground formations into which the holes are drilled must be capable of retaining their shape during drilling and concrete placement operations and no groundwater should be present.

If there are concerns about the presence of groundwater, the slurry displacement method specifications may need to be utilized. Cast-In-Drilled-Hole piles are discussed in more detail in Chapters 6, Cast-In-Drilled-Hole Piles, and Chapter 9, Slurry Displacement Piles. Special consideration piles, such as those for changeable message signs (CMS), are discussed in Chapter 13, Micropiles.

5-4 Driven Piles

Driven piles typically consist of three different types: (1) concrete, (2) steel, and (3) timber. A general description of each type is given below. Driven piles are discussed in more detail in Chapter 7, Driven Piles.
## Table 5-1. Driven Pile Types

<table>
<thead>
<tr>
<th>TYPE OF PILE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
</table>
| Driven Piles – Concrete | Driven concrete piles come in a variety of sizes, shapes, and methods of construction. In cross section, they can be square, octagonal, round, solid or hollow. These piles generally vary in sizes from 10 to 60 inches in diameter and consist of precast prestressed concrete.  
  Caltrans has standard details for splicing precast concrete piles but it is a difficult, time consuming, and expensive procedure. Hence, this almost precludes the use of precast piles where excessively long piles are required to obtain necessary bearing.  
  The unit cost to furnish concrete piles is usually lower than the steel equivalent. But this cost is often offset by the requirement for a larger crane and hammer to handle the heavier pile. This is particularly true when there are a small number of piles to drive. |
| Driven Piles – Steel | Steel piling includes structural shape piles and pipe piles. Structural shape piles are generally used for lower capacity piles shown in the Standard Plans. Pipe piles are generally used for high capacity piles. The pipe section is a standard alternate for structural shape Class 90 and 140 piling, but is seldom used.  
  Although steel piling is relatively expensive on a “per foot” furnish basis, it has a number of advantages. Structural shape steel piles come in sizes varying from HP 8 x 36 to HP 14 x 117 rolled shapes or may consist of structural steel plates welded together. They are available in high strength and corrosion-resistant steels. They can penetrate to bedrock where other piles would be destroyed by driving. However, even with “H” piles, care must be taken when long duration hard driving is encountered as the pile tips can be damaged or the intended penetration path of the pile can be drastically deflected. Using a reinforced point on the pile can sometimes prevent this type of damage. Due to the light weight and relative ease of splicing, they are useful where great depths of unstable material must be penetrated before reaching the desired load carrying stratum and in locations where reduced clearances require use of short sections. They are useful where piles must be closely spaced to carry a heavy load because they displace a minimal amount of material when driven.  
  Steel pipe piling comes in sizes varying from 10 to 120 inches in diameter, for heavy walled pipe that are driven directly with the hammer to thin walled or step-taper pipes which are driven with a mandrel. The steel shell may have a flat bottom or be pointed, and may be step-tapered or a uniform section. Steel pipe piling may or may not be filled with reinforced concrete.  
  Splice details are shown on the Standard Plans and contract plans for contracts that permit the use of steel piling. Pile welding work requires the submittal and authorization of a Welding Quality Control Plan. The requirements for the Welding Quality Control Plan are addressed in the contract specifications. Sometimes “H” piles must be driven below the specified tip elevation before the nominal driving resistance is attained. This can present an administrative problem (cost) if the length driven below the specified tip elevation is significant. Steel lugs are always recommended to reduce this problem. |

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2 2010 SS, Section 11-3.02, *Welding Quality Control*, or Special Provisions for contracts using 2006 SS.
welded to the piles are commonly used to solve this problem. This subject is covered in detail in Bridge Construction Memo 130-5.0, *Steel H-Pile Lugs*.

<table>
<thead>
<tr>
<th>TYPE OF PILE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Driven Piles – Wood</td>
<td>Untreated timber piles may be used for temporary construction, revetments, fenders and similar work; and in permanent construction where allowed by the contract specifications. They are also sometimes used for trestle construction, although treated piles are preferred. Timber piles are difficult to extend, hard to anchor into the footing to resist uplift, and subject to damage if not driven carefully. Timber piles also have a maximum allowable bearing capacity of 90 kips, whereas most structure piles are designed for at least 140 kips.</td>
</tr>
</tbody>
</table>

### 5-5 Micropiles and Alternative Piles

Micropiles are currently used when the contract specifications allow their use. The State standard micropile design or proprietary micropile systems may be used. At the time of publication, the Alternative Piling specifications were being rewritten, and there are no currently approved alternative piling systems. Refer to Chapter 13, *Micropiles*, and Appendix D, *Pier Column & Type I Pile Shaft*, for additional information.
CHAPTER 6

Cast-In-Drilled-Hole Piles

6-1 Description

Few terms are as self-descriptive as the one given to the Cast-In-Drilled-Hole (CIDH) pile. They are simply reinforced concrete piles cast in holes drilled to predetermined elevations. Much experience has been gained with this pile type because of its extensive use in the construction of bridge structures. While they probably are the most economical of all commonly used piles, their use is generally limited to certain ground conditions.

CIDH piling can be grouped into two categories:

1. CIDH piling placed in “dry” conditions, usually without inspection pipes (dry method).

2. CIDH piling placed under slurry or with the use of temporary casing to control groundwater, always with inspection pipes (wet method).

This chapter is applicable for both the dry and wet method of CIDH pile construction. Chapter 9, Slurry Displacement Piles, provides supplemental information on the wet method of CIDH pile construction. Note that piling with a diameter greater than 24”, which is de-watered with the help of a temporary casing, requires inspection pipes even if the piling is placed in “dry” conditions.

The ground formation in which the holes for CIDH piles are to be drilled must be of such a nature that the drilled holes will retain their shape and will not cave in before or during concrete placement. Because of cave-ins and concrete placement difficulties, these piles are not recommended for use as battered piles. Other pile types should be considered where groundwater is present, unless dewatering can be done with a reasonable effort and concrete can be placed without a permanent casing. If groundwater or caving conditions are present, the piles can be constructed by the slurry displacement method if permitted in the contract specifications. The slurry displacement method is described in detail in Chapter 9, Slurry Displacement Piles.

The Standard Specifications use three definitions to classify the condition of a drilled hole. A dry hole is defined as a drilled hole that requires no work, such as pumping or other means, to keep it free of water. This means there must be no standing or accumulating water in the bottom of the drilled hole, although the material at the bottom of the drilled hole may be damp or wet. A dewatered hole is defined as a drilled hole

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where water may be present, but accumulates at a rate of less than 12 inches per hour and can be controlled by pumps or other means to reduce the amount of accumulated water to 3 inches or less at the time of concrete placement. The dry method of construction can be used for dry or dewatered holes. A wet hole is defined as a drilled hole where water accumulates at a rate of more than 12 inches per hour or where a temporary casing is used to reduce the rate of water accumulation to less than 12 inches per hour. For a wet hole, inspection pipes are required and the wet method of construction is almost always necessary.

6-2 Specifications

The contract specifications contain the information necessary to administer the construction of CIDH piles. Standard Specification Section 49 contains information on the construction methods. Section 52 contains information on pile bar reinforcement. Section 90 contains information on the concrete mix design, transportation of concrete, and curing of the concrete used for CIDH piles.

The Special Provisions contain project-specific requirements and revised standard specifications. Because the CIDH pile specifications are continually updated and ground conditions vary from project to project, it is very important that the engineer carefully review the Special Provisions and discuss with the Contractor.

6-3 Drilling Equipment

The drilling auger is the most commonly used drilling tool for drilling holes for CIDH piles. Augers may be used in a variety of soil and rock types and conditions.

There are two basic varieties of augers—the standard short section (Figure 6-1) and continuous flight (Figure 6-2). Both have flights of varying diameter and pitch.

6-3.1 Continuous Flight Augers
Continuous flight augers have flight lengths that are longer than the hole to be drilled. They are generally lead-mounted. The power unit is located at the top of the auger and it travels down the leads with the auger as the hole is drilled. Drilling is performed in one continuous operation. As the auger moves down the hole, the drilling action of the flights forces the drill cuttings up and out of the hole. Hence, much material has to be shoveled away from around the drilled hole. Continuous flight augers are most commonly used for short piles, such as those used to support soundwalls or standard retaining walls, or for pre-drilling driven piles. They may also be used where overhead clearance is not a problem.

6-3.2 Short Flight Augers
Short flight augers are powered by “Kelly Bar” units fixed to the drill rig. The lengths of these augers generally vary between 5 and 8 feet. The auger is attached to the end of the Kelly Bar and, as drilling progresses; the auger (and material carried on the flights) must be removed frequently. After the auger is removed from the drilled hole, the material is “spun” off the flights onto a spoil pile and the operation is repeated. Short flight augers are generally used for smaller diameter piles (less than 48” in diameter), although they have been successfully used for larger diameter piles.

6-3.3 Single Flight and Double Flight Augers

There is a variety of different auger shapes/styles used in different situations. Augers may be single flight (Figure 6-3) or double flight (Figure 6-4). Double flight augers are better balanced than single flight augers and are more useful when alignment and location of the drilled hole are important due to clearance or right-of-way problems. Soil augers are equipped with a cutting edge that cuts into the soil during rotation. The drill cuttings are carried on the flights as the auger is removed from the drilled hole and are then “spun” off. The pitch of the flights can vary and should be chosen for the type of material encountered. Soil augers may not work well in cohesionless materials, as the soil may not stay on the flights during auger extraction. They may also have issues in highly cohesive materials where the auger may become clogged.
6-3.4 Rock Augers
Rock augers (Figure 6-5) differ from soil augers in that they are equipped with high-strength steel cutting teeth that can cut through soft rock. These augers typically have flights with a very shallow pitch so that rock pieces, cobbles and boulders can be extracted. For this reason, rock augers are generally the preferred tool for drilling in materials that have a high concentration of cobbles or boulders.

6-3.5 Drilling Buckets
Drilling buckets (Figure 6-6) are drilling tools used when augers are not able to extract material from a drilled hole. This can happen when wet materials or cohesionless materials are encountered. Drilling buckets may also be appropriate when heavy gravel or cobbles are encountered. Drilling buckets have a cutting edge that forces material into the...
bucket during rotation. When the drilling bucket is full, the bucket is spun in the direction opposite of drilling, which closes the built-in flaps. This prevents the cuttings from falling out of the bucket. The bucket is then extracted from the drilled hole and emptied.

6-3.6 Cleanout Buckets
Cleanout buckets (Figure 6-7) are specialized drilling buckets that are used to clean loose materials from the bottom of a drilled hole and to flatten the bottom. This ensures that the tip of the pile is founded on a firm flat surface. These buckets have no cutting teeth but are similar to drilling buckets in other aspects. Figures 6-6 and 6-7 show the difference between the drilling bucket and the cleanout bucket. Specialized cleanout buckets can be used to extract loose materials when groundwater or drilling slurry is present. These buckets, referred to as “muckout” buckets, allow fluid to pass through them while retaining the loose materials from the bottom of the drilled hole.

![Figure 6-6. Drilling Bucket.](image1)

![Figure 6-7. Cleanout Bucket.](image2)

6-3.7 Core Barrels
Core barrels (Figure 6-8) are used to drill through hard rock formations, very large boulders, or concrete. This type of drilling tool consists of a steel cylinder with hard metal cutting teeth on the bottom. Rock cores are broken off and extracted from the drilled hole as a single unit, or may be broken up with a rock breaker and then extracted with a drilling bucket or clamshell.
6-3.8 Down-Hole Hammers

Down-hole hammers (Figure 6-9) are also used to drill through hard rock formations. This type of drilling tool uses compressed air or hydraulic-powered percussion drilling heads to pulverize the formation and blow the resulting debris from the drilled hole.

Figure 6-9. Down-Hole Hammer.

6-3.9 Rotators and Oscillators

Rotators (Figure 6-10) and oscillators (Figure 6-11) are specialized drilling equipment used to advance a drilled hole through difficult ground formations. Each machine uses a hydraulic-powered apparatus to simultaneously rotate and push down on a drilling casing. Drilling casings are sections of steel pipe, usually 20 feet in length, designed specifically for the rotator or oscillator model, with attachments for cutting teeth or splicing of additional sections. Additional sections of drilling casing are attached as the drilled hole
is advanced to tip. As the drilled hole is advanced, the materials within the drilling casing are extracted using a clamshell or drilling bucket. The major difference between a rotator and an oscillator is that the rotator rotates the drilling casing in one direction, while the oscillator rotates the drilling casing in two directions, never making a complete rotation in either direction. The advantage provided by the rotator and oscillator is the drilling casing provides a temporary casing that preserves the integrity of the drilled hole, even in unstable or wet ground formations. The drilling casing remains in the drilled hole until pile concrete is placed, at which time the drilling casing is extracted from the drilled hole in a similar manner as any other temporary steel casing as described below.

![Rotator](image)
6-3.10 Reverse Circulation Drilling Equipment

Reverse circulation drilling equipment (Figure 6-12) is used to advance a drilled hole through difficult wet ground formations. The advantage of reverse circulation is that very deep holes can be advanced without the need to cycle in and out of the hole with the drill tool to remove cuttings. The drilled hole must be full of water or other drilling fluids. The drilling head is self-contained and is driven hydraulically or by compressed air. As the hole is advanced, the drill cuttings are suspended in the water or drilling fluid. The water or drilling fluid is continuously circulated out of the drilled hole, where the drill cuttings are removed and disposed, and then re-circulated back into the drilled hole to repeat the process.
Temporary steel casings (Figure 6-13) are used to support drilled holes when unstable conditions are encountered. Various methods are used to advance steel casings into the hole. Among them, spinning the casing with the Kelly Bar while applying some vertical force, driving the casing with whatever means are available as the hole is drilled, or using a vibratory hammer. Steel casings are generally extracted from the hole in the manner specified in the contract specifications as concrete is placed.
6-3.12 Drilling Rigs
Drilling is performed almost exclusively with portable drilling rigs. These units can be self-propelled crawler mounted (Figure 6-14), truck-mounted (Figure 6-15), or crane-mounted (Figure 6-16).

Figure 6-13. Steel Casing.

Figure 6-14. Crawler Mounted Drill Rig.
6-4 Drilling Methods

Various other materials are used to supplement the drilling work. Water or other drilling fluid is sometimes added to certain ground formations to assist drilling and lifting materials from the hole. Soil may be placed back into the hole to dry out supersaturated materials. The drilling tool is used to agitate the materials so they can be extracted from the hole. This is known as “processing” the hole.

6-5 Drilling Problems

The difficulties encountered in drilling can include cave-ins, groundwater, and utilities. The following briefly describes some actions that can be taken in these situations.

6-5.1 Cave-ins

In the case of cave-ins, the following action or combination thereof may be required:

<table>
<thead>
<tr>
<th>ITEM</th>
<th>ACTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Place a low cement/sand mix and re-drill the area of the cave-in.</td>
</tr>
<tr>
<td>2</td>
<td>Use a drilling slurry (refer to Chapter 9, <em>Slurry Displacement Piles</em>).</td>
</tr>
<tr>
<td>3</td>
<td>Use a temporary casing, which is pulled when placing concrete.</td>
</tr>
</tbody>
</table>

6-5.2 Groundwater

In the case of groundwater, the following action or combination may be required:
Table 6-2. Groundwater Actions

<table>
<thead>
<tr>
<th>ITEM</th>
<th>ACTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Place a low cement/sand mix and re-drill the hole.</td>
</tr>
<tr>
<td>2</td>
<td>Drill to tip elevation, then use a pump to remove the water and clean out the bottom of the hole.</td>
</tr>
<tr>
<td>3</td>
<td>Use a drilling slurry (refer to Chapter 9, Slurry Displacement Piles).</td>
</tr>
<tr>
<td>4</td>
<td>Use a temporary casing, then use a pump to remove the water, and remove the casing during concrete placement, keeping the bottom of casing below the head of concrete.</td>
</tr>
<tr>
<td>5</td>
<td>De-water the entire area using well points, deep wells, etc. This should be thoroughly discussed with the Bridge Construction Engineer and the project Geoprofessional.</td>
</tr>
<tr>
<td>6</td>
<td>Substitute an alternative type of piling by contract change order. This should be discussed with the project Designer, the project Geoprofessional, and the Bridge Construction Engineer.</td>
</tr>
</tbody>
</table>

6-5.3 Utilities

Construction operations should proceed with caution when drilling near utilities known or thought to be in close proximity. The Contractor should contact the area Underground Service Alert (USA) or the utility company and have the utility located. The Contractor should also pothole and physically locate the utility prior to drilling. Relocation of the utility may be required. Minor adjustments in pile location might be feasible in order to avoid conflict. Any proposed revisions to the pile layout should be discussed with the Designer, Geoprofessional, Resident Engineer and the Bridge Construction Engineer.

Under certain conditions, the contract specifications allow the Contractor to propose increasing the pile diameter in order to raise the pile tip. This can be used to address problems with groundwater, cave-ins or obstructions in the lower portion of the hole. Before allowing this, the Engineer should consult with the Designer and Geoprofessional to see if this is feasible and if so, to obtain the revised tip elevation. Appropriate pay provisions are also included in the contract specifications and a change order is not required.

Ordinarily, the above drilling problems would stimulate the Contractor’s action and a change would be proposed to the Engineer. Sometimes the drilling problem is the result of unanticipated ground conditions or unanticipated utility conflicts. In such cases, a differing site condition or a buried manmade object may exist, and it will be the Engineer’s responsibility to resolve the problem.

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1 2010 SS, Section 49-3.02C(1), CIDH Pile Construction, General, or 2006 SS, Section 49-4.03, Drilled Holes.
6-6 Inspection and Contract Administration

Cast-in-Drilled-Hole piles are designed to resist compressive loads, tensile loads, and lateral loads. Most CIDH piles are designed to resist these loads using skin friction and passive earth pressure, with minimal or no contribution from end-bearing. The Designer should be contacted to determine the manner in which the pile was designed to transfer load.

The Engineer should review the contract plans, the Foundation Report and the Log of Test Borings thoroughly. If there are any discrepancies noted between the pile type shown on the contract plans, the pile type called for in the Foundation Report, and/or the soil materials/profile and groundwater level shown on the Log of Test Borings, the Designer should be contacted for clarification.

A CIDH Pile Construction Checklist is presented in Appendix K-2 to assist field personnel in preparing documents and inspecting fieldwork to ensure compliance with contract requirements.

6-6.1 Pile Installation Plan

The contract specifications require the Contractor to submit a pile installation plan to the Engineer for review and authorization. The pile installation plan should provide sufficient detail for the Engineer to grasp the means, methods, and materials that the Contractor plans to use to successfully complete CIDH pile installation. Typical requirements for all CIDH piling include the following:

<table>
<thead>
<tr>
<th>ITEM</th>
<th>PILE INSTALLATION PLAN REQUIREMENT &amp; REASONING</th>
</tr>
</thead>
</table>
| 1    | Concrete mix design, certified test data, and trial batch reports.  
**Reasoning:** CIDH pile concrete is designated by compressive strength. |
| 2    | Drilling or coring methods and equipment.  
**Reasoning:** This gives the Engineer advance knowledge of what equipment the Contractor proposes to use to drill the CIDH pile and whether the proposed equipment is appropriate. |
| 3    | Proposed methods for casing installation and removal when necessary.  
**Reasoning:** This gives the Engineer advance knowledge of whether the Contractor plans to use casing and if so, how it will be installed and removed and whether the proposed installation and removal methods are appropriate. |
| 4    | Methods for placing, positioning, and supporting bar reinforcement. |

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2 2010 SS, Section 49-3.02A(3)(b), Pile Installation Plan, or Special Provisions for contracts using 2006 SS.
<table>
<thead>
<tr>
<th>ITEM</th>
<th>PILE INSTALLATION PLAN REQUIREMENT &amp; REASONING</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Reasoning:</strong> This gives the Engineer advance knowledge of how the Contractor plans to assemble and install the pile bar reinforcement cage and whether the proposed method of installation is appropriate.</td>
<td></td>
</tr>
</tbody>
</table>
| 5 | Methods and equipment for accurately determining the depth of concrete and actual and theoretical volume placed, including effects on volume of concrete when any casings are withdrawn.  
**Reasoning:** This is necessary so the Engineer and Contractor can determine whether an unplanned event, such as a cave-in, has occurred during concrete placement. If such an event happens, the actual volume of concrete placed will be substantially different from the theoretical volume at the location of the event and the Engineer and Contractor will be able to pinpoint the location of the event for mitigation if necessary. |
| 6 | Methods and equipment for verifying that the bottom of the drilled hole is clean prior to placing concrete.  
**Reasoning:** Over 50% of all pile defects occur at the bottom of the drilled hole due to the presence of loose soil cuttings that were not removed prior to concrete placement. This gives the Engineer advance knowledge of how the Contractor plans to remove these loose materials, verify that they were removed, and whether the proposed methods of removal and verification are appropriate. |
| 7 | Methods and equipment for preventing upward movement of reinforcement, including the Contractor’s means of detecting and measuring upward movement during concrete placement operations.  
**Reasoning:** Pile bar reinforcement cages have been known to shift laterally or upward during concrete placement. This gives the Engineer advance knowledge of how the Contractor plans to prevent movement of the pile bar reinforcement cage and whether the proposed methods are appropriate. |
| 8 | Drilling sequence and concrete placement plan.  
**Reasoning:** Contractors sometimes prefer to drill some or all of the holes at one time and come back to place reinforcement and concrete at a later time, which may impact the quality of the piles. If the drilled holes are close together, this increases the risk of side blowout into adjacent holes during concrete placement. This gives the Engineer advance knowledge of the Contractor’s planned construction sequence and provoke discussion of how to prevent problems due to construction sequencing. |
| 9 | Methods for ensuring the inspection pipes remain straight, undamaged, and properly aligned during concrete placement, if inspection pipes are required.  
**Reasoning:** Inspection pipes that were properly placed during reinforcement cage construction can become misaligned when the... |
### PILE INSTALLATION PLAN REQUIREMENT & REASONING

- Cage is placed in the drilled hole and during concrete placement. This gives the Engineer advance knowledge of the Contractor’s plan to keep the inspection pipes aligned at all times until the pile concrete has set.

### 6-6.2 CIDH Pile Preconstruction Meeting

Before drilling begins, the Standard Specifications require the Contractor to schedule and the Engineer to conduct a CIDH pile preconstruction meeting. The purpose of this meeting is to establish contacts and communication protocol for the Contractor, the Engineer and their representatives involved in CIDH pile design and construction, and to afford all parties a common understanding of the construction process, acceptance testing, and mitigation of CIDH piles. Items to be discussed should include any recently revised contract specifications, the contract pay limits, the Contractor’s planned method of operation and schedule, the equipment to be used, the plan for avoiding existing utilities (if any), and safety precautions to be taken during the work. Bridge Construction Memo 130-20.0, *Cast in Drilled Hold (CIDH) Pile Preconstruction Meeting*, provides the Structure Representative with guidelines on how to conduct the meeting.

### 6-6.3 Construction

The Contractor is required to layout the pile locations at the site prior to drilling. The Engineer should verify the layout is correct prior to drilling and set reference elevations in the area so pile lengths and pile cutoff can be ascertained.

During the drilling operation, the Engineer should verify that the piles are in the correct location and drilled plumb. Usually, the Contractor will check the Kelly Bar with a carpenter’s level during the drilling operation. The Engineer should also evaluate the material encountered and compare it to the *Log of Test Borings*. If the material at the specified tip elevation differs from that anticipated, the Designer and Geoprofessional should be consulted, as a change in pile length might be needed. A written record of the drilling progress should be kept in the project daily report and the record utilized to investigate any differing site condition claims submitted by the Contractor.

When the hole has been drilled to the specified tip elevation, the Contractor should use a cleanout bucket or other authorized means as described in the pile installation plan to remove any loose materials and to produce a firm flat surface at the bottom of the drilled hole.

The depth, diameter, and plumbness/straightness of the drilled hole must be checked and verified after drilling is completed. Check the drilled hole using a suitable light, furnished by the Contractor, or a mirror. At this time, the Engineer should measure and record the length of each pile. Unless the Engineer orders the Contractor, in writing, to change the specified tip elevation, no payment will be made for any additional depth of pile below the specified tip elevation.
For large diameter piles, it may be necessary for the Engineer or the Geoprofessional to inspect the bearing surface at the bottom of the drilled hole. All pertinent requirements of the *Construction Safety Orders* and *Mining and Tunneling Safety Orders* must be met before constructing or entering a drilled hole. Note that for CIDH piles over 20 feet in depth and 30 inches or larger in diameter, *CalOSHA Mining and Tunneling Safety Orders* apply. *Construction Procedure Directive CPD 04-6*, which is included in Appendix B, *Contract Administration*, and Bridge Construction Memorandum (BCM) 14-5.0, *Tunnel Safety*, address this.

Check the pile bar reinforcement cage clearances and spacers immediately after the cage is placed and secured in the drilled hole. In addition, the reinforcing cage must be adequately supported, as described in the pile installation plan and some means must be devised to ensure concrete placement to the proper pile cutoff elevation.

Immediately before placing concrete, check the bottom of the drilled hole for loose materials or water. Loose materials and small amounts of water can be removed with a cleanout bucket before placing the pile bar reinforcement cage. Large amounts of water may need to be pumped out. It is important to note that it may be necessary to remove the pile bar reinforcement cage to accomplish this. Failure to do so could affect the quality of the pile.

Those involved in the work should thoroughly review the contract specifications before fieldwork begins. Applicable portions of Standard Specifications Section 90, *Concrete*, should also be reviewed with respect to concrete mix design, consistency of the concrete mix, and concrete curing requirements.

### 6-7 Pile Defects

The drilling problems mentioned previously, if not corrected, can cause CIDH piles to be defective. There are also problems that can occur during concrete placement or casing removal that can cause defective CIDH piles.

#### 6-7.1 Drilling Problems

The following drilling problems can cause pile defects:

<table>
<thead>
<tr>
<th>ITEM</th>
<th>DRILLING PROBLEM/RESULTING PILE DEFECT</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A cleanout bucket is not used to clean up the bottom of the drilled hole.</td>
</tr>
</tbody>
</table>

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3 2010 SS, Section 49-3, *Cast-In-Place Concrete Piling*, or 2006 SS, Section 49-4, *Cast-In-Place Concrete Piles*. 

---
<table>
<thead>
<tr>
<th>ITEM</th>
<th>DRILLING PROBLEM/RESULTING PILE DEFECT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>Result:</strong> This can result in the pile bearing on soft material. For CIDH piles designed for end bearing, this flaw can seriously compromise the value of the pile. This defect is shown in Figure 6-17.</td>
</tr>
</tbody>
</table>
| 2    | A tapered auger is used to advance the drilled hole to the specified tip elevation but a cleanout bucket is not used to flatten the bottom of the hole.  
   **Result:** Concrete may crush at the tip of the pile, which would reduce its capacity and possibly cause differential settlement. There may also be soft material at the tip of the drilled hole, which would cause the problems mentioned previously. This defect is also shown in Figure 6-17. |
| 3    | The drilling operation smears drill cuttings on the sides of the drilled hole.  
   **Result:** This can result in the degradation of the pile’s capacity to transfer loads through skin friction. This may be critical if the pile is designed as a tension pile. This condition is most likely to occur in ground formations containing cohesive materials. This defect is shown in Figure 6-18. |

These problems are preventable. Adherence to the contract specifications and timely inspection will ensure the best quality pile and mitigate most of these problems.
6-7.2 Concrete Placement Problems
The following concrete placement problems can cause pile defects:

Table 6-5. Pile Defects – Concrete Placement Problems.

<table>
<thead>
<tr>
<th>ITEM</th>
<th>PLACEMENT PROBLEM/RESULTING PILE DEFECT</th>
</tr>
</thead>
</table>
| 1    | A cave-in at a location above the top of concrete or sloughing material from the top of the drilled hole occurs during concrete placement.  
*Result:* Degraded concrete at the location, thus reducing the capacity of the pile. This defect is shown in Figure 6-19. |
| 2    | The Contractor tailgates concrete into the drilled hole without the use of a hopper or “elephant trunk” to guide it. The concrete falls on the pile bar reinforcement cage or supporting bracing and segregates.  
*Result:* Defective concrete, thus reducing the capacity of the pile. This defect is shown in Figure 6-20. |
| 3    | A new hole is drilled adjacent to a freshly poured pile or concrete is placed in a drilled hole that is too close to an adjacent open drilled hole.  
*Result:* This can result in the sidewall blowout of a freshly poured pile into the adjacent drilled hole. This would probably cause the pile bar reinforcement cage to buckle. This defect is shown in Figure 6-21. |
| 4    | The Contractor does not remove groundwater from the drilled hole.  
*Result:* Groundwater mingles with the concrete leading to defective concrete at the bottom of the pile. If the pile were designed for end bearing, the capacity would be reduced. This defect is shown in Figure 6-22. |

As with the drilling problems, most of these placement problems are preventable. Adherence to the contract specifications and timely inspection will prevent most of these problems. However, if a cave-in occurs during concrete placement, the Contractor may need to remove the pile bar reinforcement cage and concrete, and then start over.
Figure 6-19. Pile Defects—Cave In.  Figure 6-20. Pile Defects—Concrete Segregation.
6-7.3 Casing Removal Problems

The following casing removal problems can cause pile defects:

Table 6-6. Pile Defects – Casing Removal Problems.

<table>
<thead>
<tr>
<th>ITEM</th>
<th>CASING REMOVAL PROBLEM/PILE DEFECT</th>
</tr>
</thead>
</table>
| 1    | The Contractor waits too long to pull the casing during concrete placement. This may result in three problems:  
|      | 1. The concrete sets up and comes up with the casing as shown in Figure 6-23(a).  
|      | 2. The concrete sets and the casing cannot be removed as shown in Figure 6-23(b).  
|      | 3. The concrete sets up enough so that it cannot fill the voids left by the casing as it is removed, as shown in Figure 6-23(c). The first problem may result in a void being formed in the pile at the bottom of the casing. It is possible that the suction created may cause a cave-in at this location. The second and third problems result in the loss of the pile’s capacity to transfer skin friction to the ground. |
Historically, problems with casings have produced the worst type of CIDH pile defects. Again, these problems are preventable. Adherence to the contract specifications and timely inspection will prevent most of these problems. Caltrans recommends the slump of the concrete placed in the pile to be at the high end of the allowable range. Research has shown that concrete with higher fluidity will consolidate and fill in the voids better than concrete with lower fluidity. As there is an increased risk in pouring piles with temporary casings, under certain circumstances, piles poured with this method need to undergo non-destructive testing prior to acceptance. The CIDH pile contract specifications require that all CIDH piles constructed with the use of temporary casings to control groundwater undergo acceptance testing prior to acceptance. The pile testing methods used to test piles constructed by the slurry displacement method (as described in Chapter 9, *Slurry Displacement Piles*) would be used in this circumstance.

Figure 6-23(a)  Figure 6-23(b)  Figure 6-23(c)

Pile Defects - Casing Problems.

6-8 Safety
As with all construction activities, the Engineer should be aware of safety considerations associated with the operation. As a minimum, the Engineer must review the Construction Safety Orders that pertain to this work. A tailgate safety meeting should be held to discuss the inherent dangers of performing this work before the work begins.

The primary and obvious hazard encountered with CIDH pile construction is the open drilled hole. Common practice is to keep the drilled hole covered with plywood or steel plates, especially if the drilled hole is left open overnight. This provides protection, not only for the construction crew working in the area, but also the public. In urban areas, more stringent measures may be required to secure the site.

As with any other type of operation, use common sense safety practices when working around this equipment. If you do not need to be there, stay away from the equipment. If a crane-mounted drilling rig is used, check the crane certificate.

In addition, footing excavations should be properly sloped or shored as discussed in Chapter 4, Footing Foundations. Imposed loads, such as those from cranes and transit mix concrete trucks, must be kept a sufficient distance from the edge of the excavation. If the Contractor intends to place equipment of this type adjacent to the excavation, the load must be considered in the shoring design and/or in determining the safe slope for unshored excavations. Additional information on excavations can be found in the Caltrans Trenching and Shoring Manual.

Worker and public safety must be enforced during drilling and excavating operations. The Contractor must have a Fall Protection plan and fall protection equipment must be used when working near open holes. Personnel not directly involved in the construction operation should not stand next to an open hole to avoid falling in or if the edge collapses. For CIDH piles over 20 feet in depth and 30 inches in diameter, Cal-OSHA Mining and Tunneling Safety Orders apply. Construction Procedure Directive CPD 04-6 addresses this and is included in Appendix B, Contract Administration.
Driven Piles

7-1 Introduction

Driving piles for structure foundations has occurred for centuries. Originally, timber was used for piles. In 1897, the first concrete piles were introduced in Europe, and the Raymond Pile Company drove the first concrete piles in America in 1904. These new concrete piles were designed for 30 tons and over. Currently, steel H-Piles and pipe piles are also used. These piles can be expensive but their ability to transfer greater loads has made them economical, particularly in large structures.

Pile driving is the operation of forcing a pile into the ground thereby displacing the soil mass across the whole cross section of the pile. Historically, the oldest method of driving a pile, and the method most often used today, is by use of an impact type hammer.

The first hammers known to be used were drop hammers which were used exclusively until the invention of the steam engine, which eventually resulted in steam hammers. Subsequent technological advances have led to the development of air, diesel, hydraulic powered impact hammers, plus vibratory and sonic hammers. Modern day requirements...
for construction have also resulted in various adaptations of the aforementioned pile driving techniques.

This chapter is intended to outline specifications, equipment, techniques, and safety items that a bridge engineer can expect to encounter during typical pile driving operations.

### 7-2 General Specifications

The following is a partial list of some of the more important pile driving specifications. Before starting a project, the Engineer should thoroughly review the Standard Specifications for general requirements and the special provisions for information tailored to the needs of the specific project.

Typical sections of the Standard Specifications (SS) to be reviewed are as follows:

- Earthwork (SS, Section 19).
- Piling (SS, Section 49).
- Wood and Plastic Lumber Structures (SS, Section 57).

The following are taken from the Standard Specifications and should be reviewed as applicable:

- In embankment areas where piles are to be placed or driven, do not use material containing rocks, broken concrete, or other solid materials larger than 4 inches in greatest dimension.

- For bridge footings to be constructed in embankment, construct the embankment to the grading plane elevation and extend the finished slope to the grading plane before:
  1. Excavating for footings.
  2. Driving piles or drilling holes for Cast-in-Place (CIP) piles.

- Where an embankment settlement period is specified, and before the end of the settlement period, do not:
  1. Excavate for abutments, bent footings, wingwalls, or retaining wall footings.
  2. Drive foundation piles or drill holes for CIP piles.

- Piling must have sufficient length to attain the specified trim elevation shown and extend into the pile cap or footing.

- Install driven piles using an authorized impact hammer. The impact hammer must be:

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1. 2010 SS, Section 19-6.02A, Materials General, or 2006 SS, Section 19-6.01, Embankment Construction, Placing.
2. 2010 SS, Section 19-6.03A, Construction General, or 2006 SS, Section 19-6.01, Placing.
4. 2010 SS, Section 49-1.01D(1), Quality Control and Assurance General, or 2006 SS, Section 49-1.03, Determination of Length.
1. Steam, hydraulic, air, or diesel.
2. Able to develop sufficient energy to drive the pile at a penetration rate of not less than 1/8 inch per blow at the nominal driving resistance shown.5

- For piles to be driven through embankments constructed under the Contract, drive piles through predrilled holes where the depth of the new embankment at the pile location is in excess of five feet. The hole diameter must be at least 6 inches larger than the greatest dimension of the pile cross section. After driving the pile, fill the space around the pile to the ground surface with dry sand or pea gravel.6
- Except for piles to be load tested and sheet piles, drive piles to at least the nominal driving resistance and the specified tip elevation shown.7

The preceding specifications indicate that there are two different pile driving acceptance criteria: (1) A specific pile tip penetration, and (2) a prescribed bearing value. In all but a few cases, both of these criteria must be met in order to accept the pile.

### 7-3 Pile Driving Definitions

The following is a partial list of some of the definitions unique to the pile driving trade. These are the most common terms used and should be of benefit to those new to pile driving work. Refer to Figures 7-2 through 7-8 for the illustration of the defined terms.

<table>
<thead>
<tr>
<th>TERM</th>
<th>DEFINITION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anvil</td>
<td>The bottom part of a hammer that receives the impact of the ram and transmits the energy to the pile.</td>
</tr>
<tr>
<td>Butt of Pile</td>
<td>The term commonly used in conjunction with the timber piles—the upper or larger end of the pile, the end closest to the hammer.</td>
</tr>
<tr>
<td>Cushion Blocks</td>
<td>Usually plywood pads placed on top of precast concrete piles to eliminate spalling.</td>
</tr>
<tr>
<td>Cushion Pad</td>
<td>A pad of resilient material or hardwood placed between the drive cap insert, or helmet, and drive cap adapter.</td>
</tr>
<tr>
<td>Drive Cap Adapter</td>
<td>A steel unit designed to connect specific type of pile to a specific hammer. It is usually connected to the hammer by steel cables.</td>
</tr>
<tr>
<td>Drive Cap Insert</td>
<td>The unit that fits over the top of pile, holding it in line and connecting it to the adapter.</td>
</tr>
<tr>
<td>Drive Cap System</td>
<td>The assembled components used to connect and transfer the energy from the hammer to the pile.</td>
</tr>
<tr>
<td>Follower</td>
<td>An extension used between the pile and the hammer that transmits blows to the pile when the pile head is either below the reach of the hammer (below the guides/leads) or under water. A follower is usually a section of pipe or “H” pile with connections that match both the pile hammer and the pile. Since the follower may absorb a</td>
</tr>
</tbody>
</table>

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5 2010 SS, Section 49-2.01C(2), Driving Equipment, or 2006 SS, Section 49-1.05, Driving Equipment.
6 2010 SS, Section 49-2.01C(4), Predrilled Holes, or 2006 SS, Section 49-1.06, Predrilled Holes.
7 2010 SS, Section 49-2.01A(4)(b), Pile Driving Acceptance Criteria, or 2006 SS, Section 49-1.08, Pile Driving Acceptance Criteria.
<table>
<thead>
<tr>
<th>TERM</th>
<th>DEFINITION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hammer Energy</td>
<td>The amount of energy available to be transmitted from the hammer to the pile. Usually measured in foot-pounds.</td>
</tr>
</tbody>
</table>
| Leads        | A wooden or steel frame with one or two parallel members for guiding the hammer and piles in the correct alignment. There are three basic types of leads:  
- Fixed, which are fixed to the pile rig at the top and bottom. Refer to Figure 7-4.  
- Swinging, which are supported at the top by a cable attached to the crane. Refer to Figure 7-5.  
- Semi-Fixed or Telescopic, which are allowed to translate vertically with relation to the boom tip. Refer to Figure 7-6.                                                                                                                                                                                                 |
Figure 7-2. Drive Cap System.
Figure 7-3. Typical Pile Rig Configuration.
Figure 7-4. Fixed Lead System.
Figure 7-5. Swinging Lead System.
Figure 7-6. Semi-Fixed Lead System.
Figure 7-7. Lead Configurations for Battered Piles.
7-4 Hammer Types

Many different types of pile driving hammers are used in the pile industry today. In the past, single acting diesel hammers were used on most projects. With the onset of retrofit work and new construction in areas with low overhead clearances, the use of double/differential acting hammers and hammers that require only a limited overhead clearance are finding their way to the job site. Site specific construction challenges, be it limited space, noise levels, or unusual tip or bearing requirements will tend to dictate the type of hammer used.

The pile hammer is not only the production tool for the Contractor, it is also a measuring device for the Engineer. The energy transmitted to the pile advances it toward the
specified tip elevation. The amount of energy and the penetration per blow can be used to determine the bearing capacity of the pile. A working knowledge of pile hammers, their individual parts and accessories, and their basis for operation and the associated terminology is essential for the Engineer.

Following is a partial list of different types of hammers available today with a brief description of their limiting characteristics.

7-4.1 The Drop Hammer
Although the drop hammer was invented centuries ago, it is still in use today. Although modernized somewhat, the basic principle of operation remains the same. A weight is lifted a measured distance by means of a rope or cable and allowed to freefall, or drop, and strike a pile cap block. The available potential energy is calculated by multiplying the weight and the distance of the fall.

One variation of the drop hammer currently finding its way to the job site is one that requires only a minimal amount of headroom. The idea utilizes a closed-ended pipe pile with a large enough diameter to allow the drop hammer run inside the pipe’s walls. The hammer impacts onto a “stop” built into the bottom, inside of the pipe pile. As the pile is driven, the impact occurs near the tip of the pile. In fact the pile is actually pulled down into position in lieu of being pushed. This configuration minimizes the need for the additional overhead clearance (leads, crane, etc.).

Drop hammers are not typically used and are permitted only when allowed by the special provisions.

When using a drop hammer the Engineer should:

<table>
<thead>
<tr>
<th>NO.</th>
<th>ITEM DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Ensure that you have the correct weight for the hammer being used. If in doubt, have it weighed.</td>
</tr>
<tr>
<td>2</td>
<td>Ensure the drop hammer lead sections are properly aligned and that all lead connections are properly tightened.</td>
</tr>
<tr>
<td>3</td>
<td>Ensure, while in use, that the hoist line is paying out freely.</td>
</tr>
</tbody>
</table>
7.4.2 Single Acting Steam/Air Hammer
The single acting steam/air hammer is the simplest powered hammer. Invented in England by James Nasmyth in 1845, it has been used in this country since 1875.

As shown in Figure 7-10, the hammer consists of a heavy ram connected to a piston enclosed in a chamber. Steam or air is supplied to lift the ram to a certain height. The lifting medium is then exhausted and the ram falls by its own weight. The rated energy of the single acting steam/air hammer is calculated by multiplying the ram weight (total weight of all moving parts: ram, piston rod, keys, slide bar, etc.) times the length of fall (stroke).

These hammers have a stroke of 30 to 40 inches and operate at 60 to 70 strokes per minute. They are rugged and deliver a relatively low velocity heavy blow. The only necessary changes in operation from steam to air are a change in the general lubrication and the hose line specification.

When using a single acting steam/air hammer the Engineer should:
Table 7-3. Single Acting Steam/Air Hammer Actions.

<table>
<thead>
<tr>
<th>NO.</th>
<th>ITEM DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Have the manufacturer’s current specifications for the type and model of hammer being used.</td>
</tr>
<tr>
<td>2</td>
<td>Ensure all required parts of the hammer are intact and in good operating condition.</td>
</tr>
</tbody>
</table>

**Figure 7-10. Single Acting Steam/Air Hammer.**

7-4.3 Double Acting Steam/Air Hammer

The double acting steam/air hammer employs steam or air, not only to lift the piston to the top of its stroke, but also to accelerate the piston downward faster than by gravity alone. The additional energy put into the downward stroke by the compressed air/steam increases the effectiveness of the hammer. The advantage of the double-acting hammer is that stroke lengths can be reduced making them ideal in low overhead clearance situations. The stroke typically ranges from 10 to 20 inches, or about half that of a single-
acting hammer. The blow rate is more rapid than the single acting hammer, somewhere between 120 and 240 blows per minute. Refer to Figure 7-11. The rated available energy of the double acting steam/air hammer is calculated by multiplying the ram weight times the length of stroke and adding the effective pressure acting on the piston head during the downstroke.

In addition to being an ideal hammer in low overhead situations, this type of hammer does not use a cushion block between the ram and the anvil block. Another advantage is that some of these hammers are entirely enclosed and can be operated submerged in water. With this type hammer, it is essential that the hammer is operating within the manufacturer’s specifications. Since pressure is used to drive the hammer, it’s imperative that operating pressures are known. The pressures recorded will correlate to an impact energy found on a chart/table provided with the hammer.

When using a double acting steam hammer the Engineer should:

<table>
<thead>
<tr>
<th>NO.</th>
<th>ITEM DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Have the manufacturer’s current specifications for the type and model of hammer being used.</td>
</tr>
<tr>
<td>2</td>
<td>Ensure all required parts of the hammer are intact and in good operating condition.</td>
</tr>
<tr>
<td>3</td>
<td>Have chart available declaring rated energy vs. operating speed of hammer.</td>
</tr>
</tbody>
</table>

Figure 7-11. Double Acting Steam/Air Hammer.
7-4.4 Differential Acting Steam/Air Hammer (External Combustion Hammer)

The differential acting steam/air hammer is similar to a double acting hammer. Compressed air/steam is introduced between large and small piston heads to lift the ram to the top of its stroke. The valves are then switched so that the medium (motive fluid) used to lift the ram accelerates it in its down stroke. Refer to Figure 7-12. When hydraulic fluid is used as a motive fluid it is called a double/differential acting hydraulic hammer.

The rated striking energy delivered per blow by a differential acting steam/air hammer is calculated by (1) adding the differential force due to the motive fluid pressure acting over the large piston head (2) to the weight of the striking parts and (3) multiplying this sum by the length of the piston stroke in feet. The differential force results from the fluid pressure acting on the top piston head surface minus the same pressure in the annulus acting on the bottom surface and is equal to the area of the small piston head times the fluid pressure. This type of hammer uses a cushion block between the ram and the helmet.

Figure 7-12. Differential Acting Steam/Air Hammer.
When using a differential acting steam/air hammer the Engineer should:

**Table 7-5. Differential Acting Steam/Air Hammer Actions.**

<table>
<thead>
<tr>
<th>NO.</th>
<th>ITEM DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Have the manufacturer’s current specifications for the type and model of hammer being used.</td>
</tr>
<tr>
<td>2</td>
<td>Ensure all required parts of the hammer are intact and in good operating condition.</td>
</tr>
<tr>
<td>3</td>
<td>Have chart available declaring rated energy vs. operating speed of hammer.</td>
</tr>
</tbody>
</table>

### 7-4.5 Diesel Pile Hammer

In the early 1950’s a new type of pile driving hammer was introduced - the Diesel Hammer. Basically, it is a rudimentary one-cylinder diesel engine. It is fed from a fuel tank. The tank and fuel pump are mounted directly on the hammer, in contrast to air and steam hammers, which require an external energy source. Simple to operate, diesel hammers are commonly used on most bridge contracts today.

#### 7-4.5.1 Single Acting Diesel Hammers.

The fundamental makeup and operation of all diesel hammers are similar. They consist of a cylinder-encased ram, an anvil block, a lubrication system, and a fuel injection system that regulates the amount of fuel to each cycle. New models added a variable fuel metering system that can change the energy delivered by the ram, thereby making them more versatile for varying soil conditions. The energy imparted to the driven pile is developed from gravitational forces acting on the mass of the piston. Refer to Figure 7-13. The operational cycle of the single acting diesel hammer is shown on Figure 7-14 and is described in the following paragraphs.

To start operations, a cable from the crane lifts the ram. At the top of the stroke, the lifting attachment is “tripped” and the ram (piston), is allowed to drop. The ram falls by virtue of its own weight and activates the cam on the fuel injector that injects a set amount of fuel into the cup-shaped head of the impact block. As soon as the falling ram passes the exhaust ports, air is trapped in the cylinder ahead of the ram, and compression begins. The rapidly increasing compression pushes the impact block (anvil) and the helmet immediately below it against the pile head prior to the blow.

Upon striking the impact block with its spherically shaped leading end, the ram drives the pile into the ground and, at the same time atomizes the fuel which then escapes into the annular combustion chamber. The highly compressed hot air ignites the atomized fuel particles and the ensuing two-way expansion of gases continues to push on the moving pile while simultaneously recoiling the ram.

As the upward flying ram clears the exhaust ports, the gases are exhausted and pressure equalization in the cylinder takes place. As the ram continues its upward travel, fresh air is sucked in through the ports, thoroughly scavenging and cooling the cylinder. The cam on the fuel injector returns to its original position allowing new fuel to enter the injector.
for the next working cycle. The operator may stop the hammer manually by pulling a trigger, which deactivates the fuel supply.

The diesel hammer is difficult to keep operating when driving piles in soft material. Large downward displacements of the pile absorb most of the energy; therefore, little remains to lift the ram high enough to create sufficient compression in the next downstroke to ignite the fuel. To resume operation, the cable hoist must again raise the ram.

It is generally accepted that the energy output of an open-end diesel hammer is equal to the ram weight times the length of stroke. This combination ignores any component of the explosion that acts downward. In production pile driving, the stroke is really a function of the driving resistance, the pile rebound, and the combustion chamber pressure. The combustion chamber pressure is affected by the general condition of the hammer as well as the fuel timing and the efficiency of combustion. Accordingly, manufacturer’s energy ratings are based upon the hammer operating at refusal with almost all the energy of combustion developing the upward ram stroke leaving just the weight of the ram and the stroke left to determine energy.

Diesel hammers are very versatile. They may be connected to almost any set of leads. They do not require an additional energy source, such as steam or air so the size of the pile crew can be reduced. On occasion, piles are driven with crews of as few as three workers, including the crane operator. These hammers typically operate within a speed of 40 to 60 blows per minute and can have strokes in excess of 10 feet. Although these hammers will drive any type of pile, their stroke is dependent on soil conditions. Hard driving in harder soils results in increasing stroke lengths, thus providing increasing hammer energies; while easy driving in softer soils results in lower stroke lengths and lower hammer energies. It should be noted that diesel hammers are deemed to be noisy and are viewed as environmentally unfriendly by some as they tend to spew oil and grease and emit “unsightly” exhaust.

When using a diesel hammer the Engineer should:

### Table 7-6. Single Acting Diesel Hammer Actions.

<table>
<thead>
<tr>
<th>NO.</th>
<th>ITEM DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Have the manufacturer’s current specifications for the type and model of hammer being used.</td>
</tr>
<tr>
<td>2</td>
<td>Ensure all required parts of the hammer are intact and in good operating condition.</td>
</tr>
<tr>
<td>3</td>
<td>Have chart available declaring rated energy vs. operating speed of hammer.</td>
</tr>
<tr>
<td>4</td>
<td>Be aware of the actual stroke of the hammer during driving and that it will vary depending on soil resistance.</td>
</tr>
</tbody>
</table>
Figure 7-13. Single Acting Diesel Hammer.

Figure 7-14. Operational Cycle for Single Acting Diesel Hammer.
7- 4.5.2 Double Acting Diesel Hammer

The double acting diesel hammer is similar in its operations to other double acting hammers. The top of the cylinder is capped so that pressures can be developed on the downward stroke. The energy transferred is more than just a function of gravity. As the ram nears the top of its upward stroke, air is compressed in a “bounce chamber”. This halts the upward flight of the ram as pressure increases. The downstroke energy now becomes a function of both gravity and the internal pressure generated in the “bounce chamber”. The hammers have a stroke that is typically around 3 to 4 feet and operate at a much higher/quicker blow rate compared to the single acting diesel hammer. Refer to Figure 7-15.

These hammers normally have a manually operated variable fuel injector, which is controlled by the crane operator. Unless the control is wide open, the energy delivered is difficult to determine. The rated energy needs to be computed from a formula incorporating the length of the free fall downstroke of the ram multiplied by the sum of its weight and adding the effects of changes in pressures and volumes of air in the bounce/scavenging chambers of the hammer. Manufacturers have plotted the solutions to the formulae for each model of hammer for various pressure readings in the bounce chamber.

Figure 7-15. Double Acting Diesel Hammer.
When using a double acting diesel hammer the Engineer should:

<table>
<thead>
<tr>
<th>NO.</th>
<th>ITEM DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Have the manufacturer's current specifications for the type and model of hammer being used.</td>
</tr>
<tr>
<td>2</td>
<td>Ensure all required parts of the hammer are intact and in good operating condition.</td>
</tr>
<tr>
<td>3</td>
<td>Ensure the energy chart made available by the manufacturer is the correct one for the model of hammer being used and that there has been a recent calibration or certification of the bounce chamber gauge.</td>
</tr>
</tbody>
</table>

7-4.6 Vibratory Driver/Extractor

Vibratory pile drivers/extractors could be likened to a mini-stroke, high blow rate hammers. However, the familiar vibratory pile drivers in standard use today do not contain linearly reciprocating weights or rams. Instead, they employ two balanced rotating weight sets, which are eccentric from their centers of rotation. Moving in opposite directions, they impart a vibration that is entirely vertical. This motion is transmitted to the pile through the hydraulic clamps of the driving head. The pile in turn transmits the vibratory action to the soil allowing the soil granules to be more readily displaced by the pile tip. The same action works even more effectively for extracting piles. Refer to Figure 7-16.

The effectiveness of a vibratory unit is dependent upon the interrelationship of the performance factors inherent to the unit. The larger the eccentric moment, the more potential vibratory force the driver possesses. In order to realize this potential force, the driver must operate with the proper frequency and amplitude.

With heavier piles, there is a higher vibratory weight supported by the hammer. This tends to reduce the amplitude. So as piles get larger, it is necessary to use drivers with larger eccentric moments. The non-vibratory weight has the effect of extra weight pushing the pile downward.

Vibratory drivers are most effective in granular soil conditions, but recent developments and new techniques have also made them effective in more cohesive soils. They can handle a variety of piling, including steel sheets, steel pipe, concrete, timber, wide flange sections, “H” piles, as well as caissons. They do not create a large amplitude ground vibration compared to impact pile hammers discussed previously. This makes the vibratory hammer desirable in areas where excessive ground motions could possibly cause damage to adjacent structures.

The contract specifications† prohibit the use of the vibratory hammer for driving permanent contract piles because there is no way to determine the amount of energy delivered to the pile. However, contractors frequently use vibratory hammers to install

† 2010 SS, Section 49.2.01C(2), Driving Equipment, or 2006 SS, Section 49-1.05, Driving Equipment.
temporary works (i.e. placing and extracting sheet piles for shoring, etc.). These hammers are also used to extract piles.

Although vibratory hammers cannot be used when there is a nominal driving resistance requirement, the vibratory hammer has occasionally been permitted to install a bearing pile to a point above the expected final penetration. An impact hammer authorized for this operation is then placed upon the pile to drive it to acceptable bearing and final penetration values. A situation where this technique is useful is where alignment of a pile is critical. The vibratory hammer allows the operator to minimize the rate of penetration of a pile, thereby allowing for more precise alignment of a pile as it gets started into the ground.

There have been comparisons made in the recent past indicating variances in bearing capacities of piles when comparing a pile driven to the same elevation with a vibratory hammer and one driven with an authorized impact hammer. Items of interest and discussion include the set of the pile and the disturbance of the soil mass. The vibration of the pile against the soil may reduce the amount of skin friction on the pile leading to lower nominal resistances than what would have occurred if the pile were driven without vibratory means. This condition may be temporary. Depending on the soil, the skin friction may return in full or in part as the soil remolds or sets over time.

When a request is made to use a vibratory hammer to start a pile, the Engineer should:

<table>
<thead>
<tr>
<th>NO.</th>
<th>ITEM DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Be aware of specific pile requirements and limitations stated in the contract specifications.</td>
</tr>
<tr>
<td>2</td>
<td>Discuss the proposal with the Bridge Construction Engineer, the Designer, and the Geoprofessional.</td>
</tr>
</tbody>
</table>
7-4.7 Hydraulic Hammers
A hydraulic hammer incorporates an external energy source to lift the hammer to the top of its stroke. For the single acting hydraulic hammer, the free-falling piston provides the energy induced into the pile, much the same as a drop hammer or a single acting diesel hammer. The rated energy for the differential acting hydraulic hammer is found by means similar to other differential acting hammers. Refer to the previous section on differential acting steam/air hammers.

The theories of energy delivery and transfer vary between differential hydraulic hammers. For example, one particular hydraulic hammer manufacturer utilizes a ram made of composite material. In this case, it is made of lead wrapped in steel. The theory behind the lead ram is that a heavier weight falling a similar distance should produce blows with longer impact durations. This longer impact duration produces a compression wave that is low in amplitude and long in duration. It is thought that this type of blow is more efficient in terms of delivering driving energy to the tip of the pile (relative to a lightweight hammer with a longer stroke).
The hydraulic hammer has a variable stroke, which is readily controlled from a detached control box either located in the cab with the crane operator or otherwise. With the control box the stroke can be varied, finitely (reported to be in the centimeter range), such that the stroke can be optimized so that it matches the dynamic spring constant of the hammer and pile. Manufacturers have stated that the ability to vary the stroke and frequency enables these hammers to perform more efficiently than other types of hammers.

The general theory behind the hammer is as follows: Every ram body, depending on material and cross sectional area, has its own dynamic spring constant. Likewise, each pile, based on different materials and sizes, has its own dynamic spring constant or acoustic impedance. As the dynamic spring constants for the pile and the hammer converge, higher efficiencies can be achieved. Energy will be transmitted through the pile to the tip with fewer losses and at lower internal stresses. Essentially all the hammer energy will go into moving the pile since the losses in the pile were minimized. The greatest efficiency is achieved when the hammer impedance is the same as the pile impedance. If this were to occur, a pile cushion would be unnecessary and driving would be further optimized.

The manufacturer data sheets for these types of hammers state the following:

<table>
<thead>
<tr>
<th>NO.</th>
<th>ITEM DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Hammer efficiencies in the range of 80% to 98%, while saying that diesel hammers have efficiency in the range of 30% to 40%.</td>
</tr>
<tr>
<td>2</td>
<td>Due to the increased efficiency of the hammers and because more energy is transmitted through the hammer, there is less internal stress of the pile, less pile damage, etc.</td>
</tr>
<tr>
<td>3</td>
<td>They claim the operation to be quieter than the typical diesel hammer.</td>
</tr>
<tr>
<td>4</td>
<td>The typical exhaust of the diesel hammer is eliminated, since only the motor driving the hydraulics is the source of exhaust.</td>
</tr>
<tr>
<td>5</td>
<td>Avoids diesel hammer problems of soft ground starting and operating in extreme climates.</td>
</tr>
</tbody>
</table>

7-4.8 General Hammer Information
The contract specifications require the Contractor furnish an authorized hammer with sufficient energy to drive piles at a penetration rate of not less than 1/8-inch per blow at the required bearing value. In effect, this specification places a lower limit on the hammer size because hammer size, in most cases, is related to energy. An upper limit is not specified; however some hammers may be too large for the intended use and may damage the pile during installation.

Economics often dictate the selection of hammer size and type. Large hammers provide vast amounts of energy that will advance the pile quickly and reduce driving time. They also help achieve specified tip elevations when hard driving is encountered, thus enabling...
completion of the work without the need of supplemental measures such as jetting or drilling. On the other hand, heavy hammers require heavy leads and heavy cranes; the result being decreased mobility and increased equipment costs. Another consideration is that larger hammers deliver more energy to the pile. Hence, the probability of pile damage (heavy spalling, buckling, or other) increases as the hammer size increases. Ram impact velocity is another important factor. In general, a large ram weight with a short stroke and low velocity at impact will not produce the magnitude of pile stress that a light ram with a long stroke and high velocity will induce. Generally, at constant driving energy, the driving stress on the pile will decrease as the ram weight increases. Though there are situations where the bigger hammer may be too big and will overstress the pile. However, the option to run a bigger hammer at less than the maximum capacity, with a shortened stroke, may help, as the impact durations are different. Refer to the section on hydraulic hammers for more information on impact duration.

7-5 Nominal Resistance/Bearing Capacity

Pile driving formulas have been developed over the years to determine the nominal resistance of driven piles. There are many different (at least 450) pile driving formulas, the more notable of these being the Gates, Hiley, Pacific Coast Uniform Building Code, Janbu, and the ENR. Refer to Appendix E for examples. They have been empirically developed through testing and research. They utilize known information such as the energy delivered per blow, the resistance to the movement of the pile per blow, pile penetration, and some acknowledgement or estimates of the unknown or unquantifiable that serves to drive and/or resist the pile. All of the driving formulas make use of the conservation of energy theory:

\[(\text{HAMMER ENERGY}) - (\text{ENERGY LOSSES}) = (\text{WORK PERFORMED})\]

Soil resistance multiplied by pile penetration represents work performed, hammer stroke multiplied by ram weight represents hammer energy, and various factors and/or constants in driving formulas are derived to represent energy losses in the piling system. The desired objective is to account for the most significant energy losses so that soil resistance can be estimated. Some of the energy losses associated with pile driving are hammer combustion and mechanical inefficiency, hammer and pile cushion restitution, dynamic soil resistance and pile flexibility. No pile driving formula accounts for all energy losses, and the major difference between formulas is which losses each considers.

The contract specifications require the bearing value of driven piles be determined using the Gates formula as follows (Refer to Appendix E, Driven Piles, for examples):

\[11\] 2010 SS, Section 49-2.01A(4)(b), Pile Driving Acceptance Criteria, or 2006 SS, Section 49-1.08, Pile Driving Acceptance Criteria.
\[ R_u = (1.83 \times (E_r)^{1/2} \times \log_{10}(0.83 \times N)) - 124 \]

Where:
- \( R_u \) = the nominal resistance in kips.
- \( E_r \) = the manufacturer’s rating for foot-pounds of energy developed by the hammer at the observed field drop height.
- \( N \) = the number of hammer blows in the last foot (maximum value for \( N \) is 96).

This formula is appropriate for piles with a nominal driving resistance of 600 kips or less. Acceptance criteria for piles that require higher capacities than the Standard Plan piles may be determined by other methods. The other methods for determining the load-bearing capacity of a pile depend on detailed knowledge of how energy is transmitted to a pile during driving. These exercises are much more detailed than the pile driving formulas. These methods and procedures typically obtain more accurate representations of the pile’s bearing capacity and can be categorized into three areas: (1) Pile Load Testing, (2) Wave Equation Analysis of pile driving, and (3) Dynamic Pile Driving Analysis (PDA). The processes are explained in detail in the next chapter but a brief description of each one follows.

7-5.1 Pile Load Testing

The most accurate way to determine the axial capacity of a pile is to perform a static load test on it. The method is time consuming and expensive so it is reserved for locations where the underlying geology is variable and complex. Load tests are useful in determining the capacities of large diameter piles as the traditional method of using pile-driving formulas loses accuracy as the diameter of the pile increases. Typically, the load test pile is pushed and pulled by hydraulics that are attached to a resisting beam to a point were design loads or ultimate capacity is achieved. Refer to Chapter 8, Static Pile Load Testing and Pile Dynamic Analysis, for information that is more detailed.

7-5.2 Dynamic Analysis by Wave Equation

Wave equation analysis is used to create site-specific model of the interaction of the pile, hammer and soil. It is a one-dimensional finite difference analysis method, which models the transmission of a hammer’s impact wave down a pile and into the soil. Several versions of the program are available. The program used by Caltrans is one of the most widely used. It was developed by a company called GRL and is called Wave Equation Analysis of Piles (GRL WEAP).

Wave equation analysis models the pile and the driving system as well as the different soil lenses that the pile is expected to drive through. The soil is modeled as a series of elastic plastic springs and linear dashpots. The relative sizes of the springs and masses depend on the actual soil properties as shown on the Log of Test Borings (LOTB). Driving system characteristics are embedded in the program and pile characteristics, such as diameter and wall thickness are input by the user. After modeling, a dynamic analysis is performed. Wave equation analysis has been used for drivability studies, hammer acceptance studies, and to develop site-specific curves that relate nominal resistance with
pile blow counts and energy. The wave equation analysis method has been shown to provide a more accurate indication of actual nominal resistance than by pile driving equations.

7-5.2.1 Drivability Study
The wave equation analysis can be used as a drivability study during the design phase to validate design assumptions for wall thickness on pipe piles and hammer sizes and types. Geoprofessionals from Geotechnical Services, Foundation Testing Branch, create the driving system model. The input information consists of soil characteristics taken from the Log of Test Borings, pile length, and other material properties of the pile obtained from the Designer. In addition, hammer data, such as type and cushion properties for the different hammers likely to be used in the actual construction operations, is input.

The output information provides the internal stresses of the pile as it travels through the varying strata and as it approaches the specified tip elevation. The output also gives information on driving rates for specific hammers through the different soil strata. The model is run using several different hammer sizes and types. The results are presented in a report that shows how the different hammers will drive piles through the different soil strata. This analysis also offers the Designer the opportunity to change pile types, sizes, or thicknesses should the drive analysis show that pile-driving difficulties could be overcome with changes in the pile characteristics.

7-5.2.2 Hammer Acceptance Study
Hammer Acceptance studies are done after the contract is awarded. The Standard Specifications require the Contractor to submit information and wave equation analysis for driving systems proposed for use on the project under two circumstances; (1) when the Special Provisions require a driving system submittal, or (2) if the ram stroke for the proposed hammer is not visible during driving. This information is used by the Foundation Testing Branch to perform wave equation analysis for comparison. Essentially a drivability study is performed using the actual hammer information instead of assumed values. From this information, the Engineer can decide if the proposed driving systems will drive the pile to the specified tip elevation and reach the nominal resistance without overstressing the pile during driving. The results of the study might also show that the chosen driving system is not efficient. Either way the results of the drivability study are used by the Engineer as a basis for authorizing the hammer submittal.

7-5.2.3 Acceptance Curve Study
The studies described above use theoretical or empirical information to develop a model that gives a pretty accurate indication of what is likely to be encountered in the field. Gathering additional information while driving an actual pile can refine this model. Pile Dynamic Analysis (PDA) equipment can be used to record and process information.
gathered from stress and strain gauges attached directly to the pile. The information can be recorded during initial driving and during re-drives to determine increased capacities over time. The information from the PDA can be analyzed using the Case Pile Wave Analysis Program (CAPWAP) to estimate capacity. On some larger projects with complex soils, a static load test might also be performed to refine CAPWAP even further. The pile capacity as determined by CAPWAP is used to refine the original WAVE model.

Acceptance curves are developed from outputs of the refined models. The curves correlate pile capacity to blow counts and hammer energy/driving rate. They are site specific and may even be foundation specific. The Engineer uses the curves in the field to determine the nominal resistance of a driven pile. The curves are used in place of the acceptance criteria outlined in the contract specifications. The curves may also be used to provide criteria for field revisions to the specified tip elevation when compression controls the design. Refer to Appendix E, Driven Piles, for samples of acceptance curves.

Another situation where acceptance curves are useful is in situations where the ground conditions during driving are not what controls the design. Examples of these are foundations that require the installation of driven piles in scour sensitive areas, through liquefiable soils, or through large layers of re-moldable clays. In these instances, piles need to be driven through materials that will provide skin friction resistance during driving but not under the extreme event limit state. If driving through re-moldable clays, skin friction is lost during the driving operation, but then returns over time.

Pile load tests, WAVE analysis and CAPWAP runs have been performed in the design phase and the construction phase to provide additional information and confidence to the Designer and Geoprofessional. These types of analysis are normally done on large projects but in recent years have been done on projects that use large diameter piles. The correlation of nominal resistance to pile driving formulas is not very effective for large diameter piles so these additional measures are needed.

Piles driven in re-moldable clays, such as Bay Mud found in the San Francisco Bay Area, lose virtually all their skin friction during driving. The skin friction returns with time as the pore water pressures are redistributed. The driven pile will actually achieve greater capacity over time as the skin friction returns. As such, piles driven to specified tip on the day of driving might not achieve nominal resistance but may do so days and sometimes hours later. Acceptance curves provide new criteria for the piles thereby eliminating the need to perform expensive and time consuming re-drives or the need to extend the piles.

During the process to develop acceptance curves it may become apparent that there is a need or opportunity to revise the specified tip elevations shown on the contract plans. When this is done during construction, the special provisions will outline the

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13 2010 SS, Section 49-2.01A(4)(b), Pile Acceptance Criteria, or 2006 SS, Section 49-1.08, Pile Acceptance Criteria.
administrative process to be followed. Often the special provisions prohibit the procurement of piles until pile load tests are completed and revised tip elevations are provided. That way piles and rebar cages can be fabricated to the correct length and any required splices kept to a minimum.

7-5.3 Manufacturer’s Energy Ratings
Generally each manufacturer publishes a catalog or brochure for their hammers. It outlines operating specifications, including any specific equipment that is required for the safe operation of the hammer. Manufacturer’s specifications such as ram weight, stroke, blows per minute, and the minimum required steam or air pressure are important as they all relate to the energy that the hammer is capable of delivering under ideal conditions. Manufacturers calculate hammer energy differently. Some use ram weight multiplied by the stroke. At one time, Delmag calculated a hammer’s energy as a function of the amount of fuel injected but now use the weight of the ram times the stroke. Other manufacturers include the effects of additional parameters such as fuel ignited and the effect of the bounce chamber. In any case, a hammer’s rated maximum energy is the rating when the pile hammer is operating at or near refusal. It does not consider losses and is essentially the amount of potential energy, in foot-pounds, capable of being delivered by any one blow.

The Engineer uses the manufacturer’s maximum rated energy, for a given stroke, as an indication of the driving capability of the hammer. It is used in the Gates formula as required by the contract specifications. It is important to know that the manufacturer’s given energy rating should not be used “blindly”. The actual potential energy needs to be verified by measuring the stroke of single acting diesel hammers and by comparing the operations of the hammer with the manufacturer’s operating specifications for other hammer types. Just because a hammer is operating properly doesn’t mean that it is operating at maximum efficiency.

As stated previously, manufacturers rate their hammers by determining the amount of energy that can potentially be transferred to the pile. They do not specify the amount of kinetic energy that is actually delivered by a hammer at the head of the pile after undergoing losses. These losses occur in the transfer of energy through the driving system and can vary from hammer to hammer. The ratio of the maximum rated energy provided by the manufacturer to the actual energy delivered to the pile is the hammer’s efficiency. An accurate determination of the actual available energy of any given hammer is difficult as there are many things that can have an effect on the efficiency of the system. Factors such as wear and tear, age, type of cushion, improper adjustment of valve gear, poor lubrication, fuel setting, unusually long hoses, minor hose leaks, binding in guides, and minor drops in steam or air pressure can all affect the performance of a hammer.

It is necessary to have a working knowledge of hammer operations. The Engineer must ensure that the accepted hammer on the job is operating properly and is capable of producing the manufacturer’s “rated energy” (or potential energy, at the top of its stroke).
Material presented in this manual and material found in other technical publications will supplement this knowledge. However, there is no substitute for field experience. The Engineer is advised to look into the mechanical aspects of the pile driving operation when the Contractor starts assembling the equipment and driving begins.

7-5.4 Battered Piles

Adjustments must be made when driving battered piles since the path of the ram is not plumb. One method of adjusting for battered piles is to use the modified Excel spreadsheet shown in Appendix E, which incorporates the pile batter into the Blows vs Stroke chart. Another method is to adjust the ram stroke to represent the vertical fall. For example, an observed ram stroke of 7 feet for a 1:3 battered pile indicates the ram is moving 7 feet along the path of the pile. The vertical fall height is less, only 6.64 feet (7 feet x 3/3.162). It is mathematically incorrect to apply the correction to the hammer blows, since the relationship between ram stroke and hammer blows is not linear. Refer to the examples in Appendix E, Driven Piles.

A similar adjustment must be made for double acting and differential hammers. However, in determining the change in energy due to the batter, compensate only for that portion of the energy attributed to the free fall of the ram. Energy delivered by differential action or pressure imparted on the downward stroke should remain constant.

7-6 Preparing to Drive Piles

A Driven Piling Construction Checklist is presented in Appendix K-1 to assist field personnel in preparing documents and inspecting fieldwork to ensure compliance with contract requirements.

Pile driving techniques (including solutions to problems) are normally developed with time and experience. It is the intent of this section to provide some insight into the areas where problems can develop. With this potential knowledge, the Engineer is thus enabled to potentially eliminate them prior to their occurrence.

The Engineer should review the following lists before pile driving begins and while pile driving is underway. These lists are by no means complete, as new and different construction challenges will develop with each and every project.

Advance preparation to begin well before mobilization of pile driving equipment:

<table>
<thead>
<tr>
<th>NO.</th>
<th>ITEM DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Review the plans, special provisions, Standard Specifications, and Foundation Report for requirements on pile type, required bearing and penetration, drilling or predrilling depths, (critical with tension piles as well as compression piles), tip protection, pile lugs and limitations on hammer types, or other specific limitations or requirements.</td>
</tr>
</tbody>
</table>
### Table 7-11. Field Preparation Tasks Prior to Driving Piles.

<table>
<thead>
<tr>
<th>NO.</th>
<th>ITEM DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Confirm pile layout and batter requirements. The Contractor is to locate the position of the piles in the footing. The Engineer is to check the layout only. Do not lay out piles for the Contractor.</td>
</tr>
<tr>
<td>2</td>
<td>Confirm pile materials, tips, and lugs. Refer to the Materials Checklist later in this chapter.</td>
</tr>
<tr>
<td>3</td>
<td>Confirm the hammer type. If the hammer has a variable energy setting, check the setting to ensure the proper energy will be obtained. Some of the newer diesel hammers have four settings giving a range of 46% to 100% maximum energy. Ultimately the fuel setting is up to the Contractor. However, the Engineer should be aware of what the setting is and why it is in place.</td>
</tr>
<tr>
<td>4</td>
<td>Verify the reference elevation.</td>
</tr>
<tr>
<td>5</td>
<td>Layout and mark piles for logging. Mark additional reference points near the anticipated tip elevations so that monitoring can take place at smaller increments.</td>
</tr>
<tr>
<td>6</td>
<td>Locate a good place to inspect operations. Notify the pile foreman of location and signals to be used.</td>
</tr>
</tbody>
</table>

### Table 7-12. Field Preparation Tasks When Pile Driving Starts.

<table>
<thead>
<tr>
<th>NO.</th>
<th>ITEM DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Verify the pile location at the start of driving.</td>
</tr>
<tr>
<td>2</td>
<td>Verify plumbness or batter of the pile at the start of and during driving.</td>
</tr>
<tr>
<td>3</td>
<td>Monitor and log the blow count, stroke, and penetration (Refer to the Logging of Piles section later in this chapter).</td>
</tr>
<tr>
<td>4</td>
<td>Stop driving at proper bearing and penetration.</td>
</tr>
<tr>
<td>5</td>
<td>Be prepared to stop the driving if it appears that additional driving will damage the pile or if it appears the piles may run long.</td>
</tr>
</tbody>
</table>
After completion of driving piles:

**Table 7-13. Field Tasks After Completion of Pile Driving.**

<table>
<thead>
<tr>
<th>NO.</th>
<th>ITEM DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Verify proper pile cutoff.</td>
</tr>
<tr>
<td>2</td>
<td>Prepare copies of pile logs to be sent to Structure Construction in Sacramento in accordance with Bridge Construction Memo 3-7.0, Pile Records.</td>
</tr>
</tbody>
</table>

### 7-6.1 Verification of Hammer Energy

Several verification methods are available in the field to determine the amount of hammer energy that a hammer delivers to a pile in any one blow or over a short period of time. For single acting diesel, steam, or air hammers, the simplest method is to measure the stroke of the hammer and multiply this by the weight of the ram. While this method may underestimate the complexities of pile driving and energy transfer, it is the simplest method available for use in the field. To determine the stroke for diesel hammers, measure the depth of ram below the top of the cylinder before driving and add that to the height the top of the ram rises above the cylinder during driving. To determine this height, paint is often applied in one-foot intervals on the trip carriage above the cylinder. However, some hammers have rams with identifiable rings that are visible during driving. The location of the rings normally is shown on the manufacturer’s brochure.

The maximum rated stroke for maximum rated energy for many hammers can be found on pile hammer manufacturer websites.

Another method of determining the actual ram stroke of an open-end diesel hammer is accomplished by measuring the ram stroke from the blow rate. The equation involved with this method is sometimes called the Saximeter equation. Saximeter is a trade name for a device used for remote measuring of the stroke of an open-end diesel hammer or the measurement of the hammer speed. This method is simplified by simply counting the blows per minute. An example is available in Appendix E.

For Air and Steam hammers, check the boiler or air capacity of the outside energy sources. This should be equal to or greater than that specified by the hammer manufacturer. Gages that indicate steam and air pressures are required by the contract specifications. Verify the system is using the proper hose size recommended for the particular steam and air hammers. The hoses should comply with the manufacturer’s specifications. All hoses should be in good condition (no leaks).

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14 2010 SS, Section 49-2.01C(2), *Driving Equipment*, or 2006 SS, Section 49-1.05, *Driving Equipment.*
7-6.2 Materials Checklist

7-6.2.1 Precast Concrete Piles

Table 7-14. Precast Concrete Piles Checklist.

<table>
<thead>
<tr>
<th>CHECK ITEM</th>
<th>CHECK DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Check for damage, cracks, chips, etc. Check the date the pile was cast. This date is written, along with the release number, directly on the surface of the pile. The contract specifications require piles be at least 14 days old before driving.</td>
</tr>
<tr>
<td>2</td>
<td>Lifting anchors for piles in a corrosive environment are to be removed to a depth of one inch and the hole filled with epoxy. Piles driven in a non-corrosive environment must have the anchors removed along the portion of pile above the final ground line in accordance with the contract specifications.</td>
</tr>
</tbody>
</table>

Discuss with the Contractor the type and method of rigging planned to lift precast prestressed concrete piles. The Contractor is to provide the necessary equipment so as to avoid appreciable bending of the pile or cracking of the concrete. If the Contractor materially damages the pile, the pile must be replaced at the Contractor’s expense in accordance with the contract specifications.

Check the lifting procedure to ensure that the pile is not overstressed at any time during picking. The maximum permissible allowable stress is as follows:

\[
\text{Allowable Stress} = 5\sqrt{f'c} \text{ PSI tension}
\]

Measure piles and paint the necessary one-foot marks so blow counts can be determined. Check the ends of the piles. Prestressing steel should be flush with the pile head and covered with zinc primer. The head of the pile should be square.

When driving concrete piles, make sure that the cushion blocks are maintained in good condition. Failure to do so may increase the risk of damaging the piles during driving. If the driving is hard, the cushions may need to be changed once or twice per pile.

7-6.2.2 Steel Piles

If the piles are to be spliced, the Contractor must have welder(s) qualified prior to performing the welds. They must be qualified in accordance with the Welding Quality Control Plan and the AWS D1.1, Structural Welding Code. Assistance may be obtained by calling Materials Engineering and Testing Services (METS).

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15 2010 SS, Section 49-2.04C(1), Construction General, or 2006 SS, Section 49-1.07, Driving.
16 2010 SS, Section 49-2.04B(2), Fabrication, or 2006 SS, Section 49-3.01, Precast Prestressed Concrete Piles Description.
17 2010 SS, Section 49-2.04C(2), Handling, or 2006 SS, Section 49-3.03, Handling.
Some welders will have qualification tests that were performed by a private testing laboratory. Prequalification can be accomplished in this instance by forwarding a copy of the test reports to the nearest Transportation Laboratory office where they will verify the welder’s qualifications.

It is obvious that all of the aforementioned takes time. Hence, it is extremely important that determination of welder qualification be made as early as possible. Keep in mind that just because a person holds a welding certification, it does not mean you do not have to inspect the welding work.

Early contact with METS representatives in Los Angeles, Vallejo, or Sacramento is encouraged, as they can be very helpful. Reference should also be made to Section 180, (Welding), of the Bridge Construction Records and Procedures Manual.

### Table 7-15. Steel Piles Checklist.

<table>
<thead>
<tr>
<th>CHECK ITEM</th>
<th>CHECK DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Check for proper diameter, pipe pile thickness or flange and web dimensions. Paint one-foot marks and lengths on the piles. The Contractor may assist in this.</td>
</tr>
<tr>
<td>2</td>
<td>Check welded joints for any sign of improper welding. When piles are to be spliced, a Welding Quality Control Plan will be required in accordance with the contract specifications.</td>
</tr>
</tbody>
</table>

#### 7-6.2.3 Timber Piles

Check the butt and tip diameters to ensure compliance with the contract plans. Treated timber piles must be driven within 6 months after treatment.

Piles must have protective steel straps at 10-foot centers. Three additional straps are placed at the tip and two at the butt. Straps are to be approximately 1-1/4 inches wide and 0.03 inch in nominal thickness per the contract plans.

The Contractor is also required to restrain the pile during driving from lateral movement at intervals not exceeding 20 feet measured between the head and the ground surface. Make sure the Contractor is equipped for this.

#### 7-6.3 Logging of Piles

Structure Construction practice is to log at least one pile, in its entirety, per footing. There are advantages to doing a more comprehensive logging of the piles. One situation is when, during easy driving, the piles are not achieving the necessary blow counts at specified tip. The Contractor will request to restrike or re-tap them later. A good log of the piles within the footing will help the Engineer to determine how many piles might require a restrike or re-tap to prove bearing. If all the piles drove in a similar manner, it might be possible to restrike or re-tap as few as 10% of the piles that did not originally

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18 2010 SS, Section 11-3.02, *Welding Quality Control*, or Special Provisions for contracts using 2006 SS.
achieve bearing. If the piles all drove differently, a restrike or re-tap of all of the piles may be required. The following is a discussion of factors affecting pile log data.

Typically when pile driving begins, the driving resistance of the pile is very low. The stroke of the hammer will be proportional to this pile resistance (low resistance equals low rebound energy). As a result, the energy delivered to the pile will be different from the manufacturer’s rated energy value. Keeping careful track of blows per foot and actual stroke is necessary. If this difference is not taken into account, the log will be misleading when the values are put in the Gates Formula and bearing values are computed at various depths of driving. This procedure should be followed all the way to the final tip penetration.

With double acting steam or air hammers, check the gages for proper pressure during the driving operation. In addition to measuring the actual stroke, it is important that the blow rate be verified.

Underwater and closed system hammers are difficult to inspect and can be throttled by the operator. The full open position should be used to obtain maximum energy. Be sure to pick a fixed reference point as close to the pile as practical when logging piles or determining final blow count. This can be accomplished several ways:

1. Mark the pile with one foot marks and note the blows passing a fixed point near the pile (leads, reference point, lath driven near the pile, water surface, or other).
2. Mark the lower part of the leads with one foot marks and observe passage of a fixed point of the pile.

Site conditions often dictate how this is done, so improvise as necessary. Modifications must also be made to obtain blow counts over smaller increments.

If a precast concrete pile is undergoing hard driving and experiences a sudden drop or movement, this could indicate a fracture of the pile below ground. Driving should stop and an investigation of the soundness of the pile should be made. Piles that are damaged should be extracted. However, this is not always possible. Frequently, driving a replacement pile next to the rejected one can solve this problem. However, the effect of this change could impact the footing design so the Designer should be consulted when this option is used.

Be aware of the water level in the pile when driving hollow pipe piles in water. A phenomenon known as a water hammer can develop during driving. The increase in pressure from the water hammer could split the pile. To prevent this, the pile may need to be pumped free of water after seating and before driving.

Another problem that can occur with pipe piles has to do with what is called a soil plug. When driving hollow piles, there is a tendency for the soil to plug within the pile as it is being driven. This is common in cohesive materials. When this does occur the pile will drive as if it is a displacement (closed-end) pile. There are many implications if this happens. Among the possibilities include the possible overstressing of a pile as well as
misleading blow counts. Center relief drilling may be needed to remove the plug so that the specified tip elevation can be reached.

7-7 Driving Challenges

Problems with driving can vary in nature and cause. In general there are three categories of problems: (1) hard driving, (2) easy driving, and (3) pile alignment. The causes typically are the soil is too hard or soft, the type of hammer used is inappropriate for the soils encountered, or the pile type being used is inappropriate. The following is an outline of various driving problems that can be encountered. The types of problems described are, by no means, a complete listing of all possible problems.

7-7.1 Difficult or Hard Driving

Hard driving is a term used to describe piles that have achieved nominal resistance but have difficulty reaching the specified tip elevation. This may happen when the soils are dense or when the hammer size or type cannot penetrate a particular soil lens or is inappropriate for the work in general. A review of the special provisions, Foundation Report, and LOTB should give an indication as to whether or not hard driving is to be expected. The pile placement plan should address the means and methods proposed to address hard driving.

The Standard Specifications and special provisions discuss what can be done to address this condition. For example, the contract specifications state: If necessary to attain the specified tip elevation shown and where authorized, you may drill holes with a diameter not greater than the least dimension of the pile to the specified depth before driving the piles. For driven piles, shells, or casings, the contract specifications also require the use of special driving tips, heavier pile sections, or other measures authorized by the Engineer, to assist in driving or prevent damage to a pile through a hard layer of material.

Hard driving and pile refusal are often interrelated as refusal can be considered the ultimate form of hard driving. Unfortunately, there are many definitions for the term refusal. Some popular interpretations range from:

1. Twice the required blow count.
2. Ten or more blows per inch.
3. No penetration of the pile under maximum driving energy.

Regardless of any specific definition, refusal is essentially the point where additional measures are needed to advance the pile to the specified tip elevation. These measures can be as simple as verifying the efficient operation of the hammer or more time-consuming like drilling or spudding.

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19 2010 SS, Section 49-2.01C(3), Drilling, or 2006 SS, Section 49-1.05, Driving Equipment.
20 2010 SS, Section 49-2.01C(5), Driving, or 2006 SS, Section 49-1.05, Driving Equipment.
The size and type of hammer used to drive the pile play a part in having and/or resolving a hard driving issue. One should keep in mind that proper hammer sizing is not accomplished simply by meeting the minimum energy requirement required to drive the piles to the nominal driving resistance. It is important to be aware that the hammer needs to overcome the anticipated soil resistance and impedance to achieve the specified tip elevation. Other issues, such as the dynamic response of soils and the relative weights of the hammer and the pile, if not properly considered, may be the root cause of hard driving. A Wave Equation Analysis can capture many of these parameters and is often required on projects driving high capacity piles.

Hard driving is not always a permanent condition and can also be the result of a pressure bulb that has developed near the pile tip. This can occur in saturated sandy materials when pore water pressures build up during driving, but can dissipate over a relatively short period of time. Driving these types of piles in stages may remedy this situation.

Sometimes the means and methods of construction may increase the likelihood of experiencing hard driving. Soil densification or consolidation can occur when driving displacement piles in a cluster for a building or bridge footing or abutment. A revised driving sequence will often alleviate this problem. This can often be a trial and error process. Driving from one side of the footing in a uniform heading helps as does driving from the center in a uniform outward pattern. Both of these procedures should mitigate the issue and increase the likelihood of driving piles without issues.

Sometimes other construction methodologies are required to address hard driving. These methods include drilling, attaching driving tips, and spudding. These methods are typically used when economics dictate this to be the best solution or when larger hammers cannot be utilized because of their potential to overstress the pile.

Drilling a starter hole to facilitate the advancement of a driven pile is known as drilling to assist driving. The contract specifications state the hole drilled must not be larger than the least dimension of the pile to be driven. This method has the potential to impact pile capacity, particularly for those that utilize skin friction. Often the amount or depth of drilling is limited to address this. There should be information in the plans, the Foundation Report or the special provisions that outlines these restrictions.

Driving tips strengthen the tip of the pile so that it can penetrate through obstructions and dense soil lenses. Cutting shoes are another form of driving tip that allows piles with thinner wall thicknesses to be driven through dense soil lenses. Closed ended steel piles may require a conical tip to facilitate driving and mitigate damage to the pile.

Spudding is another method used to assist the penetration of piles through dense lenses of material. It involves the use of a heavy or stout section to drive, break, or cut through a

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21 2010 SS, Section 49-2.01C(3), Drilling, or 2006 SS, Section 49-1.05, Driving Equipment.
lens of hard material. The spud is removed after this is achieved and the production pile driven in its place to the specified tip elevation.

Except for timber piles, the term hard driving or difficult driving may be subject to individual interpretation as there is no language in the specifications that defines it. Steel or concrete piles have no measures specified to mitigate hard driving at predetermined blow count levels. However, the Contractor is required to employ the measures described above to obtain the required penetration and is also required to use equipment that will not result in damage to the pile.

For projects utilizing the 2006 or older Standard Specifications, the contract specifications outline what to do when hard driving is encountered in timber piles. When the blow count for timber piles exceeds either 2 times the blow count required in one-foot, or 3 times the blow count required in 3-inches for the nominal driving resistance, additional means are required to achieve the specified tip elevation. These may include drilling or changing hammers to one with a heavy ram striking at a low velocity.

Physical damage to the pile, even when it is below ground, is fairly easy to determine. Impending damage and/or high driving stresses are not as easy to pinpoint. In situations of high driving resistance, the Engineer is advised to investigate pile stresses. This can be done with Pile Driving Analysis (PDA) equipment.

Because of the many variables involved, each hard driving issue must be evaluated on its own merit. There is no substitute for engineering judgment in this area. It should also be remembered that these issues are somewhat common and there is a broad base of experience within Structure Construction.

Piles typically are designed to meet four different design criteria: tension, compression, lateral, and scour. When compression controls the design, the Engineer has the flexibility to raise tip elevations to address hard driving. However, these tips should only be revised to the elevation of the next controlling criteria, and as confirmed by the Design Team, as discussed in Chapter 3, Contract Administration.

While it may be important to make a distinction between hard driving that was anticipated and what was not, it is in the best interest of all parties to work toward resolution of the issue quickly and efficiently in order to mitigate impacts to the project. There have been occasions where pile penetration to the specified tip elevation cannot be accomplished, despite everyone’s best efforts. When this situation occurs, the Engineer needs to be proactive in finding an alternative solution. This includes conversation and meetings with Structure Design and Geotechnical Services to find an alternative tip elevation, method or design to address the challenge.

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22 2006 SS, Section 49-1.07, Driving.
### 7-7.2 Soft Piles and Re-Drive

The *Standard Specifications* require the Contractor to satisfy requirements for minimum nominal driving resistance and specified tip elevation. A pile that drove *soft* is a pile that has been driven to the specified tip elevation but has not obtained the minimum driving nominal resistance. There are several options that can be explored when this occurs:

- Continue driving until the minimal nominal penetration can be achieved.
- Install pile lugs on H-Piles as discussed in Bridge Construction Memo 130-5.0, *Steel H Pile Lugs*.
- The pile can be *re-driven* sometime after initial driving, typically a matter of days, with the expectation that the pile has *set up* over time.

There are advantages and disadvantages to selecting any of these options. The first two options require field welding of steel piles so a welding quality control plan will most likely need to be created or revised for this work. Another issue is that the locations of field splices in piles may be limited to certain zones along the pile. Some pile designs have a no-splice zone or a no-field splice zone in the upper portion of the pile. This is because the loads and subsequent risks of plastic hinging are high. As such, the contract plans or special provisions may not allow field welding an extension on to a pile as the splice may fall within this zone.

The third option is a re-drive, also known as a re-strike or re-tap, of the pile. To do this, pile driving is stopped when the pile is several inches to one foot above the specified tip elevation. The pile is then driven the remaining distance at a later time. This allows the soil time to *set-up* around the pile. The time required for *set-up* depends on the soil and is anywhere from a day to a week. This option is effective in cohesive soils but not so much in submerged and saturated sands and gravels as there is little cohesion in these soil structures.

The Gates formula is still used for pile acceptance during a re-drive. However, it is important to note that the formula uses the number of hammer blows it takes to drive the last foot to determine nominal driving resistance. Typically piles can only be driven several inches before *set-up* is lost. Since the distance driven in a re-drive is less than one foot, the number of blows per foot used in the Gates formula will need to be extrapolated from the field results based on the number of blows over the driven length of the re-drive before *set-up* is lost. The number of blows to complete driving the pile to specified tip after *set-up* is lost are not counted. The extrapolated blows-per-foot value will be used to determine nominal driving resistance in the Gates formula.

Following are some ground conditions and the expected outcome after re-driving to address soft piles:
Table 7-16. Ground Conditions and Expected Re-Drive Outcomes.

<table>
<thead>
<tr>
<th>CONDITION</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Loose submerged fine uniform sand. Driving temporarily produces a quick condition. Re-drive will probably not indicate any change in capacity.</td>
</tr>
<tr>
<td>2</td>
<td>Cohesive soil. Driving temporarily breaks down the soil structure, causing it to lose a part of its compressive strength and shear value. Re-drive should indicate increased capacity.</td>
</tr>
<tr>
<td>3</td>
<td>Saturated coarse-grained pervious material. May display high driving resistance, but on re-drive will lose capacity as compared to the initial driving. This could be due to changes in pore water pressure within the soil mass.</td>
</tr>
</tbody>
</table>

On contracts where soft driving in clay materials is anticipated, specific re-drive guidelines are frequently given in the special provisions. The set-up period is usually set at a minimum of 12 hours. In addition, only a fixed percentage of the piles are re-driven (10% or a minimum of 2 per footing). However, when re-drive requirements are not listed in the special provisions, the Engineer can still utilize this methodology. Ultimately, the Engineer must be satisfied that all of the piles have achieved the bearing capacity, and should therefore re-drive a sufficient number of piles to ensure this.

Re-driving is a tool that the Engineer can use in an attempt to obtain an acceptable pile even though the contract specifications may not discuss re-drives or specify elapsed time before attempting a re-drive. Trial and error methods may have to be employed to figure out the appropriate time to wait before re-driving. It is the Engineer’s responsibility to determine what criteria will be used to determine pile acceptability. At times piles will not attain minimum bearing at specified tip, even when re-driven. When this happens the only option is to splice on additional length and continue driving to a point where the nominal driving resistance is achieved.

Issues with soft piles frequently occur in steel H piles. When overdriving is excessive, lugs or “stoppers” can be welded on the pile to mitigate the problem. If lugs are not required by the contract, they can be added by change order. Bridge Construction Memo 130-5.0, Steel H Pile Lugs, covers this in detail.

7-7.3 Alignment of Piles
The Engineer needs to verify that each pile is placed in the correct location and that the alignment is plumb or at the required batter. This should occur often during the first part of the drilling or driving of each pile and periodically thereafter. This is extremely important when swinging leads are used for pile driving as these leads lack the guides that fixed leads have. Alignment corrections should be made if the pile begins to move out of line. In certain instances, driving may need to be stopped so that the pile can be pulled and re-driven correctly.
While the contract specifications state, *The Engineer rejects piles materially out of line,* there is no tolerance provided in the specification that define when a pile truly is *materially out of line.* Some contracts have specific tolerances outlined in the *Special Provisions* that define the criteria for acceptable alignment and/or plumbness of the piles. This is usually due to special considerations in the design of the structure and to clarify the Designer’s intent. Each situation should be analyzed separately and *engineering judgment* used in making final determination as to the acceptability of any misaligned piles.

### 7-7.4 Overdriving

Occasionally the Contractor will want to overdrive prefabricated piles to avoid cutting piles to grade. This can be allowed in most circumstances. However, no payment is allowed for the additional length driven below the specified tip elevation unless it is part of an ordered change to the specified tip elevation. This subject is discussed in Bridge Construction Memo 130-6.0, *Measurement and Payment for Piling.*

### 7-8 Safety

The potential for accidents to occur during pile driving operations may be greater than for any other construction operation. The pile driving crane rigged with a set of heavy leads and a hammer is unwieldy enough; add to it a long pile and a high potential for danger exists. These risks increase when the hammer is in operation as all the parts are moving and support equipment such as a steam or high-pressure line are at capacity.

The following are some of the items that individuals inspecting piles should be aware of, especially personnel new to construction:

<table>
<thead>
<tr>
<th>ITEM NO.</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Stand away from the pile when it is being picked and placed in the leads. Sometimes the pile when dragged will move in a direction not anticipated.</td>
</tr>
<tr>
<td>2</td>
<td>Stand as far away from the operation as practical while still inspecting the work.</td>
</tr>
<tr>
<td>3</td>
<td>Keep clear of any steam, air, or hydraulic lines.</td>
</tr>
<tr>
<td>4</td>
<td>Watch the swing of the rig so as not to be hit by the counterweight.</td>
</tr>
<tr>
<td>5</td>
<td>Wear safety glasses. There is a high incident of flying debris during the driving operation (dirt from piles, concrete from piles and steel chips).</td>
</tr>
<tr>
<td>6</td>
<td>Keep an eye on the operation in progress. Look out for falling tools and materials from the pile butts. Watch the rig in case the leads start to fall or the rig starts to tip.</td>
</tr>
<tr>
<td>7</td>
<td>Hearing protection is required due to high noise levels.</td>
</tr>
<tr>
<td>8</td>
<td>Have a planned route for rapid escape. If required to move quickly there will not be time to look around first.</td>
</tr>
</tbody>
</table>

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23 2010 SS, Section 49-2.01C(5), *Driving,* or 2006 SS, Section 49-1.07, *Driving.*
<table>
<thead>
<tr>
<th>ITEM NO.</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>Wear old clothes. Park your car and stand upwind when possible. Diesel oil does not wash out of clothes!</td>
</tr>
<tr>
<td>10</td>
<td>Look where you are walking. The protective covers may not be securely in place over the predrilled holes.</td>
</tr>
<tr>
<td>11</td>
<td>Welding must not be viewed with the naked eye. Shield eyes when in the vicinity of a welding operation and wear appropriate shaded eye protection when near this work.</td>
</tr>
</tbody>
</table>
CHAPTER 8

Static Pile Load Testing and Pile Dynamic Analysis

8-1 Introduction

Chapter 1, *Foundation Investigations*, of this Manual explained how Geotechnical Services performs a foundation investigation for all new structures, widenings, strengthening, or seismic retrofits. Under normal circumstances, the Geoprofessional assigned to perform the investigation is able to gather enough information to recommend a pile type and tip elevation that is capable of supporting the required loads on the recommended pile foundation. However, there are situations where subsurface strata are variable, unproven, or of such poor quality that additional information is needed in order to make solid pile foundation recommendations. In these situations, Static Pile Load Testing and/or Pile Dynamic Analysis (PDA) will be recommended. Information obtained from the testing and/or PDA will be used to verify design assumptions or modify foundation recommendations.

Personnel from Geotechnical Services, Foundation Testing Branch, perform Static Load Testing and PDA on Caltrans projects. Once the testing is completed, written reports summarizing the findings are transmitted to the Engineer. Ideally, these tests would be performed in the Design Phase; however, they are often done in the Construction Phase.

8-2 Reasons for Static Load Testing and Pile Dynamic Analysis

Static Load Tests measure the response of a pile under an applied load and are the most accurate method for determining pile capacities. They can determine the ultimate failure load of a foundation pile and determine its capacity to support the load without excessive or continuous displacement. The purpose of such tests is to verify that the load capacity in the constructed pile is greater than the nominal resistance (Compression, Tension, Lateral, etc.) used in the design. The best results occur when pile load tests are performed in conjunction with Pile Dynamic Analysis (PDA). The tests give the Geoprofessional information needed to allow the use of a more “rational” foundation design.

Static load tests may be recommended when piles are installed in soils with variable geologies or poor quality soils and can be used to validate design assumptions or to provide sufficient information to modify the design tip elevations. They are often recommended for Cast-In-Drilled-Hole (CIDH) piles installed in unproven ground.
formations as there is no other means to determine capacity. They provide more accurate information than can be obtained from pile driving formulas and may demonstrate that driven piles can be safely loaded beyond the capacities obtained from these formulas.

Pile load tests are expensive to perform but provide value to a structure. The FHWA Publication No. FHWA-SA-91-042, *Static Testing of Deep Foundations*, provides the following recommendations on when to perform a pile load test. They are as follows:

- When there is a potential for large cost savings. Typically on large projects with similar strata and pile types.
- When the safe loading condition is in doubt, due to limitations of an Engineer’s experience base, or unusual site or project conditions.
- When soil or rock conditions vary considerably from one portion of a project to another.
- When the design load is significantly higher than typical design loads.
- When time-related soil capacity changes are anticipated (i.e. soil setup & relaxation.)
- To determine the length of precast friction piles to avoid splices.
- When new or unproven pile types or installation methods are to be used.
- When existing piles will be used to support a new structure with heavier loads.
- To obtain a reliable value for tensile and lateral pile resistance.
- When, during construction, the load carrying capacity of the pile differs significantly from what was predicted from pile driving formulas and PDA.

In lieu of doing a static load test, PDA can be used to establish criteria for pile acceptance and to verify design assumptions. It can determine soil resistance, hammer efficiency/performance, and stresses in the pile during driving. Caltrans practice is to use PDA to establish pile acceptance criteria for non-standard driven piles up to 36 inch diameter. PDA in combination with static pile load testing is used to establish pile acceptance criteria for driven piles larger than 36-inch diameter. Refer to Bridge Construction Memo 130-4.0, *Pile Driving Acceptance Criteria*.

The information obtained from the PDA can also be used by other programs to determine the bearing capacity of the pile. Combining these results with those from the pile load test increases the accuracy when determining the bearing capacity.

### 8-2.1 Static Pile Load Tests

The static pile load test gives the most accurate indication of the capacity of the in-place pile. It is performed using a reaction method. The test procedure involves applying an axial load to the top of the test pile with one or more hydraulic jacks. The reaction force is transferred to the anchor piles that go into tension in the case of a static load test in compression; or into compression in the case of a static load test in tension. Various forms of instrumentation are installed onto the test and anchor piles so that an accurate measurement of the test pile displacement can be obtained. Redundant systems are used to ensure accuracy of the various measurements.
A five-pile test group (four anchor piles and one test pile) is used for all static load tests in compression and for most tension tests (Figure 8-1). Occasionally, a three-pile test group (two anchor piles and one test pile) is used for static load tests in tension (refer to Appendix F, *Pile Dynamic Analysis, Static Pile Load Testing and Field Acceptance Criteria*). Loads are applied in increments; typically equal to 5% of the design load. Each increment of load is held for a predetermined time interval. The test procedure is conducted in accordance with ASTM 1143, *Procedure A: Quick Test*. The pile capacity acceptance or failure criteria will be determined by the parameters established by the structure design and geotechnical engineer.

![Figure 8-1. Static Pile Load Test (Five-Pile Array).](image)

The Static Pile Load Test causes a failure along the soil/pile interface. This failure generally occurs well before the ultimate structural capacity of the pile is reached. Once the test is complete, the pile is returned to a no-load condition and can be incorporated into the foundation of a structure. The only permanent effect of a pile load test on a driven pile is the downward displacement of the test pile. The same effect would be achieved if a pile hammer drove the pile the additional distance. The previous statement, while true for driven piles, may not be the case for Cast-in-Place piles, and rock sockets in particular, as these piles will not behave the same once the bond between the concrete and the soil has been broken.

Once the pile load testing is completed, personnel from the Foundation Testing Branch compile and review the load test data. The test data is used to produce a plot of load versus pile displacement. The ultimate capacity of the test pile is determined using graphical or analytical procedures. A summary report is then forwarded to the Engineer, along with any recommended changes or modifications to the design.
Static Pile Load Testing exceeds the standards stated in ASTM D1143, Procedure A: *Quick Test*, for static load testing in compression, and ASTM D3689 for static load testing in tension. Both the compression and tension load tests each take approximately 4 to 8 hours to complete.

The Foundation Testing Branch has four static axial pile load test systems of varying maximum load capacity:

- 1,000,000-pound Load Test System
- 2,000,000-pound Load Test System
- 4,000,000-pound Load Test System
- 8,000,000-pound Load Test System

Requests for Static Load Tests are made to the Foundation Testing Branch on the Pile Load Test (PLT) Request Form. A copy of this form is included in Appendix F, *Pile Dynamic Analysis, Static Pile Load Testing and Field Acceptance Criteria*, and is available for download.

### 8-2.2 Pile Dynamic Analysis (PDA)

The dynamic analysis refers to the use of a device called the *Pile Driving Analyzer*. The *Pile Driving Analyzer* consists of a portable computer that collects and analyzes strains and accelerations measured by instrumentation attached to the pile being driven.

The *Pile Driving Analyzer* operator inputs parameters related to the physical characteristics of the pile before the pile analysis begins. Data to describe the surrounding soil and its damping resistance is also entered. The *Pile Driving Analyzer* is capable of analyzing the stress wave produced along the length of the pile by each blow of the hammer during the driving operation. By analyzing the shape of the wave trace, the *Pile Driving Analyzer* is able to measure pile stresses generated during driving. During installation, damage to a pile can often be detected by the *Pile Driving Analyzer*. The data retrieved during the analysis can be used to determine the location or depth of a crack in a concrete pile and to the point of buckling in a steel pile.

The *Pile Driving Analyzer* accurately measures the energy delivered to the pile during driving. This energy rating can be compared to the manufacturer’s rated value to provide an indication of the hammer’s actual performance efficiencies. Low or unusual delivery of energy to the pile may indicate issues such as a pre-ignition problem within the hammer, inefficient hammer combustion, misalignment of the follower or helmet, or the use of an inappropriate pile hammer cushion.

Pile Dynamic Analysis is believed to be reliable for piles driven in granular soils. For finer grained soils, such as silts and clays, this method may be less reliable because these
soils offer significantly larger damping resistance to the piles during driving and may be difficult to model accurately.

Information retrieved by the Pile Driving Analyzer is also used to predict a pile’s static load capacity. The dynamic analysis is performed on production piles as specified in the special provisions and on the test and/or anchor piles used for a Static Load Test if applicable. Piles monitored using the Pile Driving Analyzer are usually driven a predetermined distance above the specified tip before the analysis begins. At that time, the driving stops to allow personnel from the Foundation Testing Branch to attach the necessary instrumentation to the pile. The instrumentation is attached 1-1/2 to 2 pile diameters from the top of the pile. Once installed, the Contractor resumes driving the pile. The first few blows are done slowly to allow the Pile Driving Analyzer operator to ensure that the instrumentation is attached correctly and that the data is transmitted to the Pile Driving Analyzer computer. Driving then continues until the specified tip elevation is reached. In some soils, typically cohesive soils, the piles may increase in capacity or set-up over time. When this is anticipated, the tip of the pile is left approximately one foot above the specified tip elevation.

After the set-up period has elapsed, the pile is ready for a re-drive. The timeframe for set-up is usually overnight but can be longer. Before the re-drive, Pile Driving Analyzer instrumentation is once again attached to the pile, and the last foot of the pile marked in increments of one tenth of a foot. The pile is hit for a few blows to make sure that the instrumentation is working properly. The pile is then driven for several inches or the remainder of the one-foot length. The capacity of the pile is determined from PDA or through pile driving equations. The new bearing capacity is compared to the one prior to set-up to determine the increase in capacity over that period of time. The concept of pile capacities increasing during a set-up period is discussed fully in Chapter 7, Driven Piles.

Under normal circumstances, dynamic analysis is used in conjunction with static load testing to determine the adequacy of foundation piles. As with Static Load Testing, personnel from the Foundation Testing Branch are assigned the responsibility for performing PDA on Caltrans projects. Requests for PDA are submitted to the Foundation Testing Branch on the Pile Dynamic Analysis Test Request Form. A copy of this form is included in Appendix F, Pile Dynamic Analysis, Static Pile Load Testing and Field Acceptance Criteria, and is available for download.2

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8-3 Contract Administration of Static Pile Load Testing & Pile Dynamic Analysis

At the beginning of any project requiring Static Pile Load Testing and/or Pile Dynamic Analysis, the Engineer should do a thorough review of the contract plans, special provisions, Standard Specifications, and Bridge Construction Memo 130-2.0, *Pile Load Tests*, to make themselves aware of the contract requirements.

It is the Engineer’s responsibility to coordinate the Static Pile Load Testing and Pile Dynamic Analysis with the Foundation Testing Branch. Early contact and good communication with them is important, as it will ensure that the process flows smoothly. The Contractor’s schedule for the installation of the piles should be obtained as early as possible. This schedule should then be forwarded to the Foundation Testing Branch. Details relating to the logistical needs of the testing work crew should also be discussed with the Foundation Testing Branch and the necessary information relayed to the Contractor.

The contract specifications state the Contractor needs to perform extra work to assist in the set-up and performance of the Static Pile Load Testing. As such, a change order will need to be written to compensate for these expenses. This is not the case with dynamic analysis as it is paid under the contract item for piling or as indicated in the special provisions. The Contractor should be notified as early as possible of the specific equipment and personnel assistance required by the Foundation Testing Branch in order to complete the Static Pile Load Testing or PDA operations.

In general, for a Static Pile Load Test, the Contractor will need to provide a crane and operator for the lifting and placement of the testing equipment from the Caltrans transport trailers on to the pile array, and for returning the equipment to the trailer once the testing is complete. The crane will need to be capable of lifting and placing the appropriate load test beam atop the pile test groups. Occasionally, a 54,000-pound or larger beam is used for load testing. The actual beam size to be used should be confirmed with the Foundation Testing Branch. The Foundation Testing Branch will supply all necessary rigging. The Contractor will need to provide a welder, welding machine and cutting torches to assist in the installation of the testing equipment. Specific logistical needs and project-specific issues should be discussed with personnel from the Foundation Testing Branch to ensure that efficient coordination of the test set-up is accomplished.

The contract specifications state no piles represented by a load test pile may be drilled, cast, cut to length, driven, nor any reinforcement cages fabricated until the required Static Load Testing is completed. In addition, the Engineer needs to ensure that the area of the Static Load Testing and/or PDA is dry and free of debris. A safe working area should be

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3 2010 SS, Section 49-1.01D(3), *Load Test Piles*, or 2006 SS, Section 49-1.04, *Load Test Piles*. 

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established around the test piles, and any of the Contractor’s operations that conflict with the work of the testing work crews should be suspended until the testing is complete.

Static Pile Load Testing on concrete piles cannot begin until the concrete reaches a compressive strength of 2,000 Pounds per Square Inch (PSI), except for precast concrete piles, which cannot be driven until 14 days after casting. Additional cement or Type III (high early) cement may be used at the Contractor’s expense, but only for load test piles not incorporated into the work.

The contract specifications state the Engineer will not require more than 15 days to perform each static load test unless otherwise provided in the special provisions. This is important since Caltrans will be responsible for any additional costs or delays to the schedule should the testing take longer or should it not start on the day requested. As such, early and effective communication with the Foundation Testing Branch is essential.

8-4 Inspection Requirements During Static Load Testing and PDA

As with production piles, it is very important that the Engineer ensure that all piles to be used for Static Pile Load Testing and PDA are driven or constructed in accordance with the contract plans and contract specifications. The Engineer should discuss and confirm the load test pile array set-up with the Foundation Testing Branch well in advance of the work, even if the contract plans do adequately describe the test pile set-up.

Test piles must be installed plumb and to the specified tip elevation shown on the contract plans. All the piles (anchor and test piles) in each test group need to be logged for the full length of driving. For drilled piles, a soil classification record should be kept for the full length of each. If any of the driven piles have a low bearing value at the specified tip elevation (less than 50% of required), then the Engineer should contact the Foundation Testing Branch, the Designer and Geoprofessional to see if a revision to the specified tip elevation is appropriate. Changes to the specified tip elevation of test and/or anchor piles will necessitate a contract change order.

Additional work on the anchor and test piles is required to facilitate the test apparatus. These details are included in the Standard Plans and may also be shown on the contract plans. If the details are inappropriate for the piles or are unclear, contact the Designer and/or the Foundation Testing Branch. The reactions in the load test are substantial and proper bearing is essential. Therefore the top of CIDH test piles must be level and troweled smooth to ensure full contact/bearing of the load test reaction beam.

The contract plans or special provisions may require the anchor piles be constructed to tip elevations lower than the test pile, as an added precaution, to ensure that the piles don’t pull out during the test. This issue should be discussed with the Foundation Testing
Branch. Any changes to the lengths of the piles from those shown on the contract plans will warrant a contract change order.

If a construction project includes Pile Dynamic Analysis, the contract specifications\[4\] state when the piles to be analyzed are to be made available for State personnel so that the necessary preparations for these piles can be made before they are driven. A technician from the Foundation Testing Branch will need access to the piles to prepare them for the attachment of the necessary instrumentation. The Engineer needs to ensure that the Contractor provides assistance to the technician as necessary to maneuver the piles.

Once the load testing crew arrives on the jobsite, the Engineer will need to have copies of the pile driving logs, soil classification record (for CIDH piles), *Log of Test Borings*, and *Foundation Plan* available for their use. When the Static Pile Load Testing and/or Pile Dynamic Analysis is completed, the Foundation Testing Branch will provide a report that states whether the testing confirmed design assumptions or whether changes to the production piles will be necessary. These changes are normally made without the need for additional load tests. If an additional test is required, the Engineer should be sure to document any delays to the Contractor’s operations. If additional testing is required, Caltrans will be responsible for additional costs incurred by the Contractor. Substantial pile revisions (as a result of poor test results) could have a substantial impact on administrative aspects of the contract. Changes could be such that item prices for pile work are no longer valid, and an item price adjustment may be necessary.

Again, it is very important that the Engineer set up a good line of communication with the Foundation Testing Branch in the early stages of the project. The goal should always be to have a clear understanding of what coordination needs to be done in order to properly install the test piles and set up the load testing equipment without significant delays to the project. Good coordination is also important as it allows the static load testing work crews to perform the tests efficiently and on schedule.

\[4\] 2010 SS, Section 49-1.01D(4), *Dynamic Monitoring*, or 2006 Special Provisions.
9 Slurry Displacement Piles

9.1 Introduction

A slurry displacement pile is a Cast-In-Drilled-Hole (CIDH) pile where a drilling fluid is introduced into the excavation concurrently with the drilling operation. The drilling fluid, also referred to as slurry or drilling slurry, is used to prevent caving of unstable ground formations and intrusion of groundwater into the drilled hole. The drilling slurry remains in the drilled hole until it is displaced by concrete, which is placed under the drilling slurry through a rigid delivery tube.

Because the slurry displacement method, also referred to as the wet method, is a specific construction method for the construction of CIDH piles, the reader is advised to review Chapter 6, Cast-In-Drilled-Hole Piles, as it contains information about inspection duties and responsibilities of the Engineer for construction of all CIDH piles. This chapter contains modifications to inspection duties and responsibilities of the Engineer necessary for the construction of CIDH piles using the slurry displacement method.

9.1.1 History

The use of drilling slurries is commonly associated with methods used by the oil well drilling industry for more than 100 years, which provided much of the technical and practical knowledge concerning their use in drilled foundation applications. Use of the slurry displacement method for constructing drilled shafts began in Texas in the years following World War II. This early method involved the use of soil-based drilling slurries to advance drilled holes deeper than they could have without, after which a casing was used to stabilize the drilled hole for shaft construction. In the 1960’s, processed clay mineral slurry was introduced as a means of eliminating the need for casing to stabilize the drilled hole. However, the properties of the mineral drilling slurries were not controlled. Initial information on the properties of mineral drilling slurries was obtained from the Reese and Touma Research Report, which was a cooperative research program conducted in 1972 by the University of Texas at Austin and the Texas Highway Department. Due to the numerous failures that occurred, in the mid-1970’s more attention was paid to the physical properties of mineral drilling slurries and appropriate methods of preparing and recirculating drilling slurries.

There are still many unknowns about the use of drilling slurries, among them the effect of the drilling slurry on the ability of a pile shaft to develop skin friction. Research on the use of mineral slurry has indicated potentially adverse effects of mineral slurry on drilled
shaft side resistance; however, this impact can be avoided by following proper construction procedures. On the other hand, research has also shown that development of side resistance does not appear to be a problem with the use of synthetic slurry, with the possible exception of hard rock where the sides of the drilled hole are smooth. The design method used by Caltrans for determining the pile capacity adequately accounts for possible loss of pile capacity when drilling slurry is used. Research funded in part by the Federal Highway Administration (FHWA) is ongoing at universities around the United States. Caltrans has also conducted research on several contracts in recent years, which lead to the development and continual improvement of contract specifications for use of the slurry displacement method of CIDH pile construction.

Processed clay mineral slurries are considered environmentally hazardous and are difficult to dispose of. In the 1980’s, the drilled shaft industry began a trend towards the use of synthetic drilling slurries. These drilling slurries are less hazardous to the environment and are easier to dispose of.

Caltrans first used the slurry displacement method on a construction contract in 1984 and has increasingly used this method since then. A change in Caltrans seismic design philosophy has resulted in the use of more and larger CIDH piles. Because of this, ground conditions have become less of a factor in the pile type-selection process. Other factors such as lower construction costs and construction in urban environments with restricted access and noise limitations have also led towards the expanded use of CIDH piles. Because of these factors, Caltrans started inserting the slurry-displacement method specifications into all contracts with CIDH piles in 1994.

9-2 Slurry Displacement Method

The slurry displacement method of construction is similar to that of ordinary CIDH pile construction until groundwater or caving materials are encountered. When groundwater or caving materials are encountered during the drilling operation, the Contractor must have a plan in place to decide whether to use a casing to stabilize the drilled hole, dewater the drilled hole, or drill the hole and place concrete under wet conditions using the slurry displacement method. In most cases, the site conditions are known to be wet or unstable. These conditions should have been shown on the Log of Test Borings (LOTB) or in the Foundation Report. Sometimes experience on adjacent projects may also give an indication of the site conditions.
Drilling slurries are generally introduced into the drilled hole as soon as groundwater or caving materials are encountered. As drilling continues to full depth, the drilling slurry is maintained at a constant level at least 10 feet above the piezometric head until the tip elevation of the drilled hole is reached (Figure 9-1(a)). Because the drilling operation mixes soil cuttings with the drilling slurry, it is necessary to remove the soil cuttings from the drilling slurry. Depending on the type of drilling slurry used, removing the soil cuttings may be accomplished by physically cleaning the drilling slurry, or by allowing a settlement period for the soil cuttings to settle out of the drilling slurry (Figure 9-1(c)). If the drilling slurry is cleaned such that its physical properties are within the specified limits for the particular type of drilling slurry, the bottom of the drilled hole is cleaned of any settled materials using a cleanout bucket (Figure 9-1(d)). Since the action of the cleanout bucket may cause soil cuttings to contaminate the drilling slurry, cleaning the bottom of the drilled hole and the drilling slurry may take several iterations. Additional cleanings of settled materials from the bottom of the drilled hole may be performed with a cleanout bucket, pumps, or an airlift. After the final cleaning has been accomplished, the drilling slurry is retested to make sure its properties are within the specified limits. Once the drilling slurry is ready, the pile bar reinforcement cage may be placed. The slurry is again retested immediately prior to concrete placement and the bottom of the hole is re-cleaned using a pump or airlift. Once the slurry’s physical properties are within the specified limits and the bottom of the hole is clean, concrete is placed, either by a rigid tremie tube or by a rigid pump tube delivery system. Concrete is placed through the tube(s), starting at the bottom of the drilled hole (Figure 9-1(e)).
delivery tube is maintained at least 10 feet below the rising head of concrete. As concrete is placed, the displaced drilling slurry is pumped away from the hole and prepared for reuse or disposal. Concrete placement continues until the head of concrete rises to the top of the pile and is then wasted until all traces of settled material or drilling slurry contamination in the concrete are no longer evident. Under circumstances where contaminated concrete cannot be wasted from the top of the pile, such as having a pile construction joint within a permanent casing below grade, pile concrete is placed to a predetermined level above the planned concrete placement elevation, and the contaminated concrete above the planned concrete placement elevation is either mucked out immediately after placement or chipped out at a later time.

9-3 Principles of Slurry Usage

All slurries keep excavations open by the use of positive hydrostatic pressure. In order to exert hydrostatic pressure against the walls of an excavation, a pressure transfer medium must be present. With mineral slurries (e.g. bentonite mud) the deposited filter cake of clay solids on permeable formations is the pressure transfer mechanism (the thing against which the hydrostatic pressure can push). In the case of properly formulated synthetic slurries, the pressure transfer mechanism is the zone of viscous permeation that surrounds the excavation. This zone is preferably permeated (and plugged) by viscous polymer slurry. The depth of the zone around the excavation can be inches or feet.

Positive hydrostatic pressure refers to the excess pressure exerted by a column of fluid against the interstitial or pore pressure of a soil layer (Figure 9-2(a)). A column of water 33 feet tall exerts a hydrostatic pressure of 1.0 atmosphere or 14.7 pounds per square inch. It has been determined by experience that a positive hydrostatic pressure of about 6 to 7 feet of water head is normally sufficient to keep an excavation open. This is equivalent to 0.2 atmospheres or about 3 pounds per square inch. A more useful way to consider 3 pounds per square inch is that it equals 432 pounds per square foot of excavation wall area. This is apparently sufficient to keep most holes open when proper operating practices are in use.

“Positive hydrostatic pressure” also refers to hydrostatic pressure above and beyond that exerted inward on an excavation by ground water (Figure 9-2). Thus if the static ground water table is at 15 feet below ground level, and if we want to maintain a column of slurry 7 feet higher than that, we will need to keep the slurry level at 8 feet below ground level. If excessive fluid loss is not a concern, we may want to keep the hole full of fluid, but this is probably not necessary in most cases. Excessive hydrostatic pressure can accelerate non-useful, unwanted loss or permeation of slurry into granular permeable soil layers.
As mentioned previously, the filter caking process created by mineral or solid-laden slurries is called filtration. When drilling slurry is applying positive hydrostatic pressure to the sides of the drilled hole, some of the drilling slurry and soil cuttings may be forced out of the excavation and into the ground formation. When this material enters the formation, particles of the drilling slurry may be trapped or “filtered” by the individual soil grains of the formation. This process results in the development of filter cakes on the sides of the drilled hole. These filter cakes are referred to as “mudcakes” and help to temporarily stabilize the sides of the drilled hole.

The filtration process is dependent upon many variables. These include the nature of the ground formation, the type of mineral drilling slurry used, the amount of time the drilling slurry is in the drilled hole, the presence of contaminants or groundwater in the ground formation, and the chemical additives used in the drilling slurry, just to name a few. The

Figure 9-2. Positive Hydrostatic Pressure.
nature of the ground formation and the amount of time the drilling slurry is in the drilled hole are the two important variables.

The nature of the ground formation has an effect on the thickness of the filter cake that mineral slurries or other solids-laden slurries develop on the sides of the drilled hole. In general, thicker cakes will form on permeable granular ground formations, such as sands. Since the pore spaces between the individual soil grains are larger, drilling slurry with entrained soil particles can infiltrate further into the ground formation driven by the same positive hydrostatic pressure (Figure 9-3). Eventually, the infiltration slows as drilling slurry and particles build up against and beyond the exposed faces of the permeable formations. In tighter ground formations, such as dense sands and cohesive soils, the pore spaces between the individual soil grains are much smaller. The drilling slurry particles tend to fill in the pore spaces at the exposed wall face preventing further infiltration (Figure 9-4). Drilling slurry cannot be forced into the ground formation by positive hydrostatic pressure. This causes the build-up of the filter cake to cease; resulting in a thinner filter cake than would be observed in looser ground formations.

![Figure 9-3. Filtration–Loose Ground Formation.](image)

The amount of time that the drilling slurry is in the drilled hole also has a direct effect on the thickness of the filter cake that develops on the sides of the drilled hole. As long as positive hydrostatic pressure is continuous, the build-up of filter cake will continue so long as the infiltration continues. In general, the longer the drilling slurry is present in the drilled hole, the more filter cake will accumulate on the sides of the drilled hole. Sometimes this results in the presence of excess filter cake buildup, which must be removed before concrete can be placed in the drilled hole.
The important thing to remember about filtration is that it mainly pertains to mineral slurries or other solid-laden slurries and its filter cake helps to temporarily stabilize the sides of the drilled hole before concrete is placed. Filter cake is not meant to be left in place during concrete placement operations. If the filter cake is thin enough, the rising column of concrete will scrape it off the sides of the drilled hole. However, if the filter cake has excessive thickness, the rising column of concrete may not scrape all of it off the sides of the drilled hole. The remaining filter cake may act as a slip plane between the pile concrete and the sides of the drilled hole, resulting in the reduced skin friction capability of the pile. Excess filter cake must be removed prior to concrete placement.

With synthetic slurries, these fluids permeate and exert hydrostatic pressure against the walls of an excavation in order to keep the excavation open during drilling or digging and concrete placement. Synthetic slurries consist of very long, chain-like hydrocarbon molecules (polymers) that do not deposit a conventional wall cake or filter cake as with mineral slurries because the fluids are not laden with fine plate-shaped particles, such as bentonite.

Instead, a properly prepared synthetic polymer slurry permeates granular soils to a relatively shallow penetration around an excavation with long, hair-shaped strands of slurry molecules (Figure 9-5). This permeation has a gluing effect and stabilizes an excavation due to drag forces and cohesion formed from the binding of the soil particles in the formation by the polymer strands that tend to keep the soil particles in place.
The phrase “properly prepared” refers to slurry that is well-dispersed, lump-free, and viscous enough to impede filtration into granular formations. In some cases partially-hydrated, dry synthetic polymer (viscous slurry full of “pearls” of incompletely dissolved dry synthetic polymer product) may be useful in plugging coarse granular soils and appears to be more effective than emulsion synthetic polymers at controlling unwanted excessive fluid loss. These long chain polymers also inhibit hydration, swelling and distortion of clay components or layers in the soil formation.

9-4 Sampling and Testing Drilling Slurry

Sampling and testing of drilling slurry is an important quality control requirement. Responsibility for testing and maintaining drilling slurry of high quality is placed on the Contractor by the contract specifications. The Engineer is responsible for performing quality assurance testing on the drilling slurry.

The apparatus used to sample drilling slurry must be capable of sampling the drilling slurry at a given elevation in the drilled hole without being contaminated by drilling slurry from a different elevation in the drilled hole. This is necessary because the contract
specifications require the drilling slurry to be sampled at different levels in the drilled hole. The sampler must also be large enough to contain enough drilling slurry to perform all the required tests. The apparatus generally consists of a hollow tube with caps positioned above and below the tube on a cable that is used to lower the sampler into the drilled hole (Figure 9-6). Once the sampler has been lowered to the desired level, the drilling slurry contained in the hollow tube (at that level) is contained by activating the caps so that the ends of the tube are sealed. The sampler is then removed from the drilled hole and the drilling slurry contained is tested.

Figure 9-6. Slurry Sampler Schematic.
One of the responsibilities of the Contractor is to verify that the sampler used seals properly. The Engineer may require the Contractor to verify this before allowing the construction of slurry displacement piles to commence.

The primary engineering reason for testing drilling slurries is to make sure that no suspended material in the drilling slurry settles out during concrete placement. A secondary reason for testing drilling slurries is to control their properties during the drilling of the hole. This helps to stabilize the drilled hole. Drilling slurries that have physical properties within the parameters described in the contract specifications should have negligible settlement of suspended materials during concrete placement provided the pile’s bar reinforcement cage and concrete are placed promptly.

The contract specifications set parameters for some of the physical properties of drilling slurries. The four specified physical properties are density, sand content, pH, and viscosity.

9-4.1 Density
Density, or unit weight, is a function of the amount of solids held in suspension by the drilling slurry. Since mineral slurries will hold solids in suspension for long periods, the allowable density value is higher than that permitted for synthetic slurries and water, which do not hold solids in suspension as well. Its viscosity may affect the density of the drilling slurry since a more viscous fluid will suspend more solids. The reason for having an upper limit on the allowable density value is that drilling slurries with higher densities are unstable with respect to their ability to suspend solids. These solids could settle out during concrete placement and cause pile defects.

Figure 9-7. Density Test Kit.
Density is tested using the test kit shown in Figure 9-7 in conformance with the test method described in Section 4 of the American Petroleum Institute (API) Recommended Practice for Field Testing Water-based Drilling Fluids (ANSI/API Recommended Practice 13B-1). Access to this test method is available through Structure Construction Intranet under Field Resources/ASTM, AWS Spec downloads and searching for Document Number API RP 13B-1.

9-4.2 Sand Content
Sand content is an important parameter to keep under control, particularly just prior to concrete placement. Sand is defined as any material that will not pass through a No. 200 sieve. Since mineral slurries will hold sand particles and other solids in suspension, the allowable sand content value is higher than that permitted for synthetic slurries and water, which do not hold these solids in suspension as well. The primary reason for setting an upper limit on the sand content value is to prevent significant amounts of sand from falling out of suspension during concrete placement. A secondary reason for setting an upper limit on the sand content value is that high sand content can increase the amount of filter cake on the sides of the drilled hole in mineral slurries. This increased filter cake might have to be physically removed before concrete could be placed in the drilled hole. Allowing the filter cake to remain would decrease the skin friction value of the pile, thereby reducing the pile capacity.

Figure 9-8. Sand Content Test Kit.

Sand content is tested using the test kit shown in Figure 9-8 in conformance with the test method described in Section 9 of the ANSI/API Recommended Practice 13B-1. Access to this test method is available through Structure Construction Intranet website under Field Resources/ASTM, AWS Spec downloads and searching for Document Number API RP 13B-1.

9-4.3 pH Value
The pH value of drilling slurry is important to ensure as its value indicates whether or not the drilling slurry is functioning properly. Mineral slurries that have pH values outside the allowable range will not fully hydrate the clay mineral and will not develop the expected viscosity. Synthetic slurries that are mixed in water having pH values outside the allowable range may not become viscous at all. Even though drilling slurries may be mixed in a controlled environment (such as in a mixing tank), they will be affected by acids and organic material from the groundwater or the soil once it is introduced into the hole. Mineral slurries may flocculate and form a thick, soft filter cake if the slurry becomes too acidic or too alkaline. Synthetic slurries may lose their viscosity and their ability to stabilize the sides of the drilled hole if the slurry becomes too acidic or too alkaline.

The pH value of drilling slurry is tested using either a pH meter or pH paper.

9-4.4 Viscosity
Viscosity refers to the “thickness” of the drilling slurry. This property is measured to determine whether the drilling slurry is too “thick”, allowing the suspension of more solids than permitted, which would affect the density and sand content values. On the other hand, some soils may require drilling slurry with a higher viscosity during drilling to permit the formation of filter cake or to stabilize the sides of the drilled hole in loose ground formations such as gravels. Thinner drilling slurry tends to flow through a loose ground formation without building a filter cake or providing stability. After the hole is drilled and a filter cake has formed or the sides of the drilled hole have stabilized, the drilling slurry can be thinned as required prior to concrete placement.

Figure 9-9. Marsh Funnel Viscosity Test Kit.
The viscosity of drilling slurry is tested using the test kit shown in Figure 9-9 in conformance with the test method described in Section 6 of the ANSI/API Recommended Practice 13B-1. Access to this test method is available through Structure Construction Intranet under Field Resources/ASTM, AWS Spec downloads and searching for Document Number API RP 13B-1.

9-5 Types of Slurry

It is important to note that the type of drilling slurry to be used will depend on the ground conditions encountered. Use of different types of drilling slurries may be necessary to drill through different types of ground formations. It is conceivable that different types of drilling slurries may need to be used on the same contract because of varying ground conditions within the highway right-of-way. Some of the factors that influence the decision of what type of drilling slurry to use include economics, ground and groundwater contamination, ground temperature, air temperature, and the type of ground formation being drilled through.

Ground conditions can also have an effect on drilling slurry behavior. Some of these include acidity or alkalinity of groundwater, grain size of the soil, velocity of groundwater flow through the ground formation, cementation and cohesion of soil, and the presence of rock or clay structures in the ground formation. The drilling slurry’s physical properties can be adjusted to account for some of these conditions, or chemical additives may be necessary.

Because most drilling slurries are difficult and expensive to dispose of, they are often reused. Occasionally, drilling slurry is reused on another pile after completion of the previous pile. Sometimes, the drilling slurry is reused on or from another project.

The reuse of drilling slurries requires careful planning on the Contractor’s part. Drilling slurries must be cleaned before they are reused. For mineral slurries, this is accomplished through the use of de-sanding units and chemical additives. For synthetic slurries, this is accomplished by allowing the contaminants to settle out.

The contract specifications do not prohibit the reuse of drilling slurry. However, it still must meet the physical property requirements of the contract specifications. Drilling slurries will degrade over time (usually measured in months). If a Contractor proposes to reuse drilling slurry from a different contract, the Engineer may want to have the physical properties of the drilling slurry tested prior to placement in the drilled hole.

The types of drilling slurries that are permitted for use by Caltrans are detailed in the following sections. Three types of drilling slurries are permitted: water, mineral, and synthetic polymer.

2 http://onramp.dot.ca.gov/hq/oscnet
9-5.1 Water

Water may be suitable as drilling slurry under the right conditions. Most drilling contractors will try to use water as drilling slurry if the ground conditions are right because it is inexpensive. However, use of water as drilling slurry is limited to ground formations that are strong enough not to deform significantly during drilling. The water level in the drilled hole must be maintained at least 6 to 7 feet above the groundwater level in order to maintain positive effective stress on the sides of the drilled hole. This is the only means of stabilization provided to the sides of the drilled hole since water does not control filtration.

The 2010 Standard Specifications state that water may only be used as drilling slurry when a casing is used for the entire length of the drilled hole. Although water has been allowed as drilling slurry in the past by the contract specifications, history has shown that water was inappropriately chosen as drilling slurry for use in holes drilled in unstable ground formations, or even when a fully cased hole is terminated in an unstable ground formation. This resulted in many defective piles that required repair. Because of continuing problems encountered using water as a drilling slurry, water may only be used when a casing is used for the entire length of the drilled hole and the Geoprofessional authorizes its use during the project design phase. This change is stated in the 2015 Standard Specifications, which allow the use of water as a drilling slurry only when specified in the Special Provisions.

The question that arises from these limitations is why the contract specifications allow the use of water as drilling slurry at all. Retaining the limited use of water as a drilling slurry allows a Contractor, who attempts to dewater a drilled hole using a temporary casing and is unable to do so for whatever reason, to have the option of using the water in the drilled hole as a drilling slurry to prevent unstable conditions at the bottom of the drilled hole and to be able to place concrete. Water may also be used as drilling slurry when a Rotator or Oscillator is used to advance the drilled hole, provided the hole terminates in a stable formation.

The physical properties of water used as drilling slurry are not as critical as with other types of drilling slurries. Water is capable of suspending sand and silt only for short periods, usually less than 30 minutes. This allows soil cuttings to settle to the bottom of the drilled hole fairly rapidly. Since the pH of water used as drilling slurry is not important and water will not become more viscous unless a contaminant is introduced, the contract specifications set parameters for density and sand content only. Testing these parameters verifies that most of the suspended material has settled before final cleaning of the drilled hole and concrete placement.

Water used as drilling slurry can be easily disposed of onsite after settlement of all suspended materials has occurred unless hazardous materials have contaminated the water.

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3 2010 SS, Section 49-3.02B(6)(d), Water Slurry, or Special Provisions for contracts using 2006 SS.
9-5.2 Mineral

Mineral slurries are processed from several different types of clay formations. Although there are a number of different types of clay formations available, the most commonly used consist of bentonite and attapulgite clay formations.

Bentonite (Figure 9-10) is manufactured from a rock composed of clay minerals, named after Fort Benton, Wyoming, where this particular type of rock was first found. Its principal active constituent is the clay mineral montmorillonite, which hydrates in water and provides suspension of sands and other solids.

Bentonite slurry is a mixture of powdered bentonite and water. Bentonite slurry will flocculate (destabilize) in the presence of acids and ionized salts and is not recommended for ground formations where salty water is present without the use of chemical additives.

Attapulgite comes from a clay mineral that is native to Georgia. It is processed from the clay mineral palygorskite, and is similar in structure to bentonite. However, it does not hydrate in water, will not flocculate in the presence of acids and ionized salts, and can be used in ground formations where salty water is present. Slurries made from attapulgite do not control filtration well, and tend to deposit thick filter cakes on the faces of permeable soils. Due to the transportation expenses and rare usage of this type of slurry in California, its application in Caltrans projects is unlikely.

Mineral slurries stabilize the sides of the drilled hole by positive hydrostatic pressure and by filtration. Mineral slurries will penetrate deeper into more open formations, such as gravels, and will form thicker filter cakes in these formations. While filtration is desirable, a thick filter cake is not desirable because it is necessary to remove it before concrete placement. Continuous agitation or recirculation of the mineral slurry with...
removal of sand and other soil solids will help reduce the thickness of the filter cake by reducing the amount of suspended material in the mineral slurry. The contract specifications\(^4\) require the removal of “caked slurry” from the sides and bottom of the drilled hole before concrete is placed. “Caked slurry” is considered to be an excessively thick filter cake that has formed on the sides or bottom of the drilled hole. The amount of filter cake that forms on the sides and bottom of the drilled hole depends on so many variables. Research of the effect of filter cake on the ability of the pile to transfer load through skin friction has not been completed. Structure Construction defines excessively thick filter cake as a filter cake that has formed in a drilled hole where mineral slurry has been continuously agitated or recirculated in excess of 24 hours or a filter cake that has formed in a drilled hole where mineral slurry has been un-agitated in excess of 4 hours. Due to the fact that each site is different, some engineering judgment should be exercised before implementing this definition. There are other indicators that can be used to assist the Engineer in making a judgment on the amount of filter cake present on the sides and bottom of the drilled hole. One indicator is the level of mineral slurry in the drilled hole. If the mineral slurry level is difficult to maintain at the required level in the drilled hole, this is an indicator that the mineral slurry is continuously being driven into the ground formation through the sides of the drilled hole. This means that filter cake build-up is continuing and it is likely that the thickness of the filter cake is excessive. However, if the mineral slurry level is stable in the drilled hole, this is an indicator that the mineral slurry has clogged up the ground formation on the sides of the drilled hole. This means that the filter cake buildup would have ceased and it is likely that the thickness of the filter cake is not excessive. Removal of excessively thick filter cake is accomplished by slightly over boring the full length of the drilled hole.

The contract specifications\(^5\) require that mineral slurries be mixed and fully hydrated in mixing tanks prior to placement in the drilled hole. Mixing and hydration of mineral slurries usually requires several hours. One way to determine that the mineral slurry is thoroughly hydrated is to take Marsh funnel viscosity tests at different time intervals. In general, mineral slurries will achieve their highest viscosity value when they have fully hydrated. Once the viscosity test values have stabilized at their highest level, the mineral slurry can be assumed to be fully mixed and fully hydrated, providing that the mineral slurry is smooth, homogeneous and not flocculated or “clabbered”.

The physical properties of the mineral slurry should be carefully monitored while the mineral slurry is in the drilled hole. The mineral slurry’s density, sand content, and viscosity should be tested and the values maintained within the limits stated in the contract specifications\(^5\). This will prevent excessive suspended materials and keep the filter cake thickness on the sides of the drilled hole to a minimum. The mineral slurry’s pH should be tested and maintained within the limits stated in the contract specifications to prevent flocculation or destabilization. It should be noted that it usually takes the

\(^4\) 2010 SS, Section 49-3.02B(6)(b), Mineral Slurry, or Special Provisions for contracts using 2006 SS.
\(^5\) 2010 SS, Section 49-3.02B(6)(b), Mineral Slurry, or Special Provisions for contracts using 2006 SS.
Contractor some time to get the mineral slurry’s properties within the limits stated in the contract specifications. The important factor is to verify that the mineral slurry’s properties are within the limits stated in the contract specifications prior to concrete placement.

While mineral slurries are present in the drilled hole, they must be agitated in order to maintain their physical properties and to reduce the amount of filter cake buildup on the sides of the drilled hole. In order to accomplish this, the contract specifications require mineral slurries to be agitated by either of two methods: (1) the mineral slurry is to be continuously agitated within the drilled hole, or (2) the mineral slurry is to be recirculated and cleaned. Either of these methods will provide the necessary continuous agitation of the mineral slurry. The method that is chosen will depend on the cleanliness of the mineral slurry in the drilled hole. This is typically influenced by the ground conditions encountered.

Recirculation and cleaning of mineral slurries is accomplished by removing the mineral slurry from the drilled hole, running it through specialized cleaning equipment, and then placing the cleaned mineral slurry back in the drilled hole. To meet all of the specification requirements, a slurry “plant”, which is approximately the size of a railroad boxcar, must be located adjacent to the work area (Figure 9-11). The slurry plant contains screens, shakers, de-sanding centrifuges (Figure 9-12), and agitators, and is capable of mixing, storing, and cleaning the mineral slurry. Figure 9-13 shows a typical recirculation and cleaning process. It is very important to remove the mineral slurry from the bottom of the drilled hole. This is because excessive amounts of suspended materials will eventually settle to the bottom of it. These materials must be removed in order to fully clean the mineral slurry. Typically, it will take several hours to completely clean the mineral slurry of sand and other suspended materials.

Figure 9-11. Mineral Slurry Plant.

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6 2010 SS, Section 49-3.02B(6)(b), Mineral Slurry, or Special Provisions for contracts using 2006 SS.
Figure 9-12. De-sanding Centrifuges.

Usually, in order for the mineral slurry to meet the physical property requirements of the contract specifications, the mineral slurry will require recirculation and cleaning during and after the drilling operation. Occasionally, without any action on the part of the Contractor, the mineral slurry will meet the physical property requirements of the contract specifications during and after the drilling operation, in which case continuous agitation of the mineral slurry in the drilled hole is acceptable. However, the contract specifications also require that any mineral slurry that is continuously agitated in the drilled hole and exceeds the physical property requirements must be recirculated and cleaned.

7 2010 SS, Section 49-3.02B(6)(b), Mineral Slurry, or Special Provisions for contracts using 2006 SS.
Figure 9-13. Recirculation and Cleaning Schematic.

Should the mineral slurry’s properties change dramatically during the drilling operation, chemical additives are available that can reduce the filter cake thickness, modify the mineral slurry’s pH, and increase the mineral slurry’s viscosity. Additives that reduce the filter cake thickness and increase the mineral slurry’s viscosity include organic colloids such as CMC or starch. Additives that lower the mineral slurry’s pH include pyrophosphate acid (SAPP). Additives such as soda ash and caustic soda (sodium hydroxide) can increase the slurry’s pH and reduce water hardness. Additives that decrease the mineral slurry’s viscosity, reduce gelatin, and improve filter cake quality include tannins, polyphosphates, lignosulfonates, and acrylates. Caltrans has little experience with chemical additives, and their use should be discussed with Structure Construction in Sacramento before authorization is given for their use.

Mineral slurries may be used in most types of ground formations. They work best in cohesionless sands and open gravels. Caution must be taken when using mineral slurries in cohesive materials because they may contain clays that can be incorporated into the mineral slurry and rapidly change the mineral slurry’s physical properties. In addition, these cohesive materials can reduce filtration, and filter cakes may not form.

Disposal of mineral slurries can be difficult. Due to their particulate nature, they are hazardous to aquatic life and cannot be disposed of on site or at locations where they can...
enter State waters. The contract specifications require that any materials resulting from
the placement of piles under mineral slurry be disposed of outside the highway right-of-
way. Because they often contain chemical additives, mineral slurries can be considered to
be hazardous materials that must be disposed of in landfills. This can be very expensive
and can defeat the economic advantage of using the slurry displacement method over
other means of construction of CIDH piles.

9-5.3 Synthetic

Since the 1980’s, synthetic drilling slurries have gained wide acceptance in the
construction industry. The main advantage of synthetic slurries is that they are easier and
cheaper to dispose of than mineral slurries and do not require slurry plants to physically
clean the slurry. Synthetic slurries are grouped into three groups: (1) naturally occurring
polymers, (2) semi-synthetic polymers, and (3) synthetic polymers. Synthetic polymers
are either dry or emulsified.

Synthetic drilling slurry systems must undergo a rigorous approval process before being
allowed for use on Caltrans projects. The synthetic drilling slurry products that are
approved by Caltrans at the present time are synthetic polymers mixed with water to
prepare viscous slurries for CIDH piles and other foundation elements. These slurries
have been shown to have no deleterious effects on concrete-to-rebar bonding, concrete
compressive strength and other aspects of the foundation construction processes. The
contract specifications currently allow the use of four brands of synthetic slurries. These
are: Super Mud, manufactured by PDSCo, Inc.; SlurryPro CDPTM, manufactured by KB
International LLC; Shore Pac®, manufactured by CETCO Construction Drilling
Products; and Terragel (previously named NovagelTM), manufactured by Geo-Tech
Services, LLC. Because drilling slurries are products used to facilitate construction of
CIDH piles and are not incorporated into the permanent work, these products are not
listed on the Caltrans Approved Products List.

Super Mud is an emulsified (water-in-oil, liquid form) synthetic polymer product. A
liquid form of Super Mud is currently approved for use on Caltrans projects. No other
form is approved (Figure 9-14).

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8 2010 SS, Section 5-1.20B(4), Contractor-Property Owner Agreement, or 2006 SS, Section 7-1.13,
Disposal of Material Outside the Highway Right of Way.
SlurryPro CDP™ is a dry form synthetic polymer slurry product. A dry granular form of SlurryPro CDP™ is currently approved for use on Caltrans projects. No other form is approved. (Figure 9-15)

Shore Pac® is a dry form synthetic polymer slurry product. A dry granular form of Shore Pac® is currently approved for use on Caltrans projects. No other form is approved (Figure 9-16).
Terragel (previously named Novagel) is a dry form synthetic polymer slurry product. A dry granular form of Terragel is currently approved for use on Caltrans projects. No other form is approved (Figure 9-17).

Synthetic slurries must be thoroughly mixed but do not require additional time to hydrate. This is because these slurries can achieve effectively complete hydration in a short time. Water used to mix with the synthetic polymer should have a pH in the range of 8 to 11 in order to properly disperse the polymer. A more acidic pH will retard hydration of the slurry, causing poor performance. A mixing tank is usually required in order to regulate the water. The manufacturers of the approved synthetic slurries recommend tank mixing, but mixing directly into the drilled hole by introducing these products into the flow of water is also acceptable to the manufacturers.
The physical properties of synthetic slurries should be carefully monitored during drilling of the hole and before concrete placement. Because these slurries in general do not suspend particles, the permissible density and sand content values are generally lower than those allowed for mineral slurries. The density and sand content values should be tested and the values maintained within the limits stated in the contract specifications to allow for quick settlement of suspended materials. The synthetic slurry’s pH value should be tested and maintained within the limits stated in the contract specifications to prevent destabilization of the slurry. The allowable limits described in the contract specifications for density, sand content, and pH vary between Super Mud, SlurryPro CDP™, Shore Pac®, and Terragel due to the extensive research that had been done by the slurry system manufacturers during the Caltrans approval process.

The synthetic slurry’s viscosity value has a higher level of importance than that of mineral slurry. The viscosity value should be tested and maintained within the limits stated in the contract specifications to prevent destabilization of the sides of the drilled hole. However, synthetic slurries at high viscosities may be capable of suspending sand particles for longer than expected periods, causing the density and sand content values to increase above their allowable limits. For this reason, caution must be practiced when using synthetic slurries at high viscosities so that particulate settlement on the head of concrete during concrete placement can be prevented. The allowable limits described in the contract specifications for viscosity vary dramatically between Super Mud, Shore Pac®, Terragel, and SlurryPro CDP™. This is due to the extensive research that had been done by the manufacturers during the Caltrans approval process. SlurryPro CDP™ and Terragel are approved for very high viscosity values (>70 sec/quart) during drilling operations to further ensure stability of the drilled hole. Only one synthetic slurry, Terragel, with a very high viscosity value up to 104 sec/quart is approved for use during concrete placement.

In general, synthetic slurries will break down when they come in contact with concrete. This is advantageous as long as the synthetic slurry is clean and the rising head of concrete is the only concrete in contact with the synthetic slurry. However, if concrete is allowed to intermingle with the synthetic slurry, the synthetic slurry may break down and cause the sides of the drilled hole to destabilize.

The contract specifications also require the presence of a manufacturer’s representative to provide technical assistance and advice on the use of their product before the synthetic slurry is introduced into the drilled hole. The Engineer must authorize the manufacturer’s representative. Assistance on authorization of a manufacturer’s representative may be obtained from Structure Construction in Sacramento. The manufacturer’s representative can provide assistance with slurry property testing, can test the water to be used for contaminants that may adversely affect the properties of the synthetic slurry and the stability of the drilled hole, and can give advice in the proper disposal of the slurry.

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9 2010 SS, Section 49-3.02B(6)(c), Synthetic Slurry, or 2006 Special Provisions.
The manufacturer’s representative may also recommend the use of chemical additives to adjust the synthetic slurry to the existing ground conditions. Some chemical additives are approved for use with each manufacturer’s approved slurry system. Other chemical additives that are not approved may be proposed for use. Use of chemical additives should be discussed with Structure Construction in Sacramento before authorization is given for their use.

The contract specifications also require the manufacturer’s representative to be present until the Engineer is confident that the Contractor has a good working knowledge of how to use the product. Once this occurs, the manufacturer’s representative can be released. This can usually be accomplished within the completion of one pile.

Synthetic drilling slurries can be used in most types of ground formations. However, the contract specifications state that synthetic slurries cannot be used in soils classified as “soft” or “very soft” cohesive soils. There are two reasons for this. First, synthetic slurries will encapsulate and cause settlement of clay particles from the soil cuttings. These encapsulated clay particles are similar in appearance and size as sand particles and will cause excessively high false readings of the sand content test value. This problem may also occur in soils that are only slightly cohesive. To overcome this problem, the Contractor should use a dilute bleach solution or dilute acid solution instead of water to dilute the slurry sample and wash the fines through the #200 mesh screen during the sand content test. This will avoid agglomeration of clay particles so they will wash through the #200 mesh screen. Second, the synthetic slurry manufacturers have not completed the research necessary to show that their products function properly in soils defined as “soft” or “very soft” cohesive soils. If this research is successfully completed, the contract specifications may be amended to remove this limitation.

Disposal of synthetic slurries is somewhat easier than disposal of mineral slurries. The manufacturers of the approved synthetic slurries are attempting to get approval for different disposal techniques. However, until they do so, the contract specifications require all material resulting from the placement of piles, including drilling slurry, must be disposed of outside of the highway right-of-way as described in the contract specifications unless otherwise permitted by the Engineer. The Engineer may allow disposal by other means if the proper permits are secured or permission is obtained from the appropriate regulatory agency. Other means of disposal include placing the synthetic slurry in a lined drying pit and allowing it to evaporate. The dried solids then can be disposed of in a similar fashion as other jobsite spoils. Synthetic slurries can also be broken down to the viscosity of plain water with chemical additives, allow time for solids to settle out, and then be disposed of as clarified waste water. Permission must be obtained from the responsible authority, usually the local California Regional Water Authority.

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10 2010 SS, Section 49-3.02B(6)(c), Synthetic Slurry, or Special Provisions for contracts using 2006 SS.
11 2010 SS, Section 5-1.20B(4), Contractor-Property Owner Agreement, or 2006 SS, Section 7-1.13, Disposal of Material Outside the Highway Right of Way.
Quality Control Board or the local sanitation district, for this type of disposal. The dried solids can be disposed of as mentioned above.

9-6 Equipment

The equipment used to construct CIDH piles by the slurry displacement method is not much different than that used to construct CIDH piles by ordinary means. However, there are some differences in the drilling tools, drilling techniques, cleaning techniques, and use of casings.

The primary reason that modified drilling tools and drilling techniques are used has to do with the way drilling slurries work. The drilling contractor must be careful not to do anything that would disturb the positive hydrostatic pressure provided by the drilling slurry on the sides of the drilled hole. The drilling tool can produce rapid pressure changes above and below it, similar to the effect of a piston, if it is lifted or lowered too quickly. When these pressure changes are produced, the drilled hole can collapse (Figure 9-18). This problem can be remedied through the use of drilling tools that allow the drilling slurry to pass through or around the tool during lifting and lowering. For augers, special steel teeth are added to over bore the drilled hole so the diameter of the drilled hole is larger than the diameter of the auger. For drilling buckets and cleanout buckets, special steel teeth are added to over bore the drilled hole, or the bucket itself may be vented. Even with these modifications, the drilling technique must be modified so that the drilling tool is not lowered or raised too rapidly through the drilling slurry.

For reverse-circulation drills, rapid pressure changes due to raising or lowering the drill head are reduced considerably. The drill stem acts as an airlift that removes drill cuttings from the bottom of the hole as it is being excavated. This allows the drill to remain in the hole and, barring malfunctions, eliminates the need to raise or lower it until the excavation is complete.
The techniques used to clean the bottom of the drilled hole are also modified for use in drilling slurries. The initial cleaning of the bottom of the drilled hole is done with a cleanout bucket so that the bottom of the drilled hole has a hard flat surface (Figure 9-19). However, as sand particles settle out of suspension in the drilling slurry, additional cleanings may be required. These additional cleanings can be accomplished with a cleanout bucket, the combined use of a cleanout bucket and pumps, or with a device known as an airlift (Figure 9-20). The airlift device operates with air that is supplied to the bottom of the drilled hole by an air compressor. This causes the settled sand particles to be lifted off the bottom of the drilled hole and vented.
For projects that utilize reverse circulation drills, typically the drill head is left at the specified tip and allowed to spin for a certain amount of time. This allows the airlift built into the drill stem to remove all large and small particles from the bottom of the drilled hole. Once the drill stem and drill head are removed from the hole, it may be necessary to remove more fine particles that may have settled out of the slurry during removal of the drilling equipment. For these settled particles, a separate smaller airlift or pump is typically used.
The use of temporary casing may be appropriate in certain situations when the slurry displacement method is used. Temporary casing may be necessary if a dry loose material stratum or a loose material stratum with flowing groundwater is encountered during drilling (Figure 9-21). Even drilling slurries with viscosity values at the allowable maximum limit may not be able to stabilize a drilled hole in these situations. It may be necessary to place temporary casing only where the dry loose material strata or the loose material strata with flowing groundwater is located and use mineral or synthetic drilling slurries to stabilize the remainder of the drilled hole. Another option is to place a full-length temporary casing in the drilled hole and use water as the drilling slurry, if allowed by the specifications, in order to avoid a quick condition at the bottom of the drilled hole.
9-7 Specifications

Because of the nature of slurry displacement construction, visual inspection of the drilled shaft is not possible for much of the time. Most of the drilling and concrete placement is done “in the blind”. As a result, the contract specifications for this work are quite stringent in an attempt to minimize the risks and to ensure that the pile has structural and geotechnical integrity. Some of the more critical requirements of the contract specifications are discussed in the following sections.

9-7.1 Minimum Pile Diameter Requirements

Only piles 24 inches in diameter or greater may be constructed by the slurry displacement method. This is because a pile with a lesser diameter does not contain enough room for the pile bar reinforcement cage, inspection pipes, and the large concrete delivery tubes. If
a contract specifies the use of piles with a diameter of less than 24 inches, the Contractor may propose to increase the diameter of the pile to at least 24 inches by the provisions described in the contract specifications12 if use of the slurry displacement method of construction is desired. However, the diameter of the pile bar reinforcement cage would have to be increased from the original size in order to accommodate the items mentioned above.

9-7.2 Concrete Compressive Strength and Consistency Requirements
Before any pile construction work using the slurry displacement method can begin, the Contractor must demonstrate the concrete mix design can meet the required compressive strength requirements and consistency requirements. This is accomplished by producing a concrete test batch in accordance with the contract specifications13. The concrete test batch must demonstrate the proposed concrete mix design achieves the specified nominal slump at the time of placement. For piles where the concrete placement operation is expected to be 2 hours or less, the test batch must demonstrate that the proposed concrete mix design achieves a slump of at least 7 inches after twice the time of the proposed concrete placement operation. For piles where the concrete placement operation is expected to be longer than 2 hours, the test batch must demonstrate that the proposed concrete mix design achieves a slump of at least 7 inches after the time plus 2 hours of the proposed concrete placement operation. The intent of this specification is to make sure the first load of concrete placed in the drilled hole will remain sufficiently fluid as it rises to the top of the pile. The concrete must also have a high fluidity in order to flow through the pile bar reinforcement cage, to compact and consolidate under its own weight without the use of vibration, and to deliver high lateral stresses on the sides of the drilled hole in order to keep the drilled hole from collapsing as the drilling slurry is displaced and the filter cake (in the case of mineral slurries) is scoured from the sides of the drilled hole by the rising column of concrete. The concrete test batch and compressive strength requirement gives the Engineer and the Contractor the opportunity to observe how the concrete mix will behave before it is used.

9-7.3 Slurry Testing and Cleaning Requirements
During pile construction work, the contract specifications require the Contractor to sample and test the drilling slurry in order to control its physical properties. The contract specifications also require that each type of drilling slurry be sampled and tested at different intervals and locations.

9-7.3.1 Mineral
For mineral slurries, samples must be taken from the mixing tank for testing prior to the mineral slurry’s introduction into the drilled hole. Once the mineral slurry has been introduced into the drilled hole, the contract specifications require the mineral slurry to

12 2010 SS, Section 49-3.02C(1), Construction General, or 2006 SS, Section 49-4.03, Drilled Holes.
13 2010 SS, Section 49-3.02A(4)(c), Concrete Test Batch, or Special Provisions for contracts using 2006 SS.
14 2010 SS, Section 49-3.02B(6)(b), Mineral Slurry, or Special Provisions for contracts using 2006 SS.
undergo either recirculation or continuous agitation in the drilled hole. The Contractor must address which method of agitation will be used in the Pile Installation plan.

If the recirculation method is used, the contract specifications require the mineral slurry to be cleaned as it is recirculated. This is done using a slurry plant, which stores, recirculates, and cleans the mineral slurry. Samples for testing must be taken from the slurry plant storage tank and the bottom of the drilled hole. As the mineral slurry is recirculated and cleaned, samples must be taken every two hours for testing until the test values for the samples taken at the two testing locations are consistent. Once the test samples have consistent test values, the sampling and testing frequency may be reduced to twice per work shift. As the recirculation and cleaning process continues, the properties of the mineral slurry will eventually conform to the specification parameters. Once the test samples have properties within the specification parameters, the bottom of the drilled hole can be cleaned.

If the continuous agitation in the drilled-hole method is used, the contract specifications do not require the mineral slurry to be physically cleaned. Samples for testing are taken at the mid-height and at the bottom of the drilled hole. As the mineral slurry is continuously agitated, samples are taken every two hours for testing. If the samples at the two locations do not have consistent test values, the mineral slurry must be recirculated. This means that the continuous agitation in the drilled-hole method is failing to keep the suspended particles in the mineral slurry from settling. This is also an indication that the mineral slurry is not clean enough to meet the specification parameters. Therefore, the Contractor must abandon this method and use the recirculation method. However, if the test samples do have consistent test properties within the specification parameters, the bottom of the drilled hole can be cleaned.

Once the bottom of the drilled hole has been initially cleaned, recirculation or continuous agitation in the drilled hole may be required to maintain the specified properties of the mineral slurry. Usually the initial cleaning will stir up the settled materials at the bottom of the drilled hole, thus requiring the mineral slurry to be re-cleaned so it meets the requirements of the contract specifications. Several iterations may be required before both the mineral slurry and the bottom of the drilled hole are clean. To verify the cleanliness of the mineral slurry, the contract specifications require additional samples to be taken for testing. Samples are taken at the mid-height and at the bottom of the drilled hole. Once the test samples show the mineral slurry’s properties to be within the specification parameters, and there is no settled material on the bottom of the drilled hole, the last cleaning of the bottom of the drilled hole can be considered to be the final cleaning. At this point, the pile bar reinforcement cage can be placed. The contract specifications require that samples for testing be taken just prior to concrete placement to verify the properties of the mineral slurry. Samples are taken at the mid-height and at the bottom of the drilled hole. If the test samples have consistent test properties within the specification parameters, concrete may be placed. Otherwise, additional cleaning of the mineral slurry and removal of settled materials from the bottom of the drilled hole may be required.
The reason for testing mineral slurries at different levels is to make sure the mineral slurries are well mixed and have consistent physical properties throughout the length of the drilled hole. The mineral slurry’s physical properties should be the same at both locations. This indicates that the mineral slurry is completely mixed and that any sand or particles contained are in suspension.

9-7.3.2 Synthetic

For synthetic slurries, sampling for testing must be conducted before, during, and after the drilling operation, and as necessary to verify and control the physical properties of the slurry. Samples are taken at the mid-height and at the bottom of the drilled hole. Once the drilling operation has been completed, additional samples for testing must be taken. When the synthetic slurry’s physical properties are consistent at the two sampling locations and meet the requirements of the contract specifications, the bottom of the drilled hole can be cleaned.

Synthetic slurries are cleaned by allowing for an un-agitated settlement period, usually of about 30 minutes in length. Because synthetic slurries in general will not suspend sands, the sands will settle to the bottom of the drilled hole during the settlement period.

Once the bottom of the drilled hole has been initially cleaned, further settlement periods may be required. Usually, the initial cleaning will stir up the settled materials at the bottom of the drilled hole, thus requiring the synthetic slurry to be re-cleaned so it meets the requirements of the contract specifications. Several iterations may be required before both the synthetic slurry and the bottom of the drilled hole are clean. To verify the cleanliness of the synthetic slurry, the contract specifications require additional samples to be taken for testing. Samples are taken at the mid-height and at the bottom of the drilled hole. Once the test samples show the synthetic slurry’s properties to be within the specification parameters, and there is no settled material on the bottom of the drilled hole, the last cleaning of the bottom of the drilled hole can be considered to be the final cleaning. At this point, the pile bar reinforcement cage can be placed. The contract specifications require that samples for testing be taken just prior to concrete placement to verify the properties of the synthetic slurry. Samples are taken at the mid-height and at the bottom of the drilled hole. If the test samples have consistent test properties within the specification parameters, concrete may be placed. Otherwise, additional settlement periods and removal of settled materials from the bottom of the drilled hole may be required.

The reason for testing synthetic slurries at different levels is to make sure the synthetic slurries are well mixed and have consistent physical properties throughout the length of the drilled hole.

The intent of these specifications is to ensure that the drilling slurry is properly mixed in order to provide stability to the drilled hole and to control the amount of suspended materials in the drilling slurry that may settle during placement of the pile bar reinforcement cage and concrete.
9-7.4 Pile Acceptance Testing Access Requirements
During pile construction work, the contract specifications\ref{footnote15} require the installation of inspection pipes at specific intervals around the perimeter of the pile bar reinforcement cage. This is necessary to provide access for acceptance testing.

9-7.5 Pile Concrete Placement Requirements
During pile construction work, the contract specifications\ref{footnote16} require that concrete must be placed through rigid tremie tubes with a minimum diameter of 10 inches or through rigid pump tubes. The tubes are required to be capped or plugged with watertight plugs that will disengage once the tubes are charged with concrete. The tip of the concrete placement tube is required to be located a minimum of 10 feet below the rising head of concrete.

The concrete placement operation for a CIDH pile constructed under drilling slurry is an operation that requires much preplanning. Before the work begins, the contract specifications\ref{footnote17} require the concrete mix design to meet the trial batch requirements for compressive strength concrete. The concrete mix must contain at least 675 pounds of cementitious material per cubic yard. It is also important to ensure that the clear rebar spacing is not less than 5 inches, except at locations of PVC inspection pipes where the clear spacing between adjoining vertical rebars should be no less than 8.5 inches as explained in Section 9-8, Inspection and Contract Administration. The Designer should be contacted if this is not the case. A concrete test batch is also required to show the concrete mix design meets the consistency requirements of the contract specifications. The concrete consistency requirements are to ensure that the concrete will remain fluid throughout the length of the pour. The Engineer does not allow the Contractor to exceed the maximum allowable water requirement to achieve this goal. Chemical admixtures will most likely be necessary. It is also important for the concrete mix to be properly proportioned to prevent segregation and excess bleed water due to the high fluidity of the concrete.

The method of concrete placement should not allow the intermingling of concrete and drilling slurry. The contract specifications allow placement of concrete through rigid tremie tubes, or through rigid tubes connected directly to a concrete pump. In order to prevent intermingling of concrete and drilling slurry, the concrete placement tubes must be capped with a watertight cap or plugged such that the concrete will not come into contact with the drilling slurry within the concrete placement tube. The cap or plug should be designed to release when the placement tube is charged with concrete. Charging the placement tube with concrete must not begin until the capped or plugged tip of the placement tube is resting on the bottom of the drilled hole. Once the placement tube has been charged, the pour is initiated by lifting the tip of the placement tube 6

\begin{footnotes}
\item[15] 2010 SS, Section 49-3.02A(4)(d)(ii), Vertical Inspection Pipes, or Special Provisions for contracts using 2006 SS.
\item[16] 2010 SS, Section 49-3.02C(8), Placing Concrete Under Slurry, or Special Provisions for contracts using 2006 SS.
\item[17] 2010 SS, Section 49-3.01B(1), Materials General, or Special Provisions for contracts using 2006 SS.
\end{footnotes}
inches above the bottom of the drilled hole. This allows the concrete in the placement tube to force the cap or plug out of the placement tube and discharge.

Once the pour has started, it is important to place the concrete at a high rate until the tip of the placement tube is embedded in the concrete. If concrete placement operations slow or stop before the tip of the placement tube is embedded in concrete, there is nothing to prevent the intrusion of drilling slurry into the placement tube. If this happens, the likely result will be a defect at the tip of the pile.

In accordance with the contract specifications, when concrete placement begins, the tip of the concrete placement tube must not be raised from 6 inches above the bottom of the drilled hole until a minimum of 10 feet of concrete has been placed in the pile. After this level is reached, the tip of the concrete placement tube must be maintained at a minimum of 10 feet below the rising head of concrete. The best way to verify that the tip of the concrete placement tube is being maintained at this is for the Contractor to mark intervals of known distance on the placement tube and to measure the distance from the top of the pile to the rising head of concrete with a weighted tape measure.

If for some reason concrete placement is interrupted, such that the placement tube must be removed from the concrete, the placement tube must be cleaned, capped, and pushed at least 10 feet into the concrete head before restarting concrete placement in accordance with the contract specifications. Concrete placement continues in this manner until the rising head of concrete reaches the top of the pile. Concrete is then wasted until all traces of particle settlement and drilling slurry contamination are no longer evident. Under circumstances where contaminated concrete cannot be wasted from the top of the pile, such as having a pile construction joint within a permanent casing below grade, pile concrete is placed to a predetermined level above the planned concrete placement elevation, and the contaminated concrete above the planned concrete placement elevation is either mucked out immediately after placement or chipped out at a later time.

Vibration of the pile concrete is not necessary because concrete with high fluidity self-consolidates under the high hydrostatic pressure provided.

The intent of these specifications is to prevent the concrete from intermingling with the drilling slurry during concrete placement and avoid pile defects.

9-8 Inspection and Contract Administration

The reader is advised to review this section in Chapter 6, Cast-In-Drilled-Hole Piles. All inspection and contract administration information listed therein, with the exception of items that are precluded by the presence of slurry in the drilled hole, are applicable to

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18 2010 SS, Section 49-3.02C(8), Placing Concrete Under Slurry, or Special Provisions for contracts using 2006 SS.
CIDH piles constructed using the slurry displacement method. This section outlines the additional requirements for CIDH piles constructed using the slurry displacement method.

The contract specifications require the Contractor to submit to the Engineer a Pile Installation Plan for review and authorization. The Pile Installation Plan should provide sufficient detail for the Engineer to grasp the means, methods, and materials the Contractor plans to use to successfully complete pile installation. Typical requirements include those listed in Chapter 6, Cast-In-Drilled-Hole Piles, as well as additional requirements including the following:

Table 9-1. Additional Pile Installation Plan Requirements.

<table>
<thead>
<tr>
<th>ITEM</th>
<th>PILE INSTALLATION PLAN REQUIREMENT &amp; REASONING</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Concrete batching, delivery, and placing systems, including time schedules and capacities. Time schedules will include the time required for each concrete placing operation at each pile. <em>Reasoning:</em> This gives the Engineer advance knowledge of how, when, and how long it will take for the Contractor to place concrete in each pile and whether the proposal is appropriate. Time schedules are also necessary to determine the amount of time required for the concrete test batch.</td>
</tr>
<tr>
<td>2</td>
<td>Concrete placing rate calculations. If requested, base calculations on the initial pump pressures or static head on the concrete and losses throughout the placing system, including anticipated head of slurry and concrete to be displaced. <em>Reasoning:</em> This gives the Engineer additional knowledge of how the Contractor proposes to place concrete in each CIDH pile and is considered supplementary information for Item 1. This information is especially important for large deep piles as it will be used to verify whether the proposed concrete delivery system has enough pressure to displace the anticipated head of slurry and the fluid concrete placed in the pile.</td>
</tr>
<tr>
<td>3</td>
<td>Suppliers’ test reports on the physical and chemical properties of the slurry and any proposed slurry chemical additives, including Material Safety Data Sheets (MSDSs). <em>Reasoning:</em> This gives the Engineer advance knowledge of the slurry and any chemical additives that the Contractor proposes to use and whether the proposal is appropriate for each pile.</td>
</tr>
<tr>
<td>4</td>
<td>Slurry testing equipment and procedures. <em>Reasoning:</em> This gives the Engineer advance knowledge of the slurry testing equipment and procedures to verify that they are in accordance with the requirements of the contract specifications.</td>
</tr>
<tr>
<td>5</td>
<td>Methods of removal and disposal of excavation, slurry, and contaminated concrete, including removal rates. <em>Reasoning:</em> This gives the Engineer advance knowledge of the means the Contractor proposes to use for disposal of spoils from CIDH pile construction and whether the proposal is appropriate and in conformance with the contract specifications.</td>
</tr>
<tr>
<td>6</td>
<td>Methods and equipment for slurry agitating, recirculating, and cleaning.</td>
</tr>
</tbody>
</table>

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19 2010 SS, Section 49-3.02A(3)(b) *Pile Installation Plan,* or Special Provisions for contracts using 2006 SS.

20 2010 SS, Section 5-1.20B(4) *Contractor-Property Owner Agreement,* or 2006 SS, Section 7-1.13, *Disposal of Material Outside the Highway Right of Way.*
In order to facilitate pile testing, the contract specifications require the installation of inspection pipes (Figure 9-22). Before the rebar cage is placed in the drilled hole, the Engineer should verify the inspection pipes are installed inside the spiral or hoop reinforcement and are at least 3 inches away from the vertical reinforcement of the pile bar reinforcement cage. Figure 9-23 shows a typical inspection pipe layout and spacing pattern within the pile bar reinforcement cage. These pipes must be placed in a straight alignment, securely fastened in place, and be watertight. These pipes permit the insertion of a Gamma-Gamma Logging test probe that measures the density of the pile concrete. The most commonly used test probe is 1.25 inches in diameter and 54 inches in length. If the inspection pipes are not placed in a straight alignment or are not securely fastened, the test probe will not fit in the inspection pipe. One way of testing the pipe would be to try to deflect it by hand. If it can be deflected by hand, it may be deflected by the placement of concrete. It is also recommended that the Contractor install a rigid rod in each inspection pipe prior to concrete placement to ensure that the inspection pipes remain straight during and after concrete placement. Inspection pipes need to be filled with water prior to or immediately after completion of the concrete placement. The reason for this is to prevent the inspection pipe from separating from the pile concrete (debonding) or overheating during the curing process. This helps keep the inspection pipe intact so that it can be used for crosshole sonic logging at a later point if necessary. Once the inspection pipe has separated or had airspace created between it and the pile concrete, crosshole sonic logging can no longer be performed because the airspace registers as an anomaly.

The contract specifications also require the Contractor to log the locations of any inspection pipe couplers and submit the log to the Engineer. This is necessary because inspection pipe couplers show up as areas of lower density when a gamma ray scattering test is performed. Testing personnel can ignore these areas if they are aware of the coupler locations.

21 2010 SS, Section 49-3.02A(4)(d)(ii), Vertical Inspection Pipes, or Special Provisions for contracts using 2006 SS.
Figure 9-22. Inspection Pipes.

Figure 9-23. Location of Inspection Pipes within the Pile.
The Engineer will notify Geotechnical Services, Foundation Testing Branch (FTB) as soon as the proposed pile concrete placement date is known, in accordance with the provisions of Bridge Construction Memos 130-1.0, *Foundation Testing Branch* and 130-10.0, *Testing of CIDH Piling*. This places the FTB on notice that acceptance testing will be required and approximately when it will be needed.

The Engineer should be present when the slurry manufacturer’s representative is on site to verify the slurry is mixed, placed, tested, and disposed of or cleaned in accordance with the provisions of the authorized Pile Installation Plan and the contract specifications. The Engineer should also perform side-by-side slurry tests with the Contractor or manufacturer’s representative at least once per project. Slurry testing equipment is available from the Bridge Construction Engineer.

During drilling operations, the Engineer should monitor the height of the slurry in the drilled hole to verify that positive hydrostatic pressure is being maintained on the sides of the drilled hole.

Prior to placement of concrete, the Engineer should verify the properties of the slurry are within the specification requirements and that the bottom of the drilled hole is clean in accordance with the provisions of the authorized Pile Installation Plan. This is very important because settled materials left at the bottom of the pile cause over 50% of all pile defects.

Concrete placement warrants continuous inspection. The Engineer should verify that all equipment needed to measure the height of the concrete placed in the pile, the depth of the concrete placement tube within the head of concrete, and the volume of concrete placed in the pile in accordance with the provisions of the Pile Installation Plan are on site and ready for use. During concrete placement operations, the Engineer should verify that the concrete placement tube is always at least 10 feet below the free surface of the in-place concrete. The specifications require the Contractor to maintain a log of concrete placement for each pile and to deliver the completed log to the Engineer after completion of concrete placement in each pile. This log is used to pinpoint any potential problem zones within the pile that may have occurred during concrete placement. Potential problem zones are denoted on the log by marked differences between the actual amount of concrete placed and the theoretical amount of concrete that should have been placed at the same elevation within the pile.

### 9-9 Pile Acceptance Testing

After concrete placement and before acceptance testing is performed, the inspection pipes must be checked by the Contractor for blockages and straightness by passing a rigid cylinder (dummy probe) that is the same size and shape as the gamma ray scattering test
probe in accordance with the contract specifications. The Contractor must demonstrate to your satisfaction that the dummy probe passes freely through each inspection pipe without the application of any force. For reasons mentioned below, the FTB will not test any inspection pipe that cannot pass the dummy probe. Inspection pipes that cannot accept the dummy probe must immediately be refilled with water. They must also be replaced with a 2 inch diameter cored hole the full length of the pile. The Engineer should discuss this requirement with members of the Division of Engineering Services (DES) CIDH Pile Committee before any coring is performed. The reason the inspection pipes must be refilled with water is to eliminate pipe-to-concrete debonding problems that can render crosshole sonic logging unusable.

![Gamma-Gamma Logging Test Schematic](image)

**Figure 9-24. Gamma-Gamma Logging Test Schematic.**

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\[22\] 2010 SS, Section 49-3.02A(4)(d)(ii), *Vertical Inspection Pipes*, or Special Provisions for contracts using 2006 SS.
Determining the soundness of slurry displacement piles is important. There are a number of methods that may be used to test the soundness of these piles. One method is the use of external vibration, which measures stress wave propagations in the pile using either internal or external receivers. This requires a variety of expensive electronic gear and skilled operators, as well as the placement of instrumentation on the pile bar reinforcement cage prior to concrete placement. Another method uses an acoustical technique, which is commonly referred to as crosshole sonic logging. This involves lowering sender and receiver probes into the inspection pipes to measure the velocity of sonic waves through the concrete. Defective concrete is registered by the increased amount of time it takes for the sonic wave to be received by the receiver probe, as opposed to the shorter amount of time it takes for the sonic wave to be received across a solid medium (sound concrete). A third method would be to core the pile and recover the physical cores for inspection. This method may be the most conclusive, but is very time consuming and is destructive. A fourth method uses a radiographic technique called Gamma-Gamma Logging (GGL) (Figure 9-24).

The contract specifications state the GGL method of testing piles constructed using the slurry displacement method is used to determine acceptance of the pile, in accordance with the provisions of California Test 233.

During GGL testing, scatter counts are taken and compared to counts taken on a standard containing the same material being tested. By this means, relative densities can be ascertained. In general, the lower the scatter count, the denser the material. The nuclear probe used in these tests contains a source that is relatively weak and its effective range of sensitivity is limited to a 3-inch radius of concrete around the inspection pipe. Because of the nature of the data acquired, proper assessment or determination of the existence of defective concrete or voids is subject to interpretation of the results. Typical testing consists of continuous counts taken as the test probe is raised from the tip of the pile at 10 to 12 feet per minute. This procedure requires about 2 hours to log all of the inspection pipes for a 4 foot diameter pile 100 feet in length.

Because GGL uses a nuclear probe, radiation safety is of paramount importance. The FTB is permitted to use this testing equipment by State and Federal government regulators by establishing and following strict radiation safety protocols. These protocols include no unauthorized persons within 25 horizontal feet of the nuclear probe and no possibility of getting the nuclear probe stuck in the inspection pipe due to inspection pipe misalignment. The FTB can lose their permit to use nuclear materials if radiation safety protocols are not followed. Consultant testers under the auspices of the FTB must also follow strict radiation safety protocols. Radiation safety is the primary reason the Contractor is required to pass a dummy probe through each inspection tube prior to the FTB arriving on site to test the piles.

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23 2010 SS, Section 49-3.02A(4)(d)(iii), Gamma-Gamma Logging, or Special Provisions for contracts using 2006 SS.
The contract specifications also state that crosshole sonic logging or other means of inspection may be used to further evaluate a rejected pile. Typically, crosshole sonic logging or other means of inspection are used to complement the results of GGL testing and are only performed after GGL testing has been performed and the pile has been rejected.

All test methods used to accept CIDH piles constructed under slurry are performed by Caltrans personnel from Geotechnical Services, Foundation Testing Branch (FTB), or by consultant personnel under the auspices of the FTB. The results of such testing, which include a recommendation of acceptance or rejection of the pile, are reported to the Engineer in writing. An example of these results can be found in Appendix G, Slurry Displacement Piles. Further information on pile acceptance testing may be found at the FTB web page.

The Engineer has the responsibility for accepting or rejecting a pile based on the recommendations of the FTB. If the pile is accepted, the inspection pipes may be cleaned and grouted, and the pile is complete.

### 9-10 Defective Piles

What causes piles constructed by the slurry displacement method to be defective? One of the primary reasons for pile defects is a problem caused by the presence of settled materials at the bottom of the drilled hole. These are materials that were held in suspension by the slurry that settled out of suspension either before or during the concrete placement operation. These materials can also be the result of improper cleaning of the base of the drilled hole. These materials can be trapped at the bottom of the pile by concrete placement as shown in Figure 9-25(a) or they can be enveloped and lifted by the fluid concrete only to become caught by the pile bar reinforcement cage or against the sides of the drilled hole and not displaced by the fluid concrete as shown on Figure 9-25(b). These materials can also fall out of suspension and settle onto the head of concrete during concrete placement, become enveloped by the concrete as it rises, and attach to the pile bar reinforcement cage or the sides of the drilled hole as previously described.

During pile testing, these deposits will register as areas of lower density than that of sound concrete. Excessive amounts of settled materials can occur in mineral slurries that were not properly cleaned or agitated and carry inordinate amounts of suspended materials. Excessive amounts of settled materials can occur in synthetic slurries when not enough time is allowed for the materials to settle out before the final cleaning of the bottom of the drilled hole or if the synthetic slurry becomes contaminated from clay-particle encapsulation.

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24 2010 SS, Section 49-3.02A(4)(d)(iv), Rejected Piles, or Special Provisions for contracts using 2006 SS.
Another reason for pile defects is due to improper drilling slurry handling. If mineral slurries are not properly mixed and are not allowed to properly hydrate, they can form balls or clumps that can become attached to the pile bar reinforcement cage and not be removed by concrete placement as is shown in Figure 9-26. Mineral slurries that remain in the drilled hole for too long can form a filter cake that is too thick for the fluid concrete to scour off the sides of the drilled hole as is shown in Figure 9-27. Mineral and synthetic slurries that carry an excessive load of suspended materials can be subject to precipitation if an unexpected chemical reaction takes place. This is possible if the concrete is dropped through the drilling slurry.

![Diagram of pile defects](image)

**Figure 9-25 (a) (b). Defects from Settled Materials.**

- **a. Materials at Bottom of Pile**
- **b. Materials Caught by Cage or Against the Side**
A third reason for pile defects is concrete mix design and placement problems. The most common defect of this type occurs when an insufficient amount of slurry-contaminated concrete is wasted from the top of the pile during concrete placement, resulting in a defective pile top. To avoid this type of defect, it is recommended that the volume of concrete to a depth of one pile diameter within the pile be wasted. A less common defect can occur when the seal between the head of concrete and the drilling slurry is lost. This is because entrapment of drilling slurry within the concrete is almost inevitable under this circumstance (Figure 9-28). If the concrete placement tube loses its seal and allows concrete from the placement tube to drop through the drilling slurry onto the head of concrete, the drilling slurry and any settled material on the head of concrete could be trapped between the concrete layers, causing a pile defect. Typically this occurs when the concrete placement tube is removed too rapidly and pulled out of the concrete head.

Another less common defect can occur if the concrete head begins to set, resulting in the concrete “folding” over as it rises through the pile bar reinforcement cage and entrapping drilling slurry and any settled materials as previously described. Yet another type of pile defect can result due to concrete mix design problems. The Engineer should not permit...
the use of excess water in the concrete mix design or allow additional water to be mixed with the concrete at the jobsite to provide the necessary fluidity. This may result in severe bleed water from the concrete after placement, which could indicate segregation and subsidence of the pile concrete. This may cause the entire pile to be defective. If excess free water in the concrete is present when synthetic slurries are used, the excess free water will attract the polymer chains from the drilling slurry into the concrete and produce a material contaminant known as oatmeal at the concrete-slurry interface. This material can potentially be caught on the pile bar reinforcement cage and cause pile defects.

![Diagram of concrete placement problems](image)

**Figure 9-28. Defect from Concrete Placement Problems.**

These types of problems can be avoided if the Contractor and the Engineer closely follow the parameters specified in the contract specifications. These specifications help to ensure the proper mixing and properties of drilling slurries, the proper qualities of the concrete mix design, and the proper methods of concrete placement.

If the FTB recommends rejection of the pile and the Engineer rejects the pile, the Contractor must be informed in writing that the pile is rejected and given a copy of the
GGL test results. The contract specifications also require suspension of the placement of concrete under drilling slurry until written modifications to the method of pile construction are submitted to and authorized by the Engineer. This is to prevent additional failures due to the method of pile construction.

9-11 Pile Mitigation and Acceptance

9-11.1 What Happens When a Pile is Rejected
Once a pile has been rejected and the Engineer determines the rejected pile requires repair, the Contractor has several options. The defect can be accessed and repaired, the pile can be supplemented, the pile can be replaced, or the Contractor may propose a solution that allows the pile to remain in place. The Contractor’s proposal is submitted to the Engineer in the form of a Pile Mitigation Plan.

When a pile has been rejected, the Engineer should confer with the FTB and decide if the FTB will perform crosshole sonic logging on the rejected pile. Typically, the GGL test report will indicate whether the FTB will perform crosshole sonic logging. Crosshole sonic logging is used to further delineate the nature of the defective area within the rejected pile. Generally, this test method is used to determine whether the defective area is within the core of the pile or at the perimeter surrounding the bar reinforcement cage. If crosshole sonic logging is performed, the results of this test should be made available to the Contractor to aid in the preparation of their Pile Mitigation Plan.

The Contractor may also perform an investigation on the rejected pile. They may perform their own non-destructive testing or may core the pile to further determine the nature of the defective area of the rejected pile. The Contractor should submit the results of their investigation to the Engineer and use the results of their investigation in preparation of their Pile Mitigation Plan.

9-11.2 Pile Mitigation Methodologies
There are several ways to mitigate a pile once it has been determined to have anomalies and has been rejected. Mitigation of defective CIDH piles can be grouped into four methodologies:

1. Unearth and Recast (Basic Repair).
2. Pressure Grout (Grouting Repair).
3. Structural Bridging.

The following sub-sections address how to take corrective action on a rejected pile.  

26 2010 SS, Section 49-3.02A(4)(d)(iv), Rejected Piles, or Special Provisions for contracts using 2006 SS.
9-11.3 Repairs

9-11.3.1 Basic Repair
Basic repair is simply the mechanical removal and replacement of any concrete within the defective zone of the pile, as defined by the pile acceptance test results. When a basic repair is performed within 5 feet of the top of the pile, it is known as a Simple repair, as defined in Bridge Construction Memo 130-11.0, Simple Repair of CIDH Piling. Typically, a basic repair is used to mitigate pile defects caused by not wasting enough concrete from the top of the pile during concrete placement. However, basic repairs can be performed deeper down the length of the pile, provided shoring is in place to permit access to the defect. Should the Contractor propose a basic repair below 5 feet from the top of the pile, the Engineer must consult with the Geoprofessional to assess the effect of accessing the defect upon the skin friction capacity of the pile.

9-11.3.2 Grouting Repair
Grouting repairs are used to mitigate defective concrete within the pile. These repairs, when applicable, can be performed at any location within the pile, but are generally not performed within 5 feet of the top of the pile, since it is more effective to use a basic repair at this location. Grouting repairs are performed using three types of grouting procedures: (1) permeation grouting, (2) replacement grouting, or (3) compaction grouting.

Several operations are common to permeation and replacement grouting repairs. First, the Contractor must access the defective area. This is usually done through the existing inspection pipes. Generally, the inspection pipe is removed at the defective area using a high-pressure water jet, which cuts the inspection pipe into small pieces that are then flushed out through the top of the inspection pipe. Once the inspection pipe has been removed at the defective area, the Contractor will wash the defective area using high-pressure water jets and observe the discharge for soil, fragmented concrete, or other contaminants. After the initial washing operation is complete, the Contractor will evaluate the defective area using water flow testing.

Water flow testing is used to assess the nature of the defective area and determine whether permeation or replacement grouting is appropriate. If water can be injected into a defective area at low pressure and relatively high volume, then permeation grouting may be the appropriate grouting repair technique. If the defective area is large enough and permeable, communication with other inspection pipes may be observed, meaning the water injected into one inspection pipe may return to the ground surface through adjacent inspection pipes. Water may also flow into the soil formation if the defective area extends to the edge of the pile concrete. However, if water cannot be injected into a defective area, replacement grouting may be the appropriate grouting repair technique. This is an indicator that the defective area is contained within the pile concrete and the concrete surrounding the defective area is sound.
Once water flow testing has been conducted, the Contractor will typically flush the defective area using low pressure flowing water to remove any remaining loose material. The Contractor may then use a down-hole camera or other means to verify loose materials were adequately removed from the defective area.

**9-11.3.2.A Permeation Grouting.**
Typically used to repair a “soft tip” within the pile concrete, to increase frictional resistance along the side of the pile, or to address corrosion issues at the side of the pile. Usually, permeation grouting is used to repair defects caused by excessive settled materials not removed from the bottom of the drilled hole prior to concrete placement. First, the inspection pipe is removed at the location of the defective area. Then the area is washed with high-pressure water jets to remove any contaminants and loose materials. The discharge from the washing operation is evaluated. Generally, permeation grouting is recommended only if soil is present in the washing discharge or water flow testing verifies the permeability of the defective area. High-pressure grout injection is performed, usually through one of the inspection pipes, with the grout permeating the soil or concrete formation, displacing any pore water that may be present, resulting in a solid matrix of cement grout and defective concrete or soil. Permeation grouting is only successful if the pore water present in the formation can be forced out by the grout, meaning that the pore water must be able to escape into the adjacent soil or through an adjacent inspection pipe. Permeation grouting also requires the presence of sufficient confining pressure to conduct grouting operations without grout returning to the surface. For these reasons, permeation grouting is not recommended for repair of defects completely within the pile or within 10 feet of the ground (or working) surface.

**9-11.3.2.B Replacement Grouting.**
Typically used to repair a void area or an area of unconsolidated concrete within the pile. Typically, replacement grouting is used to repair defects caused by concrete placement problems. As with permeation grouting, access to the defective area is usually provided through the existing inspection pipes. First, the inspection pipe is removed at the location of the defective area. Then the area is washed with high-pressure water jets to remove any contaminants and loose materials. This generally results in the creation of a void within the pile concrete. The discharge from the washing operation is evaluated. Generally, replacement grouting is recommended only if soil is not present in the discharge or water flow testing indicates the defective area is impermeable. All water resulting from the washing operation must be removed from the void prior to placement of grout. This is typically done with compressed air. Grout is then pumped into the void, in effect, “replacing” the voided area with grout. Replacement grouting cannot be used if the grout has a means of escaping the void area. If a side of the void area includes the side or bottom of the drilled hole, replacement grouting generally cannot be used to repair the pile defect.
**9-11.3.2.C Compaction Grouting.**

Typically used to enhance the load-bearing capacity of the soil at the tip of the pile. Because of the inherent difficulty of employing grouting methods below the tip of a pile, compaction grouting is only used when the pile defect consists of a “soft tip” and end bearing is required in the design. Access to the defective area is usually provided through the existing inspection pipes. Generally, only the bottom of the inspection pipe is removed and the area below the inspection pipe is not washed or flushed. Grout is then pumped at high pressure into the loose soil formation at the tip of the pile, resulting in a “bulb” of soil-grout matrix at the tip of the pile. In order to be successful, compaction grouting must be performed through each inspection pipe.

On some projects, “tip grouting” is specified to enhance the end bearing capacity of the pile. Compaction grouting differs from tip grouting in that tip grouting is shown on the contract documents and has additional pipes and other equipment for grout access to the tip of the pile, whereas compaction grouting is a remedial procedure. The problem that arises concerning compaction grouting and tip grouting is there is no way to evaluate the pile end bearing capacity gained as a result of these operations.

Depending on the nature and number of defective areas within the pile, one or more of the grouting procedures described above may be required.

**9-11.4 Structural Bridging**

Structural bridging can be performed to increase the structural strength of a defective pile without complete removal of the defect. It is typically performed by coring the center of the pile and installing a structural steel section, rebar cage, or combination of both, cast into the central portion of the pile. This additional member may be designed to restore the structural strength of the pile to meet the project requirements. It may also be possible to extend a central drilled section into the bearing formation below pile tip to increase the geotechnical strength of the pile.

**9-11.5 Supplemental and Replacement Piles**

Occasionally, piles can be so riddled with defects that repair of the pile is not feasible. If the Engineer determines repair of the rejected pile is not feasible, the contract specifications require the Contractor to submit a pile mitigation plan for replacement or supplementation of the rejected pile. If space exists, the Contractor may propose to place supplemental piles to enhance the load-bearing capacity of the defective pile. If there is no space available for supplemental piles, the Contractor may be required to remove the defective pile and replace it.

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9-11.6 Pile Mitigation Plan Development and Authorization Procedures

Once a pile is rejected and it is determined that the pile requires mitigation, the contract specifications require the Contractor to submit a Pile Mitigation Plan for review and authorization. A Pile Mitigation Plan is required for any type of repair proposed by the Contractor, or when supplemental or replacement piling is necessary.

The Association of Drilled Shaft Contractors (ADSC), which is an industry group composed of member drilling contractors, has developed several Standard CIDH Pile Anomaly Mitigation plans. These plans are intended to address the most common types of pile anomalies encountered, which generally consist of 80-90% of all pile anomalies. These plans encompass Basic Repair, Permeation Grouting and Replacement Grouting repair methods. Caltrans has authorized these plans for statewide use. The Engineer should expect to receive a Standard CIDH Pile Anomaly Mitigation plan when applicable. The Standard Plan should be incorporated into the Contractor’s Pile Mitigation Plan submittal to address the pile-specific Pile Mitigation Plan requirements of the contract specifications. The intent of the Standard CIDH Pile Anomaly Mitigation plans is to reduce the amount of time needed for the Contractor to develop the Pile Mitigation Plan and for the Engineer and DES Pile Mitigation Plan Review Committee to review and authorize the Pile Mitigation Plan.

To aid the Engineer, a copy of these Caltrans-authorized standard mitigation plans can be obtained by contacting Structure Construction in Sacramento or accessing the Intranet website.

Development and review of the Pile Mitigation Plan is a shared responsibility between the Engineer, the Contractor, the FTB and the DES Pile Mitigation Plan Review Committee.

9-11.7 Responsibilities of the Engineer

The Engineer is responsible for the following:

<table>
<thead>
<tr>
<th>ITEM</th>
<th>RESPONSIBILITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Arranging for acceptance testing with the FTB. Based on the results of acceptance testing, accept or reject the pile and notify the Contractor in writing and supply the Contractor with a copy of the test results.</td>
</tr>
<tr>
<td>2</td>
<td>Once the pile has been rejected, determine in consultation with the FTB whether additional acceptance testing will be performed. Typically the pile acceptance test report will indicate if Caltrans intends to perform any additional testing. If additional acceptance testing is performed, notify the Contractor in writing and supply the Contractor with a copy of the test results. The Contractor may perform their own additional testing.</td>
</tr>
<tr>
<td>3a</td>
<td>Use the Pile Design Data Form provided with the test results to determine whether the pile requires mitigation for structural, geotechnical, or corrosion reasons. If the pile requires</td>
</tr>
</tbody>
</table>

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28 2010 SS, Section 49-3.02A(3)(g), Mitigation Plans, or Special Provisions for contracts using 2006 SS.
29 [http://onramp.dot.ca.gov/hq/oscnet/FTB_Resources.htm](http://onramp.dot.ca.gov/hq/oscnet/FTB_Resources.htm)
mitigation, discuss with the CIDH Pile Mitigation Committee, the Designer, the Geoprofessional, and the Corrosion Engineer and come to a consensus on acceptable mitigation methodologies (Unearth and recast, pressure grout, structural bridging, supplement or replacement).

3b If the results of the discussion described in Item 3a determine that mitigation is not required, notify the Contractor in writing that mitigation is not required. Per the contract specifications, the Contractor can either mitigate the pile or accept an administrative deduction for the pile as described in the contract specifications.

4 If the pile requires non-standard mitigation (i.e. structural bridging, supplement or replacement), hold a CIDH Pile Non-Standard Mitigation meeting with the Contractor as described in the contract specifications and Bridge Construction Memo 130-21.0, CIDH Pile Non-Standard Mitigation Meeting.

5 Upon receipt of the Contractor’s Pile Mitigation Plan, review the plan to ensure that it includes all of the requirements listed in the contract specifications. If the plan does not include all of the requirements, return the plan to the Contractor for resubmittal. Once the Contractor submits a Pile Mitigation Plan that includes all of the requirements listed in the contract specifications, send the plan to the FTB and the DES Pile Mitigation Plan Review Committee for technical review.

6 Upon the recommendation of the FTB and the DES Pile Mitigation Plan Review Committee, either return the Pile Mitigation Plan to the Contractor for resubmittal or authorize the Pile Mitigation Plan.

9-11.8 Responsibilities of the Contractor
The Contractor is responsible for developing and submitting the Pile Mitigation Plan to the Engineer for review and authorization. The Contractor develops the plan using the acceptance testing results and the Pile Design Data forms provided by the Engineer, and in accordance with the outcome of the Non-Standard Mitigation Meeting (if applicable). Pile Mitigation Plans must contain the following information:

Table 9-3. Pile Mitigation Plan – Responsibilities of the Contractor.

<table>
<thead>
<tr>
<th>ITEM</th>
<th>REQUIREMENT &amp; REASONING</th>
</tr>
</thead>
</table>
| 1    | Designation and location of the rejected pile.  
      | **Reason:** Self-explanatory. |
| 2    | Review of the structural, geotechnical, and corrosion design requirements of the rejected pile.  
      | **Reason:** This information is provided to the Contractor by the Engineer via the Pile Design Data form. Requiring the Contractor to address the pile design requirements in the Pile Mitigation Plan ensures that the Contractor understands why the pile requires mitigation and that the proposed mitigation addresses the deficiencies in the pile to meet the design requirements. |
| 3    | Step-by-step description of the mitigation work to be performed, including drawings if necessary.  
      | **Reason:** This gives the Engineer advance knowledge of the means and methods the Contractor proposes to employ to mitigate the pile and to assess whether the proposed means and methods are sufficient to mitigate the deficiencies in the pile to meet the design requirements. |

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30 2010 SS, Section 49-3.02A(4)(d)(iv), Rejected Piles, or Special Provisions for contracts using 2006 SS.  
31 2010 SS, Section 49-3.02A(3)(g), Mitigation Plans, or Special Provisions for contracts using 2006 SS.
<table>
<thead>
<tr>
<th>ITEM</th>
<th>REQUIREMENT &amp; REASONING</th>
</tr>
</thead>
</table>
| 4    | Assessment of how the proposed mitigation work will address the structural, geotechnical, and corrosion design requirements of the rejected pile.  
**Reason:** This is an expansion on Item 2 above. Requiring the Contractor to address the pile design requirements in the Pile Mitigation Plan ensures that the Contractor understands why the pile requires mitigation and that the proposed mitigation addresses the deficiencies in the pile to meet the design requirements. |
| 5    | Methods for preservation or restoration of existing earthen materials.  
**Reason:** Some mitigation methods, such as Basic Repair, may disturb the soil around the pile. Disturbance of the soil may affect the skin friction load carrying capacity of the pile. This requirement ensures that the Contractor considers the effect of the proposed mitigation upon the skin friction load carrying capacity of the pile. |
| 6    | List of affected facilities. Include methods and equipment to be used for the protection of these facilities during mitigation.  
**Reason:** There may be existing facilities, such as utilities, around or above the pile. This requirement ensures that the Contractor takes these facilities into account when developing the proposed mitigation. |
| 7    | In accordance with the contract specifications  
For Shop Drawings, the State assigned contract number, bridge number, full name of the structure as shown on the contract plans, District-County-Route-Postmile Post, and the Contractor’s (and Subcontractor’s if applicable) name on each sheet.  
**Reason:** This requirement ensures that the Pile Mitigation Plan is developed specifically for the rejected pile in question and that all sheets of the plan can be identified and referenced for that specific pile. |
| 8    | List of materials with quantity estimates for the mitigation work and a list of personnel with qualifications who will be performing the mitigation work.  
**Reason:** This requirement ensures that the Contractor is aware of how much material to have on site when the mitigation work is performed. It also ensures that the Contractor uses personnel who have done mitigation work in the past and are familiar with the mitigation procedures proposed in the Pile Mitigation Plan. |
| 9    | The seal and signature of an engineer who is registered as a Civil Engineer in the State, if required by the contract specifications.  
**Reason:** This ensures that the Pile Mitigation Plan is developed or at least reviewed by a registered civil engineer. This is necessary to ensure that the structural, geotechnical, and corrosion design requirements of the rejected pile are met. |
| 10   | For rejected piles to be repaired, an assessment of the nature and size of the anomalies in the rejected pile.  
**Reason:** Using the information provided in the acceptance test report, the log of concrete placement and other available information, the Contractor should be able to assess the nature of the anomalies in the rejected pile. This is necessary to determine what type of repair method is appropriate. The size of the anomaly may determine whether a basic or grout repair is appropriate. The nature of the anomaly, be it an area of unconsolidated concrete within the pile or a soil inclusion at the side of the pile, may determine whether a basic or grout repair is appropriate. |
| 11   | For rejected piles to be repaired, provisions for access for additional pile testing, if requested.  
**Reason:** Generally, pile mitigation methods utilize the existing inspection pipes for accessing the defective zone. This is almost always the case for grout repairs. The repair generally renders the inspection pipe unusable for the purposes of additional acceptance testing. Should the pile require additional acceptance testing after mitigation work has been performed, the Contractor should be able to access the pile with the existing inspection pipes. |

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32 2010 SS, Section 5-1.23, *Submittals*, or Special Provisions for contracts using 2006 SS.  
33 2010 SS, Section 49-3.02A(3)(g), *Mitigation Plans*, or Special Provisions for contracts using 2006 SS.
performed; the Contractor must address how to preserve the existing inspection pipes or provide new access, usually new-cored holes.

<table>
<thead>
<tr>
<th>ITEM</th>
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</tr>
</thead>
</table>
| 12   | For rejected piles to be replaced or supplemented, the proposed location and size of additional piles.  
  **Reason:** When rejected piles have to be supplemented or replaced, the Contractor is responsible for the design of these additional piles. The Engineer has to evaluate the design impact of the location and size of additional piling on the structure being constructed and any existing facilities or new facilities to be constructed during the life of the contract. |
| 13   | For rejected piles to be replaced or supplemented, structural details and calculations for any modification to the structure to accommodate the replacement or supplemental piles.  
  **Reason:** See Item 12 above. |

**9-11.9 Responsibilities of the DES Pile Mitigation Plan Review Committee**

The DES Pile Mitigation Plan Review Committee is responsible for the following:

**Table 9-4. Pile Mitigation Plan–Responsibilities of the DES Pile Mitigation Plan Review Committee.**

<table>
<thead>
<tr>
<th>ITEM</th>
<th>RESPONSIBILITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Provide advice to the Engineer regarding pile mitigation procedures and methods.</td>
</tr>
<tr>
<td>2</td>
<td>Coordinate with the FTB to provide a technical review of the Pile Mitigation Plan submitted by the Engineer. Advise the Engineer in writing whether the Pile Mitigation Plan should be authorized or returned to the Contractor for correction and resubmittal.</td>
</tr>
<tr>
<td>3</td>
<td>Act as keeper of information for all CIDH piles constructed under slurry.</td>
</tr>
</tbody>
</table>

Once all responsibilities of completion and review of the Pile Mitigation Plan have been completed, the Engineer authorizes the Pile Mitigation Plan.

After authorization of the Pile Mitigation Plan, the Contractor can proceed with the work of mitigating the pile in the field.

**9-11.10 What to Expect in the Field During Pile Mitigation**

Personnel involved with the pile mitigation work and inspection of the pile mitigation work should be thoroughly familiar with the details of the authorized Pile Mitigation Plan. Evaluation of the acceptability of the pile mitigation work is dependent upon whether the procedures described in the authorized Pile Mitigation Plan were followed.

A good Pile Mitigation Plan will allow for alternatives should the initial procedure not work. For example, if it is determined during the pile mitigation work that replacement grouting is no longer appropriate because soil was encountered in the flushing discharge, the Pile Mitigation Plan should allow an alternative for permeation grouting. Occasionally, actual conditions in the field determine that grouting repair is no longer appropriate, and the whole mitigation effort may have to be abandoned and a revised Pile Mitigation Plan submitted for authorization.
For grouting repairs, the Contractor should monitor and record observations of inspection pipe removal, the nature of the discharge from the washing operation, the pressure and flow rate of water flow testing, photos or video from the down-hole camera, and the volumes and pressures of grout placement.

For grouting repairs, the Engineer should be present to monitor the results of inspection pipe removal, assessment of the defective area of the pile, all flushing operations, and any grouting repair work performed.

For basic repairs, the Engineer should be present to verify the Contractor only removes the soil for which removal has been authorized in the Pile Mitigation Plan. The Engineer should also verify the Contractor has removed all contaminated or deleterious materials from the defective area of the pile. Finally, the Engineer should verify the Contractor replaces the soil around the repaired pile as prescribed in the authorized Pile Mitigation Plan.

9-11.11 Procedures for Authorizing the Pile Mitigation Work Performed in the Field and Pile Acceptance

The authorized Pile Mitigation Plan addresses how the pile mitigation work will be accepted. Generally, the pile can be accepted if the mitigation work is performed in accordance with the provisions of the authorized Pile Mitigation Plan. However, there are circumstances when the pile must be retested. Procedures for access for retesting are provided in the authorized Pile Mitigation Plan.

For all types of pile mitigation, once the mitigation work is complete, the contract specifications require the Contractor to submit a Mitigation Report to the Engineer for review. The Mitigation Report should contain information on the Contractor’s observations recorded during the mitigation work, including grout volumes and pressures if a grouting repair was performed. It is especially important that any deviations from the authorized Pile Mitigation Plan be included in the Mitigation Report. This is necessary so the Engineer can determine whether the deviations resulted in an effective repair. The results of any retesting should also be included in the Mitigation Report.

Once the pile mitigation work is accepted, any remaining open inspection pipes are grouted and the pile can be accepted.

9-12 Safety

Safety concerns to be considered during the construction of CIDH piles by the slurry displacement method are similar to those to be considered when CIDH piles are constructed by ordinary means. For specific information, refer to Chapter 6, *Cast-in-
Drilled-Hole Piles. However, there is one additional item that requires further attention; and that is the drilling slurry.

Some of the components of drilling slurries, especially chemical additives, are considered hazardous materials. It is advisable to avoid skin contact and to avoid breathing in vapors. The Construction Safety Orders require the Contractor to provide Material Safety Data Sheets (MSDS) for all drilling slurries and chemical additives. The Engineer should obtain these MSDS as part of the submittal of the Pile Installation Plan. During the tailgate safety meeting prior to CIDH pile construction, be sure to discuss the contents of the MSDS and discuss how Caltrans employees, the Contractor’s employees, and any manufacturer’s representatives that may be present will adhere to the safety precautions.

During construction, do not permit the use of drilling slurries or chemical additives for which a MSDS has not been submitted.
CHAPTER 10

Pier Columns

10-1 Description

Pier columns are an extension of the column or pier into bedrock material and are usually the same size, or slightly larger, than the column or pier. They are ideally suited to canyons or hillside areas where there are limitations on the usual footing foundations; i.e., the need for approximately level topography and level underlying stratum. Footing foundations constructed in steep slopes are very costly because of the tremendous amount of excavation required.

Pier columns are primarily a Cast-In-Drilled-Hole (CIDH) pile, except the means of excavation is something other than the conventional drilling method. The following is taken from Caltrans Memo to Designers, Section 3-1 Deep Foundations, “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 contract plans. See Bridge Design Details page 7-20 and Bridge Design Aids Chapter 11 for details.”

See Appendix D, Pier Column & Type I Pile Shaft, for pay limits sketch.

As mentioned above, pier columns are primarily CIDH piles, but pier columns will have contract pay items for structure excavation and structure concrete. Pier columns can also be referred to as pile shafts. Caltrans outlines the design of pier columns in Bridge Design Aids (BDA), Chapter 12. Also, Federal Highway Administration (FHWA) has useful information on drilled shafts.
10-2 Specifications

Pier column information in the special provisions, contract plans, and the Standard Specifications should be reviewed prior to the start of work. The contract specifications state the concrete cementitious material content. The contract specifications also state the requirements for placing pier column concrete. Construction of pier columns is an excellent topic for the preconstruction conference, especially in regard to safety and excavation plans.

Almost all pier columns will have neat line excavation limits specified on the contract plans. Any excavation outside these neat lines must be filled with concrete. The Contractor should be reminded of this requirement prior to the start of work. It should also be pointed out that care must be used in constructing the access road and/or work area around the pier column(s) so that these excavations do not extend below the top of the neat line areas. The contract plans also specify no splice zones and ultimate splice zones for the main column reinforcement and for the main pile reinforcement. It is very important that the Contractor adheres to the rebar splice requirements.

10-3 Construction Methods

Methods and equipment used for construction of pier columns are dictated by several major factors. Among them is access to the work area, which is determined by the topography, and adjacent facilities such as existing structures, roads, and streambeds, and also by the type of equipment required to do the work. The cross sectional area of the pier shaft, depth of excavation, and the nature and stability of the material to be excavated are other major factors affecting the method and type of equipment to be used.

The above factors vary significantly from project to project. Hence, there is a wide variation in construction methods and equipment used by contractors on different projects. Methods that have been used in the past include using a hoe-ram, jackhammer, or Cryderman (“shaft mucker”). Others have used chemical rock splitting. The most common method used is blasting with explosives. Rotators and oscillators may also be used to perform this work. For additional information on these tools refer to Chapter 6, Cast-In-Drilled-Hole Piles.

10-4 Excavation

One of the first orders of work, after access roads are constructed to the pier column site, is to establish survey control points. These points should be placed so that they not only
provide control during excavation operations, but also can be used for pier column construction.

After establishing survey control points, excavation operations begin. Usually, soft material is excavated by conventional methods, such as a Gradall, flight auger, clam bucket, and hand work. Hard material encountered in otherwise soft material requires other means such as blasting. Since blasting is the most commonly used excavation method, it merits further discussion.

Typically, the first phase of a pier column excavation operation with blasting utilizes a line drill along the perimeter of the shaft to create holes along the neat line dimensions of the excavation (the Contractor may elect to line drill slightly outside the neat line dimension). A line drill is an air-track compressor type drill rig that uses 2-1/2 to 5 inch diameter drill bits in 20 foot lengths. The holes are usually drilled on 12 inch centers with additional holes placed inside the perimeter if needed. The holes are then blown out and filled with sand or pea gravel to facilitate blasting at different levels. Next, blasting mats, tires, dirt, etc. are placed to protect existing facilities from flyrock. A galvanometer should be used to check for shorts in the wiring prior to blasting. After the blasting is completed, the Contractor removes the loose material. Blasting and excavation usually occur in stages until reaching the bottom pier column elevation. Hand work to some degree is required at the bottom of all pier columns.

10-5 Problem Areas

Because of the wide range of variables associated with pier columns, different problems can be expected with each project. Listed below are items common to most projects. All represent potential problems that must be addressed in order to successfully install pier columns.

<table>
<thead>
<tr>
<th>ITEM</th>
<th>POTENTIAL PROBLEM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alignment</td>
<td>It can be difficult to maintain plumb drilled holes if extensive predrilling techniques are used. Consequently, the Contractor may elect to predrill the outside shaft dimensions.</td>
</tr>
<tr>
<td>Surveying</td>
<td>Be prepared to improvise. Access to the site and methods employed by the Contractor may require unique solutions. Work should be monitored as it progresses.</td>
</tr>
<tr>
<td>Access</td>
<td>The Contractor must provide safe access to the site and inside the pier. Depending on excavation depth, this could vary from ladders to boatswain’s chairs to suspended personnel cages to other means (review the Construction Safety Orders). Often this work will fall under Cal-OSHA’s Division of Mines and Tunnels.</td>
</tr>
<tr>
<td>Blasting</td>
<td>A thorough review of the Contractor’s blasting plan, if blasting is the option used to remove the bedrock material, is advised. Blasting should only be done by a licensed person with a Department of Industrial Safety (DIS) permit. This individual should</td>
</tr>
</tbody>
</table>
supervise placing, handling, blasting, and storage of explosive materials. Provisions must be made for handling traffic. Restrictions on the transportation of explosives must be enforced. Protection must be provided for existing facilities, utilities, etc. A galvanometer should be used to check for shorts in the wiring prior to blasting. Blasting mats, tires, dirt, etc. should be used to prevent flyrock from being scattered beyond expected limits. Proper warning signs should be provided along highways and roads near the blast site. No explosive material should be left in the area overnight. If it cannot be avoided, leave a guard overnight in the area. During the blast, guards should be placed at selected locations to prevent individuals from entering the blast area. Beware of “misfires.” In general, this operation is under the control of the licensed blaster. The Geoprofessional from Geotechnical Services should be consulted whenever blasting is contemplated. If you have any questions on the responsibility of Caltrans in regards to blasting, contact the Caltrans District Construction Safety Officer. Refer to Appendix D, Pier Column & Type I Pile Shaft, for sample blasting specifications.

Crane Safety

Lifting pier column rebar cages into the excavated hole may require more than one crane. Proper lifting plans must be enforced. Lane closures may be required when working next to traffic lanes. Additional safety precautions are required when working near overhead electrical lines and in windy areas.

Shoring

Shoring is required in all areas that are not solid rock. In almost all cases, special designs are required in accordance with the contract specifications. Shoring systems can consist of concrete lining, steel or concrete casing, box-type shields, rock bolts, and steel or timber lagging. Refer to the Caltrans Trenching and Shoring Manual for shoring design and details.

Geology

Be prepared for unanticipated ground conditions, such as soil instability, groundwater, fissures, or simply material of lesser quality than that assumed for design purposes. Revisions may be necessary.

Concrete

Common to all mined shafts is the requirement that concrete be placed against the undisturbed sides of the excavation. The length of shaft contact could vary from a planned length in the lower portion of the shaft to the entire length of the shaft. The special provisions for these projects will usually require a minimum side contact area (generally 50%) with certain allowances for shoring left in place or to allow for concrete flow through stay-in-place casings. In other instances the shoring or lagging has to be removed as the concrete is placed. These provisions tend to complicate concrete placing operations and therefore care must be exercised to do the job properly. Close inspection is mandatory.

10-6 Safety

Extreme caution is absolutely necessary in order to protect not only personnel working in the area, but the general public as well, since the potential for serious injury is ever present.

Safety railing and barriers must be erected near the shaft perimeter and adequate protection must be provided for personnel working inside the shaft. Workers must wear

3 2010 SS, Section 7-1.02K(6)(b), Excavation Safety, or 2006 SS, Section 5-1.02A, Excavation Safety Plans.
full body harness and be tied off when working adjacent to the shaft perimeter. Crane lifting plans may be required when erecting rebar cages and column forms. The contract specifications require a temporary support system for supporting column forms and column bar reinforcement. Material Safety Data Sheets (MSDS) are needed when slurries are used. Also, traffic handling plans and lanes closures may be required when constructing pier columns.

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4 2010 SS, Section 52-1.01C(3)(b), Temporary Support System, or Special Provisions for contracts using 2006 SS.
11-1 Introduction

Chapter 5, *Pile Foundations-General*, classifies ground anchors as special case foundations. Sub horizontal ground anchors, sometimes referred to as tiebacks, are used for earth retaining structures where it is not feasible to excavate and construct a footing foundation or pile cap for a conventional retaining wall. Vertical ground anchors, sometimes referred to as tension piles or tiedowns, are used generally for seismic retrofitting or existing footings where uplift and overturning must be prevented.

11-2 Sub Horizontal Ground Anchors

Sub horizontal ground anchors are used for temporary and permanent structures. The use of sub horizontal ground anchors with sheet pile or soldier beam shoring permits taller walls and deeper excavations than are possible with cantilever type construction—up to 35 feet or so depending on soil properties versus 15 feet for cantilever construction. Walls can be built much higher than 35 feet by using high strength sheet pile or soldier beams with multiple rows, or tiers, of sub horizontal ground anchors.

11-2.1 Components

Sub horizontal ground anchors are constructed by drilling holes at a slight angle (usually 15 degrees) off the horizontal axis. Afterwards a special prestressing system is installed and the tip portion, known as the bonded length, is grouted. The bonded length acts as an anchorage by distributing the prestressing force to the surrounding soil. The unbonded end is secured with an anchor head. Refer to Figure 11-1 for a sub horizontal ground anchor schematic. Refer to Bridge Standard Detail Sheet (XS) 12-040 for sub horizontal ground anchor details.

The following list describes various sub horizontal ground anchor components:

<table>
<thead>
<tr>
<th>COMPONENT</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestressing Steel – Support Member</td>
<td>This transfers load from the wall reaction to the anchor zone and is generally a prestress rod or strand.</td>
</tr>
<tr>
<td>COMPONENT</td>
<td>DESCRIPTION</td>
</tr>
<tr>
<td>-----------------</td>
<td>----------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Bond Length</td>
<td>The portion of prestressing steel fixed in the primary grout bulb through which load is transferred to the surrounding soil or rock. Also known as the anchor zone.</td>
</tr>
<tr>
<td>Unbonded Length</td>
<td>The portion of the prestressing steel that is free to elongate elastically and transmit the resisting force from the bond length to the wall face.</td>
</tr>
<tr>
<td>Anchorage</td>
<td>This consists of a plate and anchor head or threaded nut and permits stressing and lock-off of the prestressing steel.</td>
</tr>
<tr>
<td>Grout</td>
<td>This provides corrosion protection as well as the medium to transfer load from the prestressing steel to the soil or rock.</td>
</tr>
</tbody>
</table>

Figure 11-1. Sub Horizontal Ground Anchor Schematic.
In addition to enabling the construction of higher/taller walls and deeper excavations, subhorizontal ground anchors serve another useful purpose. The system provides an open unrestricted work area adjacent to the wall and inside the excavation since the only part of the system that projects beyond the wall is the relatively small anchorage device.

For permanent structures, the Contractor is responsible for providing a sub horizontal ground anchor system that conforms to the design requirements shown on the contract plans and meets or exceeds the testing requirements specified in the contract. Sub horizontal ground anchor shoring designs are often proprietary and require sophisticated engineering techniques and calculations submitted by the contractors and consultants. The designed bonded length is based on site specific soil parameters/mechanical properties. In accordance with the contract specifications, the Contractor submits sub horizontal ground anchor shop drawings and design calculations to Structure Design, Documents Unit, for distribution, review, and authorization, and notifies the Engineer of the submittal. The Designer, Geoprofessional, staff specialist for Earth Retaining Systems in Structure Policy and Innovation, the DES Prestressing Committee and Structure Construction field personnel all review the shop drawings. The Designer authorizes the shop drawings based on the recommendations of the individual units reviewing the drawings. These individuals and groups can be consulted for help in answering any questions that may arise in the field during construction. In addition, the Structure Construction Substructure Technical Team is also available to provide assistance.

A Ground Anchor Wall Construction Checklist is presented in Appendix K-6 to assist field personnel in preparing documents and inspecting fieldwork to ensure compliance with contract documents.

The contract specifications state the requirements for performance and proof testing of sub horizontal ground anchors. The record of readings from the performance and proof tests performed to verify the adequacy of the system shall be documented by the Contractor and provided to the Engineer. Structure Construction field personnel witness all performance and proof testing of the sub horizontal ground anchors.

### 11-2.2 Sequence of Construction

Sequence of sub horizontal ground anchor construction is as follows:

<table>
<thead>
<tr>
<th>SEQUENCE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Drill the holes to the required length and diameter.</td>
</tr>
<tr>
<td>2</td>
<td>Install the prestressing steel unit. (Strands or Bar)</td>
</tr>
<tr>
<td>3</td>
<td>Place primary grout.</td>
</tr>
<tr>
<td>4</td>
<td>Complete performance and proof tests (refer to section on testing later in this chapter).</td>
</tr>
</tbody>
</table>

---

1 2010 SS, Section 46-1.01C(2), Shop Drawings, or Special Provisions for contracts using 2006 SS.

2 2010 SS, Section 46-2.01D(2), Load Testing, or Special Provisions for contracts using 2006 SS.
### SEQUENCE | DESCRIPTION
--- | ---
5 | Lock-off and stress.
6 | Place secondary grout.

Note: Each step must comply with the contract specifications before proceeding to the next step.

#### 11-2.3 Safety

Check the Contractor’s construction sequence against the authorized shop drawings. As excavation proceeds from the top down, look for signs of failure in the lagging or changes in the soil strata.

Sub horizontal ground anchors systems use powerful hydraulic rams to prestress or post-tension the system. Safety concerns are similar to those encountered with other prestressing operations. Structure Construction employees should not stand behind the hydraulic ram or cross it while stressing is taking place. The *Prestress Manual* and the *SC Code of Safe Practices* should be consulted for additional safety considerations.

#### 11-3 Vertical Ground Anchors

Vertical ground anchors are similar to sub horizontal ground anchors although they act in the vertical plane. They can be used where site conditions do not allow traditional piles to achieve 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 sufficient skin friction to resist uplift or tensile loads required by the design. Vertical ground anchors are especially 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.

A schematic of a prestressing bar vertical ground anchor for a retrofit application is shown in Figure 11-2. Refer to *Bridge Standard Detail Sheets (XS) 12-030-1 and 12-030-2* for vertical ground anchor details.

The Contractor is responsible for providing the vertical ground anchor system that conforms to the design requirements shown on the contract plans and the testing requirements specified in the contract. After selecting a vertical ground anchor system, the Contractor sends the shop drawings and calculations to Structure Design, Documents Unit, for distribution, review, and authorization similar to the process outlined above for sub horizontal ground anchors.

The record of readings from the performance and proof tests must be documented by the Contractor and provided to the Engineer. Structure Construction field personnel witness all performance and proof testing of the vertical ground anchors.
Figure 11-2. Vertical Ground Anchor Schematic.
11-3.1 Sequence of Construction

Sequence of vertical ground anchor construction is as follows:

<table>
<thead>
<tr>
<th>SEQUENCE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Drill the hole the required depth and diameter.</td>
</tr>
<tr>
<td>2</td>
<td>Install the prestressing strands or bar.</td>
</tr>
<tr>
<td>3</td>
<td>Place primary grout.</td>
</tr>
<tr>
<td>4</td>
<td>Complete performance and proof tests (refer to section on testing later in this chapter).</td>
</tr>
<tr>
<td>5</td>
<td>Lock-off and stress.</td>
</tr>
<tr>
<td>6</td>
<td>Place secondary grout.</td>
</tr>
</tbody>
</table>

Note: Each step must comply with the contract specifications before proceeding to the next step.

11-4 Testing of Ground Anchors

Ground anchors require testing of the in-place anchors. Performance tests are done on a predetermined number of anchors, and proof tests are required on all of the remaining anchors. If the test results indicate that the anchors are not achieving capacity, additional monitoring and testing is required. If they do not pass at that point, a revision to the original design will be required. The redesign should be discussed with the Designer. The specific requirements for testing are provided in the contract specifications, the following is a general explanation of the required tests.

11-4.1 Performance Tests

A performance test involves incremental loading and unloading of a production anchor to accurately verify that the design loads will be safely carried by the system, that there is sufficient free length to allow for elastic elongation, and the residual movement of the anchor after stressing is within tolerable limits. Unless otherwise specified in the contract, at a minimum, performance tests are required for at least two but not less than 10% of ground anchors in footings, and at least three but not less than 5% of ground anchors in walls. Do not wait until many ground anchors have been installed before performance testing is conducted as the purpose of these tests is to verify the installation procedure selected by the Contractor. It is in the best interest of both parties to begin testing early and before a large number of anchors have been installed. The contract specifications state the testing and acceptance criteria for each anchor that is performance tested and the number of performance tests required at each location.

A proof test involves incrementally loading a production anchor to verify the design capacity can be safely carried and that the free length is as specified. The proof test is a

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3 2010 SS, Section 46-2.01D(2), Load Testing, or Special Provisions for contracts using 2006 SS.
single cycle test where the load is applied in increments until the specified maximum load value is reached. The contract specifications state the testing and acceptance criteria for each anchor that is proof tested.

### 11-4.2 General Acceptance Criteria – Performance & Proof Tests

**Table 11-4. Acceptance Criteria – Performance Tests.**

<table>
<thead>
<tr>
<th>CRITERIA</th>
<th>PERFORMANCE TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Achieve test results that indicate the anchor is capable of supporting 100% of the factored test load for the anchor shown on the contract plans.</td>
</tr>
<tr>
<td>2</td>
<td>The measured elastic movement exceeds 80% of the theoretical elongation of the unbonded length plus the jacking length at the maximum test load.</td>
</tr>
<tr>
<td>3</td>
<td>The creep movement between one and 10 minutes is less than 0.04 inch.</td>
</tr>
</tbody>
</table>

**Table 11-5. Acceptance Criteria – Proof Tests.**

<table>
<thead>
<tr>
<th>CRITERIA</th>
<th>PROOF TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Achieve test results that indicate that the anchor is capable of supporting 100% of the factored test load for the anchor shown on the contract plans.</td>
</tr>
<tr>
<td>2</td>
<td>The pattern of movement is similar to that of adjacent performance tested ground anchors.</td>
</tr>
<tr>
<td>3</td>
<td>The creep movement between one and 10 minutes is less than 0.04 inch.</td>
</tr>
</tbody>
</table>

The contract specifications outline acceptance criteria for these tests, however a performance tested or proof tested ground anchor which fails to meet the third criterion will be acceptable if the maximum load is held for 60 minutes and the creep curve plotted from the movement data indicates a creep rate of less than 0.08 inch for the last log cycle of time between 6 and 60 minutes.

### 11-4.3 General Construction Control

**Table 11-6. Ground Anchor General Construction Tasks.**

<table>
<thead>
<tr>
<th>ITEM</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mill certification should be provided for the steel tendons.</td>
</tr>
<tr>
<td></td>
<td>a) Check the steel for damage.</td>
</tr>
<tr>
<td></td>
<td>b) Ensure that grease completely fills the free length plastic tube.</td>
</tr>
<tr>
<td></td>
<td>c) Securely tape the bottom of the free length.</td>
</tr>
<tr>
<td></td>
<td>d) Compare the actual free length dimensions versus the dimension specified.</td>
</tr>
<tr>
<td>2</td>
<td>Double corrosion protection anchors should be completely fabricated before being delivered to the project. Bar anchors are installed full-length into the hole. Record the actual free and bond length for each installed anchor.</td>
</tr>
<tr>
<td>3</td>
<td>Tendons must be equipped with centralizers. These centralizer devices are absolutely necessary to center the tendon in the hole and to prevent the tendon from laying on the side of the drilled hole where incomplete grout cover will cause loss of capacity and future corrosion.</td>
</tr>
</tbody>
</table>

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4 2010 SS, Section 46-2.01D(2), Load Testing, or Special Provisions for contracts using 2006 SS.
5 2010 SS, Section 46-2.01D(2)(c), Acceptance Criteria, or Special Provisions for contracts using 2006 SS.
Grout tubes are frequently tied to the tendon before inserting in the hole. This helps to ensure that there are no voids in the grout.

Testing—check to ensure the tendon is concentrically located in the center hole of the jack and load cell before testing begins. Poor alignment of the testing apparatus will cause eccentric loading on the load cell and jack, which will give erroneous readings. Deflections at the anchor head should be measured with a dial gauge.

### 11-5 Soil Nails

Soil nailing is a technique used to reinforce and strengthen an existing embankment (Figure 11-3). It can also be used to reinforce excavations to allow steeper cuts and/or deeper excavations. The fundamental concept is that soil can be effectively reinforced by installing closely spaced grouted steel bars, or “nails”, into a slope or excavation as construction proceeds from the original ground to the bottom of the excavation or from the top down. Unlike ground anchors, the soil nail bars are not tensioned when they are installed and are grouted along the entire length of the nail. They are forced into tension as the ground deforms laterally in response to the loss of support caused by the excavation. The grouted nails increase the shear strength of the overall soil mass and limit displacement during and after excavation. Soil nails are bonded along their full length and are not constructed with a permanent unbonded length, as are ground anchors. A typical soil nail is shown in Figure 11-4.

Soil nailing is a cost-effective alternative to conventional retaining wall structures for most soils. However they are not practical in loose materials or plastic soils.

Common soil nail wall applications include the following:

<table>
<thead>
<tr>
<th>APPLICATION</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Temporary and permanent walls for excavations.</td>
</tr>
<tr>
<td>2</td>
<td>Cut slope retention for roadway widening and depressed roadways.</td>
</tr>
<tr>
<td>3</td>
<td>Bridge abutments – addition of traffic lanes by removing end slopes from in front of existing bridge abutments.</td>
</tr>
<tr>
<td>4</td>
<td>Slope stabilization.</td>
</tr>
<tr>
<td>5</td>
<td>Repair or reconstruction of existing structures.</td>
</tr>
</tbody>
</table>

Soil nail wall construction is sensitive to ground conditions, construction methods, equipment, and excavation sequencing. For soil nail walls to be most economical, they should be constructed in ground that can stand unsupported on a vertical or steep slope cut of 3 to 6 feet for at least one to two days, and can maintain an open drilled hole for at least several hours.
Figure 11-3. Soil Nail Schematic.
Figure 11-4. Soil Nail Detail.
11-5.1 Sequence of Construction
Soil Nail Wall construction sequence is as follows:

Table 11-8. Soil Nail Construction Sequence.

<table>
<thead>
<tr>
<th>SEQUENCE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Excavate a vertical cut to the elevation of the soil nails.</td>
</tr>
<tr>
<td>2</td>
<td>Drill the hole for the nail.</td>
</tr>
<tr>
<td>3</td>
<td>Install and grout the soil nail tendon.</td>
</tr>
<tr>
<td>4</td>
<td>Place the geocomposite drain strips, the initial shotcrete layer, and install the bearing plates and nuts.</td>
</tr>
<tr>
<td>5</td>
<td>Repeat process to final grade.</td>
</tr>
<tr>
<td>6</td>
<td>Place the final facing (for permanent walls).</td>
</tr>
</tbody>
</table>

11-5.2 Engineer’s Responsibility
The Structure Representative must ensure the soil nail wall is being built in accordance with the contract. Caltrans is responsible for reviewing and authorizing the shop drawings and construction details. The review process is similar to that of ground anchors. One important difference between ground anchor designs and those of soil nails is that of design responsibility. Ground anchors have a grouted length that is designed or determined by the Contractor while soil nail walls do not; they are grouted full length.

Prior to construction, the planned alignment, depth, and layout of the soil nails must be checked in the field for any possible discrepancies. As with any work involving soils or rock, good daily diaries and records must be maintained for all field activities.


11-5.3 Contractor’s Responsibility
The Contractor is responsible for constructing the soil nail wall in accordance with the contract. The Contractor is also responsible for submitting complete details of the materials, procedures, sequences, and proposed equipment to be used for constructing the soil nail assemblies and for constructing and testing the test soil nail assemblies. The Contractor must furnish a complete test result to the Engineer for each soil nail assembly tested.

11-5.4 Testing of Soil Nail Walls–Verification, Proof, and Supplemental
The contract specifications should be consulted for the specific test requirements for your project. Testing involves stressing the nails to simulate design load conditions. The following is a general description of the required tests.

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6 2010 SS, Section 46-3.01D(2), Load Testing, or Special Provisions for contracts using 2006 SS.
11-5.4.1 Verification Nails

Verification nails, sometimes referred to as test nails, are not production nails and are meant to be “sacrificial”. They are installed in the same manner as production nails but have an area that is not grouted or bonded. Verification tests should be performed before excavation is continued below the level of the test nail. Once the test is performed, the remainder of the drilled hole is filled with grout. The location of test nails is determined by the Designer and shown on the contract plans. Refer to Figure 11-5 for a test nail detail.

The contract specifications state verification testing is performed to test the soil nail for creep and maximum load. The test involves incrementally loading the test soil nail assembly to the test load, holding it for an hour and then loading the nail to 150% of the test load. Movement of the soil nail end is measured and recorded to the nearest 0.001 inch at each increment of load, including the ending alignment load, relative to an independent fixed reference point. The contract specifications outline acceptance criteria for the verification test nails. The nails need to fulfill these criteria before moving forward with construction of the rest of the wall. Should the nails not meet the criteria, additional tests may be necessary. The nails may fail due to constructability issues or insufficient length. In either case, additional verification tests will be required. The Contractor will need to provide a Log of Test Borings of the material removed from the holes for the additional verification test nails. This information should be provided to the Designer and Geoprofessional to help resolve this issue and determine whether the Contractor’s means and methods are the cause of test nail failure or if the soil nails require redesign.

7 2010 SS, Section 46-3.01D(2)(b)(ii), Verification Test, or Special Provisions for contracts using 2006 SS.
8 2010 SS, Section 46-3.01D(2)(c), Acceptance Criteria, or Special Provisions for contracts using 2006 SS.
Figure 11-5. Verification/Test Nail Detail.
11-5.4.2 Proof Testing
Proof testing is performed on production nails that are shown on the contract plans and is used to measure creep. The contract plans indicate a specific number of proof tests to be performed at locations shown. The proof testing loading schedule, as well as the acceptance criteria for proof tests, are different than those for verification tests and are outlined in the contract specifications.

11-5.4.3 Supplemental Testing
Supplemental testing is done on a specified number of soil nails designated for proof testing (up to one-half of the proof-tested soil nails) and is performed immediately after the completion of proof testing. Supplemental testing is used to ensure pullout failure does not occur. The supplemental testing loading schedule and acceptance criteria are outlined in the contract specifications.

11-5.5 Safety
The soil nail wall should be monitored during construction for movement and for signs of failure. Occasionally, poor material will be encountered as the excavation continues downward. This differing condition may require a change to the contract plans or safety provisions in the construction method.

Personnel working around soil nail operations must wear the required Personal Protection Equipment (PPE) to include eye protection and ear plugs.

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9 2010 SS, Sections 46-3.01D(2)(b)(iii), Proof Test; 46-3.01D(2)(c), Acceptance Criteria; or Special Provisions for contracts using 2006 SS.
10 2010 SS, Sections 46-3.01D(2)(b)(iv), Supplemental Test; 46-3.01D(2)(c), Acceptance Criteria; or Special Provisions for contracts using 2006 SS.
12-1 General

A cofferdam is a retaining structure, usually temporary in nature, which is used to retain water and support the sides of excavations where water is present. These structures generally consist of: (1) vertical sheet piling, (2) a bracing system composed of wales, struts, or tiebacks, and (3) a bottom seal course to keep water from piping up into the excavation or to prevent heave in the soil. Cofferdams differ from braced excavations or shoring in that they are designed to control the intrusion of water from a waterway and/or the ground.

A seal course is a concrete slab poured under tremie to block the intrusion of water into the bottom of an excavation. The limits of the cofferdam are the limits of the seal course and the thickness is calculated to address engineering considerations such as pressures from differential hydrostatic head at the bottom of footing elevation.

12-2 Sheet Piles and Bracing

There are three basic materials used for the construction of sheet piles: wood, concrete, and steel. Wood sheet piling can consist of a single line of boards or “single-sheet piling” (Figure 12-1), but it is suitable for only comparatively small excavations where there is no serious ground water problem.

![Figure 12-1. Single Sheet Piling.](image)

In saturated soils, particularly in sands and gravels, it is necessary to use a more elaborate form of sheet piling which can be made reasonably watertight with overlapping boards spiked or bolted together, such as the “lapped-sheet piling” or “Wakefield” system (Figure 12-2).
“Tongue and groove” sheet piling (Figure 12-3) is also used. This is made from a single piece of timber that is cut at the mill with a tongue and groove shape.

Precast concrete sheet piles (Figure 12-4) are normally used in situations where these members are going to be incorporated into the final structure or are going to remain in place after they fulfill their purpose. Caltrans does not normally encounter precast concrete sheet piling in structure work. However, it is usually made in the form of a tongue and groove section. They vary in width from 18 to 24 inches and in thickness from 8 to 24 inches. They are reinforced with vertical reinforcing steel bars and hoops in much the same way that is done with precast concrete bearing piles. This type of sheeting is not perfectly watertight; however the spaces between the piles can be grouted to try to address this.

In order to provide a more watertight precast concrete sheet pile, two halves of a straight steel web sheet pile, which has been split in half longitudinally, are cast into the concrete pile during fabrication (Figure 12-5).
Steel sheet piling is most commonly used in the field. It is available in a number of different sizes and shapes. The shape provides bending strength and each end is fabricated with an interlock (connection between sheets) that provides alignment and interconnectivity between sheets. Each steel company that manufactures sheet piling has its own shape and form of interlock. The simplest shape is known as the “straight-web” (Figure 12-6). These are made in various widths ranging from about 15 to 20 inches. The web thickness varies from about 3/8 to 1/2 inch. The straight-web sheet piling is comparatively flexible and it requires a considerable amount of bracing in deeper excavations where lateral loads from waterways and soils are large.

![Figure 12-6. Straight-Web Steel Sheet Piling.](image1)

In order to provide greater resistance to bending, the steel companies have developed sheet piles in a variety of shapes. One type is known as the “arch-web” section (Figure 12-7), where the center of the sheet is offset to provide a greater moment of inertia in the cross section. A “deep-arch” section (Figure 12-8) provides an even greater stiffness. It is similar to the “arch-web” except that the offset in the web is considerably larger. A third type, known as the “Z-Section” (Figure 12-9) has considerably greater stiffness than that of the “deep-arch” and is used in deeper excavations.

![Figure 12-7. Arch-Web Steel Sheet Piling.](image2)

![Figure 12-8. Deep-Arch Steel Sheet Piling.](image3)
The choice of the type of steel sheet pile to be used on a given project depends largely on the kind of service in which it will be used. The straight-web is comparatively flexible so it requires a considerable amount of bracing to resist large lateral loads in excavations. However, its cross section allows it to be used in locations where space is an issue and where a deep-arch or Z-Section will not fit in between the excavation limits and an obstruction or Right-of-Way line.

The composition of the bracing system inside the cofferdam will depend upon the forces that system must resist, the availability of materials, and the costs connected with the system. Ground anchors can be used in large land cofferdams where a system of cross bracing is impractical.

12-3 Excavation

Cofferdams in waterways are typically excavated with a submerged clamshell bucket, with the excavation elevations being checked by sounding. In the case of pile foundations, it is often advisable to over-excavate a predetermined amount to compensate for possible heave of the foundation material caused by driving piles; displacement piles in particular. This is done to eliminate the need for excavation after driving. If excavation is needed, care needs to be taken so as not to damage any of the driven piles.

To ensure the stability of the excavation, a seal course is used to control the influx of water into the excavation from the bottom due to hydrostatic head differentials. The contract plans show where seal courses are required. As in many other areas of our work, there are times when engineering judgment should be used to make decisions. Depending on the types of soils and the depth of the excavation in relation to that of the water table, the cofferdam may be dewatered without constructing a seal course while still allowing construction of the footing in the dry. The decision to use a seal course that is shown on the contract plans, or to revise its thickness, is the responsibility of the Engineer. Discussions about the need for a seal course or revisions to thickness need to take place early so that design considerations for the cofferdam can be addressed.
Seal courses for cofferdams might not be shown on the contract plans but may be needed to facilitate construction and provide a quality product. If a seal course is not shown on the contract plans and the Contractor elects to use one to control and remove water from the excavation, the work must be done in accordance with the provisions of the contract specifications.

12-4 Seal Course

The contract specifications state that a seal course should be used when the Engineer determines it is impossible or inadvisable to dewater an excavation prior to pouring concrete. As the name implies, a seal course seals the entire bottom of a cofferdam and prevents subsurface water from entering the cofferdam. It also controls the expansion of soils that have a tendency to expand or heave. Sealing the bottom of the cofferdam allows cofferdams to be dewatered and permits the construction of footings, columns or other facilities in the dry. The seal course is a concrete slab placed underwater by the tremie placement method and is constructed thick enough so that its weight is sufficient to resist uplift from hydrostatic forces. The friction bond between the seal course concrete, the cofferdam, and piles if present, also helps resist uplift. A seal course is a construction tool and in terms of importance to the designed structure it has no structural significance.

Following the installation of the cofferdam and prior to dewatering, the soil is excavated to the elevation of the bottom of the seal course, and the piles are driven. The seal course is poured under tremie and allowed to cure. The cofferdam is dewatered after the seal course has cured. A small area of the seal course can be left low for the placement of a pump to remove water that seeps into the excavation prior to the placement of footing concrete.

Information about seal courses for a project can be found in the contract plans. Additional information may be found in the Foundation Report or RE Pending File. As previously discussed, when seal courses are shown on the contract plans, the decision about the need for the seal course and its thickness rests with the Engineer. This decision is based on conditions encountered on the jobsite. The contract specifications also contain provisions for adjusting excavation item quantities if seal courses are adjusted or eliminated. Additional information about seal courses can be found in Bridge Construction Memo 130-22.0, Seal Courses. Bottom of footing elevations should not be revised as a result of eliminating or revising seal courses unless shown on the contract plans or addressed in the special provisions.

1 2010 SS, Section 19-3.03D, Water Control and Foundation Treatment, or 2006 SS, Section 19-3.04, Water Control and Foundation Treatment.
2 2010 SS, Section 51-1.03D(3), Concrete Placed Under Water, or 2006 SS, Section 51-1.10, Concrete Deposited Under Water.
3 2010 SS, Section 19-3.04, Payment, or 2006 SS, Section 19-3.07, Measurement.
12-4.1 Concrete Deposited Underwater (Tremie Placement Method)
The Tremie Placement Method is a name given to the method of placing concrete underwater through a pipe or tube, known as a tremie, or with a concrete pump. The tremie can either be rigid or flexible. The purpose of the tremie is to enable continuous placement of concrete, monolithically, underwater without creating turbulence. Essentially the water is displaced by a slowly moving concrete mass.

To accomplish this, it is imperative that the discharge end of the tremie be kept embedded in the concrete. It is also imperative that the concrete have good flow characteristics. Concrete placement can be accomplished by either a tremie supported and maneuvered by a crane or the discharge end of a concrete pump. Frequently contractors will use multiple-tremie systems with each hopper supported by bracing or walkways in the cofferdam. In this case, tremie spacing is controlled by the flow characteristics of the concrete.

A typical tremie operation begins with the tremie pipe being lowered into position with a plug or other device fitted into the pipe as a physical barrier between the water and concrete. Concrete is charged into the pipe to a sufficient height to permit gravity flow. The flow itself is started by slightly lifting the pipe. Once started, the concrete flow must be continuously maintained through the pipe. The operation continues until completion. The tremie pipe remains immersed in concrete during placement. Some factors that ensure success for this operation are:

Table 12-1. Seal Course Tremie Concrete Placement.

<table>
<thead>
<tr>
<th>FACTOR</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Tremie concrete must have a slump of between 6 and 8 inches, per the contract specifications.</td>
</tr>
<tr>
<td>2</td>
<td>Concrete must contain a minimum of 675 pounds of cementitious material per cubic yard, per the contract specifications.</td>
</tr>
<tr>
<td>3</td>
<td>Concrete placement and the maneuvering of the tremie pipe must be done smoothly and deliberately.</td>
</tr>
<tr>
<td>4</td>
<td>Concrete delivery must be adequate and timely.</td>
</tr>
<tr>
<td>5</td>
<td>The concrete mix design should be geared to good flow characteristics.</td>
</tr>
</tbody>
</table>

12-4.2 Seal Course Inspection
A Cofferdam and Seal Course Construction Checklist is presented in Appendix K-3 to assist field personnel in preparing documents and inspecting fieldwork to ensure compliance with contract requirements.

In addition to the usual concrete placement requirements, such as access and suitability or adequacy of equipment, sufficient soundings of the bottom of the excavation should be

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4 2010 SS, Section 90-1.02G(6), Quantity of Water and Penetration or Slump, or 2006 SS, Section 90-6.06, Amount of Water and Penetration.
5 2010 SS, Section 51-1.02B, Concrete, or 2006 SS, Section 90-1.01, Description.
taken to verify as-built elevations so that deficiencies can be addressed. Particular care should be given to the perimeter of the cofferdam and the pile locations, as excavation is somewhat difficult in these areas. If not completely excavated, ground elevations in these areas will be higher than those in easier to reach areas, which will result in a thinner than anticipated seal course. Soundings can be accomplished using a flat plate of suitable size and weight on the end of a rod or rag tape.

Sounding devices can also be used to determine the nature of the material (soft or firm). During the pour, soundings are again used to verify the elevation of the top surface of concrete. Because of the type of operation, surface irregularities can be expected, particularly in pile footings. The important thing is to check for proper thicknesses throughout and to address any excessively low spots.

Of the various devices available to plug the end of the tremie, an inflated rubber ball is about the most practical. A tip plug can cause long tremie pipes to float and should be used with caution.

12-4.3 Thickness of Seal Course
A chart for determining the thickness of seal courses is included in Appendix I. Certain safeguards or safety factors are built into this chart. For example, seal courses in pile footings are constructed one foot thicker than required to allow for surface irregularities and the bond friction between sheet piling and concrete is disregarded. The bond friction between seal course concrete and foundation piles is limited to 10 pounds per square inch (PSI). Minimum thickness of seal course concrete is 2 feet. This subject is also covered in Bridge Construction Memo 130-22.0, Seal Course and Bridge Design Aids, Seal Course included in Appendix I, Cofferdams and Seal Courses.

12-5 Contractor’s Responsibility
Cofferdams fall under the category of temporary features or measures necessary to construct the work. As such, the Contractor is responsible for the proper design, construction, maintenance, and removal of cofferdams. The Contractor is required to submit shop drawings and calculations to the Engineer for authorization in accordance with the contract specifications. The Contractor is also required to comply with the applicable sections of the Construction Safety Orders (Sections 1539-1543) and the provisions of Section 6705 of the California Labor Code. Refer to the Trenching and Shoring Manual for additional information on braced or shored excavations.

The Contractor has the option of constructing a seal course to control water when one is not shown on the contract plans. In these situations the Contractor is responsible for determining the thickness and the performance of the seal course. The seal course

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6 2010 SS, Section 5-1.23B(2), Shop Drawings, or 2006 SS, Section 5-1.02, Plans and Working Drawings.
thickness and curing requirements in the contract specifications do not apply to optional contractor-designed seal courses. However, the successful performance of the seal course, if used, will be solely the responsibility of the Contractor.

12-6 Engineer’s Responsibility

The Engineer is responsible for performing an independent analysis, or check, of the Contractor’s cofferdam and for authorizing the shop drawings. In situations where a seal course is shown on the contract plans, the Engineer is responsible for making the decision as to whether or not a seal course is needed.

The Engineer should be familiar with the applicable information in the contract specifications and Bridge Construction Memos 2-9.0, Footing and Seal Course Revisions, and 130-22.0, Seal Courses.

12-7 Dewatering

The contract specifications require a minimum cure period of 5 days (at concrete temperatures of 45° F or more) before dewatering may begin. Dewatering can present some anxious moments since the cofferdam and the seal course will be put to the test.

Dewatering is sometimes conducted in stages, particularly for a deeper cofferdam. Intermediate bracing systems may need installed before proceeding deeper. Depending on the particular design, these internal braces maintain the stability of the system. Details of dewatering and internal bracing placement should be included in the cofferdam plans. A review of contract specifications for water pollution control should be made before dewatering operations start.

Sheet pilings are not watertight and minor leaks can be expected as the cofferdam is dewatered. These leaks are ordinarily not a problem and occur along the joints between adjacent sheets. Sawdust, cement, or other material can be used to plug these types of leaks. Dropping the material into the water adjacent to the leaking sheets usually corrects

7 2010 SS, Section 19-3.03(D), Water Control and Foundation Treatment, or 2006 SS, Section 19-3.04, Water Control and Foundation Treatment.
9 2010 SS, Section 51-1.03D(3), Concrete Placed Under Water, or 2006 SS, Section 51-1.10, Concrete Deposited Under Water.
10 2010 SS, Section 13-4.03G, Dewatering, or Special Provisions for contracts using 2006 SS.
this as the flow through the leak carries the fine material to the problem area and seals the crack or opening. A sump built into the surface of the seal outside of the footing limits is also helpful in keeping the work area reasonably dry.

Prior to proceeding with footing work, all high spots in the seal course have to be removed. All scum, laitance, and sediment must also be removed from the top of the seal. This work can be very time consuming and expensive. It can be reduced significantly if care is taken during the placement of the seal course.

12-8 Safety

Cofferdam work presents safety problems similar to braced excavations. Among them are limited access, limited work areas, damp or wet footing, and deep excavations. Provisions must be made for safe access and egress in terms of adequate walkways, rails, ladders, or stairs into and out of the lower levels. The *Trenching and Shoring Manual* goes into those issues in depth and should be consulted prior to working around cofferdams.

Additional considerations apply to cofferdams, as they tend to occur within a waterway, in which case additional safety regulations may apply. These include provisions for flotation devices, boats, warning signals, and suitable means for a rapid exit. The *Construction Safety Orders* and project-specific *Code of Safe Practices* should be consulted for specific requirements.
13 Micro piles

13-1 Introduction

This chapter provides information on the design applications and construction of micro piles on Caltrans projects.

13-2 Micro piles


13-2.1 Micropile Definition and Description

A micropile is a small-diameter (typically less than 12 inches), drilled and grouted replacement pile that is typically reinforced. A replacement pile is one that is placed or constructed within a previously drilled borehole, thus replacing the excavated soil. A micropile is constructed by drilling a borehole, placing reinforcement, and grouting the hole. Micropile construction uses similar equipment and techniques as those used for ground anchors and soil nails as described in Chapter 11, Ground Anchors & Soil Nails. Many contractors who specialize in drilling and grouting ground anchors and soil nails also construct micropiles. Micropiles are also known as root piles, pin piles, needle piles, and minipiles.

Micropiles can withstand axial compression and tension loads and some lateral loads. Depending upon the design concept employed, micropiles may be a substitute for conventional piles or as one component in a composite soil-pile interaction mass. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, soil, and the environment. They can be installed in access-restrictive environments and in all soil types and ground conditions. Since micropiles provide little lateral resistance, their use on Caltrans projects has been limited to retrofit work and for the construction of retaining and sound walls.

Since the installation procedure causes minimal vibration and noise and can be used in conditions of low headroom, micropiles are often used to underpin existing structures.
Underpinning is the process of strengthening and stabilizing the foundation of an existing structure. It is accomplished by extending the foundation in depth or in breadth so it either rests on a stronger soil stratum, or distributes its load across a greater area. Specialized drilling equipment is often required to install the micropiles from within existing basement facilities or through existing bridge footings.

Most of the applied load on conventional cast-in-place replacement piles is structurally resisted by the reinforced concrete; increased structural capacity is achieved by increased cross-sectional and surface areas. Micropile structural capacities, by comparison, rely on high-capacity steel elements to resist most or the entire applied load. The special drilling and grouting methods used in micropile installation allow for high grout-ground bond values along the grout-ground interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors. Due to the small pile diameter, any end bearing contribution in micropiles is generally neglected. The grout-ground bond strength achieved is influenced primarily by the ground type and grouting method used, i.e., pressure grouting or gravity feed. The role of the drilling method is also influential, although less well quantified.

13-2.2 Applications
Micropiles are most commonly used in two general applications, (1) structural support, where micropiles are loaded directly and where micropile reinforcement resists the majority of the applied load and (2) in-situ reinforcement, where micropile elements circumscribe and internally reinforce the soil to theoretically make a reinforced soil composite that resists applied loads.

Structural Support includes:
- Foundations for new structures.
- Seismic retrofitting.
- Underpinning of existing foundations.

In-situ reinforcement includes:
- Slope stabilization and earth retention.
- Ground strengthening and protection.
- Settlement reduction.
- Structural stability.

Micropiles were originally developed for underpinning existing structures. The underpinning of existing structures may be performed for many purposes:
- To arrest and prevent structural movement.
- To upgrade load-bearing capacity of existing structures.
- To repair or replace deteriorating or inadequate foundations.
- To add scour protection for erosion-sensitive foundations.
- To raise settled foundations to their original elevation.
- To transfer loads to a deeper strata.
13-2.3 Caltrans Applications
AASHTO added a section on micropiles to their design specifications in 2010 (5th edition). But while the rest of the country sees the value, Caltrans limits the use of micropiles due to the lateral demand requirements. The lateral load capacity of micropiles is small because their size is too small to develop any real bending moments. Micropiles can resist lateral load, but not that much. A large quantity of micropiles would be required, too many of them, to resist a significant amount of lateral load.

Caltrans is currently using micropiles for seismic retrofits, earth retention, and foundations for new structures, but mostly retaining and sound walls. Micropiles have also been utilized recently in a Caltrans project for the foundation support of the arch bridge struts for the Spanish Creek Bridge located in Plumas County. (Bridge No. 09-0077, Contract EA 02-373104, 02-Plu-70 KP 56.5/57.2). Refer to Appendix J, Micropiles, for a case study.

13-2.4 Seismic Retrofit
Caltrans has used micropiles for seismic retrofitting of existing highway bridge structures. The existing bridge foundations are retrofitted to increase the capacity to resist tension/uplift forces resulting from a seismic event.

A recent Caltrans retrofit project using micropiles was at the Richmond San Rafael Bridge located in the San Francisco Bay Area (Bridge No. 28-0100, Contract EA 04-0438U4, 04-Mrn-580-PM 6.22.) The micropiles were completed in 2005. Refer to Appendix J for a case study.

Micropiles may be economically feasible for bridge foundation retrofits having one or more of the following constraints:
- Restrictions on footing enlargements.
- Vibration and noise restrictions.
- Low headroom clearances.
- Difficult access.
- High axial load demands in both tension and compression.
- Difficult drilling or driving conditions.
- Hazardous soil sites.

Because of their high slenderness ratio (length/diameter), micropiles may not be acceptable for conventional seismic retrofitting applications in areas where liquefaction may occur, given the current standards and assumptions on support required for long slender elements. However, the ground improvement that can be induced by the use of micropiles may ultimately yield an improved earthquake mitigation foundation system.

13-2.5 Earth Retention
The ability of micropiles to be installed on an incline provides designers an option for achieving the required lateral capacity. Near the town of Duncan Mills in Sonoma County
in the San Francisco Bay Area, a micropile retaining wall was constructed in 2007 to stabilize the soil and roadway (Contract No. 04-1S2804, 04-Son-116 PM 3.2.) The wall has two rows of micropiles. The front row was vertical using steel pipe as reinforcement and the interior row was at an angle/incline using 2-#36 epoxy coated bundled rebar. Refer to Appendix J, *Micropiles*, for a case study.

13-2.6 **Foundations for New Structures (Retaining Walls)**

In 2007, construction started on a retaining wall on Rte 74 in District 12, Orange County (Contract No. 12-043214, 12-Ora-74 PM 13.3/16.6.) Micropiles support the retaining wall, concrete barrier slab, and concrete barrier. Ground anchors are also used to support the retaining wall. Refer to Appendix J, *Micropiles*, for a case study.

Also, on Rte 1, San Mateo County near the city of Pacifica in the San Francisco Bay Area, construction began in 2007 on a retaining wall supported by micropiles (Contract 04-1123U4, 04-SM-1 KP 61.2/64.9). The retaining wall (with barrier and chain link fence) is on a steep cliff facing the Pacific Ocean. A pedestrian sidewalk runs parallel to the barrier and chain link fence. On one portion of the wall, the micropiles are battered in opposite directions providing lateral support. Refer to Appendix J, *Micropiles*, for a case study.

13-2.7 **Construction and Contract Administration**

The contract specifications describe all submittal requirements and construction requirements for micropiles. Depending on the project location, the micropile design, and the Contractor, different drilling and grouting techniques may be used. The Contractor is required to submit all micropile shop drawings and a step-by-step procedure describing all aspects of micropile installation for authorization.

When required, verification and proof load tests are performed by the contractor. The Department may verify the test loads using a Department-furnished load cell. The Structure Representative will coordinate with the Foundation Testing Branch (FTB) for any Caltrans-required load tests or verification of test loads. The grouting operation can be very messy so the storm water pollution prevention plan (SWPPP) must be enforced and all best management practices (BMPs) implemented.

A *Micropile Construction Checklist* is presented in Appendix K-5 to assist field personnel in preparing documents and inspecting fieldwork to ensure compliance with contract requirements.

13-2.8 **Measurement and Payment**

The 2010 Standard Specifications handle payment quite differently from the 2006 Standard Specifications. In the 2010 Standard Specifications, the contract item number "sets forth" the governing specification section and all work mentioned in the specification section is included in the contract item. Nothing should be said about the

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1 2010 SS Section 49-5, *Micropiling*, or Special Provisions for contracts using 2006 SS.
measurement or payment unless there is something that needs further explanation. Detailed payment information is no longer included in the specifications. So, for micropiles, the contract item "Micropile" is measured and paid per micropile (EA). When this contract item is included in the Engineer's estimate for a project, it sets forth the Standard Specifications. Any work mentioned in Section 49-5 will be included in the payment for the Micropile contract item, unless otherwise stated in Section 49-5 or in the contract special provisions.

No payment will be made for micropiles that are damaged, either during installation, or after the micropiles are complete in place. No payment will be made for additional excavation, backfill, concrete, reinforcement, nor other costs incurred from footing enlargement resulting from replacing rejected micropiles.

13-2.9 Safety
All personnel must wear the proper personal protection equipment (PPE) during drilling and grouting operations to include eye protection, earplugs, and hardhat. Life vests are required when working near water. Safe access must be provided by the Contractor when working on slopes or within trenches. Be cautious and avoid slipping or falling when working near slopes. Caltrans field engineers should not stand too close to the work when the micropile reinforcement and steel pipe is hoisted into place.

2 2010 SSP Section 49-5, Micropiling.
Specialty Piles and Special Considerations for Pile Foundations

14-1 Introduction

This chapter provides information on specialty piles and special considerations for pile foundations.

Specialty piles include: Alternative Piling, Continuous Flight Auger Piling and other pile types that are under consideration for use by Caltrans.

Special considerations for other pile foundations include use of CIDH piling for overhead sign structure foundations, tip grouting of CIDH piling, Type II CIDH piling, Soldier Piling and other considerations used or proposed for use on Caltrans projects.

14-2 Specialty Piles

14-2.1 Alternative Piling

For projects using the 2006 Standard Specifications, the special provisions allow the Contractor to propose substitute alternative pile systems instead of the micro piling shown on the contract plans. The use of alternative pile system is contingent upon approval of working drawing submittal and successful performance of the alternative pile system once it has been load tested. There are four vendors who have on file generalized working drawings for given alternative pile systems that have been successfully tested and approved by the Engineer. These alternative pile systems are:

- DBM Micropile System
- Malcolm Micropile
- Nicholson Pin Pile
- Tubex Grout Injection Pile

During the development of the 2010 Standard Specifications, reserved sections 49-5, Micropiling, and 49-6, Alternative Piling, were created. However, the corresponding Standard Special Provisions 49-5 and 49-6 were not published, because there were a number of significant items that needed revisions, among them the fact that the four previously approved alternative pile systems have not been evaluated using load resistance factor design. It was decided that these specifications should not be published...
or used until these items were addressed. A new micropile Standard Special Provision 49-5 has been developed and was issued in 2013. However, a new alternative piling specification has not yet been developed. Hence, for projects utilizing the 2010 Standard Specifications, *Alternative Piles* are not authorized for use at this time.

### 14-2.2 Continuous Flight Auger Piling

Continuous flight auger piling uses a hollow continuous flight auger to drill and lift spoils from the drilled hole. Once drilling is complete, concrete is introduced into the drilled hole through the hollow continuous flight auger and placed as the auger is removed from the drilled hole. Steel reinforcement is stabbed into the wet concrete upon completion of concrete placement.

Advantages of continuous flight auger piling include minimal site disturbance, reduced spoils, the ability to drill through nearly all geomaterials, including contaminated soils, and no need for nondestructive evaluation.

Disadvantages of continuous flight auger piling include no ability to assess the axial capacity of the in-place piling without performing static load testing, the quality of the concrete placed, or the quality of the steel reinforcement placement.

Caltrans is considering use of continuous flight auger piling in locations where appropriate, such as standard retaining wall or soundwall piling, but no decision has been made to date.

### 14-2.3 Other Specialty Piling

Other specialty piling includes types that have been used rarely or not at all on Caltrans projects, including stone columns, soil-mixed auger-cast piling, helical piling and secant piling. Should one of these or other type of specialty piling be specified for your project, the reader is encouraged to consult with the Designer of the specialty piling, members of the DES Substructure Technical Committee, and perform an Internet search for additional information on the specialty piling.

### 14-3 Special Considerations

#### 14-3.1 Overhead Sign Structure Pile Foundations

Overhead Sign Structures (OHS) are overhead signs attached to a steel truss that are typically supported by a combination of a single post with a large diameter Cast-In-Drilled-Hole (CIDH) pile foundation. Figure 14-1 shows a 5-foot diameter pile with a minimum length of 22 feet for Changeable Message Sign (CMS) Model 500.

Construction of an OHS foundation is difficult when groundwater is encountered. If there is groundwater, then the slurry displacement method is usually required as described in Chapter 9, *Slurry Displacement Piles*. 
The contract specifications outline all the requirements. However, plans for CIDH piles for OHS foundations are found in the 2010 Standard Plans. These plans do not show the location of the PVC inspection pipes, which means the pile reinforcement cage requires reconfiguration to accommodate the inspection pipes in accordance with the requirements of *Bridge Memo to Designers 3-1, Attachment 2*, when the slurry displacement method is used.

Another consideration is the presence of the signpost anchor bolts. The anchorage system is typically heavy and must be positioned precisely to accommodate the signpost. This is further complicated if the foundation pile is constructed using the slurry displacement method. Use of the permissible construction joint shown on the OHS foundation plan.
sheet should be encouraged when the pile is constructed using the slurry displacement method, which allows the signpost anchorage system to be placed after the pile concrete is placed below the construction joint. Structure Representatives are advised to consider all of these factors during review of the Contractor’s pile installation plan.

Contractors that are smaller operations or inexperienced typically have difficulty meeting slurry displacement method submittal and construction requirements. Structure Representatives need to thoroughly communicate all requirements. The CIDH pile preconstruction meeting is a good forum to initially discuss slurry displacement method requirements.

A Log of Test Borings (LOTB) might not be included in small OHS projects, making it difficult to anticipate the presence of groundwater, indicating no foundation investigation was performed. If this is the case, corrosion information might not be available for corrosion evaluation of an anomalous pile. A proactive Structure Representative can obtain LOTB as-built drawings from the nearest bridge structure location. The proactive Structure Representative should review the LOTB as-built drawings and share the information with the Contractor. As-built drawings are available at District Headquarters and online on the Intranet [Bridge Inspection Records Information System (BIRIS) and Document Retrieving System (DRS)].

Personnel safety must be enforced during drilling and excavating operations. A full-body harness should be used when working near open holes. In order to avoid falling in or collapsing edges, personnel not directly involved in the construction operation should not stand next to an open hole.

14-3.2 Tip Grouting
Tip grouting is a procedure used to enhance the end bearing capacity of a CIDH pile. The CIDH pile is constructed with a grout delivery system installed at the base of the pile reinforcement cage. Upon completion of pile concrete placement and cure, grout is pumped through the grout delivery system to consolidate the foundation material at the tip of the CIDH pile, which, in theory, enhances the end bearing capacity of the pile.

An advantage of tip grouting includes the ability to shorten CIDH piles considerably from the length that would be required considering only “skin friction” capacity of the pile. This reduction in length can be substantial and result in considerable cost savings to the project.

A disadvantage of tip grouting includes the inability to properly assess the actual end bearing capacity of the CIDH pile after tip grouting is completed without performing a static load test. There is no known methodology of calculating the enhancement of end bearing capacity using only grout pressures and grout volumes placed.

Caltrans’ position on tip grouting is that it is not authorized for use on state highway projects. However, tip grouting has been utilized on a design-build project sponsored by a
local agency. Results have been tabulated and lessons have been learned from this project, but there has been no conclusive evidence presented to date that tip grouting resulted in the piles meeting the required design capacity. Due to the potential for cost reduction, Caltrans continues to investigate tip grouting and it may be authorized for use on state highway projects in the future.

14-3.3 Type II Shafts
Foundations that use a single pile to support a column are designated as Type I or Type II shafts. A Type I shaft utilizes a single bar reinforcement cage for the pile and the column. A Type II shaft utilizes a bar reinforcement cage for the pile and one or more separate reinforcement cages for the column.

For a Type II shaft constructed using the slurry displacement method, the special consideration occurs at the tip of the column reinforcement cage(s). A construction joint is required at this location. The construction joint is a constructability enhancement designed to allow concrete to be placed in the area of pile and column reinforcement cage overlap by the dry method. However, there are several factors that must be considered at the construction, which include the following:

1. Since the location of the construction joint is typically below grade, a permanent casing suitable for worker entry is required, from the ground surface to 5 feet (2-feet if the casing is embedded in bedrock) below the level of the construction joint.
2. The pile concrete must be placed, acceptance testing performed, and any mitigation completed before column construction begins.
3. Any pile concrete that is wasted above the level of the construction joint as described in Chapter 9, Slurry Displacement Piles, must be removed to at least the location of the construction joint before column construction begins.

Structure Representatives are advised to review the project documents for the presence of Type II shafts, engage in discussions with the Designer and Contractor about design considerations and construction methods, and thoroughly vet the Pile Installation Plan before authorizing it.

Additional safety concerns with Type II shafts include the presence of an open hole for a longer period of time than with ordinary CIDH piles and that workers must enter the hole to prepare the construction joint for column construction. Fall protection and confined space safety procedures must be followed, and if the level of the construction joint is far enough below the ground surface, CalOSHA Mining and Tunneling Safety Orders may apply. Structure Representatives must ensure the Contractor has an authorized safety plan addressing these concerns – and adheres to it.

14-3.4 Soldier Piling
Soldier Piling consists of a heavy rolled structural steel shape placed in a drilled hole, which is then filled with concrete. Soldier Piles are typically used for temporary shoring
and permanent earth retaining structures. Requirements for Soldier Piling are found in the 2010 Standard Specifications\[1\].

One of the special considerations for soldier piling is the drilled hole must be dry when concrete is placed. Those of you who have built soldier piling know that this is not always the case. Tremie seals and temporary casings may be required, which can be difficult to maneuver in locations where soldier piling are typically used. Dewatering the drilled hole can also be problematic. Prudent Structure Representatives discuss the possibility of wet holes and how they will be handled with the Contractor – prior to the start of soldier pile construction.

Another special consideration for soldier piling is the specification requirement that the drilled hole have 1-inch minimum clearance to the steel piling. Typically this means the Contractor will try to drill the smallest diameter hole possible. Structure Representatives are advised to discuss the size, location, and plumbness of the drilled hole and soldier pile alignment requirements with the Contractor – prior to the start of soldier pile construction.

14-3.5 Other Special Considerations
Other special considerations for foundation construction will be added here as they are developed.

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\[1\] 2010 SS Section 49-4, Steel Soldier Piling.
APPENDIX

A Foundation Investigations

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Soil & Rock Logging, Classification and Presentation Manual A-2
Soil and Rock Logging, Classification, and Presentation Manual

2010 Edition

State of California
Department of Transportation
Division of Engineering Services
Geotechnical Services

This document is available on the DES-Geotechnical Services website:
APPENDIX

B Contract Administration

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CALTRANS • FOUNDATION MANUAL
Memorandum

To: DEPUTY DISTRICT DIRECTORS, Construction CONSTRUCTION MANAGERS SENIOR CONSTRUCTION ENGINEERS RESIDENT ENGINEERS DOLORES VALLS, Deputy Division Chief, Structure Construction

From: ROBERT PIEPLOW Chief DIVISION OF CONSTRUCTION

Date: May 10, 2004

File: Division of Construction CPD 04-6

Subject: Application of the Tunnel Safety Orders

The California Department of Transportation (Department) is required to obtain tunnel classifications from the California Department of Industrial Relations, Division of Occupational Safety and Health (Cal/OSHA), Mining and Tunneling unit before bid advertisement for all operations covered by Sections 8400 through 8469, “Tunnel Safety Orders,” of the California Code of Regulations. The intent of the tunnel classification is to inform the contractor of the potential hazards and safety precautions required.

The Tunnel Safety Orders are available via the internet at the Department of Industrial Relations web site:

http://www.dir.ca.gov/Title8/sub20.html

The Department has recently received clarification from the Cal/OSHA Mining and Tunneling unit citing which types of operations are applicable to the Tunnel Safety Orders. The Tunnel Safety Orders are applicable only when human entry will occur during a construction operation at any of the following locations:

- Tunnels: Culverts greater than 760 mm (30 inches) in diameter;
- Shafts: Excavations where the depth, (a) is at least twice the greatest cross-sectional dimension, and (b) exceeds 6.1 m (20 feet);
- Rises: Vertical or inclined underground excavation driven from bottom to top;
- Underground chambers and premises appurtenant thereto;
- Boring and pipe-jacking operations 760 mm (30 inches) in diameter or greater.

Some of the common types of activities where human entry is likely and often require a tunnel classification include:

- Pipe jacking or boring operations

“Caltrans improves mobility across California”
DEPUTY DISTRICT DIRECTORS, et al
May 10, 2004
Page 2

- Culvert rehabilitation
- Large diameter pile construction
- Pump house vaults
- Cut-and-cover operations connecting to an existing underground installation
- Well construction
- Deep structure footings/shafts
- Cofferdam excavations
- Waste slab construction

Review both on-going and already advertised projects for operations that may require a tunnel classification. Issue contract change orders on all projects where a tunnel classification is required and a tunnel classification was not included in the special provisions. Issue the change order as a “change in character of work” in accordance with Section 4-1.03C, “Changes in Character of Work,” of the Standard Specifications. If a tunnel classification has not been obtained and the contractor is currently performing an operation where the Tunnel Safety Orders are applicable, halt operations until the Cal/OSHA Mining and Tunneling unit is contacted and a tunnel classification is obtained. Processing time for a tunnel classification can take up to two weeks so contact should be made as quickly as possible. Section 8422, “Tunnel Classifications,” of the California Code of Regulations lists the information that must be submitted to the local Cal-OSHA Mining and Tunneling unit. A listing of the Cal/OSHA Mining and Tunneling unit offices is attached.

The contractor may be entitled to compensation for compliance with the requirements of the Tunnel Safety Orders only where the requirements exceed the requirements of the Construction Safety Orders. The Construction Safety Orders are available via the internet at the Department of Industrial Relations website:

http://www.dir.ca.gov/Title8/sub4.html

If you have any questions regarding this construction procedure directive, please contact Greg Berry, Safety Coordinator, Division of Construction, at (916) 654-4580.

Attachments: Mining and Tunneling unit offices

"Caltrans Improves Mobility Across California"
California Mining and Tunneling Districts

Northern District Office
2211 Park Towne Circle, Suite 2
Sacramento, CA 95825
Phone: 916-574-2540
FAX: 916-574-2542

Central District Office
6150 Van Nuys Boulevard, Suite 310
Van Nuys, CA 91401-3333
Phone: 818-901-5420
FAX: 818-901-5579

Southern District Office
464 West 4th Street, Suite 354
San Bernardino, CA 92401-1400
Phone: 909-383-4782
FAX: 909-388-7132

Revised July 2003
APPENDIX C–FOOTING FOUNDATIONS

APPENDIX

C

Footing Foundations

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Sample Spread Footing Letter to Contractor C-4
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Footing Retrofit Strategies C-6
Please note that these conversion tables are approximate. They can be used by characterizing the soil as being either predominately granular or cohesive. If possible, the conversion of the Penetration Index (N value) should be checked by using is-situ or laboratory tests.

Table C-1. Approximate Values for N, φ and Unit Weight for GRANULAR SOILS

<table>
<thead>
<tr>
<th>COMPACTNESS</th>
<th>VERY LOOSE</th>
<th>LOOSE</th>
<th>MEDIUM</th>
<th>DENSE</th>
<th>VERY DENSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relative Density, $D_d$</td>
<td>15%</td>
<td>35%</td>
<td>65%</td>
<td>85%</td>
<td></td>
</tr>
<tr>
<td>Standard Penetration Resistance, $N = \text{Blows/ft}^*$</td>
<td>4</td>
<td>10</td>
<td>30</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>Angle of Internal Friction, $\phi$</td>
<td>28</td>
<td>30</td>
<td>36</td>
<td>41</td>
<td></td>
</tr>
<tr>
<td>Unit Weight (PCF)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moist</td>
<td>100</td>
<td>95-125</td>
<td>110-130</td>
<td>110-140</td>
<td>130+</td>
</tr>
<tr>
<td>Submerged</td>
<td>60</td>
<td>55-65</td>
<td>60-70</td>
<td>65-85</td>
<td>75+</td>
</tr>
</tbody>
</table>

VERY LOOSE: A reinforcing rod can be pushed into soil several feet.
DENSE: Difficult to drive a 2x4 stake with a sledge hammer.

*N = Blows/Ft as measured by the standard penetration test (See Appendix B).

Relative Density, $D_d = \frac{e - e_{min}}{e_{max} - e_{min}} \times 100$

$e =$ existing void ration of mass being considered.

$e_{max} =$ void ratio of same mass in its loosest state.

$e_{min} =$ void ration of same mass in its most compact state.
Table C-2. Approximate Values for $N$, $q_u$ and Unit Weight for COHESIVE SOILS

<table>
<thead>
<tr>
<th>CONSISTENCY</th>
<th>VERY SOFT</th>
<th>SOFT</th>
<th>MEDIUM</th>
<th>STIFF</th>
<th>VERY STIFF</th>
<th>TIFF</th>
<th>HARD</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q_u =$ unconfined comp. strength (PSF)</td>
<td>500</td>
<td>1000</td>
<td>2000</td>
<td>4000</td>
<td>8000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Standard Penetration Resistance, $N$ – Blows/Ft *</td>
<td>2</td>
<td>4</td>
<td>8</td>
<td>16</td>
<td>32</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unit Weight (PCF) Saturated</td>
<td>100-200</td>
<td>110-130</td>
<td>120-140</td>
<td>130+</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

VERY SOFT: Exudes from between fingers when squeezed in hand.
SOFT: Molded by light finger pressure.
MEDIUM: Molded by strong finger pressure.
STIFF: Indent by thumb.
VERY STIFF: Indent by thumb nail.
HARD: Difficult to indent by thumb nail.
*N = Blows/ft as measured by the standard penetration test
(See Appendix B)

To be used only as a rough guide.
STATE OF CALIFORNIA -- BUSINESS, TRANSPORTATION, AND HOUSING AGENCY

DEPARTMENT OF TRANSPORTATION
Nevada City Construction Office
P. O. Box 691
Nevada City, CA 95959

September 10, 1991

03-NEV-49-21.9
03-295604 F-P049(95)
S. Yuba River Br.

David A. Mowat Company
Highway 49
Nevada City, CA

Gentlemen:

This letter is to clear up any possible misunderstanding about field revision of the elevation of spread footings. You are reminded that Section 51-1.03 of the Standard Specifications states that "the elevations of the bottoms of footings shown on the plans shall be considered as approximate only."

The Engineer will establish final footing elevations at the earliest time possible consistent with the progress of the work, and that you will be informed in writing of the Engineer's decision.

You are reminded that should you elect to do any work or order any materials before receiving the Engineer's decision regarding spread footing elevations, you do so at your own risk and assume the responsibility for the cost of alterations to such work or materials in the event that revisions are required.

If you have any questions about this or any other matter, please call me at (916) 265-9413.

Sincerely,

John Rodrigues
Resident Engineer

[Signature]

by David R. Keim
Structures Representative

cc: OSC
03 Const
DKDefoe
File c:\wp50\pr3\letters\09-10-91.1
METHOD FOR INSTALLATION AND USE OF EMBANKMENT SETTLEMENT DEVICES

A. SCOPE

The installation, maintenance, and data collection procedures for the various embankment settlement devices used to monitor subsurface settlement are described in this method. Analysis of the settlement data is included as a separate part of this method.

Settlement devices are used to monitor the rate and magnitude of settlement occurring at a point within or beneath an embankment during and subsequent to construction. The data obtained from these devices are used to determine the allowable loading rate during embankment construction and the appropriate time for removal of surcharge and/or commencement of permanent structure construction.

This method is divided into the following parts:

1. Fluid Level Settlement Devices
2. Pipe Riser Settlement Device
3. Settlement Data Analysis

The fluid level vented standpipe unit may be used at most locations. A sealed standpipe unit must be installed at locations where groundwater may interfere with the operation of the unit or where excess pore water is expected from the use of dredged material or wet soil in embankment construction. Where it is possible, the tube length between standpipe and indicator unit should generally be limited to a maximum 300 linear ft. Installations over longer distances can be made but are not advisable under normal circumstances since it may result in inconsistent test data. Factors such as larger size tubes, change of platform location, or changes in elevation of the water line may have to be considered (see NOTE).

NOTE: There may be job conditions with respect to terrain, long tube length between standpipe and indicator unit, or anticipated large settlements that require special installations.

The pipe riser settlement device is used for monitoring fill settlement over soft foundation soils where the fluid level settlement devices are not feasible because of flat terrain, width of embankment construction, or other features which would make installation of a fluid level type of settlement platform undesirable. The pipe riser settlement device is a direct-reading unit which is exposed for the full duration of fill construction and surcharge removal. Because of the vulnerability of this unit to damage by the contractor's operations, the pipe riser settlement device should be used only on those projects where the fluid level type of settlement device would be impractical.

B. REFERENCE

None.
Throughout the 1990’s Caltrans underwent a massive seismic retrofit program. Retrofits of footings designed and built prior to 1973 were required to address deficiencies. These retrofits required the installation of a top mat of reinforcing steel (Figure C-1) to address tensile loads at the top of the footing due to seismic forces. In some cases footing dimensions were increased and/or perimeter piles added (Figure C-2). These additional piles provide additional resistance to bending moment in the structure and provide additional restraint against rotation. Typical spread footings seismic retrofits are shown in the Figures below.

Figure C-1. Seismic Retrofit Strategy – Add Top Mat.
Figure C-2. Seismic Retrofit Strategy – Enlarged Footing.
APPENDIX

Pier Column & Type I Pile Shaft

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Figure D-1. Pier Column - Caltrans Bridge Design Details, page 7-20.
Blasting Example & Sample Special Provisions

What follows is an example of how Caltrans uses blasting in the construction of bridge foundations. In the past, blasting has been performed to facilitate the construction of spread footings and pier columns. The photograph below is an example of what can be considered a hybrid of the two. Pier W2 of the San Francisco-Oakland Bay Bridge East Span Construction project uses blasting to construct a foundation in rock. The foundation is in excess of 60’ deep and 80’ square.

Pier W2, part of the E2-T1 project (EA 04-0120L4) is part of the new San Francisco Oakland Bay Bridge. Blasting was used for structure excavation of these piers. Construction of W2 structures was complete in September 2004 at a cost of $24.1 million.

Figure D-2. SFOBB East Span Pier W2 Footing Excavations.
Special Provisions – EA 04-0120C4

The following specification was taken from the Special provisions of the above referenced contract. It is understood that projects of this type are unique and extreme in their application of engineering principles, but still utilize construction operations utilized on projects throughout the State. Note that special provisions are unique to each contract; and as such may vary from what is presented below. In some geologic regions blasting may be the only option, however Industry continues to implement new technologies. In some cases, new tools such as the rotator and/or oscillator (refer to Chapter 6, Cast-In-Drilled-Hole Piles) may be more appropriate and more environmentally considerate.

**BLASTING**

Attention is directed to, "Project Information," and "Photo Survey of Existing Facilities," of these special provisions, regarding the Blasting Demonstration Report, and photo survey of the existing facilities.

Attention is directed to "Order of Work," of these special provisions regarding transportation and use of explosives.

If the Contractor elects to use blasting for structure excavation (bridge) at Piers W2, project blasting shall conform to Sections 7-1.10, "Use of Explosives," and 19-2.03, "Blasting," of the Standard Specifications and these special provisions.

The Contractor shall control project blasting effects (fly rock, ground motion, and air noise levels) within the safe limits so as not to cause damage to neighboring improvements.

**Blasting Plan Submittal**

The Contractor shall submit a blasting plan to the Engineer detailing how he proposes to control fly rock, air noise level, and ground motion peak particle velocity. No blasting operations, including drilling, shall start until the Engineer has reviewed and approved the blasting plan.

The Contractor shall submit the blasting plan in accordance with the provisions in "Working Drawings," of the special provisions not less than 30 working days before commencing blasting activity or at any time the Contractor proposes to change the drilling and blasting methods. The Contractor shall provide 10 working days for the Engineer to complete the review of the blasting plan. In the event that additional blasting plans are required, the Contractor shall provide 5 working days for the review of each additional plan.

The blasting plan shall provide for limiting ground motion to a maximum peak particle of 100 mm/sec at the existing E1 Pier of the San Francisco Oakland Bay Bridge (Bridge No. 33-0025), and 50 mm/sec at the Torpedo Building (Building 262). Controlling fly rock, air noise levels, and ground motor peak particle velocities as specified herein shall not relieve the Contractor of his responsibility for assuring the complete safety of his operation.
The blasting plan shall indicate the type and method of instrumentation proposed by the Contractor to determine air noise levels, and ground motion peak particle velocity at the nearest improvements. The blasting plan shall also provide for a pre-blast reconnaissance survey of all adjacent improvements.

Approval of the Contractor's blasting plan or blasting procedures shall not relieve the Contractor of any of his responsibility under the contract for assuring the complete safety of his operations with respect to neighboring improvements, or for the successful completion of the work in conformance with the requirements of the plans and specifications.

If the Engineer fails to complete the review within the time allowed, and if, in the opinion of the Engineer, the Contractor's controlling operations are delayed or interfered with by reason of the delay, an extension of time commensurate with the delay will be granted as provided in Section 8-1.07, "Liquidated Damages," of the Standard Specifications.

Qualifications

The blasting supervisors (blaster in charge) shall have a minimum of 10 years experience, directly related to the specific types of blasting they are supervising.

All blasters and supervisors shall be properly qualified and licensed in accordance with applicable federal, State, and local government regulations.

The Contractor shall retain the services of an experienced seismologist or engineering consultant with at least 10 years experience in monitoring blasting operations and interpreting ground vibration, air overpressure, and water pressure amplitudes for similar construction projects.

The Contractor shall retain the services of an experienced specialist who will conduct the pre-blast inspections of private properties as specified herein. The specialist shall have performed similar pre-construction survey services on at least three projects of similar scope and complexity.

Pre-Blast Condition Survey

The Contractor shall perform a pre-blast survey of specified buildings and structures, and utilities within 100 meters or which may potentially be at risk from blasting damage. The survey method used shall be acceptable to the Contractor's insurance company. The Contractor shall perform the pre-blast survey within 30 working days in advance of the planned commencement or resumption of blasting operations and pre-blast records shall be made available to the Engineer for review. The Contractor prior to the beginning of the blast shall notify occupants of the local buildings. The pre-blast survey shall, as a minimum, contain the following:

A. The name of the person making the inspection.
B. The names of the property owner and occupants, the addresses of the property, the date and time of the inspection.
C. A complete description of the structure(s) or other improvement(s) including culverts and bridges.

D. A detailed interior inspection with each interior room (including attic and basement spaces) designated and described. All existing conditions of the walls, ceiling and floor such as cracks, holes and separations shall be noted.

E. A detailed exterior inspection fully describing the existing conditions of all foundations, walls, roofs, doors, windows, and porches.

F. A detailed listing, inspection and documentation of existing conditions of garages, outbuildings, sidewalks and driveways.

G. A detailed listing of highway signposts, light fixtures and overhead power lines.

H. A survey of any wells or other private water supplies including total depth and existing water surface levels.

The Contractor shall perform a re-survey of all locations whenever blasting operations are either terminated or suspended for a period in excess of 30 working days. The documentation may consist of either a written report, or videotape with voice narration. The videotape, if used, must include date and time displayed on the image. The Contractor shall provide copies of the pre-blast inspection report or videotape documentation to the Engineer at the time that the blasting plan is submitted.

The Contractor shall control project blasting so that vibration, flyrock, ground and vibration motion, and air noise levels do not cause damage to nearby structures including highway sign posts, light fixtures and parked vehicles, undue annoyance to nearby residents, or danger to employees on the project. The Contractor shall use controlled blasting techniques and designs and shall coordinate the traffic control during blasting operation. The Contractor shall be responsible for all damage resulting from blasting.

Vibration Control and Monitoring

When blasting within proximity of buildings, structures, or utilities that may be subject to damage from blast-induced ground vibrations, the Contractor shall control ground vibrations by the use of properly designed delay sequences and allowable charge weights per delay. Allowable charge weights per delay shall be based on vibration levels that will not cause damage. The Contractor shall perform trial blasts to select allowable charge weights per delay by measuring vibration levels. The Contractor shall select proper control method to limit over break. The trial blasts shall be carried out in conformance with the blasting test section requirements, modified as required to limit ground vibrations to a level which will not cause damage. The blasting test section requirements require that two seismographs be used, one placed on the end of the shot and one placed at 90 degrees behind the shot to establish vibration levels and their relation to the measurement location. The Contractor shall have full responsibility to control over break.

Whenever vibration damage to adjacent structures is possible, the Contractor shall monitor each blast with an approved seismograph located, as
approved, between the blast area and the structures subject to the blast site. The seismograph used shall be capable of recording particle velocities for three mutually perpendicular components of vibration in the range generally found with controlled blasting.

The Contractor shall employ a qualified vibration specialist to establish safe vibration limits. The vibration specialist shall also interpret the seismograph records to ensure that the seismograph data are utilized effectively in the control of the blasting operations with respect to the existing structures. The vibration specialist used shall be subject to the Engineer’s approval.

The Contractor shall provide vibration monitoring at the following locations:

A. Existing E1 pier of San Francisco-Oakland Bay Bridge.
B. Torpedo Building (Building 262).
C. Navy Building 1.
D. Coast Guard Building 27.

The measuring devices should be positioned at the closest face of structure or body of water to the blast site.

Data recorded for each shot shall be furnished to the Engineer prior to the next blast and shall include the following information:

A. Identification of instrument used.
B. Name of qualified observer and interpreter.
C. Distance and direction of recording station from blast area.
D. Type of ground at recording station and material on which instrument is sitting.
E. Maximum particle velocity in each component.
F. A dated and signed copy of seismograph readings record.

At the Contractor's option, shot designs may be based upon scaled distance following the chart below. The scaled distance is the ratio of distance in feet from the blast site to the site to be protected to the square root of the maximum explosive weight used for each delay of 9 milliseconds or more.

**Blast Design Table**

<table>
<thead>
<tr>
<th>Distance to site to be protected</th>
<th>Scaled distance factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 91 meters</td>
<td>22.57 m/kg(^{1/2})</td>
</tr>
<tr>
<td>91 to 1,524 meters</td>
<td>24.94 m/kg(^{1/2})</td>
</tr>
<tr>
<td>1,524 meters</td>
<td>29.4 m/kg(^{1/2})</td>
</tr>
</tbody>
</table>
Environment Protection

Sound Pressure Level (SPL) due to blasting shall not be greater than 180 dB (decibels) in the water at a distance of 10 meters from any point on the shoreline at Yerba Buena Island. The Contractor shall design blasting plan to meet SPL performance limitations and shall perform trial blasts to select allowable charge weights per delay based on measured values of SPL. The Engineer will conduct acoustical monitoring and marine mammal monitoring during all blasting activities. The safe distance for marine mammals due to blasting effects is herein referred to as the Marine Mammal Safety Zone (MMSZ). The MMSZ will be established at a 50-meter radii from the shoreline adjacent to the blasting area, and may be increased or decreased in size based on results of acoustical monitoring. The purpose of the marine mammal monitoring is to prohibit blasting activity if marine mammals are present within the MMSZ. In addition, the Engineer will monitor for Pacific herring spawning event within a 200-meter distance from the shoreline adjacent to the blasting area. If spawning is observed, blasting activity will be prohibited. Work shall not resume until the Engineer notifies the Contractor, which is expected to be approximately 14 calendar days from the time of spawning.

The Contractor shall provide two working days advance notice to the Engineer before each day he is planning to blast. The marine mammal monitoring shall commence at least 15 minutes before blasting begins. The Engineer will have the sole discretion to direct Contractor with approval to proceed with blasting operation prior to each and every blast.

The Department will conduct surveys and monitoring of bird activity before and during blasting activities as part of an agreement with the resource agencies.

Air Blast and Noise Control

The Contractor shall install an air blast monitoring system between the main blasting area and the nearest structure subject to blast damage or annoyance. The equipment used to make the air blast measurements shall be the type specifically manufactured for that purpose. Noise levels shall be held below 125 dB(A) at the nearest structure or designated location. The Contractor shall use appropriate blast hole patterns, detonation systems, and stemming to prevent venting of blasts and to minimize air blast and noise levels produced by the blasting operations. The decibel level shall be lowered if it proves to be too high based on damage or complaints. The Contractor shall furnish a permanent, signed and dated record of the noise level measurement to the Engineer immediately after each shot.

Flyrock Control

Before the firing of any blast in areas where flying rock may result in personnel injury or unacceptable damage to property, parked vehicles or the work, the Contractor shall cover the rock to be blasted with approved blasting mats, soil, or other equally serviceable material, to prevent flyrock.
If flyrock leaves the construction site and lands on private property all blasting operations will cease until a qualified consultant, hired by the Contractor, reviews the site and determines the cause and solution to the flyrock problem. Before blasting proceeds, a written report shall be submitted by the Contractor to the Engineer for approval.

**Video Recordings of Blasts**

Videotape recordings will be taken of each blast. The tapes or sections of tapes will be indexed in a manner to properly identify each blast. At the option of the Engineer, copies of videotapes of blasts will be furnished on a weekly basis. The Contractor shall keep accurate records of each blast. Blasting records shall be made available to the Engineer at all times and shall contain the following data as a minimum:

A. Blast Identification by numerical and chronological sequence.
B. Location (referenced to stationing), date and time of blast.
C. Type of material blasted.
D. Number of holes.
E. Diameter, depth and spacing of holes.
F. Height or length of stemming.
G. Types of explosives used.
H. Type of caps used and delay periods used.
I. Total amount of explosives used.
J. Maximum amount of explosives per delay period of 9 milliseconds or greater.
K. Powder factor (pounds of explosive per cubic yard of material blasted).
L. Method of firing type.
M. Weather conditions (including wind direction).
N. Direction and distance to nearest structure or structures of concern.
O. Type and method of instrumentation.
P. Location and placement of instruments.
Q. Instrumentation records and calculations for determination of ground motion particle velocity or for charge size based on scaled distance.
R. Measures taken to limit air noise and fly rock.
S. Any unusual circumstances or occurrences during blast.
T. Measures to limit over break
U. Name of contractor.
V. Name and signature of responsible blaster.

**Blasting Guards**

The Contractor shall provide sufficient blasting guards and station them around the blasting area during blasting to assure that people and structures are not endangered. Traffic during blasting shall be controlled by the Contractor. Blasting operations may be suspended by the Engineer for any of the following:
A. Safety precautions, monitoring equipment and traffic control measures are inadequate.
B. Ground motion particle velocity or air noise exceeds the limits specified.
C. Blasting control plan have not been approved.
D. Required records are not being kept.
E. Excessive outbreak as determined by the Engineer

Suspension of blasting operations shall in no way relieve the Contractor of his responsibilities under the terms of this contract. Blasting operations shall not resume until modifications have been made to correct the conditions that resulted in the suspension.

Blasting complaints shall be accurately recorded by the Contractor as to complainant, address, date, time, nature of the complaint, name of person receiving the complaint, the complaint investigation conducted, and the disposition of the complaint. The Contractor shall make the complaint available to the Engineer as soon as practical, but no later than at the beginning of the following day’s work shift.

PAYMENT

Full compensation for blasting including all the requirements as specified herein, shall be considered as included in the contract price paid per cubic meter for structure excavation (bridge) and no separate payment will be made therefor.
Blasting Photos

Figure D-3. Blasting Photos.
Pile Shaft (Type I) - Case Study

Contract No. 04-470804
04-SJ, Ala-205, 580-10.0/0.4 (near the City of Tracy)
580/205 Separation (Bridge No. 33-0693R)
Construction started in 2006.

Pile Shaft Project: Although this project has large diameter CIDH piles, this project is not considered a pier column. It does not have contract pay items for structure excavation (pier column) and structure concrete (pier column). Conventional methods were used to drill the CIDH pile.

However, the pile shaft design was chosen because limited space constraints next to the existing freeway and change of elevation differences between Abutment 1 to Abutment 8. If a pile cap foundation was chosen, then the pile caps would have had to have been excavated 10 to 20 feet beneath the existing freeway to account for the different column stiffness. This excavation would have been problematic due to space constraints in the middle of the existing freeway. Due to the limited space constraints, single column pile shaft foundations were considered easier to construct than a conventional pile cap with standard plan piles. Due to the different column lengths, isolation casing were required at certain locations to account for the different stiffness of short and long columns. Also, the claystone formation underlying the project site was conducive to drilled shaft construction since caving issues would be reduced after using temporary casings to stabilize softer/looser near surface soils (Comments by Tim Alderman, Caltrans Geologist).

The pile shaft design is designated as Type I, meaning that the pile and the column share the same bar reinforcement cage.

Description of Bridge Work: construct a 7-span cast-in-place prestressed concrete box girder bridge approximately 373 meters in length and 12.6 meters in width.
Pile shaft diameter: 1980-mm for Bents 2, 3, & 4; 2280-mm for Bents 5, 6, & 7.
Pile shaft length: 26.75 meters (Bent 4) to 39.8 meters (Bent 2)
Column heights: 8.9 meters (Bent 4); 19.2 meters (Bent 7)
Isolation casings: Bents 2, 3 & 4.

Construction Issues: Groundwater was anticipated and encountered. Two cranes were needed to lift the pile shaft column rebar into place. Windy conditions affected crane operations.

(Photos contributed by Gon Choi, Consultant Engineer.)
Figure D-4. Bents 2 & 3 Column Details – 580/205 Separation, Contract No. 04-470804.
Figure D-5. 580/205 Separation General Plan.
Figure D-6. 580/205 Separation CIDH Pile Construction 1.
Figure D-7. 580/205 Separation CIDH Pile Construction 2.
Figure D-8. 580/205 Separation CIDH Pile Rebar Cage Construction.
Figure D-9. 580/205 Separation CIDH Pile Rebar Cage Setting.
Figure D-10. 580/205 Separation CIDH Pile Concrete Placement.
Figure D-11. 580/205 Separation Column Construction.
Figure D-12. 580/205 Separation CIDH Pile and Column Construction.
APPENDIX

E Driven Piles

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Gates Formula Commentary

Projects with driven pile foundations specify the “Gates Formula” to determine nominal resistance. This change was first incorporated in the “Amendments to July 1999 Standard Specifications” and is now included in the Standard Specifications. The change is also discussed in Bridge Construction Memo (BCM) 130-4.0, *Pile Driving Acceptance Criteria*.

Why change from ENR to Gates Formula?

- Factor of safety from ENR (Engineering News Record) varies from 1/2 to 20. With low factor of safety, capacity of the pile is actually driven to be under the factored design load. Lack of capacity has resulted in excessive settlement. Extremely high factors of safety often cause damage to the pile and result in contractor claims and also is a waste of time and energy.

- California was actually one of the last States using the ENR formula.

- ENR does not properly account for down drag or the overburden effects and resistance associated with zones that may scour or liquefy.

Advantage of Gate’s Formula

- This formula predicts the static capacity of the pile significantly more accurately than the ENR Formula because it provides a significantly lower coefficient of variation.

Additionally, since the formula utilizes ultimate capacity and not an unfactored safe load, the formula can account for the effects of downdrag, scour, and liquefaction.

The Gates formula (US Customary) is:

\[ R_u = (1.83 \times (E_r)^{\frac{1}{2}} \times \log_{10}(0.83 \times N)) - 124 \]

Where:

\( R_u \) = Calculated nominal resistance/ultimate compressive capacity in kips

\( E_r \) = Energy rating of hammer at observed field drop height in foot pounds

\( N \) = Number of blows in the last foot (maximum of 96)
Additional Notes:

*Caltrans Memo to Designer 3-1* was updated in June 2014. During constructability reviews, it is very important that the Structure Construction reviewer checks the pile data table on the plan sheets for notes on downdrag and liquefaction.

A very good reference showing the differences in formulas (Gates, ENR, Haley, Janbu, etc) is the “Comparison of Methods for Estimating Pile Capacity, Report No. WA-RD 163.1”, Final Report dated August 1988, by the Washington State Department of Transportation. In lieu of that, examples of comparisons are shown below.
Pile Driving Formulas

Gates Formula

\[ P = \left( (1.83 \times (E_r)^{0.5} \times \log_{10}(0.83 \times N)) - 124 \right) z \]

Where,  
\( P \) = safe load in kips  
\( E_r \) = energy of driving in foot pounds  
\( N \) = number of hammer blows in the last foot  
\( z \) = conversion factor for units and safety with this formula

Engineering News (ENR)

\[ P = \frac{2E}{(s + 0.1)} \]

Where,  
\( P \) = safe load in pounds  
\( E \) = rated energy in foot-pounds  
\( s \) = penetration per blow in inches

This formula was derived from the original Engineering News formula for drop hammers on timber piles, which was:

\[ P = \frac{WH}{(s + c)} \]

Where,  
\( W \) = weight of ram in pounds  
\( H \) = length of stroke in inches  
\( c \) = elastic losses in the cap, pile, and soil in inches

It was modified to correct units and apply other factors to compensate for modern equipment.
Janbu Formula

\[ P = \left( \frac{WH}{k_w s} \right) z \]

Where,  
- \( P \) = safe load in pounds
- \( W \) = weight of ram in pounds
- \( H \) = length of stroke in inches
- \( s \) = penetration per blow in inches
- \( k_w \) = factor derived from the following,
  \[ k_w = C_d \left[ 1 + \sqrt{1 + \left( \frac{\lambda}{C_d} \right)} \right] \]
  \[ C_d = 0.75 + 0.15 \left( \frac{W_p}{W} \right) \]
- \( \lambda = \frac{WHL}{AEs^2} \)

where,  
- \( W_p \) = weight of pile in pounds
- \( L \) = length of pile in inches
- \( A \) = area of pile in square inches
- \( E \) = modulus of elasticity of pile in pounds per square inch
- \( z \) = conversion factor for units and safety with this formula

Hiley Formula

\[ P = \left( \frac{e_f WH}{s + \frac{1}{2}(c_1 + c_2 + c_3)} \right) \left( \frac{W + n^2 W_p}{W + W_p} \right) z \]

Where,  
- \( P \) = safe load in pounds
- \( e_f \) = efficiency of hammer (%)
- \( W \) = weight of ram in pounds
- \( H \) = length of stroke in inches
- \( s \) = penetration per blow in inches
- \( c_1, c_2, c_3 \) = temporary compression of pile cap and head, pile, and soil, respectively in inches
- \( n \) = coefficient of restitution
- \( W_p \) = weight of pile in pounds
- \( z \) = conversion factor for units and safety with this formula
Pacific Coast Formula

\[
P = \frac{E_n \left( \frac{W + kW_p}{W + W_p} \right) z}{s + \frac{PL}{AE}}
\]

Where,
- \( P \) = safe load in pounds
- \( E_n \) = energy of driving in inch pounds
- \( W \) = weight of ram in pounds
- \( W_p \) = weight of pile in pounds
- \( s \) = penetration per blow in inches
- \( L \) = length of pile in inches
- \( A \) = area of pile in square inches
- \( E \) = modulus of elasticity of pile in pounds per square inch
- \( k \) = 0.25 for steel piles
  - 0.10 for other piles
- \( z \) = conversion factor for units and safety with this formula
Comparison of Formulas

Given Problem Conditions

Hammer Data: Delmag D36-32
- Maximum Energy = 83,880 ft·lbs
- Ram Weight = 7,938 lbs
- Maximum Stroke = 10.42 ft
- Penetration or Set = 0.844 inches

Length of Pile = 80 feet  
- Assume hard driving -

| Case 1: | 12” PC/PS concrete pile |
| Case 2: | 12 BP 53 Steel Piles |

Gates Formula

For Case 1 & 2: 
\[ P = \left( \frac{(1.83 \cdot (E_j)^{\frac{1}{2}} \cdot \log_{10}(0.83 \cdot N)) - 124}{2} \right) \frac{1}{2(\frac{z}{\text{in}})} \]
\[ = \left( \frac{(1.83 \cdot 289.62 \cdot 1.072) - 124}{2(\frac{z}{\text{in}})} \right) \frac{1}{2(\frac{z}{\text{in}})} \]
\[ = \left( \frac{568.122 - 124}{2(\frac{z}{\text{in}})} \right) \frac{1}{2(\frac{z}{\text{in}})} \]
\[ = \frac{444.122 \text{ kips}}{2(\frac{\text{kip}}{\text{ton}})} \approx 111.0 \text{ tons} \]

Engineering News (ENR) Formula

For Case 1 & 2: 
\[ P = \frac{2E}{(s + 0.1)} \]
\[ = \frac{2(83,880.0 \text{ ft·lbs})}{0.844 \text{ in} + 0.1} \]
\[ = 177,712 \text{ lbs} \approx 70 \text{ tons} \]
Janbu Formula

Case 1:
\[
P = \left( \frac{WH}{k_p s} \right) z = \left( \frac{WH}{k_u s} \right) \frac{1}{3\left(2000^{lb/ton}\right)^3}
= \left( \frac{7,938\, \text{lbs} \times 10.42\, \text{ft} \times 12\, \text{in/ft}}{2.697(0.844\, \text{in})} \right) \frac{1}{3\left(2000^{lb/ton}\right)^3}
= \left( \frac{435,931\, \text{lbs}}{3\left(2000^{lb/ton}\right)} \right) \approx 72.66\,\text{tons}
\]

\[
c_d = 0.75 + 0.15\left( \frac{W_p}{W} \right)
= 0.75 + 0.15(11,600\, \text{lbs} / 7,938\, \text{lbs})
= 0.969
\]

\[
\lambda = \frac{WHL}{AES^2}
= \frac{7,938\, \text{lbs} \times 10.42\, \text{ft} \times 12\, \text{in/ft}}{(144\, \text{in}^2)(4.4 \times 10^6\, \text{lbs/in}^2)(0.844\, \text{in})^2}
= 2.111
\]

\[
k_u = c_d \left[ 1 + \sqrt{1 + \left( \lambda / c_d \right)} \right]
= 0.969 \left[ 1 + \sqrt{1 + \left( 2.111 / 0.969 \right)} \right]
= 2.697
\]

Case 2:
\[
P = \left( \frac{WH}{k_p s} \right) z = \left( \frac{WH}{k_u s} \right) \frac{1}{3\left(2000^{lb/ton}\right)^3}
= \left( \frac{7,938\, \text{lbs} \times 10.42\, \text{ft} \times 12\, \text{in/ft}}{2.581(0.844\, \text{in})} \right) \frac{1}{3\left(2000^{lb/ton}\right)^3}
= \left( \frac{455,578\, \text{lbs}}{3\left(2000^{lb/ton}\right)} \right) \approx 75.93\,\text{tons}
\]

\[
c_d = 0.75 + 0.15\left( \frac{W_p}{W} \right)
= 0.75 + 0.15(4,240\, \text{lbs} / 7,938\, \text{lbs})
= 0.830
\]

\[
\lambda = \frac{WHL}{AES^2}
= \frac{7,938\, \text{lbs} \times 10.42\, \text{ft} \times 12\, \text{in/ft}}{(15.58\, \text{in}^2)(30 \times 10^6\, \text{lbs/in}^2)(0.844\, \text{in})^2}
= 2.861
\]

\[
k_u = c_d \left[ 1 + \sqrt{1 + \left( \lambda / c_d \right)} \right]
= 0.830 \left[ 1 + \sqrt{1 + \left( 2.861 / 0.830 \right)} \right]
= 2.581
Hiley Formula

Case 1:

\[
P = \left( \frac{e_W H}{s - \frac{1}{2}(c_1 + c_2 + c_3)} \right) \left( \frac{W + n^2 W_p}{W + W_p} \right) z
\]

\[
= \left( \frac{e_W H}{s - \frac{1}{2}(c_1 + c_2 + c_3)} \right) \left( \frac{W + n^2 W_p}{W + W_p} \right) \frac{1}{2.75(\text{2000 lbs/ton})}
\]

\[
= \left( \frac{1.00(7,938 \text{ lbs})(10.42 \text{ ft} \times 12 \text{ in/ft})}{0.844 \text{ in} + \frac{1}{2}(0.37 \text{ in} + 0.32 \text{ in} + 0.10 \text{ in})} \right) \left( \frac{7,938 \text{ lbs} + (0.25^2)(11,600 \text{ lbs})}{7,938 \text{ lbs} + 11,600 \text{ lbs}} \right) \frac{1}{2.75(\text{2000 lbs/ton})}
\]

\[
= \frac{355,090 \text{ lbs}}{2.75(\text{2000 lbs/ton})} \approx 64.6 \text{ tons}
\]

Case 2:

\[
P = \left( \frac{e_W H}{s - \frac{1}{2}(c_1 + c_2 + c_3)} \right) \left( \frac{W + n^2 W_p}{W + W_p} \right) z
\]

\[
= \left( \frac{e_W H}{s - \frac{1}{2}(c_1 + c_2 + c_3)} \right) \left( \frac{W + n^2 W_p}{W + W_p} \right) \frac{1}{2.75(\text{2000 lbs/ton})}
\]

\[
= \left( \frac{1.00(7,938 \text{ lbs})(10.42 \text{ ft} \times 12 \text{ in/ft})}{0.844 \text{ in} + \frac{1}{2}(0.0 \text{ in} + 0.48 \text{ in} + 0.10 \text{ in})} \right) \left( \frac{7,938 \text{ lbs} + (0.55^2)(4,240 \text{ lbs})}{7,938 \text{ lbs} + 4,240 \text{ lbs}} \right) \frac{1}{2.75(\text{2000 lbs/ton})}
\]

\[
= \frac{662,508 \text{ lbs}}{2.75(\text{2000 lbs/ton})} \approx 120.5 \text{ tons}
\]
Pacific Coast Formula

Case 1:

\[
P = \frac{E_n \left( \frac{W + kW_p}{W + W_p} \right)^z}{s + \frac{PL}{AE}}
\]

\[
E_n \left( \frac{W + kW_p}{W + W_p} \right)^z = \frac{1}{s + \frac{PL}{AE}} \times \frac{1}{4(2000 \text{ lbs/ton})}
\]

83,880 ft \cdot \text{lbs}(12 \frac{\text{in}}{\text{lbs}}) \left( \frac{7,938 \text{ lbs} + 0.1(11,600 \text{ lbs})}{7,938 \text{ lbs} + (11,600 \text{ lbs})} \right)

= \frac{0.844 \text{ in} + \frac{P(80 \text{ ft} \times 12 \frac{\text{in}}{\text{ft}})}{(144 \text{ in}^2)(4.4 \times 10^6)}}{4(2000 \text{ lbs/ton})} \times \frac{1}{4(2000 \text{ lbs/ton})}

= \frac{468,711 \text{ in} \cdot \text{lbs}}{0.844 \text{ in} + P(1.52 \times 10^{-6} \frac{\text{in}}{\text{lbs}}) \times 4(2000 \text{ lbs/ton})}

= \frac{343,511 \text{ lbs}}{4(2000 \text{ lbs/ton})} \approx 42.94 \text{ tons}
Case 2:

\[
P = \frac{E_n \left( \frac{W + kW_p}{W + W_p} \right) z}{s + \frac{PL}{AE}}
\]

\[
= \frac{E_n \left( \frac{W + kW_p}{W + W_p} \right)}{s + \frac{PL}{AE}} \times \frac{1}{4(2000 \text{ lbs/ton})}
\]

\[
= \frac{83,880 \text{ ft} \cdot \text{lbs}(12 \text{ in/ft}) \left( \frac{7,938 \text{ lbs} + 0.25(4240 \text{ lbs})}{7,938 \text{ lbs} + (4240 \text{ lbs})} \right)}{0.844 \text{ in} + \frac{P(80 \text{ ft} \times 12 \text{ in/ft})}{(15.58 \text{ in}^2)(30 \times 10^6)}} \times \frac{1}{4(2000 \text{ lbs/ton})}
\]

\[
= \frac{743,720 \text{ in} \cdot \text{lbs}}{0.844 \text{ in} + P(2.1 \times 10^{-6} \text{ in/lbs})} \times \frac{1}{4(2000 \text{ lbs/ton})}
\]

\[
= \frac{430,395 \text{ lbs}}{4(2000 \text{ lbs/ton})} \approx 53.8 \text{ tons}
\]
<table>
<thead>
<tr>
<th>Pile Length</th>
<th>CASE 1 12&quot; PC/PS Concrete Pile</th>
<th>CASE 2 HP12x53 Steel Pile</th>
</tr>
</thead>
<tbody>
<tr>
<td>80.0 ft</td>
<td>111.0 tons</td>
<td>111.0 tons</td>
</tr>
<tr>
<td>40.0 ft</td>
<td>111.0 tons</td>
<td>111.0 tons</td>
</tr>
<tr>
<td>80.0 ft</td>
<td>70 tons</td>
<td>70 tons</td>
</tr>
<tr>
<td>40.0 ft</td>
<td>70 tons</td>
<td>70 tons</td>
</tr>
<tr>
<td>42.9 tons</td>
<td>72.7 tons</td>
<td>75.9 tons</td>
</tr>
<tr>
<td>63.5 tons</td>
<td>91.5 tons</td>
<td>92.7 tons</td>
</tr>
<tr>
<td>53.8 tons</td>
<td>120.5 tons</td>
<td>135.7 tons</td>
</tr>
<tr>
<td>73.3 tons</td>
<td>135.7 tons</td>
<td>135.7 tons</td>
</tr>
</tbody>
</table>
Example 1: Calculation of Minimum Hammer Energy

Given:

Hammer Data: Delmag D36-32
   Ram Weight = 7938 lbs
   Manufacturer’s Maximum Energy Rating = 83,880 ft-lbs

Nominal Resistance = 390 kips


From the Gates Equation,

\[ R_n = (1.83 \times (E_r)^{1/2} \times \log_{10}(0.83 \times N)) - 124 \]

Rearranging for \( N \):

\[ N = \frac{10^{\frac{R_n + 124}{1.83 \times \sqrt{E_r}}}}{0.83} \]

\[ N = \frac{10^{\frac{390 + 124}{1.83 \times 83,880}}}{0.83} \]

\[ = 10^{\frac{514}{530} - 0.83} \]

\[ = 10^{0.9698 - 0.83} \]

\[ = 0.83 \]

\[ = 11.23 \approx 11 \text{ blows/ft} \]

\[ s = \text{penetration per blow in inches} \]

\[ = N^{-1}(12 \text{ in/ft}) \]

\[ = (11.23 \text{ blow/ft})^{-1}(12 \text{ in/ft}) \]

\[ = 1.07 \text{ in/blow > 0.125 in/blow} \]

\[ \therefore \text{proposed hammer meets the minimum energy requirements of 2010 Standard Specification 49-2.01C(2), Driving Equipment.} \]
Example 2: Calculations for Establishing a Blow Count Chart

Given:

Hammer Data: Delmag 36-32
Ram Weight = 7938 lbs
Maximum Stroke = 10.42 ft

Nominal Resistance = 390 kips

Assumption(s):

\[ E_r = \text{Ram Weight} \times \text{Observed Field Drop Height} \]

Observed Field Drop Height = 6 ft

From the Gates Equation,

\[ R_v = (1.83 \times (E_r)^{1/2} \times \log_{10}(0.83 \times N)) - 124 \]

Rearranging to solve for \( N \):

\[ N = 10^{\frac{R_v + 124}{1.83 \times (E_r)^{1/2}}} \times \frac{0.83}{\log_{10}(0.83 \times N)} \]

\[ N = 10^{\frac{390 + 124}{1.83 \times 47,628}} \times \frac{0.83}{\log_{10}(0.83 \times 47,628)} \]

\[ N = 10^{\frac{514}{399}} \times \frac{0.83}{\log_{10}(0.83 \times 47,628)} \]

\[ N = 10^{1.287} \times \frac{0.83}{\log_{10}(0.83 \times 47,628)} \]

\[ N = 23.33 \approx 23 \text{ blows/ft} \]

Calculations for the chart data are completed by using the Excel spreadsheet, \textit{Pile Equation-Gates.xls}, downloaded from the SC Intranet website. See next page for calculation results of the spreadsheet.
Figure E-1. Gates Formula Excel Spreadsheet.
Example 3: Calculations for Establishing a Battered Pile Blow Count Chart

Given:

Hammer Data: Delmag 36-32
- Ram Weight = 7938 lbs
- Maximum Stroke = 10.42 ft

Nominal Resistance = 390 kips

Battered pile: 1:3

Assumption(s):
- $E_r = \text{Ram Weight} \times \text{Observed Field Drop Height}$
- Observed Field Drop Height = 9 ft

As in the previous example, rearranging the Gates Formula gives,

\[
N = \frac{10^{\left(\frac{R_e+124}{1.83}\sqrt{E_r}\right)}}{0.83} = \frac{10^{\left(\frac{390+124}{1.83}\sqrt{67,775.8}\right)}}{0.83} = \frac{10^{514/476}}{0.83} = \frac{10^{1.0798}}{0.83} = 14.48 \approx 14 \text{ blows/ft} 
\]

$\theta = \sin^{-1}\left(\frac{3}{3.16}\right) = 71.565^\circ$

$E_r = 7938 \text{ lbs} \times 9 \text{ ft} \times \sin 71.565^\circ$

$= 67,775.8 \text{ ft} \cdot \text{lbs}$

Calculations for the chart data are completed by using a MODIFIED value of $E_r$, modified as shown above for the batter angle, in the Excel spreadsheet, *PileEquation-Gates.xls*. 
Example 4: Calculations for Piles with Downdrag

The following metric example has downdrag:
(Example submitted by Joy Cheung, P.E., and Anh Luu, P.E.)

Island Parkway Overcrossing – Rte 101/Ralston Interchange
EA 04-256804, Oversight Project

The Pile Data Table from the contract plans show:
Bent 2 Piles – Class 900C Alt “X” (Pile Data Table)
Nominal Resistance (Compression) = 1250 KN
Estimate Down Drag Load = 242 KN
Ultimate Pile Capacity = \( R_u \) = Nominal resistance + 2 x downdrag

Therefore:
\[ R_u = \text{Nominal resistance} + 2 \times \text{downdrag} \]
\[ R_u = 1250 \text{ KN} + (2 \times 242\text{ KN}) = 1734 \text{ KN} \]

Contractor’s proposed hammer:
Delmag D36-32

Pile Hammer Data - (per specs, Contractor provides data)
Also see Bridge Construction Memo 130-4.0, *Pile Driving Acceptance Criteria.*
Internet: [www.pileco.com](http://www.pileco.com), [www.hmc-us.com](http://www.hmc-us.com), …etc;

Pile hammer data:
Max Energy Output = 83880 ft.lbs = 83880 \times 1.3558 = 113724.5 Joules
Ram Weight = mass = 7938 lbs = 3600.6 kg
Maximum obtainable stroke/Piston Drop = height = 10’5” = 3.18 m
Find:
**Energy rating of hammer at observed field drop height in Joules**

**It is generally accepted that the energy output of an open-end diesel hammer is equal to the ram weight times the length of stroke.**

Gravitational potential energy = mass × free-fall acceleration × height = \( m \cdot g \cdot H = E_r \)

\[ E_r = 3600.6 \text{ kg} \cdot 9.81 \cdot 3.18 = 112,323 \text{ Joules} < 113,724 \text{ (Max Energy)} \]

**For battered pile,**

\[ E_r = m \cdot g (H \cdot \sin \theta) \]

\[ N = \frac{10 \left( \frac{R + 550}{7 \sqrt{E_r}} \right)}{0.83} \]

**Number of blows per 300 millimeters (maximum of 96)**

**Set up table:**

<table>
<thead>
<tr>
<th>Hammer Type: Delmag D 36-32</th>
<th>Design Load: 625kN</th>
<th>Nominal Resistance: 1734kN</th>
<th>Max Energy 113724 Joules</th>
<th>Ram Weight 3600.6Kg</th>
</tr>
</thead>
</table>

**PISTON DROP (ft) PISTON DROP (m) ENERGY (joules) GATES**

<table>
<thead>
<tr>
<th>PISTON DROP (ft)</th>
<th>PISTON DROP (m)</th>
<th>ENERGY (joules)</th>
<th>GATES</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.417</td>
<td>3.18</td>
<td>112151</td>
<td>11</td>
</tr>
<tr>
<td>10</td>
<td>3.05</td>
<td>107661</td>
<td>12</td>
</tr>
<tr>
<td>9</td>
<td>2.74</td>
<td>96895</td>
<td>13</td>
</tr>
<tr>
<td>8</td>
<td>2.44</td>
<td>86129</td>
<td>16</td>
</tr>
<tr>
<td>7</td>
<td>2.13</td>
<td>75363</td>
<td>19</td>
</tr>
<tr>
<td>6</td>
<td>1.83</td>
<td>64597</td>
<td>23</td>
</tr>
<tr>
<td>5</td>
<td>1.52</td>
<td>53831</td>
<td>31</td>
</tr>
<tr>
<td>4</td>
<td>1.22</td>
<td>43064</td>
<td>45</td>
</tr>
<tr>
<td>3</td>
<td>0.91</td>
<td>32298</td>
<td>79</td>
</tr>
</tbody>
</table>

**Set up graph:**
A very good spreadsheet (PileEquation-Gates.xls) used to calculate blows per foot using
the Gates equation can be found on the OSC Intranet Homepage under, “Downloads/
Forms”.
Continue calculations:
Contract Specifications
--Impact Hammer Minimum Energy “not less 3mm/blow at the specified bearing
value…”

Use the Gates formula again…
\[ R_u = (7 \times (E_r)^{1/2} \times \log_{10} (0.83 \times N)) - 550 \]

Find N.
Using \( E_r = 3600.6 \text{ kg} \times 9.81 \times 3.18 = 112,323 \text{ Joules} \)
\[ R_u = 1250 \text{ KN} + (2 \times 242\text{KN}) = 1734 \text{ KN} \]

\( N = 11 \text{ blows/ 300 mm} \)
\( s = \text{Penetration per blow in millimeters} \)
\( = 300 \text{ mm/11 blows} \)
\( \approx 27.0 \text{ mm} \geq 3 \text{ mm} \quad \text{OK.} \)
Note: An upper limit is not specified for the Contractor to furnish an approved hammer having sufficient energy to drive piles at a penetration rate of not less than 1/8 inch per blow at the required bearing value.
Example 5: Estimate Hammer Stroke of a Single Acting Hammer

Given:

Hammer Data: Delmag 36-32
Ram Weight = 7938 lbs
Maximum Stroke = 10.42 ft

From Field Observations: Ram Blows per Minute (bpm) = 43

From the SAXIMETER Formula,

\[ H = 4.01 \left( \frac{60}{\text{bpm}} \right)^2 - 0.3 \]

\[ H = \text{hammer stroke in feet} \]
\[ \text{bpm} = \text{field observation of hammer blows per minute} \]

\[ H = 4.01 \left( \frac{60}{43 \text{ bpm}} \right)^2 - 0.3 \]
\[ = 4.01 \left( \frac{60}{43} \right)^2 - 0.3 \]
\[ = 7.81 - 0.3 \]
\[ = 7.51 \approx 7.5 \text{ ft} \]
Example Battered Pile Blow Count Chart

**BATTERED PILE**

<table>
<thead>
<tr>
<th>PILE CAPACITY</th>
<th>140,000 POUNDS</th>
</tr>
</thead>
<tbody>
<tr>
<td>HAMMER</td>
<td>D 30-23</td>
</tr>
<tr>
<td>PISTON WEIGHT</td>
<td>6,600 POUNDS</td>
</tr>
</tbody>
</table>

\[ E = W \cdot H \cdot \sin 71.565^\circ \]

<table>
<thead>
<tr>
<th>STROKE (FEET)</th>
<th>BLOWS PER FOOT</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>15.0</td>
</tr>
<tr>
<td>9.5</td>
<td>15.9</td>
</tr>
<tr>
<td>9</td>
<td>16.9</td>
</tr>
<tr>
<td>8.5</td>
<td>18.0</td>
</tr>
<tr>
<td>8</td>
<td>19.3</td>
</tr>
<tr>
<td>7.5</td>
<td>20.8</td>
</tr>
<tr>
<td>7</td>
<td>22.6</td>
</tr>
<tr>
<td>6.5</td>
<td>24.6</td>
</tr>
<tr>
<td>6</td>
<td>27.1</td>
</tr>
<tr>
<td>5.5</td>
<td>30.2</td>
</tr>
<tr>
<td>5</td>
<td>34.0</td>
</tr>
</tbody>
</table>
Figure E-2. Example Field Acceptance Charts.
APPENDIX

F

Pile Dynamic Analysis, Static Pile Load Testing and Field Acceptance Criteria

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APPENDIX F–PILE DYNAMIC ANALYSIS, STATIC PILE LOAD TESTING
AND FIELD ACCEPTANCE CRITERIA

State of California
Department of Transportation

Memorandum

To: JOHN ZEHNDER
Structure Representative
405/22 Separation Project

Date: December 13, 2013
File: 12-ORA-22.405-PM20.66
12-071624 (1200000036)
405-22 HOV Connector Separation
Bridge No. 55-1103E

From:
DEPARTMENT OF TRANSPORTATION
Division of Engineering Services
Geotechnical Services - MS 5

Subject

Pile Dynamic Analysis (PDA) test results: Pile 19 at Bent 3L

Introduction

This report presents the results of Pile Dynamic Analysis (PDA) performed on the Pile 19 at Bent 3L of the above-referenced project. A site location map is provided for reference in Appendix A. The Pile 19 at Bent 3L was a production pile and would be incorporated into the structure. This driven pile at Bent 3L is in the control location of Bent 2 of Bridge number 55-1103E, so the same acceptance criteria needs to be used for this location. The PDA and PLT (Pile Load Test) test results along with Acceptance Criteria for the test piles at Bent 2 of Bridge number 55-1103E were provided in the report from this Office dated August 9, 2011. Pile 19 at Bent 3L was open-ended steel pipe pile of 48 inch diameter and 0.75 inch thickness spiral weld. The total length of the subject pile was 109.75 ft.

Pile Installation Summary

The subject pile was installed at Bent 3L by using vibratory hammer and then by Delmag 62-22 open ended diesel hammer. During driving by vibratory hammer top of pile at location of jaws was damaged, so contractor did remove the damaged part of pile and resurfaced top of pile.
Because of this, the approximate distance from the top of the pile to the instruments remains 7.75 ft instead of 8.0 ft (2 \times \text{diameter of pile}). According to Structure Construction, the elevation of the ground at the location of pile installation is about 15.5 ft and first 40 ft of the pile was driven using vibratory hammer. According to Contractor, the maximum fuel setting of the hammer was used during the PDA monitoring of the pile. PDA monitoring of the pile was conducted during initial drive on November 26, 2013; and during restrike on December 11, 2013 by Jason Wahllethner, Toua Vang, Wendy Tkacheff of the Foundation Testing Branch. By mistake the instruments F3 and F4 were swapped with each other, and A1 and A2 were left on in the machine, which was corrected during data analysis. The PDA instruments were attached approximately at 7.75 feet below the top of the pile, and PDA test pile was longer than other production piles to attach the PDA instruments.

**PDA Monitoring and Analysis**

The Contractor utilized DELMAG 62-22 diesel hammer to drive the subject pile, and manufacturer’s published specifications as available in the GRLWEAP Hammer Database File are shown in Table I.

<table>
<thead>
<tr>
<th>Hammer</th>
<th>Rated Energy</th>
<th>Ram Weight</th>
<th>Max. Stroke</th>
</tr>
</thead>
<tbody>
<tr>
<td>DELMAG 62-22</td>
<td>164.6 (kip-ft)/kw</td>
<td>13.6 kips</td>
<td>12.05 ft</td>
</tr>
</tbody>
</table>

*Source: GRLWEAP Hammer Database File.

Pile Dynamic Analysis (PDA) monitoring was performed by utilizing a Pile Driving Analyzer® Model PAX 1, manufactured by Pile Dynamics Inc. and as per ASTM D4945-08. Elevation information is based on information provided by Structure Construction. Four strain sensors and two accelerometers were used to monitor the pile driving. The sensors used for the test pile are summarized in Table II.
Table II: PDA Sensor Data

<table>
<thead>
<tr>
<th>Sensor Type</th>
<th>ID No.</th>
<th>Calibration Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Accelerometer A3</td>
<td>855</td>
<td>5/14/2013</td>
</tr>
<tr>
<td>Accelerometer A3**</td>
<td>855</td>
<td></td>
</tr>
<tr>
<td>Accelerometer A4</td>
<td>837</td>
<td>5/14/2013</td>
</tr>
<tr>
<td>Accelerometer A4**</td>
<td>837</td>
<td></td>
</tr>
<tr>
<td>Strain Gauge F1</td>
<td>7343</td>
<td>6/8/2012</td>
</tr>
<tr>
<td>Strain Gauge F1**</td>
<td>7343</td>
<td></td>
</tr>
<tr>
<td>Strain Gauge F2</td>
<td>5152</td>
<td>6/8/2012</td>
</tr>
<tr>
<td>Strain Gauge F2**</td>
<td>5152</td>
<td></td>
</tr>
<tr>
<td>Strain Gauge F3</td>
<td>7346</td>
<td>6/8/2012</td>
</tr>
<tr>
<td>Strain Gauge F3**</td>
<td>6197</td>
<td>6/8/2012</td>
</tr>
<tr>
<td>Strain Gauge F4</td>
<td>5156</td>
<td>5/13/2013</td>
</tr>
<tr>
<td>Strain Gauge F4**</td>
<td>6199</td>
<td>6/8/2012</td>
</tr>
</tbody>
</table>

* Calibration required per ASTM every two years.
** Instruments used during restrike.

Measured strains and accelerations induced in the pile as a result of driving were used to determine various engineering parameters of interest. Some of the more significant attributes derived for each hammer blow include the maximum energy transferred from the hammer, maximum compressive stresses, and the blow count. Plots depicting these parameters as a function of penetration depth are presented in Appendix B. The PDA-monitoring results of driving the subject pile are summarized in Table III.

Table III: PDA Monitoring Results; Pile 19 at Bent 3L

<table>
<thead>
<tr>
<th>Approx Elevation of Pile Tip at Start of Monitoring (ft)</th>
<th>Initial Drive</th>
<th>Restrike</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-59.5</td>
<td>-83.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Transferred Energy (EMX)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near End of Initial Drive (k-ft)</td>
</tr>
<tr>
<td>Near Beginning of Restrike (k-ft)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Maximum Average Compressive Stress (CSX)</th>
</tr>
</thead>
<tbody>
<tr>
<td>During Initial Drive (ksi)</td>
</tr>
<tr>
<td>During Restrike (ksi)</td>
</tr>
</tbody>
</table>

“Caltrans improves mobility across California”
Table III (continued): PDA Monitoring Results; Pile 19 at Bent 3L

<table>
<thead>
<tr>
<th>Maximum Individual Compressive Stress (CSI)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>During Initial Drive (ksi)</td>
<td>29.7</td>
</tr>
<tr>
<td>During Restrike (ksi)</td>
<td>31.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Approx Blow Counts During Driving</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Near End of Initial Drive, 60 Blows/ft</td>
<td></td>
</tr>
<tr>
<td>During Restrike*, 200 Blows/ft</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Stroke Length (ft)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Near End of Initial Drive, 10.3 ft</td>
<td></td>
</tr>
<tr>
<td>Near Beginning of Restrike, 11.7 ft</td>
<td></td>
</tr>
</tbody>
</table>

*Extrapolated from first 1/10th of foot driving during restrict.

Maximum CSX and CSI are within allowable limits (90% of 50 ksi) during initial and restrike PDA monitoring. According to the Acceptance Criteria (Pile 10 at Bent 2, Bridge number 55-1103E report dated August 9, 2011), Pile 19 at Bent 3L during initial driving near tip of pile with approximately 60 blows/ft and 10.3 ft stroke length have reached the required capacity of 1800 kips.

Recommendations

This Office recommends Structure Construction, Geotechnical and Structure designers to review the results of PDA testing in this report and use as needed. Any questions or comments regarding the change in design or tip elevations should be directed to Structure Construction personnel or designers.
If you have any questions or comments regarding this memorandum please contact Tejinderjit Singh, P.E. at (916) 227-1052.

TEJINDERJIT SINGH, P.E.
Transportation Engineer
Foundation Testing Branch
Office of Geotechnical Support

RONNIE GU, P.E.
Senior Transportation Engineer (Acting)
Foundation Testing Branch
Office of Geotechnical Support

Attachments
C:  R. Stott - SC (Email)
    B. Alsamman - SC (Email)
    S. K. Amiri - OGDSI (Email)
    S. Kim - SC (Email)
APPENDIX A

Location Map

405/22 HOV Connector Separation
Bridge No. 55-1103E
Appendix F–Pile Dynamic Analysis, Static Pile Load Testing and Field Acceptance Criteria

Foundation Testing Branch

Bent 3L Pile 19 Testing Location

BEGIN CONSTRUCTION (ROUTE 405)
Sta "NB405" 296+03 PM 20.4

END CONSTRUCTION
Sta "E" 3333+00 PM

GENERAL SITE LOCATION MAP
405-22 HOV Connector Separation Bridge
Pile 19 at Bent 3L - PDA-Monitored Drive

Bridge No. 55-1103E
Contract No. 12-071624
12-ORA-22,405-PM20.75

CISS 48X0.75 inch Steel Pipe Piles
Installed: 11/28/2013
APPENDIX B

PILE DYNAMIC ANALYSIS PLOT

405/22 HOV Connector Separation
Bridge No. 55-1103E
APPENDIX C

Pile Information

405/22 HOV Connector Separation
Bridge No. 55-1103E
APPENDIX F—PILE DYNAMIC ANALYSIS, STATIC PILE LOAD TESTING AND FIELD ACCEPTANCE CRITERIA  
OCTOBER 2015

Memorandum

To: BINH NGO  
Oversight Structure Representative  
405 / Wilmington Avenue  
Interchange Improvements

From: DEPARTMENT OF TRANSPORTATION  
Division of Engineering Services  
Geotechnical Services - MS 5

Date: June 3, 2015

File: 07-LA-405-9 3/9.9  
07-234004 (0700000394)  
Dominguez Channel Bridge  
Bridge No. 53-1166

Subject: Pile Dynamic Monitoring Results and Bearing Acceptance Criteria: Bents 4 and 6 Control Locations

Introduction

This memorandum presents the dynamic monitoring results and bearing acceptance criteria, prepared in accordance with Section 10-1.55 “PILING” of the Special Provisions, for the control locations identified in the Special Provisions (page 227) as Bents 4 and 6 of the Dominguez Channel Bridge (Widen), Bridge No. 53-1166.

Foundation Description

The Dominguez Channel Bridge (Widen) pile foundations include open-ended Cast-In-Steel-Shell Concrete Piling (CISS 24” x 0.5”). Specified tip elevations are controlled by compression demands. Pile data specified in the Contract Plans (Appendix C) for the control location is shown below in Table 1.

Table 1: Pile Data

<table>
<thead>
<tr>
<th>Location</th>
<th>Pile Type (in.)</th>
<th>Nominal Resistance Compression (Kips)</th>
<th>Nominal Resistance Tension (Kips)</th>
<th>Cut-off Elevation(1) (ft)</th>
<th>Specified Tip Elevation (ft)</th>
<th>Pile Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bent 4</td>
<td>CISS 24 x 0.5</td>
<td>860</td>
<td>0</td>
<td>28.2</td>
<td>-87.0</td>
<td>115.2</td>
</tr>
<tr>
<td>Bent 6</td>
<td>CISS 24 x 0.5</td>
<td>720</td>
<td>0</td>
<td>27.5</td>
<td>-80.0</td>
<td>107.5</td>
</tr>
</tbody>
</table>

(1) Pile Cut-Off Elevations obtained from Log Pile Sheets.

“Provide a safe, sustainable, integrated and efficient transportation system to enhance California’s economy and livability.”
Subsurface Conditions

The subsurface conditions at the test pile locations are characterized by Geotechnical Log of Test Boring (LOTB) R-07-017 (October 2007), located between the two test piles. For a complete description of the subsurface conditions, please refer to the LOTB included in Appendix D.

Test Pile Installation

Bents 4 and 6 test piles were driven at the location of production piles. Exact location of the test piles within the bent layout is the outside pile, as marked on the Contract Plan sheet Foundation Plan included in Appendix C. Pile material consisted of A252 Grade 3 steel pipe based on field observation. The driving system consisted of an APE D46-32 single-acting diesel hammer based on field observation and discussions with field inspectors. The APE D46-32 has an approximate rated energy of 114 Kip-ft, a stroke of 11.25 feet at maximum rated energy, and a ram weight of 10.14 Kips. The driving system submitted to the project identified a Delmag D46-32 hammer. Both the APE and Delmag D46-32 hammers have an equivalent ram weight of 10.14 Kips.

Each test pile was driven in two pieces and included one field welded splice. The hammer was operated at the maximum fuel setting at the Bent 6 test pile. At Bent 4 the fuel setting was adjusted from fuel setting 3 to the maximum fuel setting during the initial drive. Pre-drilling occurred at both test pile locations prior to installation of the first pile piece. Approximately 16.7 feet of pre-drilling occurred at Bent 4 and approximately 21.4 feet of pre-drilling occurred at Bent 6 according to field inspectors. Center-relief drilling was not used during test pile installation. The depth to soil plug within the open-ended piles following driving to specified tip elevation was not measured due to the approximate 30 feet of pile extension above grade.

Table II: Test Pile Driving Information

<table>
<thead>
<tr>
<th>Location</th>
<th>Date of Initial Drive&lt;sup&gt;(3)&lt;/sup&gt;</th>
<th>Date of Restrike</th>
<th>Test Pile Length (ft)</th>
<th>Length of Pile Penetration (ft)</th>
<th>Approx. Depth to Soil Plug&lt;sup&gt;(4)&lt;/sup&gt; (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bent 4</td>
<td>5-26-15</td>
<td>5-27-15</td>
<td>117.0</td>
<td>85.7</td>
<td>n/a</td>
</tr>
<tr>
<td>Bent 6</td>
<td>5-28-15</td>
<td>5-29-15</td>
<td>109.5</td>
<td>76.4</td>
<td>n/a</td>
</tr>
</tbody>
</table>

<sup>(3)</sup> Initial drive defined as continuous driving event to within one foot of specified tip after pile splicing.

<sup>(4)</sup> Not available due to over 30 feet of pile extension above grade following drive to tip elevation.

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Pile Dynamic Monitoring

Dynamic Monitoring was conducted utilizing a Pile Driving Analyzer manufactured by Pile Dynamics, Inc. Monitoring of the test piles was conducted by Engineers Jason Wahleithner and Jeremy Peterson-Self of the Caltrans Foundation Testing Branch (FTB). Calibration dates for strain gages and accelerometers used in monitoring are listed in Table III.

**Table III. Dynamic Monitoring Sensor Information**

| Sensor Type      | ID No. | Date of Calibration
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Accelerometer A3</td>
<td>K1291</td>
<td>10-28-2014</td>
</tr>
<tr>
<td>Accelerometer A4</td>
<td>K1298</td>
<td>10-28-2014</td>
</tr>
<tr>
<td>Strain Gauge F3</td>
<td>7342</td>
<td>10-14-2014</td>
</tr>
<tr>
<td>Strain Gauge F4</td>
<td>3664</td>
<td>10-14-2014</td>
</tr>
</tbody>
</table>

(1) Calibration recommended every two years by ASTM 4945.

Monitoring was performed during the final approximate 29 feet of initial driving at Bent 4 and the final approximate 23 feet of initial driving at Bent 6. Additional monitoring was also performed during a one foot restrike at each location. Results from dynamic monitoring are summarized in Table IV and included in Appendix B. Corresponding elevations, estimated to nearest tenth of a foot, were derived from temporary bench marks placed on existing columns and identified by field inspectors.

“Provide a safe, sustainable, integrated and efficient transportation system to enhance California’s economy and livability.”
Table IV. Pile Dynamic Monitoring Results

<table>
<thead>
<tr>
<th></th>
<th>Bent 4</th>
<th>Bent 6</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Initial Drive</td>
<td>Restrike</td>
</tr>
<tr>
<td>Pile Tip Elevation At Start of Monitoring</td>
<td>(ft)</td>
<td>-57.3</td>
</tr>
<tr>
<td>Pile Tip Elevation At End of Monitoring</td>
<td>(ft)</td>
<td>-86.0</td>
</tr>
<tr>
<td>Transferred Energy: End Initial Drive [EMX]</td>
<td>(Kip-ft)</td>
<td>41.2</td>
</tr>
<tr>
<td>Transferred Energy: Begin Restrike [EMX]</td>
<td>(Kip-ft)</td>
<td>--</td>
</tr>
<tr>
<td>Peak Max Average Compressive Stress [CSX]</td>
<td>(Ksi)</td>
<td>28.6</td>
</tr>
<tr>
<td>Peak Max Individual Compressive Stress [CSI]</td>
<td>(Ksi)</td>
<td>31.8</td>
</tr>
<tr>
<td>Stroke at End of Initial Drive [STK]</td>
<td>(ft)</td>
<td>8.5</td>
</tr>
<tr>
<td>Blow Count at End of Initial Drive [BLC]</td>
<td>(Blows/ft)</td>
<td>92</td>
</tr>
<tr>
<td>Stroke at Beginning of Restrike [STK]</td>
<td>(ft)</td>
<td>--</td>
</tr>
<tr>
<td>Blow Count at Beginning of Restrike [BLC]</td>
<td>(Blows/ft)</td>
<td>--</td>
</tr>
<tr>
<td>Blow Count for Entire Restrike [BLC]</td>
<td>(Blows/ft)</td>
<td>--</td>
</tr>
</tbody>
</table>

(1) Extrapolated from blow count over first 0.1 foot of restrike.

Discussion of Dynamic Monitoring Results

The test piles appear to have been driven without damage while being monitored with a PDA. The pile driving stresses measured by the PDA during initial driving and restrike did not exceed the allowable driving stress of 40.5 ksi which represents 90% of the minimum yield stress for the A252 Grade 3 pipe material. Substantial difference between the average compressive stress of both strain gages and the peak compressive stress of any individual strain gage indicates that uneven stresses were induced in the pile. Pile and hammer alignment must be properly maintained to impart maximum energy to the pile and limit the potential for pile damage.

"Provide a safe, sustainable, integrated and efficient transportation system to enhance California's economy and livability."
Wave equation analysis was performed utilizing the pile driving analysis software CAPWAP Version 2006 in order to predict test pile capacity at the end of initial drive and the beginning of restrike. The CAPWAP derived pile capacities are presented in Table V.

**Table V. CAPWAP Derived Pile Capacity**

<table>
<thead>
<tr>
<th></th>
<th>Bent 4</th>
<th>Bent 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Capacity at End of Initial Drive (Kips)</td>
<td>775</td>
<td>827</td>
</tr>
<tr>
<td>Capacity at Beginning of Restrike (Kips)</td>
<td>1,143</td>
<td>1,294</td>
</tr>
</tbody>
</table>

**Pile Bearing Acceptance Criteria**

Bearing acceptance criteria was developed by this Office based on the results of wave equation analysis correlated to pile dynamic measurements for the test piles driven at Bents 4 and 6. Wave equation analysis was performed utilizing the pile driving analysis software CAPWAP Version 2006 and GRL-WEAP Version 2010-4. According to the Special Provisions, the bearing acceptance criteria for the Bents 4 and 6 Control Locations shall be used for field acceptance of production pile driving. Bearing Acceptance Criteria Charts depicting estimated pile nominal compressive resistance (pile capacity) as a function of blow count and hammer stroke are presented in Appendix A. Estimated setup factors were calculated from initial drive and restrike monitoring and are also provided in Appendix A.

Bearing acceptance criteria presented in this report relate driving resistance to stroke and blow count. Bearing acceptance criteria are valid only for the APE D46-32, single-acting diesel hammer when operating properly for the installation of CISS (24" x 0.5") piling. The influence of uneven or eccentric blows is not addressed by these charts but should be considered. The values identified for stroke have been correlated to effective energy transferred to the pile for the driving event monitored. This field acceptance criteria will not be valid if the performance of the hammer is altered or if the construction practice is altered in a way that affects imparted energy.

*Provide a safe, sustainable, integrated and efficient transportation system to enhance California's economy and livability.*
Recommendations

This Office recommends review of the attached dynamic monitoring results by the Dominguez Channel Bridge (Widen) geotechnical designer when considering release of production piles within the subject control zones.

The derived pile driving resistance relationship plot presented may be used to estimate pile driving resistance from observed blow count and stroke height for initial driving. The field acceptance criteria presented are pile type, site and hammer specific and should not be applied to piles outside the control zone. If the driving system is modified or if a new driving system is utilized, additional dynamic monitoring should be performed to verify dynamic characteristics and develop revised bearing acceptance criteria.

If you have any questions or comments regarding this report, please contact Jason Wahleithner at (916) 227-1059.

JASON D. WAHLEITHNER, P.E.
Transportation Engineer, Civil
Foundation Testing Branch
Office of Geotechnical Support

c:  V. Francis  – OSC (Email)
    T. Liu    – OGDS1 (Email)
    Z. Saleh  – Hill International (Email)
APPENDIX A

Bearing Curve and Setup Curve

Dominguez Channel Bridge (Widen)
Bridge No. 53-1166
Bents 4 and 6 Control Locations
Bent 4 Control Location
For Use With APE D46-32 and 24"x 1/2" CISS Pile

Bearing Acceptance Criteria Chart
Derived from Wave Equation Analysis Program (WEAP)

EA Number 07-234004
Bridge Number 53-1166
07-LA-405-9.3/9.9

Bent 4 Control Location
Dominguez Channel Bridge (Widen)
Foundation Testing Branch

Bent 4 Control Location
For Use With APE D46-32 and
24"x 1/2" CISS Pile

Percentage of Ultimate Compressive Capacity (%)

Time (Days)

Bearing Acceptance Criteria: Setup Relationship
Derived from CapWap Capacities

EA Number 07-234004
Bridge Number 53-1166
07-LA-405-9.3/9.9

Bent 4 Control Location
Dominguez Channel Bridge (Widen)
Bent 6 Control Location
For Use With APE D46-32 and
24" x 1/2" CISS Pile

Bearing Acceptance Criteria Chart
Derived from Wave Equation Analysis Program (WEAP)

EA Number 07-234004
Bridge Number 53-1166
07-LA-405-9.3/9.9

Domínguez Channel Bridge (Widen)
Foundation Testing Branch

Bent 6 Control Location
For Use With APE D46-32
and
24" x 1/2" CISS Pile

Bearing Acceptance Criteria: Setup Relationship
Derived from CapWap Capacities

EA Number 07-234004
Bridge Number 53-1166
07-LA-405-9.3/9.9

EA Number 07-234004
Bent 6 Control Location
Domínguez Channel Bridge (Widen)
The following presents an example of how to use the “Time-Setup Relationship” Curve at Bent 4. Use the same method for Bent 6 but with the setup factor from the Bent 6 time-setup relationship.

1) The production pile was driven to within 1 foot from the specified tip elevation.

2) 30 blows per foot were recorded with an 8.0 ft stroke at this depth. According to the Bearing Acceptance Criteria Chart (Bearing Graph), a Nominal Compressive Resistance of 585 kips is obtained (this is at 68% of the ultimate compressive capacity according to the time-setup relationship).

3) After a setup period of 1 day, the projected Nominal Compressive Resistance can be extrapolated as:

   585 kips x (1/0.68) = 860 kips
APPENDIX B

Pile Dynamic Monitoring Results

Dominguez Channel Bridge (Widen)
Bridge No. 53-1166

Bents 4 and 6 Control Locations
APPENDIX C

Contract Plan Pile Data Table and Foundation Plan

Dominguez Channel Bridge (Widen)
Bridge No. 53-1166
Bents 4 and 6 Control Locations
APPENDIX D

Pile Log Sheet
Log of Test Borings

Dominguez Channel Bridge (Widen)
Bridge No. 53-1166

Bents 4 and 6 Control Locations
<table>
<thead>
<tr>
<th>Foundation (f)</th>
<th>Tip Elevat (ft)</th>
<th>Blows/l</th>
<th>Blows/M</th>
<th>Equiv Stroke (in)</th>
<th>Time (s)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-58.34</td>
<td>48</td>
<td></td>
<td></td>
<td></td>
<td>Welded connection</td>
</tr>
<tr>
<td>2</td>
<td>-59</td>
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<tr>
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<td>-81</td>
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</tbody>
</table>
### PDA Testing

**Bridge Name:** Berrinquez Channel Bridge (Widen)  
**Sheet No.:** PDA-41  
**PDA No.:** 53-116c  
**Hammer Model:** APE  
**Hammer Mlde:** No. 32  
**Tip Elev.:** 57.64  
**Tip Dev.:** -7.87  
**Pile Type:** D/3  
**Pile Length:** 115.24  
**Date(s) Driven:** 5/26/15, 5/27/15  
**Inspected by:** J. Martinez

| Penetration (ft) | Tip Elev. (ft) | Blowff | Stresses/M | Time | Rema
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
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</tr>
</thead>
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<tr>
<td>25</td>
<td>-82</td>
<td>80</td>
<td></td>
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</tr>
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<td>20</td>
<td>-83</td>
<td>72</td>
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<tr>
<td>27</td>
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<td>80</td>
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<td>28</td>
<td>-85</td>
<td>85</td>
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</tr>
<tr>
<td>5.26.15</td>
<td>29</td>
<td>86</td>
<td>0.5±</td>
<td></td>
<td>Stop at 28.7± ftet</td>
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<tr>
<td>5.27.15</td>
<td>30</td>
<td>87</td>
<td>0.5±</td>
<td></td>
<td>Stop at 29.7± ftet</td>
</tr>
</tbody>
</table>

**Note:** 0.5±
### Wilmington Ave Interchange Modification at 405 Fwy
Federal Aid Project No. H4714-5403(07)
Contract No. DT-231-L4
Prelim Plan No. 0100022592

**APPENDIX F—PILE DYNAMIC ANALYSIS, STATIC PILE LOAD TESTING AND FIELD ACCEPTANCE CRITERIA**

**STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION**

**LOG PILE SHEET**

3C-4803 (Form 3C-4803 C76) (REV. 10/6/13)

**PDA Testing**

<table>
<thead>
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<th>Bridge Name</th>
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<tr>
<td>Hammer Make</td>
<td>APE</td>
</tr>
<tr>
<td>Model</td>
<td>DAB-32</td>
</tr>
<tr>
<td>Energy (ft-lbs)</td>
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</tr>
<tr>
<td>Reference Depth</td>
<td>Tip of Pile Head 107.45'</td>
</tr>
<tr>
<td>Pile Type</td>
<td>EISS</td>
</tr>
<tr>
<td>Pile TIP Elev.</td>
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<td>Pile Cut Off Elev.</td>
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<tr>
<td>Debit(s) Driven</td>
<td>05.28.15 &amp; 05.29.15</td>
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</table>

<table>
<thead>
<tr>
<th>Penetration (ft)</th>
<th>Tip Elev. (ft)</th>
<th>Slow Lift</th>
<th>Blow / Min</th>
<th>Body Strike</th>
<th>Time</th>
<th>Remarks</th>
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<tbody>
<tr>
<td>1</td>
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<tr>
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<td>-64</td>
<td>30</td>
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<td>23:24</td>
<td>Welded Connection</td>
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<td>52.9</td>
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<td>31:01</td>
<td>Elev. - 3.43' (Top of Pad)</td>
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<td>Elev. - 25.0'</td>
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<td>ELEV. - 54.06'</td>
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<td>23</td>
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<td>77</td>
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<td>24</td>
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<td>8.6 ±</td>
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</tr>
</tbody>
</table>

**CALTRANS • FOUNDATION MANUAL**

F - 42
Memorandum

To: MANO MIRZAI
Structure Representative
Fresno Field Construction Office

Date: April 3, 2013

From: DEPARTMENT OF TRANSPORTATION
Division of Engineering Services
Geotechnical Services - MS 5

File: 06-Fre,Mad-99-26.7/R31.6, R0.0/R1.6
06-442624 (06 0000 0972)
San Joaquin River Bridge (Replace)
Bridge No. 41-0090

Subject: Pile Dynamic Analysis, Pile Load Test Results, and Pile Field Acceptance Criteria:
Test Pile at Pier 3

Attached is a report from this Office containing the Pile Dynamic Analysis, Pile Load Test results, and the Pile Field Acceptance Criteria for the Test Pile at Pier 3 of the above-referenced project.

If you have any questions or comments regarding this report, please contact James L. Ta, P.E. at (916) 227-1050.

DOUGLAS E. BRITTSAN, G.E.
Senior Transportation Engineer
Foundation Testing Branch

Attachments

c: R. Stott - SC (Email)
   B. Alsaman - SC (Email)
   K. Low - SC (Email)
   N. Quiroz - SC (Email)
   R. Simmons - SD (Email)
   Q. Huang - OGDN (Email)
   W. Bertucci - OGDN (Email)
   T. Shantz - DRI (Email)
   Geodog

"Caltrans improves mobility across California"
FOUNDATION TESTING BRANCH

April 3, 2013

06-Fre,Mad-99-26.7/R31.6,R0.0/R1.6
06-442624

San Joaquin River Bridge (Replace)
Bridge No. 41-0090

Pile Dynamic Analysis, Pile Load Test Results, and Pile Field Acceptance Criteria:

Test Pile at Pier 3
April 3, 2013

**Project Information**

06-Fre,Mad-99-26.7/R31.6,R0.0/R1.6
06-442624
San Joaquin River Bridge (Replace)
Bridge No. 41-0090

**Subject**

Pile Dynamic Analysis, Pile Load Test Results, and Pile Field Acceptance Criteria:
Test Pile at Pier 3

**Introduction**

This report presents the pile dynamic analysis (PDA), pile load test (PLT) results, and pile field acceptance criteria for the Test Pile at Pier 3 of the San Joaquin River Bridge replacement structure. Static axial load testing was conducted on the 74.5-inch diameter by 1.25-inch thick Cast-in-Steel-Shell (CISS) open ended steel pipe pile with respect to two different pile installation methods. The subject Test Pile is identified on the contract plans as Pile No. 1 at Pier 3. The compression load test used four reaction anchor piles each consist of a 48-inch diameter by 1.0-inch thick steel pipe piling. The Test Pile and four anchor piles were driven using the APE D180-42 and Pileco D225-22/32, open ended, single acting diesel impact hammers. A site location map of the Test Pile is provided in Appendix A.

The Test Pile at Pier 3 is a production pile that will be incorporated into the San Joaquin River Bridge foundations. Based on the contract plans, the four anchor piles used for PLT are non-production piles that will be cut below grade and leave in-placed.

The criteria detailed herein are designed to provide pile compressive acceptance criteria for the 74.5-inch diameter CISS piling based on wave equation analysis and PDA data with correlations to the static axial test results of the measured load-displacement behavior of the Test Pile. As
such, the Test Pile at Pier 3 is intended to be representative of the CISS piling for the control locations of Piers 2 to 6 of the San Joaquin River Bridge replacement foundations.

Foundation Description

The replacement San Joaquin River Bridge is a six-span structure. Based on contract plans, Abutments 1 and 7 are designed to be supported by Class 140 Alt W piling. Piers 2 to 6 are designed to be supported by a five-column pier with CISS piling. A total of twenty-five (25) 74.5-inch diameter by 1.25-inch thick Cast-in-Steel-Shell (CISS) piles are planned to be installed at the proposed new bridge structure. The specified “designed” nominal resistance in compression for the production piles (Piers 2 to 6) at the San Joaquin River Bridge ranges from 4070 kips to 6230 kips with zero demand for tension. Nominal driving resistance ranges from 4230 kips to 6640 kips were shown on the plans (referred to Pile Data Table for specific support location).

Subsurface Conditions

Stratigraphy at the San Joaquin River Bridge load test site can be characterized by Geotechnical Boring RC-11-001 (boring dated 05-25-11). The site stratigraphy consists of granular materials of loose to medium dense to very dense silty sand, sand, silt, and sandy silt. Trace of gravel was encountered within the granular soil matrix. Very stiff to hard silty/lean clay, sandy clay, and silt were encountered and interbedded within the soil matrix. For complete description of the subsurface conditions at the test site, please refer to the Log of Test Boring provided in Appendix E.

Pile Installation Summary

The Test Pile at Pier 3 utilized for the compressive static axial load testing was a 74.5-inch diameter with 1.25-inch shell thickness, open ended steel pipe pile. Each of the four (4) anchor piles consist of a 48-inch diameter with 1.0-inch shell thickness, open ended steel pipe pile. All piling conformed to ASTM A252, Grade 3 and were manufactured by XKT Engineering, Inc. of Vallejo, CA. At Pier 3, the anchor pile (AKA Pile A, utilized for PDA monitoring) was driven with the APE D180-42 diesel hammer (max Fuel Setting during PDA monitoring) and the Test
Pile was driven with the Pileco D225-22/32 diesel hammer (Fuel Setting set from minimum to maximum during PDA monitoring) to approximate tip elevation of 52 feet (as reported). Original Grade (OG) elevation at the location of the Test Pile was estimated to be approximately elevation 237.5 feet.

Pile Dynamic Analysis - Test Pile & Anchor Pile

The contractor installed the isolation CMP casing. Clean out inside the CMP casing was provided to near casing tip. The first and second pipe sections (80 feet each) of the Test Pile were driven with the APE D180-42 hammer followed by field welding. Pile driving installations of the third-section of the steel pipe (76 feet) were utilized by both the APE D180-42 and Pileco D225-22/32 hammers. PDA monitoring (Pileco D225-22/32) was performed on March 8, 2013 for Test Pile and was driven to approximate tip elevation of 52 feet. The initial PDA monitoring of the Test Pile was referenced to penetration depths of 168.5 ft to 173.2 ft (elevation 56.7 ft to elevation 52 ft) with datum referenced to ground surface inside CMP casing (estimated at elevation 225.2 ft).

PDA monitoring was performed on February 22, 2013 for the Anchor Pile (Pile A) and was driven to approximate tip elevation of 106 feet. The initial PDA monitoring of the Anchor Pile was referenced to penetration depths of 104 ft to 130 ft (elevation 132 ft to elevation 106 ft) with datum reference at OG.

Characteristics of the Pileco D225-22/32 and APE D180-42 Diesel hammers are provided in Table 1 below that include the rated energy, Eq. max stroke, Eq. rate stroke, ram weight, and helmet/cap weight. Manufacturer’s published specifications (additional information provided by the Contractor from other project with different hammer ID number) of the diesel hammers utilized to install the piles at the test site are shown in Table 1. The below data were utilized for analysis purposes.
Table 1: Hammer Characteristics

<table>
<thead>
<tr>
<th>Hammer</th>
<th>Rated Energy* (kip-ft)</th>
<th>Ram Weight* (kips)</th>
<th>EQ. Rated Stroke* (EQ. Max Stroke)* (ft)</th>
<th>Helmet/Cap Weight** (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pileco D225-22/32***</td>
<td>555.34</td>
<td>49.584</td>
<td>11.2 {14.3}</td>
<td>45</td>
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<tr>
<td>APE D180-42</td>
<td>446.51</td>
<td>39.69</td>
<td>11.25 {16.75}</td>
<td>45</td>
</tr>
</tbody>
</table>

**Contractor Submittal.
***Per Contractor submittal, Pileco D225-32 is the same as Pileco D225-22 from GRLWEAP hammer database.

Pile Dynamic Analysis (PDA) monitoring was performed on the Test Pile for 4.7 feet of driving, utilizing a Pile Driving Analyzer® Model PAX, Serial No. 37051, manufactured by Pile Dynamics Inc. Four strain sensors and two accelerometers were used to monitor the Test Pile driving operations. All sensors were mounted on the outer surface of the pile at approximately 12.3 ft below top of pile. The sensor data for the Test Pile at Pier 3 are summarized in Table II.

Table II. PDA Sensor Data

<table>
<thead>
<tr>
<th>Sensor Type</th>
<th>ID No.</th>
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<tbody>
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<td>K1307</td>
</tr>
<tr>
<td>Accelerometer A4</td>
<td>K1300</td>
</tr>
<tr>
<td>Strain Gauge F1</td>
<td>C500</td>
</tr>
<tr>
<td>Strain Gauge F2</td>
<td>C501</td>
</tr>
<tr>
<td>Strain Gauge F3</td>
<td>C502</td>
</tr>
<tr>
<td>Strain Gauge F4</td>
<td>C494</td>
</tr>
</tbody>
</table>

Measured strains and accelerations induced in the pile as a result of driving were used to determine various engineering parameters of interest. Some of the more significant attributes derived for each hammer blow include the maximum energy transferred from the hammer to the pile, maximum pile compressive stresses, and the blow count. Plots depicting these parameters as a function of penetration depth are presented in Appendix B. PDA test results for the Anchor Pile (Pile A) and Test Pile at Pier 3 are summarized in Tables III(a) and III(b), respectively.
### Table III(a). PDA Test Results of Anchor Pile (Pile A) at Pier 3

<table>
<thead>
<tr>
<th>Approx. Elevation of Pile Tip for PDA Monitoring</th>
<th>English Units</th>
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<tbody>
<tr>
<td>Start of Initial Drive</td>
<td>132 ft</td>
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<tr>
<td>End of Initial Drive</td>
<td>106 ft</td>
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<tr>
<td>Max Transferred Energy (EMX)</td>
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</tr>
<tr>
<td>End of Initial Drive*</td>
<td>246.179 kip-ft*</td>
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<tr>
<td>Maximum Average Compressive Stress (CSX)</td>
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</tr>
<tr>
<td>Initial Drive*</td>
<td>26.246 ksi*</td>
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<td>Maximum Individual Compressive Stress (CSI)</td>
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<td>Initial Drive*</td>
<td>33.739 ksi*</td>
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<td>Actual Field Blow Counts</td>
<td>28 blows/ft</td>
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<td>End of Initial Drive</td>
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Note:
1) *Value based on full depth range of PDA data.
Table III(b). PDA Test Results of Test Pile at Pier 3

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<tr>
<th>Approx. Elevation of Pile Tip for PDA Monitoring</th>
<th>English Units</th>
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</thead>
<tbody>
<tr>
<td>Start of Initial Drive</td>
<td>56.7 ft</td>
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<tr>
<td>End of Initial Drive</td>
<td>52.0 ft</td>
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<table>
<thead>
<tr>
<th>Max Transferred Energy (EMX)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>End of Initial Drive*</td>
<td>329.06 kip-ft*</td>
</tr>
<tr>
<td>End of Initial Drive**</td>
<td>286.9 kip-ft**</td>
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<thead>
<tr>
<th>Maximum Average Compressive Stress (CSX)</th>
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</thead>
<tbody>
<tr>
<td>Initial Drive*</td>
<td>27.178 ksi*</td>
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<tr>
<td>Initial Drive**</td>
<td>26.09 ksi**</td>
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</table>

<table>
<thead>
<tr>
<th>Maximum Individual Compressive Stress (CSI)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Drive*</td>
<td>29.872 ksi*</td>
</tr>
<tr>
<td>Initial Drive**</td>
<td>27.31 ksi**</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Actual Field Blow Counts</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>End of Drive</td>
<td>149 blows/ft</td>
</tr>
</tbody>
</table>

Notes:
1) *Value based on full depth range of PDA data.
2) **Value based on last ten blows only.

The Test Pile at the San Joaquin River replacement structure project appears to have been driven without observed damage while being monitored by Pile Dynamic Analyzer (PDA). The compressive pile driving stresses measured by PDA did not exceed the allowable stresses within the pile. The maximum compressive stress (CSI) recorded at any sensor was 29.872 ksi, during the initial drive. This represents about 66% of the minimum yield stress of 45 ksi, which is below the 90% allowable for pile installation. The maximum average compressive stress over the pile cross section was observed during initial drive at 27.178 ksi, which is about 60% of the minimum yield stress of 45 ksi.
In order to impart maximum energy to the pile and limit the potential for pile damage, pile and hammer alignment must be properly maintained. The pile experienced no observed bending during installation with the subject hammer system.

It should be noted that ultimate pile capacity has not been shown to be reliably predicted by PDA for large diameter, open ended pipe piles. Therefore, ultimate capacities computed from PDA using the CAPWAP® program are not deemed as a dependable evaluation of the actual CISS pipe pile capacity.

Pile drivability is highly dependent upon soil characteristics, hammer alignment, pile length, pile handling, and adherence to the specifications and industry-accepted driving practices, so engineering judgment should be exercised when applying this information to other piles driven within the control location.

**Static Axial Compressive Load Testing**

Static axial compressive load testing was conducted on March 15, 2013 for the first PLT test and on March 21, 2013 for the second PLT test by personnel (R. Medina, R. Cosato, J. Pattison, A. Valdez and J. Ta) from the Foundation Testing Branch of Geotechnical Services. Test procedures used were in general conformance with ASTM D 1143/D 1143M - 07, contract Special Provisions and specifications developed by this Office. The test frame consisted of a five-pile group, a Test Pile with four anchor piles. Applied load was monitored at the Test Pile utilizing four (4) load cells placed between the hydraulic loading rams and the main beam. Deflection was monitored utilizing four (4) displacement transducers placed (on mirror plate) below the top of the test pile and fastened to a pair of fixed beams. For test frame layout plan and photos, please refer to Appendix E.

Based on requirements from contract Special Provisions (with modifications by the Geotechnical Designers), total of two (2) compressive axial load tests were conducted for two different types of installation methods (first test with no clean out and second test with partial pile clean out). Each test consists of two (2) loading cycles. Cycle #1 was set to 2000 kips (at 30% the nominal resistance in compression) and Cycle #2 was set to 6640 kips (nominal resistance in compression) and increased to maximum test frame capacity of 8000 kips. Table IV summarizes
the results of the static axial compressive load tests. Plot depicting load-displacement behavior for the Test Pile at Pier 3 can be found in Appendix C.

<table>
<thead>
<tr>
<th>Compression Test</th>
<th>PLT Test #1</th>
<th>PLT Test #2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Date</td>
<td>03/15/2013</td>
<td>03/21/2013</td>
</tr>
<tr>
<td>Measured Load at 1.0-inch deflection</td>
<td>4737.2 kips*</td>
<td>4630.7 kips*</td>
</tr>
<tr>
<td>Measured Demand Load</td>
<td>6640.4 kips*</td>
<td>6640.7 kips*</td>
</tr>
<tr>
<td>Displacement at Measured Demand Load</td>
<td>1.51 inches*</td>
<td>1.51 inches*</td>
</tr>
<tr>
<td>Measured Maximum Compressive Load</td>
<td>8011.7 kips*</td>
<td>8006.7 kips*</td>
</tr>
<tr>
<td>Max Displacement at Max Load</td>
<td>2.15 inches*</td>
<td>2.10 inches*</td>
</tr>
<tr>
<td>Required Nominal Compressive Resistance</td>
<td>6640 kips</td>
<td>6640 kips</td>
</tr>
</tbody>
</table>

*Data from Cycle #2.

Pile Field Acceptance Criteria

Field acceptance criteria were developed by this Office based on the results of wave equation analysis correlated to pile load test and pile dynamic measurements. Wave Equation Analysis was performed utilizing the pile driving analysis software GRLWEAP™ Version 2010. Plots depicting estimated pile nominal compressive resistance (pile capacity) as a function of blow count and hammer stroke are presented in Appendix D.

The pile field acceptance criteria are considered valid for only the specified pile type driven by the Pileco D225-22/32 hammer system (Field ID as D225-32), operating properly at up to maximum fuel setting. The influence of uneven or eccentric blows is not addressed by these charts and should be considered. If the performance of the hammer is altered, or if the construction practice is altered in a way that affects imparted energy, the pile field acceptance criteria will not be valid.

Results

This Office recommends the Structure Representative, Structural Designer, and Geotechnical Designers to review the submitted PLT report and to utilize the test results in accordance to the design requirements for the 74.5-inch diameter CISS piling at the control locations of Piers 2 to 6 of the San Joaquin River replacement structure, Bridge No. 41-0090.
If you have any questions or comments regarding this report, please contact James L. Ta, P.E. at (916) 227-1050.

JAMES L. TA, P.E.
Associate M & R Engineer
Foundation Testing Branch
Office of Geotechnical Support
APPENDIX A

LOCATION MAP OF SUBJECT PILING

San Joaquin River Bridge (Replace)
Bridge No. 41-0090

Test Pile at Pier 3
APPENDIX B

PILE DYNAMIC ANALYSIS

San Joaquin River Bridge (Replace)
Bridge No. 41-0090

Test Pile at Pier 3
APPENDIX C

STATIC AXIAL PILE LOAD TEST:
LOAD-DISPLACEMENT BEHAVIOR

San Joaquin River Bridge (Replace)
Bridge No. 41-0090

Test Pile at Pier 3
Load - Displacement Behavior
Pile Load Test
San Joaquin River Bridge (Replace)
(First PLT Test)
Test Pile at Pier 3

Bridge No. 41-0090
EA: 06-442624
06-Fre,Mad-99-26.7/R31.6,R0.0/R1.6
Compression Test Date: 03/15/2013

74.5-inch diameter by 1.25-inch thick CISS Piling (prior to pile clean out)
Pile Tip Elev. = 52 ft (Reported)
APPENDIX F–PILE DYNAMIC ANALYSIS, STATIC PILE LOAD TESTING AND FIELD ACCEPTANCE CRITERIA

OCTOBER 2015

Load - Displacement Behavior
Pile Load Test

San Joaquin River Bridge (Replace)
(Second PLT Test)
Test Pile at Pier 3

Bridge No. 41-0090
E.A: 06-442624
06-Fre,Mad-99-26.7/R31.6,R0.0/R1.6
Compression Test Date: 03/21/2013

74.5-inch diameter by 1.25-inch thick
CISS Piling (with partial pile clean out)
Pile Tip Elev. = 52 ft (Reported)

Measured load of 4630.7 kips at 1.01-inch displacement.

Measured load of 6640.7 kips at 1.51-inch displacement.

Maximum measured load of 8006.7 kips and maximum displacement of 2.10-inch.
APPENDIX D

PILE FIELD ACCEPTANCE CRITERIA

San Joaquin River Bridge (Replace)
Bridge No. 41-0090

Test Pile at Pier 3
Field Acceptance Chart: Capacity Relationship

Derived from Wave Equation Analysis Program (WEAP) and Load Test Results

San Joaquin River Bridge (Replace)
Test Pile at Pier 3

(For Stroke Range: 9.0 feet to 10.5 feet)

Bridge No. 41-0090
06-Fre, Mad-99-26.7/R31.6, R0.0/R1.6
Compression Test Date: 03/15/2013

74.5-inch x 1.25-inch CISS Pile
OG Elev. = 237.5 ft (estimated)
Pile Tip Elev. = 52 ft (reported)
Load-Displacement Behavior

Field Acceptance Chart: Capacity Relationship

Derived from Wave Equation Analysis Program (WEAP) and Load Test Results
San Joaquin River Bridge (Replace)
Test Pile at Pier 3

(For Stroke Range: 10.5 feet to 13.0 feet)

Bridge No. 41-0080
06-Fre.Mad-99-26.7/R31.6,R0.0/R1.6
Compression Test Date: 03/15/2013

74.5-inch x 1.25-inch CISS Pile
OG Elev. = 237.5 ft (estimated)
Pile Tip Elev. = 52 ft (reported)
APPENDIX E

CONTRACT PLANS AND PHOTOS

San Joaquin River Bridge (Replace)
Bridge No. 41-0090

Test Pile at Pier 3
APPENDIX

G

Slurry Displacement Piles

Table of Contents

Cast-In-Drilled-Hole (CIDH) Pile Acceptance Test Request Form G-2
Sample Report of Gamma-Gamma Logging Acceptance Test Results G-3
Sample Report of Gamma-Gamma Logging and Cross-Hole Sonic Acceptance Test Results G-16
Figure G-1. FTB CIDH Pile Acceptance Test Request Form
Memorandum

To: ALLEN KING
   Structure Representative
   SR 4 Crosstown Viaduct

Date: April 27, 2015

File: 10-SJ-4-T14.83
      10-0S1104 (1000000229)
      SR4 Crosstown Viaduct
      Bridge No. 29-0350

From: DEPARTMENT OF TRANSPORTATION
      DIVISION OF ENGINEERING SERVICES
      GEOTECHNICAL SERVICES - MS 5

Subject: Gamma-Gamma Logging Test Results: Pile 1 at Bent 7 of the SR4 Crosstown Viaduct

Introduction

This memorandum presents Gamma-Gamma Logging (GGL) test results for Pile 1(referred to as Pile 7-1) at Bent 7 of the SR4 Crosstown Viaduct. The subject pile is a Cast-In-Drilled-Hole (CIDH) concrete pile with a diameter of 108 inches. The pile contains ten (10) inspection pipes on the interior of the reinforcing steel cage. The centerline location of the pile with number designation was provided by Structure Construction and is shown on the Foundation Plan No. 2 sheet included in Appendix A. The reported elevations for top of concrete, bottom of inspection pipe, and pile tip were provided by OSC and are included in Appendix B. Pile information is summarized in Table I.

Table I. Summary of Pile Information: Pile 7-1 at Bent 7

<table>
<thead>
<tr>
<th>Description</th>
<th>Pile 7-1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specified Pile Cut-Off Elev. (ft)</td>
<td>0.00</td>
</tr>
<tr>
<td>Construction Joint Elev. (ft)</td>
<td>-15.00</td>
</tr>
<tr>
<td>Specified Pile Tip Elev. (ft)</td>
<td>-150.00</td>
</tr>
<tr>
<td>Reported Bottom of Permanent Steel Casing Elev. (ft)</td>
<td>-20.00</td>
</tr>
<tr>
<td>Reported Top of Concrete Elev. (ft)</td>
<td>-15.00</td>
</tr>
<tr>
<td>Reported Bottom of Inspection Pipe Elev. (ft)</td>
<td>-147.80</td>
</tr>
<tr>
<td>Reported CIDH Pile Tip Elev. (ft)</td>
<td>-151.50</td>
</tr>
<tr>
<td>Approx. CIDH Pile Length (ft)</td>
<td>136.50</td>
</tr>
<tr>
<td>Approx. GGL-Tested Length (ft)</td>
<td>132.11</td>
</tr>
<tr>
<td>Approx. Length of Untested Concrete at Bottom of Pile (ft)</td>
<td>4.39</td>
</tr>
</tbody>
</table>

(1) Based on information provided in the Contract Plans.
(2) Based on information provided in the Contractor’s Drilled Shaft Record II.
(3) Calculated from information provided.
(4) GGL data not collected over the bottom 1 foot of inspection pipe – See Background of this report.

"Provide a safe, sustainable, integrated and efficient transportation system to enhance California’s economy and viability"
GGL at Pile 7-1 was conducted by Technician Rocco Cosato of the Foundation Testing Branch (FTB) on April 23, 2015 within each accessible inspection pipe. Testing utilized a Mt. Sopris Model HLP-2375 Gamma-Gamma Probe with a 100-millicurie Cesium 137 source. The inspection pipes for the subject pile are numbered in a clockwise direction with Pipe 1 marked with orange paint in the field.

**Background**

GGL is generally viewed as among the most accurate non-destructive test methods used to detect anomalies in CIDH piles. Substantial drops in bulk density readings from GGL are indicative of the presence of anomalies in the material surrounding the inspection pipe. For the Mt. Sopris Model HLP-2375 Gamma-Gamma Probe used by this Office, the range of detection is approximately 3 inches into the concrete surrounding the inspection pipe.

Limitations of the GGL equipment preclude the accurate measurement of concrete density across the top one foot of the pile. Data collected in the top one foot of pile concrete is influenced by the detector exiting the concrete, and calibration of measured gamma count rate to density is not applicable in this region. The trend of measured densities immediately below the top one foot of pile concrete may assist the Engineer in evaluation of the top of pile concrete. Limitations of the GGL equipment also prevent evaluation of the concrete surrounding the bottom approximate one foot of an inspection pipe. The total length of untested concrete at the bottom of a given pile is equal to the length of pile below the bottom of the inspection pipe plus one foot.

**Discussion**

A GGL graph depicting the variation from mean bulk density versus depth for Pile 7-1 at Bent 7 is presented in Appendix C. The mean and standard deviation criteria set was derived from GGL readings of the tested lengths of the inspection pipes, excluding portions significantly impacted by reinforcement and anomalies, as applicable. The reported locations of inspection pipe couplers were considered in the analysis. For each inspection pipe, separate mean densities were calculated for the known differences in the steel reinforcement schedule. Testing was performed in the completely submerged condition using a submerged probe calibration. GGL results are presented in Table II.
Table II: Summary of GGL Test Results: Pile 7-1 at Bent 7

<table>
<thead>
<tr>
<th>Pile (Section)</th>
<th>Approx. Depth(1) (ft)</th>
<th>Approx. Elevation(2) (ft)</th>
<th>GGL Pipe(s)</th>
<th>Data Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (A-A)</td>
<td>51.4’ to 52.3’</td>
<td>-66.4’ to -67.3’</td>
<td>6</td>
<td>GGL detected an anomaly at one inspection pipe. Depth of anomaly is near the reported location of a PVC coupler. However, the anomaly’s magnitude of deviation from mean density is not consistent with a PVC coupler. May affect up to 10% of the pile cross-section at this depth range.</td>
</tr>
</tbody>
</table>

(1) Depth 0.0 feet is equal to the Reported Top of Concrete / Construction Joint Elevation of -15.0”.
(2) Calculated based on elevation information provided by OSC.

Recommendations

This Office recommends rejection of Pile 7-1 at Bent 7 based on the GGL test results. Please see the Pile Design Data Form (PDDF) in Appendix D and refer to Caltrans BRIDGE CONSTRUCTION MEMO 130-10.0 (June 30, 2014) for guidance in addressing rejected CIDH piles. This Office will conduct Cross-Hole Sonic Logging at Pile 7-1 to further evaluate the size of reported anomalies if requested by the OSC.

This Office also recommends that OSC inspect the top of pile concrete prior to completing pile construction above the reported construction joint elevation. Further, this Office recommends that pile designer’s review the condition of over 4 feet of untested concrete at the bottom of the pile.

This Office also recommends OSC review the Contractor’s plan for placing inspection pipes considering the bottom of inspection pipe elevation, as reported, indicates inspection pipes do not extend to the bottom of the reinforcing steel cage as required by the specifications.

“Provide a safe, sustainable, integrated and efficient transportation system to enhance California’s economy and livability.”
If you have any questions or comments regarding this report, please contact Jason Wahleithner at (916) 227-1059.

JASON D. WAHLEITHNER, P.E.
Transportation Engineer, Civil
Foundation Testing Branch
Office of Geotechnical Support

C:  J. Abercrombie – OSC (Email)
    N. Terzis – OSC (Email)
    Q. Huang – OGDN (Email)
    K. Hariranz – R&M Engineers (Email)
    C. Puzzi – OSC (Email)

BEN BARNES, P.E.
Senior Transportation Engineer (Acting)
Foundation Testing Branch
Office of Geotechnical Support

B. Alsamman – OSC (Email)
R. Erfanian – OSFP (Email)
M. Mihkovic – METS (Email)
C. Henderson-Kleinfelder (Email)
GEODOG

"Provide a safe, sustainable, integrated and efficient transportation system to enhance California's economy and livability."
APPENDIX A

Pile Location

SR4 Crosstown Viaduct
Bridge No. 29-0350

Pile 7-1 at Bent 7
APPENDIX B

Pile Information

SR4 Crosstown Viaduct
Bridge No. 29-0350

Pile 7-1 at Bent 7
## DRILLED SHAFT RECORD II
### INSPECTION TUBE LOG

**PROJECT/CONTRACT:** SR4 Crosstown Viaduct  
**Shaft Number:** 7-1  
**Date:** 26-Mar-15

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Inspection Tube Number</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4</td>
<td>X X X X X X X X X X X</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>8.63</td>
<td>-6.84</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>28.63</td>
<td>-20.64</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>48.63</td>
<td>-40.64</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>68.63</td>
<td>-60.64</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>88.63</td>
<td>-80.64</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>108.63</td>
<td>-100.64</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>128.63</td>
<td>-120.64</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>148.63</td>
<td>-140.64</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>149.13</td>
<td>-141.13</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>149.63</td>
<td>-141.63</td>
<td>PVC Coupler</td>
</tr>
</tbody>
</table>

**Comment:**

**Shaft Tip Elevation:** -151.5 ft  
**Top of Inspection Tube Elevation:** -16 ft  
**Top of Concrete Elevation:** -16 ft  
**Tip of Inspection Tube Extension:** -147.8 ft

**Supervisor Name:** Jerry Flores  
**Date:** 4/7/15  
**Inspector Name:**  
**Date:**

---

**Malcolm Drilling Co., Inc.**

**Caltrans • Foundation Manual**

**APPENDIX G—SLURRY DISPLACEMENT PILES**  
**October 2015**
APPENDIX C

Gamma-Gamma Logging Acceptance Test Results

SR4 Crosstown Viaduct
Bridge No. 29-0350

Pile 7-1 at Bent 7
GAMMA-GAMMA LOGGING ACCEPTANCE TEST RESULTS
SR4 Crosstown Viaduct
Pile 7-1 at Bent 7

EA 10-0S1104
Bridge No. 29-0350
10-SJ-4-T14.83
Date Tested: 4/23/15

108" Diameter CIDH Pile
Reported Cut-off Elev. = -15.00 ft
Reported Pile Tip Elev. = -151.5 ft
Winch/Probe/Source: 1328-3581-429

The mean and standard deviation were calculated using density readings from all pipes, excluding portions significantly impacted by reinforcement and anomalies, as applicable.
APPENDIX D

Pile Design Data Form (PDDF)

SR4 Crosstown Viaduct
Bridge No. 29-0350

Pile 7-1 at Bent 7
### Appendix D - Pile Design Data Form (GGL)

#### 1 Foundation Testing

<table>
<thead>
<tr>
<th>Name</th>
<th>Phone</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jason Wahldehn</td>
<td>(916) 227-1059</td>
<td>4/27/2015</td>
</tr>
</tbody>
</table>

**Anomaly Overview**
- Shaft Diameter: 108" CIDH
- Cutoff Elev. or Top of Concrete Elev: -15.00'
- Anomaly detected at one (1) inspection pipe (IP 6).

**Section A - A**
- Elev: 66.4' to 67.3'
- Depth: 51.4' to 52.3'
- Permanent Steel Casing

**Section B - B**
- Elev: Depth: 

**Reported Tip Elev:** -151.50'

**Anomaly Description**
- Section A - A: GGL detected an anomaly at one (1) inspection pipe (IP 6).
- Section B - B: 

**NOT TO SCALE**

---

#### 2 Geotechnical

**Name:**
**Phone:**
**Date:**

**Required Nominal Resistance of Pile (per contract plans):**
- Compression: _____ kip
- Tension: _____ kips

**Groundwater Elevation:**

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

**Soil and/or Rock Type:**

**Section is geotechnically acceptable:**
- Yes
- No

**Section B-B: Compression:_____ Tension:_____**

**Soil and/or Rock Type:**

**Section is geotechnically acceptable:**
- Yes
- No

**Comments:**

---

#### 3 Structural

**Name:**
**Phone:**
**Date:**

**As-Designed Capacity of Shaft**
- Section A-A: Shear: _____ Moment: _____
- Section B-B: Shear: _____ Moment: _____

**Maximum Demand of Shaft at Section A-A:**
- Shear: _____ Moment: _____
- Shaft is structurally acceptable:**
- Yes
- No

**Maximum Demand of Shaft at Section B-B:**
- Shear: _____ Moment: _____
- Shaft is structurally acceptable:**
- Yes
- No

**Comment:**

---

#### 4 Corrosion

**Name:**
**Consideration is:**
- Required
- Not Required

**Corrosion Potential at Section A-A:**
**Corrosion Potential at Section B-B:**

---

#### 5 Construction

**Sec. A-A is:**
- Acceptable with Administrative Deduction
- Unacceptable; Mitigation is Required

**Sec. B-B is:**
- Acceptable with Administrative Deduction
- Unacceptable; Mitigation is Required

**Bridge:** SR4 Crosstown Viaduct
**Bridge No.:** 29-0350
**Bent:** 7
**Dist-Co.-Rte:** 10-SJ-4-T14.83
**EA:** 10-051104
**Pile:** 7-1
**Structure Rep.:** Allen King
**Phone:** (209) 470-8619
**Fax:** n/a
APPENDIX G–SLURRY DISPLACEMENT PILES

Memorandum

To: ALLEN KING
   Structure Representative
   SR 4 Crosstown Viaduct

Date: April 30, 2015

From: DEPARTMENT OF TRANSPORTATION
   DIVISION OF ENGINEERING SERVICES
   GEOTECHNICAL SERVICES - MS 5

File: 10-SI-4-T14.83
      10-051104 (1000000229)
      SR4 Crosstown Viaduct
      Bridge No. 29-0350

Subject: Combined Gamma-Gamma Logging (GGL) and Cross-Hole Sonic Logging (CSL) Test Results: Pile 1 at Bent 7 of the SR4 Crosstown Viaduct

Introduction

This memorandum presents the combined GGL and CSL test results for Pile 1 (referred to as Pile 7-1) at Bent 7 of the SR4 Crosstown Viaduct. The subject pile is a Cast-In-Drilled-Hole (CIDH) concrete pile with a diameter of 108 inches. The pile contains ten (10) inspection pipes on the interior of the reinforcing steel cage. The centerline location of the pile with number designation was provided by Structure Construction and is shown on the Foundation Plan No. 2 sheet included in Appendix A. The reported elevations for top of concrete, bottom of inspection pipe, and pile tip were provided by OSC and are included in Appendix B. Pile information is summarized in Table 1.

Table 1. Summary of Pile Information: Pile 7-1 at Bent 7

<table>
<thead>
<tr>
<th>Specification</th>
<th>Pile 7-1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specified Pile Cut-Off Elev. (f) (1)</td>
<td>0.00</td>
</tr>
<tr>
<td>Construction Joint Elev. (f) (1)</td>
<td>-15.00</td>
</tr>
<tr>
<td>Specified Pile Tip Elev. (f) (1)</td>
<td>-150.00</td>
</tr>
<tr>
<td>Reported Bottom of Permanent Steel Casing Elev. (f)</td>
<td>-20.00</td>
</tr>
<tr>
<td>Reported Top of Concrete Elev. (f) (2)</td>
<td>-15.00</td>
</tr>
<tr>
<td>Reported Bottom of Inspection Pipe Elev. (f) (2)</td>
<td>-147.80</td>
</tr>
<tr>
<td>Reported CIDH Pile Tip Elev. (f) (2)</td>
<td>-151.50</td>
</tr>
<tr>
<td>Approx. CIDH Pile Length (f) (3)</td>
<td>136.50</td>
</tr>
<tr>
<td>Approx. GGL Tested Length (f) (10) / CSL Tested Length (f)</td>
<td>132.11 / 133.0</td>
</tr>
<tr>
<td>Approx. Length of Untested Concrete at Bottom of Pile (f) (6)</td>
<td>4.39</td>
</tr>
</tbody>
</table>

(1) Based on information provided in the Contract Plans.
(2) Based on information provided in the Contractor's Drilled Shaft Record II.
(3) Calculated from information provided.
(4) GGL data not collected over the bottom 1 foot of inspection pipe – See Background of this report.

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G - 16
Based on GGL test results as presented in “Gamma-Gamma Logging Test Results: Pile 1 at Bent 7 of the SR4 Crosstown Viaduct” dated April 27, 2015, CSL testing was conducted at Pile 1 to further define the estimated size of a reported anomaly. Engineers Jeremy Peterson-Self and Jason Wahleithner of the Foundation Testing Branch (FTB) performed the CSL testing on April 28, 2015. CSL Testing utilized the Cross-Hole Analyzer (Pile Dynamics, Inc.) Model: CHAMP in order to further evaluate the pile concrete. The inspection pipes for the subject pile are numbered with Pipe 1 marked with orange paint in the field.

Background

GGL and CSL are generally viewed as among the most accurate non-destructive test methods used to detect anomalies in CIDH piles. As each test examines different parts of the shaft cross-section, they can be used in combination to complement one another.

GGL is generally viewed as among the most accurate non-destructive test methods used to detect anomalies in CIDH piles. Substantial drops in bulk density readings from GGL are indicative of the presence of anomalies in the material surrounding the inspection pipe. For the Mt. Sopris Model HLP-2375 Gamma-Gamma Probe used by this Office, the range of detection is approximately 3 inches into the concrete surrounding the inspection pipe.

Limitations of the GGL equipment preclude the accurate measurement of concrete density across the top one foot of the pile. Data collected in the top one foot of pile concrete is influenced by the detector exiting the concrete, and calibration of measured gamma count rate to density is not applicable in this region. The trend of measured densities immediately below the top one foot of pile concrete may assist the Engineer in evaluation of the top of pile concrete. Limitations of the GGL equipment also prevent evaluation of the concrete surrounding the bottom approximate one foot of an inspection pipe. The total length of untested concrete at the bottom of a given pile is equal to the length of pile below the bottom of the inspection pipe plus one foot.

For CSL, high frequency compression waves propagating through the concrete material between the signal probe and receiver probe placed in pipe pairs is examined to evaluate concrete integrity between pipes. The propagation time of these sonic waves is a function of concrete density and wave travel path. A significant increase in wave arrival time, or reduction in apparent sonic wave velocity, is representative of the presence of anomalies in the material between the inspection pipes.

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CSL is utilized as an indicator of concrete integrity between the signal probe and receiver probe pair. CSL cannot confirm concrete integrity around the inspection pipe perimeter, as verified by GGL, and this uncertainty is one limitation of CSL testing when attempting to verify concrete homogeneity around the perimeter of an inspection pipe.

Discussion

The previously reported GGL graph depicting the variation from mean bulk density versus depth for Pile 7-1 at Bent 7 is presented in Appendix C. The mean and standard deviation criteria set was derived from GGL readings of the tested lengths of the inspection pipes, excluding portions significantly impacted by reinforcement and anomalies, as applicable. The reported locations of inspection pipe couplers were considered in the analysis. For each inspection pipe, separate mean densities were calculated for the known differences in the steel reinforcement schedule. Testing was performed in the completely submerged condition using a submerged probe calibration.

Plots presenting the CSL compression wave arrival times and signal energy as a function of depth for the subject piles are presented in Appendix D. The general criteria for analyzing CSL results are also included in Appendix E. A summary of the combined GGL and CSL analyses are presented in Table II and shown in a section view included in Appendix D.

### Table II: Summary of GGL Test Results: Pile 7-1 at Bent 7

<table>
<thead>
<tr>
<th>Pile (Section)</th>
<th>Approx. Depth(1) (ft)</th>
<th>Approx. Elevation(2) (ft)</th>
<th>GGL Pipe(s)</th>
<th>CSL Pipe Pair(s)</th>
<th>Data Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 (A-A)</td>
<td>51.4' to 52.3'</td>
<td>-66.4' to -67.3'</td>
<td>6</td>
<td>--</td>
<td>GGL detected an anomaly at one inspection pipe. Depth of anomaly is near the reported location of a PVC coupler. However, the anomaly’s magnitude of deviation from mean density is not consistent with a PVC coupler. No anomalies detected by CSL at the same depth range (Pipe Pair Combinations 5-6, 6-7, and 5-7) May affect up to 2% of the pile cross-section at this depth range.</td>
</tr>
</tbody>
</table>

(1) Depth 0.0 feet is equal to the Reported Top of Concrete / Construction Joint Elevation of -15.0'.
(2) Calculated based on elevation information provided by OSC.

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CSL was conducted for three profiles (Inspection Pipes 5-6, 6-7, 5-7) representing an inside boundary of the anomaly reported in the GGL report. The anomaly was not found in any of the three profiles indicating the actual location is likely contained to the pile perimeter adjacent to Inspection Pipe 6.

**Recommendations**

This Office recommends continued rejection of Pile 7-1 at Bent 7 based on the combined GGL and CSL test results. Please see the revised Pile Design Data Form (PDDF) in Appendix F and refer to Caltrans BRIDGE CONSTRUCTION MEMO 130-10.0 (June 30, 2014) for guidance in addressing rejected CIDH piles.

This Office also continues to recommend that OSC inspect the top of pile concrete prior to completing pile construction above the reported construction joint elevation. Further, this Office continues to recommend that pile designer’s review the condition of over 4 feet of untested concrete at the bottom of the pile.

This Office also recommends OSC review the Contractor’s plan for placing inspection pipes considering the bottom of inspection pipe elevation, as reported, indicates inspection pipes do not extend to the bottom of the reinforcing steel cage as required by the specifications.

If you have any questions or comments regarding this report, please contact Jason Wahleithner at (916) 227-1059.

JASON D. WAHLEITHNER, P.E.
Transportation Engineer, Civil
Foundation Testing Branch
Office of Geotechnical Support

BEN BARNES, P.E.
Senior Transportation Engineer (Acting)
Foundation Testing Branch
Office of Geotechnical Support

C: J. Abercrombie – OSC (Email)  
N. Terzis – OSC (Email)  
Q. Huang – OGDN (Email)  
K. Harisaz – R&M Engineers (Email)  
C. Pazzi – OSC (Email)  
B. Alsamman – OSC (Email)  
R. Erfanian – OSFP (Email)  
M. Mifikovic – METS (Email)  
C. Henderson – Kleinfielder (Email)  
GEODOG

"Provide a safe, sustainable, integrated and efficient transportation system to enhance California’s economy and livability"
APPENDIX A

Pile Location

SR4 Crosstown Viaduct
Bridge No. 29-0350

Pile 1 at Bent 7
APPENDIX B

Pile Information

SR4 Crosstown Viaduct
Bridge No. 29-0350

Pile 1 at Bent 7
<table>
<thead>
<tr>
<th>INSPECTION TUBE NUMBER</th>
<th>NOTES</th>
</tr>
</thead>
<tbody>
<tr>
<td>-1.4</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>0.63</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>2.63</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>4.63</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>6.63</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>8.63</td>
<td>PVC Coupler</td>
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<tr>
<td>10.63</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>12.63</td>
<td>PVC Coupler</td>
</tr>
<tr>
<td>14.63</td>
<td>PVC Coupler</td>
</tr>
</tbody>
</table>

COMMENT:

SHAFT TIP ELEVATION: -161.5 FT
TOP OF CONCRETE ELEVATION: +15 FT
TOP OF INSPECTION TUBE ELEVATION: -147.8 FT

SUPERVISOR NAME: Jerry Flores
DATE: 4/7/15
APPENDIX C

Gamma-Gamma Logging Acceptance Test Results

SR4 Crosstown Viaduct
Bridge No. 29-0350

Pile 1 at Bent 7
GAMMA-GAMMA LOGGING ACCEPTANCE TEST RESULTS
SR4 Crosstown Viaduct
Pile 7-1 at Bent 7

EA 10-051104
Bridge No. 29-0350
10-SJ-4-T14.83
Date Tested: 4/23/15

108" Diameter C/DH Pile
Reported Cut-off Elev. = -15.00 ft
Reported Pile Tip Elev. = -151.5 ft
Winch/Probe/Source: 1328-3551-429

The mean and standard deviation were calculated using density readings from all pipes, excluding portions significantly impacted by reinforcement and anomalies, as applicable.
APPENDIX D

Cross-Hole Sonic Logging Test Results

SR4 Crosstown Viaduct
Bridge No. 29-0350

Pile 1 at Bent 7
Crosshole Sonic Logging Test Results

SR4 Crosstown Viaduct – Pile 1 at Bent 7: Section (A-A)
Approximate Depth Range: 51.4 feet to 52.3 feet

Bridge Number: 29-0350
10-SJ-4-T14.83
Dated Tested: 4/30/2015

EA. 10-0S1104
9 ft. Diam. CIDH Pile
Reported Top of Conc. Elev.: -15.00 ft
Reported Tip Elev.: -151.50 ft

LEGEND

--- Anomalies Detected/Inconclusive by Crosshole Sonic Logging

--- No Anomalies Detected by Crosshole Sonic Logging (CSL)

⊙ Anomalies Detected by Gamma-Gamma Logging (GGL)

● No Anomalies Detected by GGL

● Inspection Pipe(s) Not Tested by GGL

● Inspection Pipe(s) Not Tested by GGL and CSL

NOT TO SCALE
<table>
<thead>
<tr>
<th>File</th>
<th>Profile</th>
<th>Start Foot</th>
<th>To Foot</th>
<th>Peak Foot</th>
<th>Energy Decrease</th>
<th>FAT Delay</th>
</tr>
</thead>
<tbody>
<tr>
<td>29-0350-B7-01</td>
<td>5-7</td>
<td>132.93</td>
<td>132.93</td>
<td>132.93</td>
<td>11.4dB</td>
<td></td>
</tr>
<tr>
<td>File</td>
<td>Pref</td>
<td>Distance in</td>
<td>Avg AT m/s</td>
<td>Avg WS ft/sec</td>
<td>Standard Dev.</td>
<td>Discrete Coeff.</td>
</tr>
<tr>
<td>----------</td>
<td>------</td>
<td>-------------</td>
<td>------------</td>
<td>---------------</td>
<td>----------------</td>
<td>-----------------</td>
</tr>
<tr>
<td>29-0350-B7-P1</td>
<td>5-6</td>
<td>28.8</td>
<td>0.169</td>
<td>10242</td>
<td>134</td>
<td>0.031</td>
</tr>
<tr>
<td>29-0350-B7-P1</td>
<td>6-7</td>
<td>28.8</td>
<td>0.163</td>
<td>10721</td>
<td>142</td>
<td>0.032</td>
</tr>
<tr>
<td>29-0350-B7-P1</td>
<td>5-7</td>
<td>55.3</td>
<td>0.315</td>
<td>12614</td>
<td>115</td>
<td>0.026</td>
</tr>
<tr>
<td>29-0350-B7-P1</td>
<td>5-7</td>
<td>55.3</td>
<td>0.311</td>
<td>12611</td>
<td>97</td>
<td>0.021</td>
</tr>
</tbody>
</table>
APPENDIX E

Cross-Hole Sonic Logging Rating Criteria

SR4 Crosstown Viaduct
Bridge No. 29-0350

Pile 1 at Bent 7
## CROSSHOLE SONIC LOGGING RATING CRITERIA

<table>
<thead>
<tr>
<th>RATING</th>
<th>INCREASE IN ARRIVAL TIME</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACCEPTABLE</td>
<td>0 – 10%</td>
<td>No signal distortion and an increase in signal arrival time of 10% or less are indicative of acceptable quality concrete.</td>
</tr>
<tr>
<td>MINOR ANOMALIES</td>
<td>10 – 20%</td>
<td>Minor signal distortion and lower signal amplitude with an increase in signal arrival time between 10% and 20% are indicative of minor contamination or intrusion and/or questionable quality concrete.</td>
</tr>
<tr>
<td>SIGNIFICANT ANOMALIES</td>
<td>More than 20%</td>
<td>Severe signal distortion and much lower signal amplitude with an increase in signal arrival time of 20% or more are indicative of water slurry contamination or soil intrusion and/or poor quality concrete.</td>
</tr>
<tr>
<td>NO SIGNAL</td>
<td></td>
<td>No signal was received. It is highly probable that a soil intrusion or other severe defect has absorbed the signal. It may also be due to debonding between the inspection tubes and concrete, especially in the portions of the piles above the water table.</td>
</tr>
<tr>
<td>WATER</td>
<td></td>
<td>A measured signal velocity of nominally V = 4,800 to 5,000 ft/sec is indicative of a water intrusion or a water-filled gravel intrusion with few or no fines present.</td>
</tr>
</tbody>
</table>
APPENDIX F

Revised Pile Design Data Forms (PDDF’s)

SR4 Crosstown Viaduct
Bridge No. 29-0350

Pile 1 at Bent 7
## Appendix D - Pile Design Data Form (GGL)

### 1 Foundation Testing

<table>
<thead>
<tr>
<th>Name: Jason Wahleithner</th>
<th>Phone: (916) 227-1059</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date: 4/29/2015</td>
<td></td>
</tr>
</tbody>
</table>

**Anomaly Overview**

- Shaft Diameter: 108" CIDH
- Cutoff Elev. or Top of Concrete Elev.: -15.00'
- Permanent Steel Casing
- Section A - A Elev.: -66.4 to -67.3' Depth: 51.4' to 52.3'

**NOT TO SCALE**

*Reported Tip Elev.: -151.50'

**Anomaly Description**

- Section A - A: GGL detected an anomaly at one (1) inspection pipe (P6). No anomalies detected by CSL. May affect up to 2% of the pile cross-section at this depth range.
- Section B - B:

### 2 Geotechnical

**Required Nominal Resistance of Pile (per contract plans):**

- Compression: ___ kip
- Tension: ___ kips
- Groundwater Elevation: ______________

- "As-Designed" nominal resistance over entire pile surface from the top to bottom elev. of anomaly / capacity loss within anomaly length (kips):
  - Section A - A: Compression ___ Tension ___
  - Soil and/or Rock Type: ______________

**Section B - B:**

- Compression ___ Tension ___
- Soil and/or Rock Type: ______________

**Section B - B:**

- Section is geotechnically: □ Acceptable □ Unacceptable
- Comments: ______________

### 3 Structural

**As-Designed Capacity of Shaft**

- Section A - A: Shear: ___ Moment: ___
- Section B - B: Shear: ___ Moment: ___

**Maximum Demand of Shaft at Section A - A:**

- Shear: ___ Moment: ___
  - Shaft is structurally: □ Acceptable □ Unacceptable

**Maximum Demand of Shaft at Section B - B:**

- Shear: ___ Moment: ___
  - Shaft is structurally: □ Acceptable □ Unacceptable
  - Comment: ______________

### 4 Corrosion

<table>
<thead>
<tr>
<th>Consideration is</th>
<th>□ Required</th>
<th>□ Not Required</th>
</tr>
</thead>
</table>

Corrosion Potential at Section A - A: ______________
Corrosion Potential at Section B - B: ______________

For anomalies between the top of pile and 3 feet below the groundwater level at the site, corrosion results listed in the Geotechnical report are used to assess the need for repair. For situations where results are not available, soil samples may be obtained adjacent to the anomaly and tested in accordance with California Test (CT) 643 (Parts 2, 3 and 4) and if necessary, CT 417 and CT 422 to determine soil corrosivity. If no corrosion is indicated, or for non-corrosive soil conditions, no consideration of corrosion potential is required.

### 5 Construction

Considering parts 2-4 of this form, Structure Rep.: ______________

- Sec. A - A is: □ Acceptable with Administrative Deduction □ Unacceptable: Mitigation is Required
- Sec. B - B is: □ Acceptable with Administrative Deduction □ Unacceptable: Mitigation is Required

<table>
<thead>
<tr>
<th>Bridge: SR4 Cross Island Viaduct</th>
<th>Bridge No.: 29-0050</th>
<th>Bent: 7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dist-Co.-Rte.: 10-SJ-4-T14.35</td>
<td>EA: 10-081104</td>
<td>Pile: 7-1</td>
</tr>
<tr>
<td>Structure Rep.: Allen King</td>
<td>Phone: (209) 470-8619</td>
<td>Fax: n/a</td>
</tr>
</tbody>
</table>
APPENDIX

H

Ground Anchors & Soil Nails

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Case Study – Soldier Pile Wall with Ground Anchors (Tieback).................. H-2
Case Study – Soil Nail Wall................................................................. H-4
Case Study- Soldier Pile Wall with Tieback Anchors

Contract No. 04-1S2724 04 - SM - Rte 84 - PM 10.2
Soldier Pile Wall with Tieback Anchors (7.7 km west of Rte 35).
Completed in June 2004. $850,000.00.

Hwy 84 runs through the Santa Cruz Mountains in San Mateo County. The forested hills form a ridge that separates the Pacific Ocean from San Francisco Bay and from Santa Clara Valley. Hwy 84, along with Hwy 1, 9, 17, and 35, are continuously being improved because of unstable slopes and erosion.

Project 04-1S2724 specified a soldier pile wall with tieback anchors. The wall prevents further erosion of the slope adjacent to a creek. The design specified 44 soldier piles with timber lagging. Two rows of tieback anchors are connected to concrete walers. A concrete barrier slab and concrete barrier with chain link fence is on top of the wall.

Figure H-1. Completed Soldier Pile Wall
Figure H-2. Soldier Pile Wall – Before and After.
Case Study - Soil Nail Wall

Contract No. 04-1123C4
04-SM - Rte 1- KP 61.2/62.2
Soil Nail Wall (South Rock Cut) near Devil’s Slide.

Description of Work:
The South Rock Cut Soil Nail Wall project, located in San Mateo County between the City of Pacifica and town of Montara, is part of the overall Devil’s Slide Tunnel and Bridge work.

The large “rock-cut” at the Tunnel’s south portal area is planned to align the highway and to provide adequate site distance. The face of the large rock-cut is designed to match the appearance of existing rock-cuts in the immediate view.

The South Rock Cut wall consists of a soil nail wall. The soil nail wall is composed of two walls separated by a 25m concrete barrier. Total length of walls is 281m (RW No. 2 is 190m long, RW No. 1 is 91m long). The soil nail assembly pay item equals 18, 860 meters.

Retaining Wall #2. Shotcrete sculpting at northern end of wall.

Figure H-3. Soil Nail Wall – Sculpted Facing.
Figure H-4. Soil Nail Wall Construction 1.
Figure H-5. Soil Nail Wall Construction 2.
Figure H-6. Soil Nail Wall Construction 3.
APPENDIX

I Cofferdams and Seal Courses

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Thickness of Seal Course I-2
Widths of Seal Course I-3
Example of Seal Course Thickness Calculation I-4
Seal Courses

A) General: The foundation report will give information relative to ground water conditions at the site and will indicate whether or not seal courses will be needed. If the foundation report indicates that seal courses will be needed the designer should show the seals on the plans, indicating the elevation of the bottoms of the seals and their thicknesses based on a careful consideration of foundation bearing value, anticipated hydrostatic head, and the permissible highest elevation of the top of the reinforced concrete footing.

B) Thickness: The figure shows the required thicknesses of seal courses for footings with and without piles.

Figure I-1. Seal Course Thickness Chart (BDA Section 2).
C) Width of Seal Courses: In some cases, particularly in the case of reaining walls, the width of the footing is a function of the height (h) of the wall above the top of the footing. When seal courses are shown on the plans to be placed below spread footings of retaining walls, the width (w) of the seal shall be the same as would be used if the seal were omitted and the retaining wall footing constructed with its bottom at the elevation shown for the bottom of the seal. If the seal is used, the width (w) of the footing slab (as constructed on top of the seal) shall be a function of the height (h) of the wall above the top of the footing slab. The designer should indicate clearly on the plans the procedure to be followed in the field in the event the elevation of the bottom of the seal is changed from that shown. Except in special cases where extremely deep footings or great seal thicknesses would be required, the above method of establishing footing dimensions shall be used.

Below is a sketch showing graphically the intent of this article.

![Diagram of Seal Course Width Chart](image)

Figure I-2. Seal Course Width Chart (BDA Section 2).
SEAL COURSE PROBLEM

Given: 14" square piles, Spacing 3'-6" by 4'-0" centers, Hydrostatic head of 15'-0".

Assume: Unit Wt. Concrete 145.0 pcf, Unit Wt. Water 64.0 pcf, Friction Pile/Seal = 10.0 psi, Friction Seal/Sheet Pile = 0.0 psi.

Calculate required thickness of concrete to resist uplift than add 1'-0" for seal course thickness.

Uplift Force = Wt. water X Head X Pile Spacing
            = 64.0 X 15.0 X 3.5 x 4.0
            = 13,440 #

Resisting Force = weight of concrete + friction (pile/seal)

Weight of concrete (1.0 foot thick) = Unit Wt. Conc. X Pile Spacing X 1.0
            = 145.0 X 3.5 X 4.0 X 1.0
            Concrete = 2,030.0 #

Friction on 1' section of pile = Perimeter X Height X 10.0 psi
            = 14.0 X 4 X 12.0 X 10.0
            Friction = 6,720.0 #

(Friction + Concrete) X Thickness = Uplift
( 2,030.0 + 6,720.0 ) T = 13,440.0
T = 13,440.0/8,750.0
T = 1.54 feet

Seal Course Thickness is 1.51 + 1.0 = 2.5 feet > 2.0 OK
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- Specification Example – Micropiles for Earth Retention (Duncan’s Mills Retaining Wall) J-2
- Case Study – Micropiles for Retaining Wall Foundation (Ortega Highway) J-6
- Case Study – Micropiles for Seismic Retrofit (Richmond-San Rafael Bridge) J-12
- Case Study – Micropiles Retaining Wall Foundation (Devil’s Slide) J-15
- Case Study – Micropiles for Bridge Foundation (Spanish Creek Bridge) J-23

## Specification Example - Micropiles for Earth Retention

**Contract No. 04-1S2804**  
04 - SON - Rte116, PM 3.2  
Duncan’s Mills Retaining Wall  

Note – this project utilized the 2006 Standard Specifications. Use the 2010 Standard Specifications for your current projects with micropiles, the current micropile specifications are very different from those described here.

**Description of Work:**

The micropile retaining wall was constructed along the eastbound shoulder of Highway 116 in Sonoma County and separates the roadway from the Russian River, which flows west approximately 15 feet below the road surface. The wall consists of a reinforced concrete cap beam and curtain wall supported on micropiles. The face of the curtain wall has an architectural surface (textured shotcrete). Type ST-30 bridge rail (modified) is on top of the wall. The length of the wall is approximately 300 feet long. The 100 micropiles are 12-inch diameter with steel
pipes installed to a depth of 50 feet and spaced 3 feet on center with another set of 100 piles set at an angle to form a buttress to stabilize the soil and the roadway. Inclinometers (slope indicators) were installed in six micropiles.

Construction Issues:

Pile production was slow at the western end of the wall due to the hard rock conditions. At another location along the wall, loose sand and ground water contributed to the caving of the drilled holes during drilling while waiting for the holes to be grouted.
Figure J-1. Duncan’s Mills Retaining Wall – Typical Cap Beam-Curtain Wall Cross Sections
Figure J-2. Duncan’s Mills Retaining Wall – Cap Beam Construction.  
Photo from Jim Cook, Sr Br Engr

Excerpts from Contract Special Provisions

Piling

General
Piling shall conform to the provisions in Section 49, "Piling," of the Standard Specifications, and these special provisions.

Micropiling
Micropiling consisting of steel pipe NPS 8 double extra strong and epoxy coated bar reinforcing steel that is grouted in place shall conform to the design requirements and layout shown on the plans and these special provisions.

Materials
Double extra strong steel pipe shall conform to the requirements of ASTM Designation: A53, Grade B. Galvanized pipe is not required.  
The stud connectors shall conform to the provisions in Section 55, "Steel Structures," of the Standard Specifications and these special provisions.
Grout shall be non-shrink type. Grout shall conform to the provisions in Section 50-1.09, "Bonding and Grouting," of the Standard Specifications.

**Working Drawings**

The Contractor shall submit complete project specific working drawings for the micropile system to the Office of Structure Design (OSD) in conformance with the provisions in Section 5-1.02, "Plans and Working Drawings," of the Standard Specifications.

No micropile shall be installed until the Engineer has approved, in writing, the working drawing submittal for micropiling.

**Construction**

Steel pipe NPS 8 double extra strong and epoxy coated bar reinforcing steel shall be installed using centralizers as shown on the plans. The pipe shall be placed vertically and grouted in place. Grout shall be injected at the bottom of the pile and may be placed before or after placing the steel pipe.

**Inclinometer Monitoring System**

**General**

The Contractor shall furnish and install an inclinometer monitoring system consisting of slope inclinometer casing at the location shown on the plans. The Contractor shall use a specialist to design and oversee installation of the instrumentation system.

**Measurement and Payment (Piling)**

Measurement and payment for the various types and classes of piles shall conform to the provisions in Sections 49-6.01, "Measurement," and 49-6.02, "Payment," of the Standard Specifications and these special provisions.

Micropiles will be measured and paid for by the meter.
Case Study – Micropile Retaining Wall Foundation

Contract No. 12-043214  
12-ORA-74  PM 13.3/16.6  
Route 74 Widening Project (Anchored Walls)  
Construction began in 2007.

Note – this project utilized the 2006 Standard Specifications. Use the 2010 Standard Specifications for your current projects with micropiles, the current micropile specifications are very different from those described here.

Description of Work

The structure work to be done consisted, in general, of constructing 13 anchored shotcrete retaining walls founded on micropiles. The anchored shotcrete walls were founded on steel pipe micropiles and capped with concrete barrier slabs and concrete barriers. The applied architectural treatment included sculptured shotcrete at various walls and stain application at all walls.

The project site is located on Route 74 (Ortega Highway), between the Orange/Riverside county line and San Juan Creek Bridge. Route 74 is a two-lane highway cut into the side of the Santa Ana Mountains along the San Juan Creek valley. The existing roadway consists of substandard 3.05 meter (10 feet) lanes and no shoulders.

The purpose of the project was to bring the lanes to the standard 3.66 meter (12 feet) width with 1.2 meter (4 feet) shoulders on each side and to increase the sight distance for this 5.3 kilometers of roadway. Since the existing roadway is cut into the mountains, it was necessary to cut further into the mountains, build viaducts, or add retaining walls on the downhill (north) side of the road in many locations. A total of 20 structures (13 anchored retaining walls, 3 sidehill viaducts, and 4 retaining walls) were planned throughout the project limits. The anchor walls are supported on micropiles.

Structure Representative Comments

The drilling operation and drilling conditions were difficult; however, the drilling was being completed rapidly. The solid rock is between 9,000 and 15,000 psi, the fractured rock is even more difficult to drill because it has a tendency to cave in and jam the drill stem. The time required to drill a 50 feet deep, 6-inch diameter anchor is approximately 1 hour. The time needed to drill a 21 feet deep, 12-inch diameter micropile is about 1.25 hours.

There were several factors affecting the anchored wall (rock anchor and micropile) drilling operation:

1. The experience of the drilling contractor.
2. The suitability of the equipment used.
3. The material characteristics of the earth at the site.
Drilling had been difficult. The "specialty" drilling subcontractor, required by the special provisions (documentation of 3 previous similar and successful installations), was directed to leave the job due to lack of performance. The special provisions also required the drilling to be done with minimal deleterious effects (airborne drilling dust) to the sensitive "environmental area" and endangered species (Arroyo Toad) in the creek 50 feet from the wall construction area. The constraints of the work area, the requirement to maintain the road open to traffic, requiring the drilling subcontractor to work at night (combined with the need to capture all dust), caused the drilling subcontractor to throw in the towel and cease operations. The drilling subcontractor had equipment that may or may not have been able to complete job.

The prime contractor, faced with this setback, started performing the drilling even though they had never done any drilling prior to this project. The contractor purchased an Austrian-made Triton drilling machine that was designed to drill vertical blast holes for mining operations and redesigned and modified it to drill horizontally. The machine created a hole using a pneumatic hammer and had the capability of capturing drill cuttings as well as using water to minimize dust. The rig was used for installing both the 6-inch diameter anchor holes 50 feet deep into hard and fractured rock, and the 12-inch diameter micropile holes.

(Comments and project photos from Victor S. Francis, P.E.)
Figure J-3. Ortega Highway Excavation and Backfill Details.
Figure J-4. Ortega Highway Micropile Details 1.

Notes:
- Equivalent size wire mesh reinforcement steel may be substituted in Stage 1 Shotcrete Construction upon approval of the Engineer.
- Anchor length is based on a design bonded length of 6.7 meters.
Figure J-5. Ortega Highway Micropile Details 2.

Notes:
- Equivalent size wire mesh reinforcement steel may be substituted in Stage 1 Shotcrete Construction upon approval of the Engineer.
- Anchor length is based on a design bonded length of 6.7 meters.
Micropile (NPS 8-XX Strong Steel Pipe) in a 300-mm dia drilled hole. On the ground –Sections of Rock Anchors to be installed later. Date: 2007.

Figure J-6. Ortega Highway Micropile Construction Photo 1.

Total wall length = 753 ft. The area is mostly comprised of very hard rock croppings. The road, Rte 74, is open to traffic. Date: 2007.

Figure J-7. Ortega Highway Micropile Construction Photo 2.
Case Study - Micropile Seismic Retrofit

Contract No. 04-0438U4 04-CC,Mrn-580-6.1/7.8,0.0/2.6
Seismic Retrofit of the Richmond-San Rafael Bridge (Br. No. 28-0100)
Work started August 2001; work completed February 2004.

Note – this project utilized the 1995 Standard Specifications. Use the 2010 Standard Specifications for your current projects with micropiles, the current micropile specifications are very different from those described here.

Description of Work:

The Richmond-San Rafael Bridge is one of the toll bridges in the San Francisco Bay Area. The Richmond-San Rafael Bridge includes two single-deck reinforced concrete approach trestles, two steel plate girder approach structures which convert from single-deck to double deck at each end of the bridge, two variable-depth, double-deck, cantilever-truss-type structures and 38 constant-depth 289 feet span, double-deck trusses which span between the two cantilever spans and between the cantilever spans and the approach structures. The structure has a combined length of approximately 21,335 feet (4.04 miles).

The bridge work on this project consisted of the replacement of the concrete trestle portion and the seismic retrofit on the rest of the structure. The seismic retrofit included constructing 481 micropiles in the substructure. The micropiles were installed underwater.

Per the special provisions, micropiles (substructure) were specified to consist of small diameter steel pipe reinforcement grouted in place and conforming to the design requirements and layout shown on the contract plans and the special provisions.

Figure J-8. Richmond-San Rafael Bridge (Photo from Caltrans Office of Geotechnical West Photo Gallery).
California Department of Transportation
Division of Maintenance

Structure Maintenance and Investigations

BRIDGE INSPECTION RECORDS INFORMATION

The requested documents have been generated by BIRIS.

These documents are the property of the California Department of Transportation and should be handled in accordance with Deputy Directive 55 and the State Administrative Manual.

Records for “Confidential” bridges may only be released outside the Department of Transportation upon execution of a confidentiality agreement.

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Figure J-9. BIRIS Cover Sheet.
Figure J-10. Richmond-San Rafael Bridge Micropile Details.
Case Study – Micropile Retaining Wall Foundation (Devil’s Slide)

Contract No. 04-1123U4, 04-SM-1 KP 61.2/64.9
South Portal Retaining Wall No.1 (retaining wall on micropiles) was completed in 2007.

Note – this project utilized the 2006 Standard Specifications. Use the 2010 Standard Specifications for your current projects with micropiles, the current micropile specifications are very different from those described here.

Description of Work

On Hwy 1, San Mateo County near the City of Pacifica in the San Francisco Bay Area, construction was completed in 2007 on the South Portal Retaining Wall No. 1, a retaining wall supported on micropiles. The retaining wall is on a steep cliff facing the Pacific Ocean. On one portion of the wall, the micropiles are battered in opposite directions providing lateral support. The retaining wall is also supported laterally with tieback anchors and with anchor bars connected to an anchor beam. On top of the wall is a concrete railing with chain link fence. A pedestrian sidewalk runs parallel to the concrete railing.

The South Portal Retaining Wall No. 1 is part of the overall work to re-align Route 1 at the south portal of the Devil’s Slide Tunnel. The micropile wall was placed to provide a future parking lot and a turn-around when the tunnel is complete. In addition, the wall provides valuable work space for construction (i.e., haul road and construction yard) without closing Hwy 1 during the tunnel construction.

Total length of wall: 103 meters.
Total micropiles: 144 piles
Length of pile: 7.5m (piles 1 through 36); 10.0m (piles 37 through 144);

Construction Issues / Comments

- Comments from Peter Lam, P.E., Assistant Structure Representative:
- The micropiles were ConTech Titan System piles.
- The micropile contractor was Condon-Johnson & Associates.
- Specs required non-shrink grout, but normal grout was allowed.
- CT Foundation Testing Branch (FTB) specified pull tests into zones. Testing was by FTB. The specs required non-shrink grout, which hydrates quicker and cost 2 to 3 times more than regular grout. Regular grout is the industry standard for micropile installation. Initially, the CT Geotechnical designer felt comfortable waving the load test requirement if non-shrink grout was used. However since regular grout was used, load testing was required. The test results came out great with little or no movement. The CT Geotechnical designer speculated that a grout beam was created below grade due to the piles being spaced so closely.
In some areas, soft soil caused grout bubbling through adjacent piles; the excess grout probably formed a grout curtain.

Micropile operation is very messy operation; proper SWPPP measures are needed.

Pile production/installation was approximately 1 pile per 30 to 40 minutes.

Comments from Jeremy Light, Assistant Structure Representative:

- The original wall design did not provide enough embedment in the retaining wall for wind load stability. The revised design specified a spread “L” footing that provided the proper stability.

- The addition of the footing to the structure satisfied the wind load requirements and enabled the Contractor to backfill the wall prior to anchor rod (Sta. 1+00 to 1+36) & tieback installation (Sta. 1+36 to 2+03). Tiebacks were installed from the outside of the wall with a reach-over drill rig. The plans called for temporary supports (Sta 1+36 to 2+03) to temporarily retain the wall during backfill operations and the footing satisfied this. Installing the tiebacks from behind the wall and using them for temporary supports was considered but tieback testing and working around the exposed tendons during the backfill operation proved to be an inefficient method of construction. The Designer initially wanted tiebacks installed & tested behind the wall but it was brought up that the tendons would be compromised by “bite” marks from the wedges as well as the exposure of the tendons during the construction operations (a temporary waler was called out in the specs to achieve this; impractical with the geometry of the site). Following this, the Designer proposed installing three sacrificial tendons for testing, but this proved to be a problem with again, the issue of providing a temporary waler to support the tieback loads. This was the main construction issue of this project, “How do we build it?” The addition of the footing, at a cost to the State in this case, proved to be a good solution.
Figure J-11. Devil's Slide – Revised Wall Design Sketch – “L” Footing.
Figure J-12. Devil’s Slide Micropile Construction Photos 1.
Figure J-13. Devil’s Slide Micropile Construction Photos 2.
Figure J-14. Devil’s Slide Micropile Construction Photos 3.
Figure J-15. Devil’s Slide Micropile Testing Photos.
Figure J-16. Devil's Slide Micropile Construction – SWPP Measures.
Case Study – Micropile New Bridge Foundation (Spanish Creek Bridge)

Contract No. 02-373104  
02-Plu-70 KP 56.5/57.2  
Spanish Creek Bridge (Replace) Br. No. 09-0077  
Construction began in 2010

Note – this project utilized the 2006 Standard Specifications. Use the 2010 Standard Specifications for your current projects with micropiles, the current micropile specifications are very different from those described here.

Description of Work:

The Spanish Creek Bridge (09-0077) replacement project consists of a newly constructed conventionally reinforced structure measuring 40 feet in width and 627 feet in length. The new structure replaces a 1933 steel truss classified as a fracture critical structure. A significant feature of this new structure is the solid concrete twin arch ribs spanning 368 feet rising 140 feet above the canyon.

The arch ribs are supported on footings at Piers 2 and 6 measuring roughly 33 feet wide, 24 feet in length and 17 feet high. Each footing is supported on a micropile foundation consisting of 77 piles oriented at an inclination of 55 degrees from horizontal, closely matching the inclination of the arch rib as it meets the footing. The micropiles are embedded in a weak rock foundation material.

The micropiles consist of a 7-inch outside diameter API N80 casing with 0.5-inch wall thickness, 20 feet in length, and with a gusseted bearing plate mounted on the upper end which is embedded within the pile cap. Through the center of the API casing there is a 1.75-inch diameter thread rod extending 40 feet to the bottom of a 10-inch diameter boring. The 7-inch casing was designed for compressive loads, whereas the threaded rod provides tensile resistance. Approximately 3500 linear feet of casing was required for this project.

Ten percent of the micropiles were compression tested by the Caltrans Foundation Testing Branch for contract compliance with a micropile test frame constructed as part of this contract.

The Federal Highway Administration’s Pub. No. FHWA-SA-97-070, Micropile Design and Construction Guidelines (June 2000) was used as a guide during the design phase.

Construction Issues:

Pile installation progressed at a rapid pace. Drilling of the 10-inch diameter borings and installation of the piling was performed during 24 hour/day continuous operations. Typically 15 piles were drilled, installed and grouted within a 24-hour period.
The planned pile spacing was 3.2 feet. A few issues were noted during drilling and grouting. While drilling adjacent to an open hole, cross-communication to the adjacent hole was occasionally noted in the form of drill tailings being expelled through an adjacent open boring. The specifications did not preclude drilling next to an open hole or drilling next to a freshly grouted hole.

Drilling of the micropiles was performed by an excavator mounted articulated down hole rotary percussive hammer. Some drifting of the borings were noted, occasionally causing binding of the 20-feet long casing during installation breaking the plastic centralizers. The use of steel centralizers would alleviate the centralizer issue. The requirement for an alignment check after drilling with a follower device would be useful in identifying boring alignment deviations.

The specifications require the use of 7-inch outside diameter API (American Petroleum Institute) N80 casing with a marked API monogram. Additionally, “Buy America” provisions were was required due to the contract being funded with Federal funds. The FHWA micropile manual substantiates their recommendation for using API casing due to the wide availability of secondary material thereby reducing installation costs. Secondary casing is cut-off surplus material from oil field work. Once the secondary casing material is deemed secondary, the heat numbers are typically ground off and mill certification reports are no longer available. Proving domestic origin becomes impossible once heat numbers are removed. On this project, the Contractor was required to order a mill run of the API N80 material due to the need for 3500 linear feet. This increased the contractor’s anticipated material costs. Project costs could be reduced if an alternative casing material was specified.

Proof testing of 10 percent of the production piling was performed to 75% of the axial nominal resistance in compression. This was a good verification of the capacity of the micropiling. Due to the micropile test frame utilizing neighboring piles as reaction piles, perimeter pilings were unable to be tested due to the lack of reaction piles for test frame mounting.
Figure J-17. Spanish Creek General Plan.
Figure J-19. Spanish Creek Footing Details No. 1.
Figure J-20. Spanish Creek Footing Details No. 2.
Figure J-21. Spanish Creek Micropile Details.
Figure J-22. Spanish Creek Micropile Test Details No. 1.
Figure J-23. Spanish Creek Micropile Test Details No. 2.
Figure J-24. API N80 Casing Prior to Threaded Rod Installation. Photo taken by Jeff Rothgery
Figure J-25. Micropile Assemblies Ready for Installation. Photo taken by Jeff Rothgery
Figure J-26. Foreground – Micropile Assembly Installation Underway with Post Grouting Tube. Background – Pier 6 Micropile Drilling with Articulated Down-hole Percussive Rotary Method. 
Photo taken by Jeff Rothgery
Figure J-27. Pier 6 Micropile Installation.  
Photo taken by Jeff Rothgery
Figure J-28. Pier 2 Installed Micropiles.  

Photo taken by Jeff Rothgery
Figure J-29. Pier 2 Installed Micropiles, Preparing for Load Testing. Photo taken by Jeff Rothgery
Figure J-30. Pier 6 Micropiles with Load Test Frame. Photo taken by Jeff Rothgery
# APPENDIX K–FOUNDATION CONSTRUCTION CHECKLISTS

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(Note: Most SC forms for recording field data are located on the Intranet in either Excel, Word or PDF format: [http://onramp.dot.ca.gov/hq/oscnet/sc_manuals/crp/vol_1/crp016.htm](http://onramp.dot.ca.gov/hq/oscnet/sc_manuals/crp/vol_1/crp016.htm))
K1. Driven Piling Construction Checklist

General Overview

In conjunction with Chapter 7, Driven Piles, a construction checklist for driven pile construction has been developed to assist field personnel in preparing documents and inspecting fieldwork to ensure compliance with contract requirements. It is important for Structure Representatives to review the contract plans and specifications, meet and go over them with Caltrans staff, conduct preconstruction meetings with the Contractor to lay out procedures, identify field problems, and other issues.

A driven pile foundation is another form of a deep foundation, typically using steel or concrete piles, driven into the ground using a hammer that produces a measured amount of energy.

The following checklist is intended to provide Caltrans personnel with a guide for the driven pile construction process. This checklist does not cover all situations of driven pile construction. If a problem does occur you are encouraged to contact your Senior Bridge Engineer, Division of Engineering Services (DES) Substructure Technical Committee or Structure Construction (SC) Substructure Technical Team.

I. Sources of Technical Information

A. Foundation Manual:
   i. Chapter 7, Driven Piles.
   ii. Appendix E, Driven Piles.

B. SC Website: http://onramp.dot.ca.gov/hq/oscnet
   i. Various form downloads, pile blow count spreadsheet, etc.

C. Various pile and hammer manufacturer web sites.

D. Bridge Construction Records & Procedures Manual:
   i. Chapter 130, Foundations.
   ii. BCM 3-7, Pile Records.

II. Sources of Project Specific Information

A. Structures Pending File:
   i. Designer Notes.
   ii. Bidder Inquiries.

B. Supplemental Project Information:
   i. Foundation Report
- Provides recommendations on construction methods.
  - Local, Regional, State, and Federal regulatory and permit specific requirements.

III. Contractual Requirements

A. Special Provisions:

B. Contract Plans:
   - Pile Data Table.
   - *Log of Test Borings*.
   - Pile Layout information.
   - Utilities.
   - Foundation Plans.

C. Standard Plans:
   - B2-5 Pile Details, class 90 & 140.
   - B2-8 Pile Details, class 200.
   - B2-9 thru B2-11 Load Test Pile Details.

D. Standard Specifications:
   - Precast Concrete Members\(^1\)
   - Welding\(^2\)
   - Water Pollution Control\(^3\)
   - Environmental Stewardship\(^4\)
   - *Piling* (SS Section 49)

IV. Job Books Setup

A. Category 9 – Welding:
   - Steel pile splicing submittals.
   - Welding inspection reports

B. Category 12 – Contractor’s Submittals:
   - Driving System submittal.
   - Shop drawings for precast concrete piles. (concrete mix designs)
   - Blow Count Spreadsheet.
   - Pile Handling Plan

C. Category 41 – Report of Inspection of Material:

---
\(^1\) 2010 SS, Section 90-4, *Precast Concrete*, or Special Provisions for contracts using 2006 SS.
\(^2\) 2010 SS, Section 11-3, *Welding*, or Special Provisions for contracts using 2006 SS.
\(^3\) 2010 SS, Section 13, *Water Pollution Control*, or Special Provisions for contracts using 2006 SS.
\(^4\) 2010 SS, Section 14, *Environmental Stewardship*, or Special Provisions for contracts using 2006 SS.
i. Materials release summary.
ii. Orange tags for piles and attach to TL-29.

D. Category 48 – Bid Item Quantity Documents:
   i. SC4803 – *Pile Quantity & Driving Record*.
   ii. SC4805 – *Log Pile Sheet*.
   iii. SC4806 – *Pile Layout Sheet*.

V. Prejob Discussion with Design & Geotechnical Services

A. The following issues should be discussed with the Designer and Geoprofessional:
   i. Why this type of foundation?
   ii. Review pile type, depth and drivability issues.
   iii. What kind of driving is anticipated?
   iv. What kinds of remedies might be available if problems are encountered?
   v. Requirements for scheduling testing, (PDA, Static Load Test, etc).

VI. Preconstruction Meeting with the Contractor

A. The following items should be discussed or reviewed with the Contractor at the preconstruction conference:
   i. Predrill elevations/depths.
   ii. Jetting requirements, including depths and methods/equipment.
   iii. Requirements for driving shoes, including installation requirements.
   iv. The need for a full-length pile if piles are to be driven with a follower.
   v. Specific pile material requirements for redundant vs. non-redundant piles.
   vi. Specific WQCP requirements as they pertain to steel pile sections.
   vii. Specific pile-splice requirements. (See WQCP requirements.)
   viii. Discuss the potential for using pile lugs, including installation requirements.
   ix. Cure time for precast piles.
   x. Pile handling requirements (Structural and Safety).
   xi. Specific pile testing requirements (PDA, Static Load Test, timing requirements).
   xii. Submittal requirements.
   xiii. SWPPP issues.

VII. Submittal Reviews

A. Typical submittals might include the following:
   i. Pile handling plan, checked for both the structural stability of the pile as well as for site specific crane/leads placement including locations of known utilities if applicable.
   ii. General safety around traffic.
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iii. Hammer information. (Check for minimum Energy, per the contract specifications5.)
v. Welding Quality Control Plan.
vi. Driving System submittal. (When required by the contract specifications.)
 vii. Shop Drawings and concrete mix designs for precast concrete piles.

VIII. Construction

A. Field Preparation:
   i. Pile Inspections
      • Verify METS-release tags release pile, or cast date marked directly on precast concrete piles by METS.
      • Check pile lift ring removal and patching requirements per the contract specifications6.
      • Verify proper pile length, longer for battered pile.
      • Mark pile at 1-foot intervals for blow count.
      • Check pile conditions: crack, rust, rebars on top, etc.
      • Verify pile layout, batter requirements. Do not lay out piles for Contractor.
      • Check for welder certification requirements if applicable.
      • Verify existence of utilities and any conflicts. Contact utility company and verify overhead power line voltages. Know minimum clearances and verify Contractor has located and marked safe travel areas for crane.
      • Verify use of full-length pile as required, one per footing location, min.
   ii. Equipment/Site Inspections
      • Check horizontal and vertical clearance requirements.
      • Inspect lead for rust or anomalies.
      • Verify hammer type/model – same one as submitted.
      • Evaluate construction plan, access, obstacles, traffic, safety, handling, storage, etc.
      • Mark/verify the reference elevation. Create an offset elevation hub for backup.
      • Measure hammer for stroke length.

B. Driving Piles:
      • Check pile driving crane requirements (Cal-OSHA Title 8 Subchapter 7 Group 13 Art. 100 Section 5031).
      • Wear eye/hearing protection. Do not stand next to driving equipment.

5 2010 SS, Section 49-2.01C(2), Driving Equipment, or 2006 SS, Section 49-1.05, Driving Equipment.
6 2010 SS, Section 49-2.04B(2), Fabrication, or 2006 SS, Section 49-3.01, Precast Prestressed Concrete Piles, Description.
ii. Talk to foreman, contingency (e.g., which piles to leave “high” if soft driving is encountered).

iii. Verify construction sequence.

iv. Bring prepared forms for recording.

v. Verify pile location at the start of driving.

vi. Verify plumbness or batter of the pile at the start and during driving.

vii. Bring hammer energy chart, pile bearing curve/table.


ix. Check the reference elevation daily. Verify proper pile cutoff after pile driving. Survey top of pile elevations for as-built plans.

C. Problems and Solutions:

i. Bearing value/penetration

• Hard driving-obstructions
  o Predrilling, Spudding, Jetting, reinforced tips.
  o Wave Analysis of Piles (WEAP) predicts internal stresses of the pile that can be used to determine blow count upper limit.
  o Check foundation reports and other supplemental project information for refusal criteria. Specified tip elevation may be raised if compression is the controlling (deepest) design tip elevation (see Pile Design Data Table). Check with the geoprofessional.
  o Heavier pile section.
  o Ensure that the hammer and its associated equipment are in good working order and stroke length measurements are accurate.

• Soft driving – restrike/retap
  o Retap 10% of piles or minimum two per footing. Check special provisions or supplemental project information for additional information.
  o Retap after 12 to 24 hours for “set up/take up”. (May take longer).
  o Count blows/inch (max. 3 to 6 inches).
  o Use “hot” hammer and verify stroke length.
  o Keep old pile cushion on.

ii. Cutoff

• Verify top of pile elevation for proper pile embedment.
• Check for maximum pile cutoff length.

iii. Misalignment

• Notify Designer of any pile misalignments, may create unintended eccentricity in pile footing design.
IX. Project Completion/ As-Builts

A. Final Record Keeping/Pay Quantities:
   i. Complete Forms (BCM 3-7.0, Pile Records).
   ii. File Pile Quantity and Driving Records in Category 48.
   iii. Send copies of Pile Quantity & Driving Record, Log Pile, and Pile Layout forms to SC HQ.

B. Record Data to As-Builts.
   i. See Foundation Manual, Chapter 3, Contract Administration, Section 3-5, As-Built Drawings and Pile Records.
   ii. See BCM 9-1.0, As-Built Plans.

C. Send all project completion records and As-Builts to the appropriate Office Associate at the following address.

   Division of Engineering Services
   Structure Construction
   1801 30th Street, M.S. 9-2/11H
   Sacramento, CA  95816
   Email: SC Office Associates@DOT

X. Forms

A. Refer to the SC Intranet, BCRP Manual Section 167 for various updated forms relating to driven pile construction.
   i. SC-4805, Log Pile Sheet.

7 http://onramp.dot.ca.gov/hq/oscnet/sc_manuals/crp/vol_1/crp016.htm
K2. Cast-In-Drilled-Hole (CIDH) Pile Construction Checklist

General Overview

In conjunction with Chapters 6, Cast-In-Drilled-Hole Piles, and 9, Slurry Displacement Piles, a construction checklist for Cast-In-Drilled-Hole (CIDH) pile construction has been developed to assist field personnel in preparing documents and inspecting fieldwork to ensure compliance with contract requirements. It is important for Structure Representatives to review the contract plans and specifications, meet and go over them with Caltrans staff, conduct preconstruction meetings with the Contractor to lay out procedures, identify field problems, and other issues.

Cast-In-Drilled-Hole piles are reinforced concrete piles cast in holes drilled to predetermined elevations. The CIDH piling can be grouped in two categories: the first is CIDH piling without inspection pipes (dry method), and the second is CIDH piles with inspection pipes (wet method). Inspection pipes are also referred to as inspection tubes.

Inspection pipes and testing are required for all CIDH piling that are 24 inches in diameter or larger, except when holes are dry or when the holes are dewatered without the use of temporary casing to control groundwater.

The following checklist is intended to provide Caltrans personnel with a guide for the CIDH pile construction process. This checklist does not cover all situations of CIDH pile construction. If a problem does occur you are encouraged to contact your Senior Bridge Engineer, Division of Engineering Services (DES) Substructure Technical Committee, Structure Construction (SC) Substructure Technical Team or the DES CIDH Pile Mitigation Committee.

I. Sources of Technical Information

A. Foundation Manual:
   i. Chapter 1, Foundation Investigations.
      • Provides information to interpret and effectively use the Foundation Report and Log of Test Borings during the administration of the project
   ii. Chapter 6, Cast-In-Drilled-Hole Piles.
      • Provides general information on equipment and construction
   iii. Chapter 9, Slurry Displacement Piles.
      • Provides extensive details on the use of slurry including general information on equipment, construction and contract administration when using slurry.
      • Provides sample reports of gamma-gamma logging test results and combined gamma-gamma logging and cross-hole sonic logging test results.
B. Bridge Construction Records and Procedures Manual:
i. BCM 130-7, CIDH Concrete Piling.
   • Criteria for accepting dry method of CIDH pile construction when
   encountering a small amount of water and a chronological outline for
   contract administration of CIDH Piling.
ii. BCM 130-8, CIDH Pile Mitigation Committee.
   • Contains information on the committee’s role and contact information.
iii. BCM 130-9, CIDH Pile Installation Plan and Concrete Test Batch.
   • See Sections VII and IX subsection A for details.
iv. BCM 130-10, Testing of CIDH Piling.
   • Information on coordinating and scheduling with the Foundation Testing
   Branch (FTB) for testing a CIDH Pile.
   • Information on contract administration when a CIDH Pile is rejected,
     including a sample letters.
v. BCM 130-11, Simple Repair of CIDH Piling.
vi. BCM 130-12, Mitigation of CIDH Piling.
vii. BCM 130-13, CIDH Pile Information Submittal.
viii. BCM 130-14, Slurry Test Kits For CIDH Piling.
     • Information to obtain a Slurry Test Kit, general equipment and testing
     information.
ix. BCM 130-15, Approved Synthetic Drilling Slurries.
x. BCM 130-20, CIDH Pile Preconstruction Meeting.
xi. BCM 130-21, CIDH Pile Non-Standard Mitigation Meeting.

C. Geotechnical Services:
i. Caltrans Soil and Rock Logging, Classification, and Presentation Manual
   • Assists in interpreting the information in the Log of Test Borings and
   communicating with Geotechnical Services.

D. Other:
i. The International Association of Foundation Drilling (ADSC-IAFD)\textsuperscript{8}
   • Drilled Shaft Inspection Manual and other technical references.
ii. FHWA:
   • Drilled Shafts: Construction Procedures and Design Methods.\textsuperscript{9}

II. Sources of Project Specific Information

A. Structures RE Pending File:
i. Designers Notes.
ii. Bidders Questions.

\textsuperscript{8} \url{www.adsc-iafd.com}
\textsuperscript{9} \url{www.fhwa.dot.gov/engineering/geotech/library_listing.cfm}
B. Supplemental Project Information
   i. Foundation Report.
      • Provides recommendations on construction methods.
   ii. Local, Regional, State, and Federal regulatory and permit specific requirements.

III. Contractual Requirements

A. Special Provisions:

B. Contract Plans:
   i. General Plan.
      • General layout and typical section.
   ii. Index To Plans
      • Provides reinforced concrete strength and type limits.
      • Pile Data Table showing pile type (size), cut off elevation, specified tip elevation and nominal resistance.
   iii. Foundation Plan.
      • Provides bottom of footing elevations and centerline bearing and stationing at abutments and bents.
   iv. Abutment Layout.
      • Provides dimensions for pile layout with respect to centerline bearing of abutments and edge of footing.
   v. CIDH Pile Details.
      • Provides reinforcement details and embedment length of column cage for Type II CIDH Piles.
   vi. Log of Test Borings (LOTB).
      • Review LOTB with respect to CIDH Pile locations. Note where the groundwater elevation is and types of soil. Also note the time of year groundwater elevations were observed. Use when reviewing the Contractors Pile Installation Plan to ensure method is appropriate. Note that Contractors may contact the Transportation Laboratory to request viewing of the rock cores collected during the geotechnical investigation.

C. Standard Specifications:
   i. Jobsite and Document Examination.10
      • The bidder is responsible to review the site of work, contract documents, and has access to Caltrans investigations of the site conditions including subsurface conditions in areas where work is to be performed.
      • This also includes prior construction project records within the project limits that have been used by or known to designers and administrators of the project.

10 2010 SS, Section 2-1.30, Job Site and Document Examination, or 2006 SS, Section 2-1.03, Examination of Plans, Specifications, Contract, and Site of Work.
ii. Water Pollution Control
iii. Environmental Stewardship
iv. Cast-In-Place Concrete Piles

IV. Job Books Setup

A. Category 12 – Contractor’s Submittals:
   i. CIDH Pile installation plan.

B. Category 41 – Report of Inspection of Material:
   i. Orange tags for the welded hoops and attach to TL-29.

C. Category 43 – Concrete and Reinforcing Steel:
   i. Mix design, certified test data, trial batch report and authorization letter sent to Contractor.

V. Preconstruction Discussion with Design & Geotechnical Services

A. Contact Structures Design and Geotechnical Services:
   i. Structures Design contacts:
      • Special provisions – at the beginning look for the RCE Stamp.
      • Contract plans – Engineer of Record is on the plans.
   ii. Geotechnical Services Contact:
      • Contact information is located in the Foundation Report.

B. Establish a relationship with Structures Design and Geotechnical Services:
   i. Structures Design:
      • Discuss the spacing of the reinforcement; if inspection pipes are required verify the clearances between inspection pipes and reinforcement meet the contract requirements for acceptance testing.
      • Discuss if Cal/OSHA Mining and Tunneling requirements apply:
        o If the CIDH Pile design provides for a construction joint at a depth greater than 20 feet, District and Structures Design should have coordinated to obtain a gaseous classification prior to PS&E.
        o If the Contractor plans to enter the shaft in a location greater than 20 feet in depth the shaft must have a gaseous classification and Cal/OSHA Mining and Tunneling requirements shall be adhered to.
        o If you have the above conditions and you do not have a gaseous classification, contact the Resident Engineer, the District Designer, and Structures Designer to obtain one.

11 2010 SS, Section 13, Water Pollution Control, Special Provisions for contracts using 2006 SS.
12 2010 SS, Section 14, Environmental Stewardship, or Special Provisions for contracts using 2006 SS.
13 2010 SS, Section 49-3, Cast-in-Place Concrete Piling, or 2006 SS, Section 49-4, Cast-in-Place Concrete Piling.
ii. Geotechnical Services:
- Performed the foundation investigation, wrote the Foundation Report and developed the LOTB.
- Should discuss the manner in which the pile was designed to transfer load (compressive, tensile and lateral) by end bearing or skin friction, most piles use skin friction.
- Verify if slurry application is appropriate for your job specific geology or groundwater situation.
- Discuss potential problem areas and risk in detail.
- Discuss expected construction methods and tooling.

VI. Preconstruction Meeting with the Contractor

A. Remind the Contractor of their responsibilities to submit a CIDH Pile installation plan per the requirements of the contract specifications and in a timely manner to allow sufficient time for review, comment and authorization by the Structure Representative.

B. Discuss any “alternative” procedures the Contractor might propose. These are especially common when groundwater is present and the Contractor wants to avoid the “wet hole” requirements, i.e., overdrill shaft diameter, sand slurry backfill, and redrill to planned diameter.

VII. Review of the CIDH Pile Installation Plan

A. Requirements to be included for all CIDH Pile installation plans:
   i. Concrete mix design, certified test data, and trial batch reports.
   ii. Drilling or coring methods and equipment.
      - Verify the proposed equipment is appropriate per the data provided in the Log of Test Borings.
      - Methods, equipment, and locations for stockpiling spoils prior to off-hauling.
      - Methods and equipment for containment, collection, removal, and disposal of groundwater.
   iii. Proposed method for casing installation and removal when necessary.
   iv. Methods for placing, positioning, and supporting bar reinforcement.
      - Review that the Contractor’s plans to assemble and install the pile reinforcement are appropriate.
      - Plan view drawing of pile showing reinforcement and inspection pipes, if required.
         - Verify that the number of inspection pipes and clearances between the pipes and reinforcement meet the contract requirements.
v. Methods and equipment for accurately determining the depth of concrete and actual and theoretical volume placed, including effects on volume of concrete when any casings are withdrawn.

vi. Methods and equipment for verifying that the bottom of the drilled hole is clean prior to placing concrete.

vii. Methods and equipment for preventing upward movement of reinforcement, including the Contractor's means of detecting and measuring upward movement during concrete placement operations.

B. Additional requirements when concrete is placed under slurry:

i. Concrete batching, delivery, and placing systems, including time schedules and capacities. Time schedules shall include the time required for each concrete placing operation at each pile.
   • Procedure for re-inserting the concrete placement tube into the concrete if the tube is withdrawn and the ‘seal’ is broken during the pour. Note the contract specifications require an End Cap and do not mention using the moveable plug or ‘pig’.

ii. Concrete placing rate calculations. When requested by the Engineer, calculations shall be based on the initial pump pressures or static head on the concrete and losses throughout the placing system, including anticipated head of slurry and concrete to be displaced.
   • This is especially important for large deep piles to verify whether the proposed concrete delivery system has enough pressure to displace the anticipated head of slurry and placed concrete.

iii. Suppliers’ test reports on the physical and chemical properties of the slurry and any proposed slurry chemical additives, including Material Safety Data Sheet.
   • Verify slurry application is appropriate for your job specific geology or groundwater situation.
   • Verify Contractor’s proposed slurry is Caltrans approved and is appropriate for the field conditions.
   • Review Contractors submitted Material Safety Data Sheets (MSDS) for all proposed drilling slurries and chemical additives.

iv. Slurry testing equipment and procedures.
   • Refer to BCM 130-14.0, *Slurry Displacement*, for testing equipment and procedures.

v. Methods of containment, collection, removal and disposal of slurry, and contaminated concrete, including removal rates.
   • Slurries should be disposed of in accordance with the contract specifications.14

vi. Methods and equipment for slurry agitating, recirculating, and cleaning.

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14 2010 SS, Section 5-1.20B(4), *Contractor-Property Owner Agreement*, or 2006 SS, Section 7-1.13, *Disposal of Material Outside the Highway Right of Way*.
APPENDIX K–FOUNDATION CONSTRUCTION CHECKLISTS
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VIII. CIDH Pile Preconstruction Meeting (BCM 130-20.0)

A. Hold this preconstruction meeting within 5 days of receipt of the Contractor’s Pile installation plan.

B. Refer to BCM 130-20.0, CIDH Pile Preconstruction Meeting, for meeting agenda and topics to be discussed with the Contractor.

C. Additional items to discuss:
   i. Request the plan view of inspection pipes and reinforcement includes the location of the centralizers for Type II CIDH Pile designs.
      • This can prevent a false positive anomaly reading during testing.
   ii. Procedure to prevent unintentional drop in slurry head during removal of Kelly Bar and drill tool.

IX. Prior to Construction

A. BCM 130-9.0 CIDH Pile Installation Plan and Concrete Test Batch:
   i. Notify the Contractor in writing whether the CIDH Pile installation plan is authorized or rejected.
   ii. When depositing concrete under slurry a test batch is required.
   iii. The Contractor shall submit CIDH Pile installation plan prior to producing the test batch and at least 15 working days prior to constructing piling
   iv. Caltrans inspector shall witness the Contractor’s test batch.
   v. Receive the test batch results and review for performance and consistency with CIDH Pile installation plan. If the performance does not match the pile installation plan, then reject the plan in writing.
   vi. Revise until plan and the test batch results are consistent. If mix design changes, a new test batch is required.
   vii. For CIDH Piles with inspection pipes send an authorized CIDH Pile installation plan and authorized mix design (with batch test results) to the CIDH Pile Mitigation Committee chairperson.

B. Safety:
   i. Review SC Code of Safe Practices and Construction Safety Orders that pertain to this work.
   ii. For CIDH Piles over 20 feet in depth, 30 inches in diameter and there is potential human entry, Cal/OSHA Mining and Tunneling Safety Orders apply.
   iii. Conduct a tailgate safety meeting prior to CIDH Pile construction. Be sure to discuss the contents of the MSDS and how to adhere to the safety precautions.
X. Construction

A. Prior to CIDH Pile Installation:
   i. Survey
      • The Contractor should submit survey request at least 2 days prior to drilling.
      • Inspectors review survey notes and field verify the stakes placed.
   ii. Utilities
      • The Contractor shall identify existing underground and overhead utilities
   iii. General
      • Field inspectors shall have an authorized copy of the Pile installation plan and concrete mix design.
      • Inspector sampling concrete shall be certified in California Test Methods 518, 533, 539, 540, 556, and 557.
   iv. Reinforcement
      • Check pile cage reinforcement and get inspection release tags for the hoops
      • Check mechanical couplers for no-splice zones and record type and location of couplers in the as-built plans.
      • Check if pile cage bracing will allow tremie to be installed.
      • If inspection pipes are required, verify they are installed per the Contractor’s authorized plan view drawing of the pile showing reinforcement and inspection pipe clearances.
      • If stray current provisions are required, verify they are installed per the contract documents.
   v. Equipment / Tooling
      • Document and photograph all equipment and tooling on the job site.
      • Verify the equipment and tooling per the Contractor’s CIDH Pile installation plan are on-site (e.g., core barrel, cleanout bucket).
      • Check Crane and Operator’s Certification are current. See CalOSHA Title 8, Subchapter 7, Group 13, Article 99 Sections 5021(a) and 5025, and Article 98 Section 5006.1.
      • If using slurry, verify the Contractor has the proper testing equipment and sampling device seal properly prior to construction.
      • Verify the Contractor has an ‘end cap’ for the concrete placement tube, as required by the contract specifications\(^\text{15}\), in the event the tube must be removed during the pour and re-inserted into the concrete.

B. CIDH Pile Installation:
   i. Drilling, dry hole
      • Requires 100% inspection.

\(^{15}\) 2010 SS, Section 49-3.02C(8), Placing Concrete Under Slurry, or Special Provisions for contracts using 2006 SS.
Each day keep a chronological timeline of the construction on the CIDH Pile and Concrete Placement Log form.

Log all materials as they are removed from the hole and check if it conforms to the Log of Test Borings.

Log all equipment/tooling changes (date, time, depth and type).

Verify depth of the hole prior to setting the reinforcement.

Verify that the bottom of the drilled hole is clean prior to setting the reinforcement and then again prior to concrete placement.

Note that dewatered holes are not classified as dry holes. If a hole has been dewatered without the use of temporary casing, the requirements for dry hole inspection may be utilized, but identify the hole as dewatered, not dry. For an uncased hole to be classified as dewatered, the unaided inflow (seepage) rate of water into the hole must be less than 12 inches per hour. Additionally, the hole must have less than 3 inches of standing water at the initiation of the concrete pour. For dewatered holes, note these values in the inspector logs.

ii. Drilling – Wet Hole (In addition to the above section i)

Verify approved slurry is being used and the Contractor conducts testing per the contract specifications.

Slurry manufacturer’s representative is required to be on site until released by the Engineer.

Monitor slurry level and maintain slurry level at least 10 feet above the groundwater surface / piezometric level.

The Contractor shall test slurry during drilling, prior to final cleaning, after final cleaning and prior to concrete placement. Compare versus contract requirements and record results; if using synthetic slurry, record the results on the Synthetic Slurry Test Record form.

iii. Drilling – Temporarily Cased (In addition to the above section i, if dewatered. If slurry is used, also consider section ii.)

Verify dimensions of the casing and its ability to fit within any permanent casing and fit the reinforcement cage within the temporary casing. Note any protrusions on the temporary casing.

If the casing is being used to facilitate drilling, verify with the Geoprofessional any detrimental effect the use of temporary casings may have on the geotechnical capacity of the pile. This is especially pertinent with rock sockets.

When a Contractor is excavating within temporary casing, check to see if the hole is advanced beyond the tip of the casing.

Consider the hydraulic balance of water outside the casing versus slurry and/or concrete head inside of the casing. Consider both during drilling and during concrete placement.

If the hole is dewatered using temporary casing, inspection pipes are still required.

iv. Reinforcement/Inspection Pipes
• Compare depth of drilled hole measured in the field versus measured length of CIDH reinforcement prior to setting reinforcement.
• Verify the Contractor’s method for placing, positioning and supporting the reinforcement is per the authorized Pile installation plan.
• Make sure the Contractor ties all PVC inspection pipes securely and the dobies are in place and secure per the authorized Pile Placement Plan.
• Make sure the Contractor has logged the location of PVC couplers and any other non-uniformity within the rebar cage.
• If Type II design with inspection pipes, verify the centralizers between CIDH Pile and Column reinforcement have sufficient clearance to the inspection pipes during GGL Testing.

v. Concrete Placement – Dry Hole
• Check concrete ticket for authorized mix design.
• Sample concrete for compressive strength per the contract specifications.
• Concrete shall not be permitted to fall from a height greater than 8 feet without the use of adjustable length pipes or tubes unless the flow is directed into the center of the hole using a hopper and not allowed to strike the reinforcement or reinforcement bracing.

vi. Concrete Placement – Wet Hole
• Check concrete ticket for authorized mix design.
• Sample concrete for compressive strength per the contract specifications.
• Verify that a stand-by pump is on site.
• Verify Contractor is prepared to record a log of concrete placement.
• Monitor slurry level and maintain slurry level at least 10 feet above the groundwater surface/piezometric level.
• Inspect the tremie tube for a backflow prevention device (PIG) to prevent slurry from entering the tube.
• Until at least 10 feet of concrete has been placed, the tip of the tremie shall be within 6 inches of the bottom of the drilled hole, and then the embedment of the tremie shall be maintained at least 10 feet below top of concrete.
• Concrete should be placed well past the top of pile to prevent slurry contaminated concrete at the top of the pile.

C. Testing:
   i. Pretest Verification
   • The Contractor checks accessibility of the inspection pipes by passing a rigid test probe 1.25 inches in diameter and 4.5 feet long through the entire length of all inspection pipes. Witness the entire probe check of the inspection pipes. Record results on the GGL Inspection Tube Verification form.
   • If test probe fails to pass through an inspection pipe, contact the FTB immediately.
• The inspection pipes must be completely dry or completely filled with water at the time of testing. Notify FTB in advance whether the inspection pipes are wet or dry so they can calibrate the test probe accordingly.

ii. Acceptance Test Request
• Fill out the CIDH Pile Acceptance Test Request Form16 and submit to the FTB.
• Coordinate with the Contractor that no work will be performed within 25 feet of FTB personnel during gamma-gamma logging.
• Send to the FTB the pile survey data, coupler logs and concrete placement logs.

iii. Pile Acceptance or Rejection
• Send a letter to the Contractor either accepting or rejecting a pile based on the FTB recommendation (See Attachment 1 of BCM 130-10.0, Testing of CIDH Piling, for sample letter).
• Complete payment for accepted piling. Do not pay for rejected piling and continue with the following steps.

XI. Pile Mitigation Plan

A. Suspend pile construction:
   i. Contractor submits revised pile installation plan to correct methods that resulted in anomalies.
   ii. Review revised pile installation plan.
   iii. Notify the Contractor when the revised pile installation plan is authorized and slurry work can resume.

B. Simple Repair:
   i. If the anomaly within the rejected pile is candidate for a “simple repair” as defined in BCM 130-11.0, Simple Repair of CIDH Piling; follow mitigation procedures presented in BCM 130-11.0.

C. Pile Design Data Form:
   i. Consult with the Designer, the Geoprofessional, and the corrosion specialist and complete the Pile Design Data Form (PDDF) included in the FTB test report.
   ii. Determine whether the rejected pile requires repair and if so, the feasibility of repairing the rejected pile.
   iii. Send a copy of the completed PDDF to the members in the CIDH Pile Mitigation Committee (refer to BCM 130-8.0, CIDH Pile Mitigation Committee) and allow 2 working days for a cursory check.
   iv. If the rejected pile does not require repair (consensus with the CIDH Pile Mitigation Committee is required) the Contractor can make these repairs for

16 http://www.dot.ca.gov/hq/esc/geotech/client_requests/reqts.html
full payment or forgo repairs and accept an administrative deduction per the contract specifications.

v. Unless otherwise stated in the contract specifications, the Engineer has 30 days to determine whether the pile requires mitigation and provide information to the Contractor. Day 1 of the 30 days shall be the first day after access has been provided to the Engineer to perform acceptance testing. If the Engineer acquires additional information that modifies the size, shape, or nature of the anomaly, the Contractor shall allow 20 additional days for the subsequent analysis.

vi. Should the Engineer determine non-standard mitigation is required, skip to Step (E).

D. Standard Mitigation Plan (BCM 130-12.0, Mitigation of CIDH Piling):
   • If the anomaly can be mitigated by basic repair or pressure grouting, refer to CT/ADSC Standard CIDH Pile Mitigation Plan. The most recent version of the plan is available at the FTB website: [http://www.dot.ca.gov/hq/esc/geotech/ft/adscmitplan.htm](http://www.dot.ca.gov/hq/esc/geotech/ft/adscmitplan.htm)

   ii. Send an appropriate letter and information to the Contractor (refer to Attachments 3 & 4 of BCM 130-10.0, Testing of CIDH Piling, for sample letters).

E. Non-Standard Mitigation Meeting (BCM 130-21.0, CIDH Pile Non-Standard Mitigation Meeting):
   i. In accordance with BCM 130-21.0, conduct meeting with the Contractor and discuss the following:
      • Pile replacement (if necessary).
      • Pile supplementation (if necessary).
      • Structural bridging (if appropriate).
   ii. Following the meeting, send an appropriate letter and information to the Contractor (refer to Attachments 3 & 4 of BCM 130-10.0, Testing of CIDH Piling, for sample letters).

F. Pile Mitigation Plan:
   i. Contractor submits Pile mitigation plan. The plan should include the following, subject to the requirements of the contract specifications:
      • The designation and location of the pile addressed by the mitigation plan.
      • A review of the structural, geotechnical, and corrosion design requirements of the rejected pile.
      • A step-by-step description of the mitigation work to be performed, including drawings if necessary.
      • An assessment of how the proposed mitigation work will address the structural, geotechnical, and corrosion design requirements of the rejected pile.

17 2010 SS, Section 49-3.02A(4)(d)(iv), Rejected Piles, or Special Provisions for contracts using 2006 SS.
• Methods for preservation or restoration of existing earthen materials.
• A list of affected facilities, if any, with methods and equipment for protection of these facilities during mitigation.
• The Caltrans contract number, bridge number, full name of the structure as shown on the contract plans, and the Contractor’s (and subcontractor’s if applicable) name on each sheet.
• A list of materials with quantity estimates and personnel, with qualifications, to be used to perform the mitigation work.
• The seal and signature of an engineer who is licensed as a Civil Engineer by the State of California. This requirement is waived for Plan ‘A’ (Basic Repair) of CT/ADSC Standard CIDH Pile Mitigation Plan. It is also waived for Plan ‘B’ (Grouting Repair) of the Standard Pile Mitigation Plan if the Engineer has determined that the pile does not require mitigation and the Contractor elects to repair the pile.

ii. For rejected piles to be repaired, the Contractor shall submit a Pile Mitigation Plan that contains the following additional information:
• An assessment of the nature and size of the anomalies in the rejected pile.
• Provisions for access for additional pile testing if required by the Engineer.

iii. For rejected piles to be replaced or supplemented, the Contractor shall submit a Pile mitigation plan that contains the following additional information:
• The proposed location and size of additional piling.
• Structural details and calculations for any modification to the structure to accommodate the replacement or supplemental piling.

iv. Unless otherwise stated in the contract specifications, the Engineer has 15 days to review the Pile mitigation plan after complete submittal has been received.
• Directly review the Pile mitigation plan if it is for simple repairs.
• Coordinate review with CIDH Pile Mitigation Committee for non-simple mitigation by sending a copy of the proposed Pile mitigation plan to FTB and the CIDH Pile Mitigation Committee Chairperson.
• Get consensus with CIDH Pile Mitigation Committee.
• Review and respond to the Contractor until the plan can be authorized.

XII. Pile Mitigation Work and Acceptance

A. Contractor performs pile mitigation work in conformance with the authorized Pile mitigation plan.
B. For piles that are repaired, the Contractor submits a Mitigation report within 10 days of completion of repair.
   i. The Engineer reviews the Mitigation report to verify the work performed is in conformance with the work described in the Pile mitigation plan.
• Send a copy of the Mitigation report to the CIDH Pile Mitigation Committee and reach a consensus of how to proceed.
  o Retest the mitigated pile if required in the Pile mitigation plan.
  o Accept the mitigated pile if consensus is reached with the CIDH Pile Mitigation Committee.
  o Require additional pile mitigation if that consensus is reached with the CIDH Pile Mitigation Committee.

C. For piles that are supplemented or replaced, proceed with acceptance testing of the supplemental or replacement piling.
  i. Mitigate or accept supplemental or replacement piling based on the acceptance test results.

XIII. Complete Payment

A. The Contractor shall provide a written request for an Administrative Deduction as described in the contract specifications in lieu of repairing anomalies that do not require mitigation.

B. Payment is made for piles described in the contract documents or as modified by CCO. Do not pay for supplemental or replacement piling.

XIV. Project Completion / As-Builts

A. On As-Built plans show:
  i. CIDH Pile tip elevation.
  ii. Type and location of couplers.
  iii. The percent of mineral admixture on the “Concrete Strength and Type Limits” per BCM 9-1.0, As-Built Plans (add a separate line for CIDH piles).

B. CIDH Pile Information form (BCM 130-13.0, CIDH Pile Information Submittal):
  i. Required for all CIDH piles when acceptance testing is performed.
     • When all CIDH Piles are complete for a given contract, complete one “CIDH Pile Information” form, include all of the CIDH Piles final results and submit to the CIDH Pile Mitigation Committee Chairperson.

C. Send all project completion records and As-Builts to the appropriate SC Office Associate at the following address.

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18 2010 SS, Section 49-3.02A(4)(d)(iv) or Special Provisions for contracts using 2006 SS.
XV. Forms

A. Refer to the SC Intranet, BCRP Manual Section 16 for various updated forms relating to CIDH Pile construction:

i. [http://onramp.dot.ca.gov/hq/oscnet/sc_manuals/crp/vol_1/crp016.htm](http://onramp.dot.ca.gov/hq/oscnet/sc_manuals/crp/vol_1/crp016.htm)

ii. CIDH Drilling & Concrete Placement Form OSC-CIDH01\(^\text{19}\)

iii. Synthetic Slurry Test Record Form OSC-SLR01\(^\text{20}\)

iv. GGL Inspection Tube Verification Form OSC-GGL\(^\text{21}\)

\(^{19}\) BCM 130-20.0, Attachment 1.3, [http://onramp.dot.ca.gov/hq/oscnet/sc_manuals/crp/vol_2/crp130.htm](http://onramp.dot.ca.gov/hq/oscnet/sc_manuals/crp/vol_2/crp130.htm)

\(^{20}\) BCM 130-20.0, Attachment 1.4, [http://onramp.dot.ca.gov/hq/oscnet/sc_manuals/crp/vol_2/crp130.htm](http://onramp.dot.ca.gov/hq/oscnet/sc_manuals/crp/vol_2/crp130.htm)

\(^{21}\) BCM 130-20.0, Attachment 1.6, [http://onramp.dot.ca.gov/hq/oscnet/sc_manuals/crp/vol_2/crp130.htm](http://onramp.dot.ca.gov/hq/oscnet/sc_manuals/crp/vol_2/crp130.htm)
K3. Cofferdam and Seal Course Construction Checklist

General Overview

In conjunction with Chapter 12, Cofferdams and Seal Courses, a construction checklist for Cofferdam and Seal Course construction has been developed to assist field personnel in preparing documents and inspecting fieldwork to ensure compliance with contract requirements. It is important for Structure Representatives to review the contract plans and specifications, meet and go over them with Caltrans staff, conduct preconstruction meetings with the Contractor to lay out procedures, identify field problems, and other issues.

The Contractor shall comply with the CalOSHA Construction Safety Orders and the authorized Storm Water Pollution Prevention Plan to reduce potential impacts to the project area.

Structure Representatives are encouraged to employ the following checklist for Cofferdam or Seal Course installations. Contact Structure Construction (SC) Earth Retaining Systems Technical Committee member listed on the SC website for additional assistance.

I. Sources of Technical Information

A. Foundation Manual:
   i. Chapter 12, Cofferdams and Seal Courses, and Appendix I, Cofferdams and Seal Courses.

B. Bridge Construction Records and Procedures Manual:
   i. BCM 2-9.0, Footing and Seal Course Revisions.
   ii. BCM 9-1.0, As-Built Plans.
   iii. BCM 122-1.0, Submitting Shoring Plans.
   iv. BCM 130-22.0, Seal Courses.

C. Bridge Design Aids:
   i. Sections 2.6 and 2.7, 11-46 and 11-53.

D. Bridge Design Details:
   i. Section 7-20.1, Footings w/ Seal Course.

II. Sources of Project Specific Information

A. Structures Pending File:
   i. Designers Notes.
B. Supplemental Project Information:
   i. Foundation Report(s)
   ii. Review regulatory agency permit requirements:
       - Department of Fish and Game, Regional Water Quality Board.
       - Army Corps of Engineers, Forest Service, Flood Control District.

III. Contractual Requirements

A. Special Provisions

B. Contract Plans

C. Standard Specifications
   i. Job Site and Document Examination
   ii. Submittals
   iii. Legal Relations and Responsibility to the Public (SS, Section 7)
   iv. Water Pollution Control
   v. Environmental Stewardship
   vi. Structure Excavation and Backfill (SS, Section 19-3)
   vii. Cofferdams
   viii. Water Control and Foundation Treatment
   ix. Concrete Placed Under Water

IV. Job Books Setup

A. Contractor’s Submittals:
   i. Seal Course/Cofferdam working drawings (Category 12).
   ii. Earthwork submittal, concrete mix design (Category 37).
   iii. SWPPP, dewatering plan, water treatment (Category 20).

B. Water Quality Testing Results, (Category 20).

C. Contractor’s Code of Safe Practice (Category 6).
V. Preconstruction Discussion with Design & Geotechnical Services

A. Review the Foundation Report(s), Log of Test Borings, As Built Plans, and Supplemental Project Information for information regarding soil conditions.

B. Discuss geotechnical design issues relative to cofferdam construction with the Geoprofessional and the Designer.

VI. Preconstruction Meeting with the Contractor

A. Discuss proposed construction methods for cofferdam and seal course with the Contractor.

B. Remind the Contractor of his responsibility to submit cofferdam shoring drawings, earthwork/excavation plan, notice of material sources and concrete mix designs.

C. Discuss the excavated material requirements and the necessity of water testing.

D. Discuss the locations of survey stakes and elevation control points to be provided by surveys for the Contractor.

E. Clarify that cofferdam plans and methods comply with local agency or permit procedures.

F. Review proposed cofferdam shoring plans and construction methods, review seal course concrete placement procedures.

G. Discuss the excavated material requirements and need for water testing.

VII. Submittal Reviews

A. Cofferdam Shoring Construction and work sequencing:
   i. Review shoring design according to excavation shoring/falsework requirements

B. Contractor’s Water Pollution Control and Dewatering Plan:
   i. Review water discharge and testing according to local agency requirements.

VIII. Construction

A. Inspection, Cofferdam:
   i. During the sheet pile shoring installation work, verify the details of the shoring plan are complied with and are consistent with the authorized plans.
ii. Ensure that the submitted Earthwork/Excavation plan working drawing proposes a realistic and detailed construction sequence that includes measures to ensure shoring and foundation stability, during all stages of excavation.

iii. Ensure the Contractor’s ability to maintain the correct pile placement. Template or survey control references should be easy for the Contractor and inspector to verify. Vertical control references are necessary to measure and determine pile lengths and pile cut off elevations.

iv. During foundation pile installation, ensure that stable excavation conditions are being maintained. Periodic checks for verification of bottom of footing elevation or bottom of seal course elevation are important to control soil loss or accumulation within the footing, due to dewatering activities and piping conditions or movement of the soil mass between the cofferdam sheet pile tips and the bottom of excavation. Changes to excavated depth should be investigated immediately to determine the cause. Dewatering prior to placement of the seal course should be keep to the minimum necessary for construction and to limit soil movement caused by pumping.

v. At the conclusion of pile driving, determine if the bottom of excavation elevation has been maintained. Changes to excavated depth will need to be corrected by additional excavation or fill to match plan grades.

B. Inspection, Seal Course Concrete:

i. Placement of concrete for the seal course should adhere to the details as specified in the contract specifications29.

ii. Every effort should be made to achieve a monolithic slab.

iii. Consistency of the seal course concrete requires a slump of 6 to 8 inches (ASTM C143), per the contract specifications30.

iv. Cure period for seal course concrete should be a minimum of 5 days, dependant on air temperature, per the contract specifications31.

v. The Contractor’s dewatering shall be in accordance with the process specified in the authorized Water Pollution Control Plan, Dewatering Plan and local water quality agency standards. Verify the methods to determine water quality and characteristics prior to discharge are being used and water quality is within allowable limits.

vi. The seepage of water through the cofferdam should be limited by sealing joints and gaps in shoring material. The top surface of the seal course should be prepared to achieve a level and sound surface, at the plan specified bottom of footing elevation.

29 2010 SS, Section 51-1.03D(3), Concrete Placed Under Water, or 2006 SS, Section 51-1.10, Concrete Deposited Under Water.

30 2010 SS, Section 90-1.02G(6), Quantity of Water and Penetration or Slump, or 2006 SS, Section 90-6.06, Amount of Water and Penetration.

31 2010 SS, Section 51-1.03D(3), Concrete Placed Under Water, or 2006 SS, Section 51-1.10, Concrete Deposited Under Water.
C. **Safety:**
   iii. Verify the constructed cofferdam has the necessary means for proper access and egress, as shown in the Contractor’s shoring submittal. The contract specifications require SAFE work site access.
   iv. The Construction Safety Orders requires posting of warning signs for evacuation. Ensure that all workers have the requisite training to comply with CalOSHA guidelines.
   v. Make certain that railing, tie off devices, ladders and safety equipment is functional and is being used by work area employees.
   vi. If overtopping by high water is possible, means shall be provided for controlled flooding of the work area. Sump pumps, well points etc.
   vii. Cofferdams in a navigable shipping channel shall be designed to be protected from vessels in transit.

IX. **Project Completion / As-Builts**

A. On the General Plan sheet, indicate horizontal dimensions and bottom elevation of seal courses. If the seal course is omitted at any particular location, it should be noted on the general plan sheet. Reference BCM 9-1.0, *As-Built Plans*, and BCM 130-22.0, *Seal Courses*.

B. Note any unusual conditions encountered and the corrective methods taken for specific locations for future consideration. Reference BCM 9-4.0, *Report of Completion of Structures*.

C. Send all project completion records and As-Builts to the appropriate Office Associate at the following address.

| Division of Engineering Services  
| Structure Construction  
| 1801 30th Street, M.S. 9-2/11H  
| Sacramento, CA 95816  
| Email: SC Office Associates@DOT |

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32 2010 SS, Section 5-1.01, *Control of Work, General*, or 2006 SS, Section 5-1.08, *Inspection*. 

**CALTRANS • FOUNDATION MANUAL**

K3-5
K4. Footing Foundation Construction Checklist

General Overview

In conjunction with Chapter 4, *Footing Foundations*, a construction checklist for Footing Foundation construction has been developed to assist field personnel in preparing documents and inspecting fieldwork to ensure compliance with contract requirements. It is important for Structure Representatives to review the contract plans and specifications, meet and go over them with Caltrans staff, conduct preconstruction meetings with the Contractor to lay out procedures, identify field problems, and other issues.

Footing foundations, also known as spread, combined or mat footings, transmit design loads into the underlying soil mass through direct contact with the soil immediately beneath the footing.

Each individual footing foundation must be sized so that the maximum soil bearing pressure does not exceed the allowable soil bearing capacity of the underlying soil mass. In addition to bearing capacity consideration, footing settlement must also be considered and must not exceed tolerable limits established for differential and total settlement. Each footing foundation must also be structurally capable of spreading design loads laterally over the entire footing area.

Footing foundations can be classified into two general categories: (1) footings that support a single structural member; frequently referred to as “spread footings” and (2) footings that support two or more structural members; referred to as “combined footings”. Footing foundations encountered in bridge construction almost always support a single structural member (abutment, column, pier or wall) and are invariably referred to as spread footings.

The following checklist is intended to serve as a stand-alone reference related solely for the spread footings construction process. If a problem or situation is encountered that is not addressed by this checklist, you are encouraged to contact your Senior Bridge Engineer, the SC Substructure Technical Team or Division of Engineering Services (DES) Substructure Technical Committee.

I. Sources of Technical Information

A. Bridge Construction Record and Procedures Manual:
   i. BCM 2-2.0, *Pre-Job Discussion with Design, Architecture and Geology.*
      - General information on the need of preconstruction discussion with all designers to clear up any problem areas prior to start of construction.
   ii. BCM 2-9.0, *Footing and Seal Course Revisions.*
      - Directs the Structure Representative to write a letter to the Contractor, prior to start of foundation excavation, to eliminate possible
misunderstanding about field revision of the elevation of spread footings and the revision or elimination of the seal course.

B. Foundation Manual:
   i. Chapter 1, Foundation Investigations.
      • Provides information to interpret and effectively use the Foundation Report and Log of Test Borings during the administration of the project.
   ii. Chapter 2, Type Selection.
      • Provides general overview of suitability of the different types of foundations including footing foundations.
   iii. Chapter 3, Contract Administration.
      • Guidance on the key actions required by the Engineer to ensure completion of the contact in accordance with all terms of the contract.
   iv. Chapter 4, Footing Foundations.
      • Covers various aspects of footing foundation construction including types, bearing capacity, settlement, construction inspection, safety and problems and solutions.

C. Outline of Field Construction Practice:
   i. Chapters 1, 2, 3, 6, 8 & 10 provide simple and brief description of the key field duties required during construction of footing foundation.

D. Caltrans Soil and Rock Logging, Classification, and Presentation Manual:
   i. Assists in interpreting the information in the Log of Test Borings and communication with Geotechnical Services.

E. Construction Manual:
   i. Chapter 3, General Provisions.
      • Section 3-5, Control of Work.
      • Section 3-6, Control of Material.
      • Section 3-404, Differing Site Conditions.
   ii. Chapter 4, Construction Details.
      • Section 4-51, Concrete Structures.
      • Section 4-52, Reinforcement.
      • Section 4-90, Concrete.
   iii. Chapter 5, Contract Administration.
      • Section 5-1, Project Records and Reports.
      • Section 5-3, Change Orders.
   iv. Chapter 6, Sampling and Testing.
      • Section 6-3, Field Tests.

F. Bridge Memo to Designer:
   i. Section 4, Footings:
• Clarifies terms and design methodology for spread footing and spread footing data table.

II. Source of Project Specific Information

A. Structures RE Pending File:
   i. Designers notes.
   ii. Bidder inquiries.
   iii. As Builts.
   iv. Constructability review comments from the Designer.
   v. Preliminary report.

B. Supplemental Project Information:
   i. Foundation Report
      • Review Spread Footing Data Table and compare with the one shown on the contract plans.
      • Note any comments concerning anticipated constructability problems.
      • Verify that Foundation report comments regarding specifications or construction issues are incorporated into the contract documents.
   ii. Local, Regional, State, and Federal regulatory and permit specific requirements.

III. Contractual Requirements

A. Special Provisions:

B. Contract Plans:
   i. General Plan:
      • General layout and typical section.
   ii. Index To Plans:
      • Provides the spread footing data table and reinforced concrete strength requirements.
   iii. Foundation Plan:
      • Provides the spread footing location and bottom elevation.
   iv. Abutment Details:
      • Provides footing dimensions and reinforcement details.
   v. Bent Details:
      • Provides footing dimensions and reinforcement details.
   vi. Log of Test Borings (LOTB):
      • Review LOTB with respect to spread footing location and elevation. Note groundwater elevations if applicable and date/season elevations were measured.

C. Standard Specifications:
i. Job Site and Document Examination
   - The bidder is responsible to review the site of work, contract documents, and has access to Caltrans investigations of the site conditions including subsurface conditions in areas where work is to be performed.
   - This also includes prior construction project records within the project limits that have been used by or known to designers and administrators of the project.

ii. Water Pollution Control

iii. Environmental Stewardship

iv. Earthwork:
   - Unsuitable Material
   - Water Control and Foundation Treatment
   - Structure Backfill
   - Payment
   - Compaction
   - Embankment Construction
   - Settlement Period

v. Concrete Structures:
   - Depth of Footing
   - Pumping
   - Placing Concrete

vi. Reinforcement (SS, Section 52, Reinforcement)

vii. Concrete (SS, Section 90, Concrete)

IV. Job Books Setup

A. Category 6 – Safety

B. Category 11 - Information Furnished at Start of Project:

33 2010 SS, Section 2-1.30, Jobsite and Document Examination, or 2006 SS, Section 2-1.03, Examination of Plans, Specification, Contract, and Site of Work.
34 2010 SS, Section 13, Water Pollution Control, or 2006 Special Provisions.
35 2010 SS, Section 14, Environmental Stewardship, or 2006 Special Provisions.
36 2010 SS, Sections 19-1.01B, Earthwork, Definitions & 19-1.03B, Unsuitable Material, or 2006 SS, Section 19-2.02, Unsuitable Material.
37 Standard Special Provisions (SSP), Section 49-5.01C(3), Shop Drawings and Calculations, or 2006 SS, Section 19-3.04, Water Control and Foundation Treatment.
38 2010 SS, Section 19-3.03E, Structure Backfill, or 2006 SS, Section 19-3.06, Structure Backfill.
39 2010 SS, Section 19-3.04, Payment, or 2006 SS, Section 19-3.07, Measurement.
40 2010 SS, Section 19-5, Compaction, or 2006 SS, Section 19-5.03, Relative Compaction (95%).
41 2010 SS & 2006 SS Section 19-6, Embankment Construction.
42 2010 SS, Section 19-6.03D, Settlement Periods and Surcharges, or 2006 SS, Section 19-6.025, Settlement Period.
43 2010 SS, Section 51-1.03C(1), Preparation, General, or 2006 SS, Section 51-1.03, Depth of Footings.
44 2010 SS, Section 51-1.03C(1), Preparation, General, or 2006 SS, Section 51-1.04, Pumping.
45 2010 SS, Section 51-1.03D, Placing Concrete, or 2006 SS, Section 51-1.09, Placing Concrete.
APPENDIX K–FOUNDATION CONSTRUCTION CHECKLISTS
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i. Environmental permits.

C. Category 12 - Contractor's Submittals:
   i. Worker Protection Plan per the contract specifications\(^{46}\) for excavations 5 feet or more in depth.

D. Category 14 - Photograph Records.

E. Category 20 – Water Pollution Control Plan or Stormwater Pollution Prevention Plan.

F. Category 37 - Initial Tests and Acceptance Tests.


H. Category 43 - Concrete and Reinforcing Steel.
   i. Mix design, certified test data, trial batch report and authorization letter sent to Contractor.

I. Category 48 - Bid Item Quantity Documents.

V. Preconstruction Discussion with Design & Geotechnical Services

A. Contact Structures Design and Geotechnical Services:
   i. Structure Design Contacts:
      • Contract Plans: Engineer of Record is listed on the plans.
      • Sometimes you may want to contact the specifications writer: Look for the RCE Stamp at the beginning of the special provisions for the specification writer.
   ii. Geotechnical Services Contact:
      • Contact information is located on the Foundation Report and the LOTB.

B. Establish a relationship with Structures Design and Geotechnical Services:
   i. Structure Design:
      • Clarify and resolve any question or concerns developed during review of the contract plans and specifications.
   ii. Geotechnical Services:
      • Performed the foundation investigation, wrote the Foundation Report and developed the LOTB.
      • Discuss foundation bearing material and ground water issues.
      • Discuss potential problem areas and risk in detail.
      • Discuss expected construction methods and tooling.

\(^{46}\) 2010 SS, Section 7-1.02K(6)(b), Excavation Safety, or 2006 SS, Section 5-1.02A, Excavation Safety Plans.
VI. Preconstruction Meeting with Contractor

A. Safety, per the contract specifications.47 Remind the Contractor of their responsibilities to submit an excavation safety plan for excavations 5 feet or more in depth per the requirements of the contract documents and in a timely manner to allow sufficient time for review/comment/approval by the Structure Representative.

B. Utilities:
   i. Protection of utilities, per the contract specifications.48

C. Environmental considerations.

D. Foundation Work:
   i. Footing and seal course revisions, BCM 2-9.0, Footing and Seal Course Revisions.
   ii. Constructability issues: Check special provisions and Foundation Report.
   iii. Required submittals.
   iv. Staking request, per the contract specifications.49
   v. Stockpiling, reuse within project limits, and/or off hauling of excavated materials.

VII. Submittal Review

A. Excavation Safety/Shoring Plan, per the contract specifications.50 The Engineer should refer to the Caltrans Trenching and Shoring Manual or go directly to Cal-OSHA website51 (when reviewing a Contractor’s excavation safety plan for compliance with Construction Safety Order Section 1541.1.

B. Contractor’s Water Pollution Control Program, per the contract specifications.52
   i. District responsibility to authorize with Structures review/comment.
   ii. Verify regulatory/agency permit requirements are addressed.

VIII. CONSTRUCTION

47 2010 SS, Section 7-1.02K(6)(b), Tunnel Safety, or 2006 SS, Sections 5-1.02A, Excavation Safety Plans, and 7-1.01E, Trench Safety.
48 2010 SS, Section 5-1.36D, Nonhighway Facilities, or 2006 SS, Section 8-1.10, Utility and Non-Highway Facilities.
49 2010 SS, Section 5-1.26, Construction Surveys, or 2006 SS, Section 5-1.07, Lines and Grades.
50 2010 SS, Section 7-1.02K(6)(b), Tunnel Safety, or 2006 SS, Sections 5-1.02A, Excavation Safety Plans, and 7-1.01E, Trench Safety.
51 http://www.dir.ca.gov/samples/search/query.htm
52 2010 SS, Section 13, Water Pollution Control, or 2006 SS, Section 7-1.01G, Water Pollution.
A. Inspection:
   i. Survey:
      • Contractor shall submit survey request per the contract specifications 53.
      • Inspectors review survey notes and field verify the stakes placed.
   ii. Utilities: Contractor shall identify existing underground and overhead utilities.
   iii. Witness the excavated material and compare with Contractor’s assumed soil properties per the Excavation Safety Plan and the LOTBs.
   iv. Buried man-made object/differing site conditions, per the contract specifications 54.
   v. Groundwater, per the contract specifications 55, which describes methods to be utilized when water is encountered in excavation and seal course are not shown on plan.
   vi. Stability of slopes and excavation noting changes in soil properties with respect to the depth of the excavation and time exposed.
   vii. Proximity to existing structures.
   viii. Conformity of foundation material, per the contract specifications 56, the Contractor shall notify the Engineer when the excavation is substantially complete and is ready for inspection. No concrete shall be placed until the Engineer has authorized the foundation.
   ix. Foundation bearing surface must be undisturbed soil or authorized alternative.
   x. Forms conform to layout before and after placement of bar reinforcement.
   xi. Reinforcement steel firmly and securely tied in place with adequate concrete cover. Re-check clearances and foundation bearing surfaces for potential loose soil caused by ironworker crews.
   xii. Shear steel hooked top and bottom and securely tied.
   xiii. Proper concrete cover over top of rebar mat.
   xiv. Concrete Placement:
      • Check concrete ticket for authorized mix design.
      • Check truck revolutions and time since mix was batched.
      • Check truck backup alarms are working properly on all concrete mix trucks.
      • Check concrete temperature.
        - Be aware of load restrictions for concrete mix trucks on existing structures.
      • Wet down rebar forms and subgrade.
      • Do not allow concrete to drop over 8 feet.

53 2010 SS, Section 5-1.26, Construction Surveys, or 2006 SS, Section 5-1.07, Lines and Grades.
54 2010 SS, Section 4-1.06, Differing Site Conditions, or 2006 SS, Section 5-1.116, Differing Site Conditions.
55 2010 SS, Section 19-3.03B(5), Water Control and Foundation Treatment, or 2006 SS, Section 19-3.04, Water Control and Foundation Treatment.
56 2010 SS, Section 19-3.03B(1), Structure Excavation, General, or 2006 SS, Section 19-3.05, Inspection.
• Monitor formwork for signs of excess deflection and/or failure due to freshly placed concrete loads.
• Sample concrete for compressive strength per the contract requirements.
• Reconsolidate concrete greater than 2.5 feet thick per the contract specifications\textsuperscript{57}.
• Verify concrete wash out is being done according to authorized SWPPP.
• Concrete curing per the contract specifications\textsuperscript{58}.

xv. Backfill inspection: 95% relative compaction per the contract specifications\textsuperscript{59}.

B. Safety:
i. Safe and authorized excavation/shoring plan for excavation over 5 feet in depth per the contract specifications\textsuperscript{60}.

ii. Verify Contractor’s excavation permit.

iii. Daily inspections by the Contractor’s competent person.

iv. Protective barrier around the excavation perimeter as required.

v. Spoil piles must be greater than 2 feet away from the excavation lip for excavations greater than 5 feet deep.

vi. Excavations may fall under the CalOSHA requirements for confined spaces.

vii. Protection against the hazards of impalement on the exposed ends of rebar.

C. Problems and solutions:
i. Addressing suitable material that has been disturbed or water damaged is the responsibility of the Contractor while unsuitable material is the responsibility of Caltrans.

ii. All disturbed or water damaged material must be removed or restored at the Contractor’s expense, to a condition at least equal to the undisturbed foundation as determined by the Engineer. To avoid or minimize the disturbance and/or water damage of the foundation surface:

• Under-excavate with mechanical equipment and excavate to bottom of footing by hand or by using a cleanup bucket.

• Divert surface water away from the excavation.

• Minimize exposure of the foundation material to the elements by constructing footings as soon as possible after excavation.

iii. Unsuitable material is defined in the contract specifications\textsuperscript{61} as incapable of being compacted; too wet to be properly compacted; or otherwise unsuitable.

\textsuperscript{57} 2010 SS, Section 51-1.03D(1), Placing Concrete, General, or 2006 SS, Section 51-1.09, Placing Concrete.

\textsuperscript{58} 2010 SS, Section 90-1.03B, Curing Concrete, or 2006 SS, Section 90-7.03, Curing Structures.

\textsuperscript{59} 2010 SS, Section 19-3.03E(1), Structure Backfill, General, or 2006 SS, Section 19-3.06, Structure Backfill.

\textsuperscript{60} 2010 SS, Section 7-1.02K(6)(b), Excavation Safety, or 2006 SS, Section 5-1.02A, Excavation Safety Plans.

\textsuperscript{61} 2010 SS, Section 19-1.01B, Definitions, or 2006 SS, Section 19-2.02, Unsuitable Material.
for the planned use. The Engineer is responsible for determining the suitability of the foundation as it relates to the design intent.

- Contact Geotechnical Services.
- Review of LOTB.
- Anticipated suitable material may be just below the excavated surface.
- Field tests to verify unsuitable material.

iv. Footing modifications: Corrective action is required whenever changes in the bottom of footing elevation are made to address disturbed, water damaged or unsuitable material. They fall into two categories: replacement of the original foundation material to achieve the original bottom of footing elevation; or revisions to the structure to address a different bottom of footing elevation.

- Excavate to a stratum with sufficient bearing capacity and replace removed material with suitable foundation materials (i.e. concrete, lean concrete, aggregate base, structure backfill etc...) based on the recommendations of the Geoprofessional and Designer.
- Lower the footing to a stratum with sufficient bearing capacity and increase the height of column or wall. This option may not be acceptable if the increase in height necessitates redesign of the column or wall.
- Increase the footing size. Settlement can’t exceed tolerable limits. Check with the Designer.
- Footing revisions due to unsuitable material will require a change order per the contract specifications62. Impact to the construction schedule must be considered.

IX. Project Completion / As-Builts

A. As Built drawings (BCM 9-1.0, As-Built Plans):
   i. Changes to footing elevations.
   ii. Location of relocation of new/existing/abandoned utilities.
   iii. Footing formed, or excavated neat (against undisturbed ground). Only note on As-Builts if placed neat.
   iv. Gravel placement to control high ground water.
   v. Concrete overpours.

B. Send all project completion records and As-Builts to the appropriate Office Associate at the following address.

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62 2010 SS, Section 19-3.03B(5), Water Control and Foundation Treatment, or 2006 SS, Section 19-3.07, Measurement.
Division of Engineering Services
Structure Construction
1801 30th Street, M.S. 9-2/11H
Sacramento, CA  95816
Email: SC Office Associates@DOT
K5. Micropile Construction Checklist

**General Overview**

In conjunction with Chapter 13, *Micropiles*, a construction checklist for Micropile construction has been developed to assist field personnel in preparing documents and inspecting fieldwork to ensure compliance with contract requirements. It is important for Structure Representatives to review the contract plans and specifications, meet and go over them with Caltrans staff, conduct preconstruction meetings with the Contractor to lay out procedures, identify field problems, and other issues.

Micropiles are a small diameter (typically less than 1 foot), drilled and grouted replacement pile that is typically reinforced. A micropile is constructed by drilling a borehole, placing reinforcement, and grouting the hole. Contractors who specialize in ground anchors and soil nails also construct micropiles since similar equipment and techniques are used.

Micropiles can withstand axial (compression and tension) loads and some lateral loads. Caltrans limits the use of micropiles due to the lateral demand requirements. Caltrans is currently using micropiles for seismic retrofits, earth retention, and foundations for new structures (retaining/sound walls).

The following checklist is intended to provide Caltrans personnel with a guide for the micropile construction process. This checklist will not cover all situations of micropile construction. Be sure to check your special provisions for additional requirements. If a problem does occur you are encouraged to contact your Senior Bridge Engineer, Division of Engineering Services (DES) Substructure Technical Committee or the Foundation Testing Branch.

**I. Source of Technical Information**

A. Foundation Manual:
   i. Chapter 13, *Micropiles*.
      • Provides micropile definition and descriptions, Caltrans application, construction and contract administration.

   i. A comprehensive review of current design and construction methods.
   ii. The presenting the guideline procedure is to help ensure that agencies adopting use of micropile technology follow a safe, rational procedure from site investigation through construction.

[6](http://www.fhwa.dot.gov/engineering/geotech/library_listing.cfm)
C. Geotechnical Services:
   i.  *Caltrans Soil and Rock Logging, Classification, and Presentation Manual* [2]  
       • Assist in interpreting the information in the *Log of Test Borings* and communicating with Geotechnical Services.

II. Source of Project Specific Information

A. Structures Pending File:
   i.  Designer’s Notes

B. Supplemental Project Information:
   i.  *Foundation Report*
      • Provides recommendations on construction and grouting methods.
   ii.  Local, Regional, State, and Federal regulatory and permit specific requirements.

III. Contractual Requirements

A. Special Provisions:
   i.  Micropiling:
      • Caltrans provides design parameters for the micropiles in the contract plans, Contractor submits shop drawings, and the Office of Structures Design has 30 days to review the shop drawings [65]. Verification and proof testing requirements are detailed in the contract specifications [66].
   ii.  Alternative Piling:
      • There are currently no alternative piling systems authorized for use on Caltrans projects.

B. Contract Plans
   i.  General Plan
      • General layout and typical section.
   ii.  Index To Plans, Foundation Plan and Micropile Details
      • What you are constructing (seismic retrofit, retaining wall and sound wall) determines where you find the information regarding grout strength, pile data table, layout, reinforcement and required loads.
   iii. *Log of Test Borings (LOTB)*
      • When reviewing the Contractor’s shop drawings for the micropile system, verify that the proposed equipment will work in the soil conditions provided in the *LOTB*.

C. Standard Specifications:

65  Standard Special Provisions (SSP), Section 49-5.01C(3), *Shop Drawings and Calculations*, or Standard Provisions for contracts using 2006 SS.
i. Job Site and Document Examination
   - The bidder is responsible to review the site of work and contract documents, has access to Caltrans investigations of the site conditions including subsurface conditions in areas where work is to be performed.
   - This also includes prior construction project records within the project limits that have been used by or known to designers and administrators of the project.

ii. Contractor – Property Owner Agreement
   - Material resulting from grouting micropiles shall be disposed of properly.

iii. Water Pollution Control

iv. Environmental Stewardship

v. Piling (SS, Section 49)

vi. Bonding and Grouting
   - Reference specifications for cement, water and admixtures.
   - Grout shall meet the requirements of California Test 541.

vii. Reinforcement (SS, Section 52).

IV. Job Books Setup

A. Category 12 – Contractor’s Submittals:
   i. Micropile shop drawings.

B. Category 41 – Report of Inspection of Material:
   i. Welding, reinforcement and couplers.
   ii. Certificate of compliance for grout.

V. Preconstruction Discussion with Structures & Geotechnical Design

A. Structures and Geotechnical Design:
   i. Discuss potential problem areas and risk in detail.
   ii. Discuss expected construction methods and tooling.
      - Method of grouting (gravity grout, pressure grout, multiple stage grouting).
      - Method of construction (open hole, temporary cased, partial permanently cased, permanently cased).
      - This is very important because micropiles can be constructed in all types of soil and by various methods. You want to ensure the Contractor is providing the product the Designer intended.

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67 2010 SS, Section 2-1.07, Job Site and Document Examination, or 2006 SS, Section 2-1.03, Examination of Plans, Specifications, Contract, and Site of Work.
68 2010 SS, Section 5-1.20B(4), Contractor-Property Owner Agreement, or 2006 SS, Section 7-1.13, Disposal of Material Outside the Highway Right of Way.
69 2010 SS, Section 13, Water Pollution Control, or Standard Provisions for contracts using 2006 SS.
70 2010 SS, Section 14, Environmental Stewardship, or Standard Provisions for contracts using 2006 SS.
71 2010 SSP, Section 49-5.02D, Grout, or 2006 SS, Section 50-1.09, Bonding and Grouting.
B. Foundation Testing Branch (FTB):
   i. The Foundation Testing Branch is a good source of information on construction methods and micropile testing since they have been involved in testing and accepting micropiles in the past.

VI. Preconstruction Meeting with Contractor

A. Remind the Contractor of their responsibilities to submit shop drawings for the Micropile System to the Office of Structures Design per the requirements of the contract specifications and in a timely manner to allow sufficient time for review, comment and authorization.

B. Remind the Contractor of their responsibilities to submit experience qualifications and the micropile installation plan per the requirements of the contract specifications and in a timely manner to allow sufficient time for review, comment and authorization.

VII. Micropile Shop Drawings

A. Depending on the project location, design, and Contractor, different drilling and grouting techniques may be used. Review the contract specifications for requirements to be included in the shop drawings. This also includes submittal review time, number of shop drawings and where to submit.

VIII. Preconstruction Micropile Submittals

A. Experience Qualifications
   i. Review and authorize the experience qualification information submitted by the Contractor.
   ii. Keep in mind this submittal is post-award, so you can’t simply reject the micropile contractor if they can’t demonstrate compliance with the contract specifications. However, you can require the Contractor bring in a work crew that does meet these requirements.

B. Installation Plan
   i. Review and authorize the micropile installation plan submitted by the Contractor. Coordinate this review with the Experience Qualifications submittal.
   ii. Identify who will be performing load testing, the Contractor or a pile testing company. There are experience qualifications for load testing in the contract specifications.

72 2010 SSP, Section 49-5.01C(3), Shop Drawings and Calculations, or Special Provisions for contracts using 2006 SS.
73 2010 SSP, Section 49-5.01C(2), Experience Qualifications, or Special Provisions for contracts using 2006 SS.
74 2010 SSP, Section 49-5.01D(2), Experience Qualifications, or Special Provisions for contracts using 2006 SS.
IX. Micropile Preconstruction Meeting

A. Hold this preconstruction meeting after 5 days of receipt of the Contractor’s micropile shop drawings and installation plan.

B. Refer to the contract specifications\(^7\) for topics to be discussed with the Contractor.

X. Construction Inspection, Submittals & Quality Assurance

A. Field Inspection:
   i. Review the authorized shop drawings and be familiar with the construction sequence for your project-specific micropiling system.
   ii. Drilling mud or chemical stabilizers are not allowed.
   iii. Mill secondary steel reinforcing elements are no longer allowed.
   iv. Steel casing or bar reinforcement shall be installed with centralizers.
   v. Grout-ground bond is how micropiles transfer load which makes the inspection of the drilling and grouting procedure very important.

B. Submittals:
   i. For each micropile, the Contractor is required to submit within 1 business day of completion for each micropile:
      • Micropile Installation Log\(^7\)
      • Grout Test Results\(^7\)
      • Load Test Data\(^7\)
   ii. Submittals must be reviewed and authorized before each micropile can be accepted.

C. Testing:
   i. Project-specific requirements depend on Geotechnical Control Zones.
   ii. The Contractor is required to perform and provide the equipment for the pile load tests. Verify requirements in the contract specifications\(^7\).
   iii. Verification Testing
      • Notify FTB in advance of verification testing and determine whether they will do additional testing on the verification load test micropile(s) to verify the test loads.

\(^{75}\) 2010 SSP, Section 49-5.01D(3), *Preconstruction Meeting*, or Special Provisions for contracts using 2006 SS.

\(^{76}\) 2010 SSP, Sections 49-5.01C(6), *Installation Log*, and 49-5.03H, *Installation Log*, or Special Provisions for contracts using 2006 SS.

\(^{77}\) 2010 SSP, Sections 49-5.01C(7), *Grout Test Results*, and 49-5.01D(4), *Grout Testing*, Special Provisions for contracts using 2006 SS.

\(^{78}\) 2010 SSP, Section 49-5.01C(8), *Load Test Data*, or Special Provisions for contracts using 2006 SS.

\(^{79}\) 2010 SSP, Section 49-5.01D(5), *Load Testing*, or Special Provisions for contracts using 2006 SS.
Verification testing is performed for tension and compression using the load schedules and acceptance criteria in the contract specifications.\textsuperscript{80} Verifies site-specific capacity for a given micropile type.

If a verification test micropile fails to meet acceptance criteria, contractors usually want to try post-grouting of the verification test micropile and retest before redesigning the micropiles. This is acceptable if the Contractor is willing to post-grout every production micropile in the wall zone represented by the verification test micropile.

iv. Proof Testing

Proof testing is performed for tension and compression using the load schedules and acceptance criteria in the contract specifications.\textsuperscript{81} Serves as Quality Assurance for the project.

The contract specifications\textsuperscript{82} specify how many micropiles are proof-tested.

Select randomly micropiles to be proof-tested, after installation, use engineering judgment.

If a micropile fails to meet proof testing acceptance criteria, the Contractor has two options for addressing the test failure. Both options involve suspending current micropile construction. Contractors tend to favor the post-grouting option.

XI. Projection Completion / As-Builts

A. As-Builts:

i. Include authorized shop drawings.

ii. Record all changes to the contract.

iii. Note which micropiles were post-grouted.

B. Send all project completion records and As-Builts to the appropriate Office Associate at the following address.

\begin{flushright}
\textsuperscript{80} 2010 SSP, Section 49-5.01D(5)(b), Verification Load Testing, or Special Provisions for contracts using 2006 SS.

\textsuperscript{81} 2010 SSP, Section 49-5.01D(5)(c), Proof Load Testing, or Special Provisions for contracts using 2006 SS.

\textsuperscript{82} 2010 SSP, Section 49-5.01D(5)(c)(i), General, or Special Provisions for contracts using 2006 SS.
\end{flushright}
K6. Ground Anchor Wall Construction Checklist

General Overview

In conjunction with Chapter 11, *Ground Anchors & Soil Nails*, a construction checklist for ground anchor wall construction has been developed to assist field personnel in preparing documents and inspecting fieldwork to ensure compliance with contract requirements. It is important for Structure Representatives to review the contract plans and specifications, meet and go over them with Caltrans staff, conduct preconstruction meetings with the Contractor to lay out procedures, identify field problems, and other issues.

The Contractor must comply with the CalOSHA Construction Safety Orders and Storm Water Pollution Prevention Plan to reduce potential impacts to the site.

Structure Representatives are encouraged to employ the following checklist for ground anchor wall construction. Contact the Earth Retaining Systems Specialist in the Office of Structure Design in Sacramento for additional assistance.

I. Sources of Technical Information

A. Foundation Manual:
   i. Chapter 11, *Ground Anchors & Soil Nails*.

B. Bridge Construction Records and Procedures Manual:
   i. Chapter 160, *Prestressed Concrete*.

C. Recommendations for Prestressed Rock and Soil Anchors (Post-Tensioning Institute).

II. Sources of Project Specific Information

A. Structures Pending File.
   i. Designers Notes.

B. Supplemental Project Information.
   i. *Foundation Report(s)*
   ii. Local, Regional, State, and Federal regulatory and permit specific requirements.

III. Specification Requirements

A. Special Provisions.
B. Contract Plans.
   i. General Plan and & Elevation
      • Check stationing, grades and bearings with District Plans.
   ii. Typical Section
      • Note ground anchor layout, spacing and inclination angles.
      • General notes.
      • Review ultimate bond stress for Test Load determination.
   iii. Structure Plans/Elevations
      • Verify wall grades, stationing and dimensions.
   iv. Foundation Plan
      • Review with respect to construction layouts, utility plans and drainage plans.
   v. Ground Anchor Details
      • Review ground anchor details for production and test assemblies, note total overall and bonded lengths.
      • Review drainage details for geotextile material placement.
   vi. Log of Test Borings
      • Review LOTB’s coring location with respect to wall layout and stationing.

C. Standard Specifications:
   i. Job Site and Document Examination
   ii. Submittals
   iii. Welding
   iv. Water Pollution Control
   v. Environmental Stewardship
   vi. Structure Excavation and Backfill
   vii. Slurry Cement Backfill
   viii. Ground Anchors
   ix. Prestressing
   x. Bonding and Grouting

83 2010 SS, Section 2-1.07, Job Site and Document Examination, or 2006 SS, Section 2-1.03, Examination of Plans, Specifications, Contract, and Site of Work.
84 2010 SS, Section 5-1.23, Submittals, or 2006 SS, Section 5-1.02, Plans and Working Drawings.
85 2010 SS, Section 11-3, Welding, or Special Provisions for contracts using 2006 SS.
86 2010 SS, Section 13, Water Pollution Control, or Special Provisions for contracts using 2006 SS.
87 2010 SS, Section 14, Environmental Stewardship, or Special Provisions for contracts using 2006 SS.
89 2010 SS, Section 19-3.03F, Slurry Cement Backfill, or 2006 SS, Section 19-3.062, Slurry Cement Backfill.
90 2010 SS, Sections 46-1, Ground Anchors and Soil Nails, General, & 46-2, Ground Anchors, or Special Provisions for contracts using 2006 SS.
91 2010 SS, Section 50-1.03B, Prestressing, or 2006 SS, Section 50-1.08, Prestressing.
92 2010 SS, Section 50-1.03B(2)(d), Bonding and Grouting, or 2006 SS, Section 50-1.09, Bonding and Grouting.
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xi. Concrete Structures (SS Section 51)
   xii. Finishing Concrete
   xiii. Reinforcement (SS Section 52).
   xiv. Shotcrete (SS Section 53).
   xv. Materials
   xvi. Portland Cement Concrete (SS Section 90,

IV. Job Books Setup

A. Contractor’s code of safe practices (Category 6).

B. Welding Quality Control Plan, Stud Welding (Category 9).

C. Contractor’s Submittals (Category 12):
   i. Ground Anchor assembly working drawings.
   ii. Earthwork submittal.
   iii. Hydraulic jack calibration chart and date of calibration.
   iv. Theoretical elongation calculations.

D. Ground Anchor Proof and Performance testing results, and Earth Stability
   Report results (Category 37).

E. Reports of Inspection of Materials (Category 41):
   i. Prestress Tendon Steel.
   ii. Sheathing (corrugated sheathing thickness.)
   iii. Corrosion Inhibitors.
   iv. Anchor assemblies (Anchor Heads, Bearing Plates, Trumpets, and
      Wedges).
   v. Steel Soldier Piling.
   vi. Treated Timber Lagging.

F. Concrete Records (Category 43):
   i. Grout, Shotcrete, and Structural Concrete Mix designs.

V. Preconstruction Discussion with Design & Geotechnical Services

A. Review Log of Test Borings and other geotechnical information for soil conditions. Discuss
   geotechnical design issues with the Geoprofessional and the Designer.

B. Discuss the possible need for use of a “grout sock” in case of drilling through
   unfavorable conditions such as shattered rock or fractured formation. A “grout
   sock” is a porous layer of filter fabric or equivalent that lines the wall of the
   hole. It is inserted into the hole or inside the temporary casing prior to the

93 2010 SS, Section 51-1.03F, Finishing Concrete, or 2006 SS, Section 51-1.18, Surface Finishes.
94 2010 SS, Section 55-1.02, Materials, or 2006 SS, Section 55-2, Materials.

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grout being pumped into the hole. This is to prevent the excessive loss of grout into cracks, fissures or voids in native materials that are encountered.

VI. Preconstruction Meeting with Contractor

A. Remind the Contractor of his responsibility to submit Ground Anchor and/or Shoring working drawings, Earthwork/Excavation plan, notice of material sources, and grout, shotcrete, and structural concrete mix designs in a timely manner.

B. Discuss the locations of survey stakes and reference points to be provided by the Engineer, and the requirements for filling out and submitting Survey Staking Requests.

C. Discuss with the Contractor location of field storage of construction materials to prevent damage. Discuss methods to control grout and other SWPP issues.

D. Discuss proposed drilling methods with Contractor, including the use of casing, grout socks, etc. Check for drilling conflicts. Is there enough horizontal clearance between the roadway and the drilling operation? Has the Contractor notified USA and checked for clearance between soldier piles, ground anchors and utilities?

E. Discuss with the Contractor in detail how they will perform the Performance and Proof testing. For example, the use of a temporary stressing chair, use of shims, recording the elongation using a dial gage or caliper, etc. Emphasize the testing procedures as specified (for time and load) will be strictly enforced to accurately measure the creep in the system. No deviations are allowed for such testing.

VII. Submittal Reviews

A. The Ground Anchor Assembly shop drawings and Earthwork/Excavation plan should be submitted to the Documents Unit. Ensure the Designer authorizes the plan submittals before any fieldwork begins.

B. Review authorized shop drawings for bond length of ground anchors, drilled hole diameter, centralizer spacing/layout, and grouting procedures.

C. Inspect site condition for equipment and crane setup. If a suspension platform is used for drilling, request and review information for load capacity requirements. Does the excavation plan work, or is a temporary shoring system necessary?
A. Materials:
   i. Coordinate with METS and verify source inspection and release of materials.
   ii. Collect release tags on the ground anchor assemblies, bearing plates etc. and verify lot numbers with Form TL-29. Visually check all ground anchor assemblies and reinforcing steel for damage and defects upon delivery and prior to use.
   iii. Visually check encapsulated tendons for compliance with the specifications and for any damage to the corrosion protections.
   iv. Measure ground anchor assemblies and ensure centralizers are installed as per authorized shop drawings.
   v. Verify compliance of geocomposite drainage materials with the contract plans/specifications.
   vi. Take photos of stored materials.
   vii. Verify storage of materials such as treated timber lagging complies with SWPPP requirements.

B. Inspection: Earthwork:
   i. Prior to the start of any wall construction, check for any variance between the actual ground surface elevations along the wall line and those shown on the contract plans.
   ii. Use survey stakes and reference points to verify wall layout line. Work with the Contractor at the beginning of each shift to check wall layout and layout holes to be drilled for the day.
   iii. Ensure that the submitted Earthwork/Excavation plan proposes a realistic and detailed construction sequence that includes measures to ensure slope stability during all stages of excavation. The named competent person shall be on site at all times during excavation/earthwork.
   iv. Ensure the Contractor’s stability analysis has been completed prior to production excavation as per the contract specifications.\(95\)
   v. Frequently ensure that stable excavation conditions are being maintained, both for general mass excavation and wall neat finish face excavation. Remind the Contractor that finish face excavation needs to be completed within the time limit shown in the project documents or authorized working drawings.
   vi. Work with Caltrans Surveys and the Contractor’s grade setter to ensure the excavation is being performed within the specified tolerances for line and grade, and that overexcavation is not taking place.
   vii. Ensure that each preceding segment of the wall is structurally complete prior to performing excavation of the next lift.

\(95\) 2010 SS, Section 19-3.01A(3)(b), Stability Test for Ground Anchor and Soil Nail Walls, or Special Provisions for contracts using 2006 SS.
viii. Ensure that no work takes place in different excavation zones, as listed in the contract specifications, until the required stability testing is complete.

C. Inspection: Drilling holes and installing ground anchors:
   i. Discuss with the Contractor prior to start of drilling operation measures to control dust and drill cuttings forced out of hole during drilling, particularly operations adjacent to traffic.
   ii. Discuss with the Contractor prior to start of drilling operation measures to control water used to flush cuttings out of drilled hole and grout expelled from drilled hole.
   iii. Verify and record drilling methods, whether rotary or percussion, and drilling progress in length per time. If boulders or similar objects are encountered, the Contractor must provide measures to advance the hole to plan dimension.
   iv. Verify and record whether casing will be used. If groundwater or soft soil is encountered, the Contractor is required to provide measures to protect holes from caving in, as well conform to the construction dewatering plan (part of excavation plan) to control water flow.
   v. Verify and record drilling spoils using standard soil classifications throughout the drilling operation. Check in with the drilling foreman on a regular basis.
   vi. Verify the diameter of the drilled hole by measuring the casing teeth and/or drill string bit.
   vii. Verify the inside diameter of the casing to verify the ground anchor, with centralizers, will fit inside casing, if applicable.
   viii. Ensure the vertical angle and drill diameter is in accordance with the authorized working drawings.
   ix. Verify the drilled hole alignment is perpendicular to the layout line at each drilled hole location or matches plan requirement.
   x. Ensure the hole is drilled to the specified depth and is clear prior to installation of the ground anchor. Adhere to project requirements regarding installation of ground anchor assemblies and maximum time permitted before grouting to ensure no caving between installation and grouting stages.
   xi. Drilling, ground anchor installation and the grouting should be done in one shift to avoid caving of the soil.
   xii. Check that centralizers are placed as per the authorized ground anchor shop drawings.
   xiii. Verify the ground anchors are installed to the correct depth without excessive force.
   xiv. Verify grout mix design proportions, test consistency, (CA Test 541). If sand is used in the grout mix design, check the penetration in accordance with CTM 533.
   xv. Verify grouting sequence. For permanent ground anchors with full length corrugated sheathing (Alternative B), verify primary grouting on inside
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of sheathing before grouting of the drilled hole outside of sheathing. Primary (initial) grouting outside of sheathing is held to a minimum of 6 inches below the trumpet for holes equal or less than 6 inches in diameter.

xvi. Following primary grouting and prior to installing trumpet, verify grout outside of sheathing is held sufficiently below trumpet.

xvii. Verify secondary grouting following successful testing and lock-off of ground anchor.

xviii. Verify final grouting takes place no earlier than 24 hours after secondary grouting.

xix. Verify Contractor is recording grout volumes and pressures during grouting, including post-grouting.

D. Ground Anchor Testing:

i. Request the jack number and corresponding gage numbers from the Contractor prior to the testing date – see SC website under Field Resources/OSM Prestress Calibration Charts for a current listing.

ii. Verify stressing equipment meets specifications (jack “ram”, pump and pressure gages) and was calibrated by Caltrans METS within the last year. Verify the length of the ram extension (stroke) is sufficient for the calculated elongation of strand anchors. The combination of fewer strands and long unbonded lengths can produce large strand elongations during testing.

iii. Review the California Prestress Manual and BCM 160-3.0, Pressure Cells, and verify equipment is working properly. Notify the Contractor that you will be monitoring loading using the pressure cell.

iv. Verify the device proposed for measuring ground anchor movement meets the requirements of the contract specifications96.

v. Verify the Contractor’s stressing chair is adequate for the proposed loads.

vi. Verify the permanent lock-off materials (wedge plates, anchor heads, shims) are adequate for the lock-off load and the maximum test load. There may be a difference between stressing heads and permanent wedge plates.

vii. Verify the Contractor is not using the ram extension for measuring ground anchor movement. The ram extension may include movement of the structure in response to the ground anchor load.

viii. Review testing and lock-off procedures:

• Identify specific ground anchors for Proof and Performance testing.
• Verify adequate grout cure (and concrete whaler cure if the ground anchor is stressed against the concrete structure).
• Verify ground anchor grouting is held a sufficient distance back from the wall.

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96 2010 SS, Section 46-1.01D(2), Load Testing, or Special Provisions for contracts using 2006 SS.
• Ensure proper arrangement of strands to achieve uniform loading of strands.
• Thread anchor head on bar or place permanent wedge plate and wedges on strands.
• Set stressing chair against bearing plate or whaler.
• Place ram on stressing chair running bar or strands through ram center hole. Place stressing head or loading plate/wedges against ram cylinder.
• Set up independent measuring device such as a tripod with dial gauge.
• Align ram along axis of the ground anchor.
• Apply alignment load to ram. Check for uniform bearing of anchor head on bearing plate and alignment of ram with strands.
• Commence with testing.

ix. Performance vs. Proof Tests:
• Performance Testing: Cyclic loading with incremental increase in maximum load for each cycle up to the factored test load shown.
• Proof Testing: Single cycle of loading to the factored test load shown.

x. Performance and Proof Tests:
• The contract plans specify which ground anchors to performance test.
• All other ground anchors are proof-tested.
• Loading:
  o Each loading increment applied within 1 minute.
  o Each loading increment held for no more than 2 minutes.
  o Movement for each load increment is noted and recorded.
  o Test load hold at the factored test load shown, held constant for 10 or 60 minutes
  o Loading schedules require a return to AL after achieving the Test Load for both proof and performance tests. Without returning to AL, the total elastic movement cannot be determined. Requirement to seat the wedges at the Test Load should not be inferred to mean that the wedges are seated after reaching the Test Load initially and prior to returning to the AL. The test should first be completed in its entirety, including returning to AL. Then the ground anchor should be stressed again to the Test Load and the wedges seated.

xi. Test Acceptance Criteria:
• Elastic movement exceeds 80% of theoretical.
• Movement at the factored test load hold is less than 0.04 inch after 10 minutes or 0.08 after 60 minutes.

xii. Failed Tests:
• Elastic movement does not exceed 80% of theoretical. Possible causes:
  o Unbonded length insufficient – should be checked by visually inspecting prior to installation.
o Insufficient loading is being applied to the strands by the testing mechanism – verify accuracy of gauge pressure with a properly calibrated pressure cell.

o Check load path from ram to strands for losses – check that wedges are properly seated within both the loading plate and permanent anchor head.

o Contact the Designer and Geoprofessional for further direction.

• Movement at test load hold exceeds 0.08 inches after 60 minute test load hold:
  o Ground anchor is rejected.

xiii. Contractor’s responsibility to address failed ground anchors:

• Elastic movement does not exceed 80% of theoretical:
  o If applied loading and load path are satisfactory, reject and replace ground anchor.

• Movement at test load hold exceeds 0.08 inch:
  o Contractor normally repeats post grouting and retests after sufficient cure time.

xiv. File all testing results within Category 37 for ground anchors.

E. Lock-Off:

i. Lock-off results in relaxation of ground anchor force to the lock-off load shown:
   • The lock-off load is specified to achieve residual capacity within the ground anchors.

ii. Lock-off conducted upon successful testing of ground anchors:
   • Ram is backed off anchor head.
   • Strands stressed to relax anchor head off shims.
   • Shims between anchor head and bearing plate removed.
   • Anchor head returned to bearing plate.
   • Perform Lift-off test.
   • For strand tendons, permanent wedges shall be fully set in the anchor head while the tendon is stressed to the factored test load shown and then locked off at the lock-off load shown.

iii. Lift-off Test:
   • Verifies force in the ground anchor.
   • Load reapplied to strands until anchor head lifts off of bearing plate.
   • Pressure/load at lift-off noted, should be within 5% of required lock-off load shown.
   • Record final force in ground anchor on test sheets.
   • Potential Problems:
     o Actual lift-off force exceeds lock-off load shown in excess of 5% tolerance:
       (i) Lock-off shim thickness used was too thin.
       (ii) Back strand wedges out of anchor head.
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(iii) Re-stress to the factored test load shown.

(iv) Re-seat the wedges at the factored test load shown. (*Requires leaving teeth marks from wedges in Strand – Consult with Designer*).

(v) Repeat lock-off & lift-off test.
   - Actual lift-off force is less than the lock-off load shown in excess of 5% tolerance:
     (i) Lock-off shim thickness used was too thin.
     (ii) Install a permanent shim between anchor head and bearing plate.

F. Testing / Stressing Summary:
   i. Stressing / Testing requires full-time, attentive inspection.
   ii. Ensure residual force in each ground anchor is per contract documents within allowed tolerances.

G. Safety:
      - Section IV Excavations.
      - Section XVII Earthwork.
   iii. Hold project-specific Safety Meeting prior to start of testing operation for all employees.

IX. Project Completion / As-Builts

A. Bridge Construction Memo 9-4.0, *Report of Completion for Structures*, applies:
   i. Do not forward post-tensioning test results – maintain within the job files.
   ii. Note/draw any modifications on the As Built drawings on the number or location of ground anchors.

B. Send all project completion records and As Builts to the appropriate Office Associate at the following address:

   Division of Engineering Services
   Structure Construction
   1801 30th Street, M.S. 9-2/11H
   Sacramento, CA  95816
   Email: SC Office Associates@DOT

I. Forms

A. Refer to the SC Intranet, BCRP Manual Section 169Forms, for various updated forms relating to ground anchor construction:

98 http://onramp.dot.ca.gov/hq/oscnet/sc_manuals/crp/vol_1/crp016.htm
K7. Soil Nail Wall Construction Checklist

General Overview

In conjunction with Chapter 11, *Ground Anchors & Soil Nails*, a construction checklist for soil nail wall construction has been developed to assist field personnel in preparing documents and inspecting fieldwork to ensure compliance with contract requirements. It is important for Structure Representatives to review the contract plans and specifications, meet and go over them with Caltrans staff, conduct preconstruction meetings with the Contractor to lay out procedures, identify field problems, and other issues.

The Contractor shall comply with the CalOSHA Construction Safety Orders and Storm Water Pollution Prevention Plan to reduce potential impacts to the site.

Structure Representatives are encouraged to employ the following checklist for soil nail wall construction. Contact the Earth Retaining Systems Specialist in the Office of Structure Design in Sacramento for additional assistance.

Soil nails provide a means to reinforce and strengthen an existing soil structure in order to achieve a slope face steeper than the natural angle of repose. Soil nails provide tensile reinforcement for soils which typically exhibit low tensile strength. They are termed “passive inclusions” as they are not pre-tensioned, but rather simply grouted in place along their full embedment into the ground. Soil nails are designed with sufficient embedment depths to adequately transfer the tensile stresses developed by the active soil mass pressures, back into stable soil structures behind the active failure planes.

I. Sources of Technical Information

A. Foundation Manual:
   i. Chapter 11, *Ground Anchors & Soil Nails*.


II. Sources of Project Specific Information

A. Structures Pending File.
   i. Designer’s notes

B. Supplemental Project Information.
   i. *Foundation Report(s)*
   ii. Local, Regional, State, and Federal regulatory and permit specific requirements.
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III. Contractual Requirements

A. Special Provisions:

B. Contract plans:
   i. General Plan and & Elevation
      • Check stationing, grades and bearings with District Plans.
   ii. Typical Section
      • Note soil nail layout, spacing and inclination angles.
      • General notes.
      • Review ultimate bond stress for Test Load determination.
   iii. Structure Plans/Elevations
      • Verify wall grades, stationing and dimensions.
   iv. Foundation Plan
      • Review with respect to construction layouts, utility plans and drainage plans.
   v. Soil Nail Details
      • Review soil nail details for production and test assemblies, note total overall and bonded lengths.
      • Review drainage details for geotextile material placement.
   vi. Log of Test Borings
      • Review LOTB’s coring location with respect to wall layout and stationing.

C. Standard Specifications
   i. Job Site and Document Examination
   ii. Submittals
   iii. Welding
   iv. Water Pollution Control
   v. Environmental Stewardship
   vi. Structure Excavation and Backfill
   vii. Slurry Cement Backfill
   viii. Soil Nails

99 2010 SS, Section 2-1.07, Job Site and Document Examination, or 2006 SS, Section 2-1.03, Examination of Plans, Specifications, Contract, and Site of Work.
100 2010 SS, Section 5-1.23, Submittals, or 2006 SS, Section 5-1.02, Plans and Working Drawings.
101 2010 SS, Section 11-3 or Special Provisions for contracts using 2006 SS.
102 2010 SS, Section 13 or Special Provisions for contracts using 2006 SS.
103 2010 SS, Section 14 or Special Provisions for contracts using 2006 SS.
105 2010 SS, Section 19-3.03F, Slurry Cement Backfill, or 2006 SS, Section 19-3.062, Slurry Cement Backfill.
106 2010 SS, Sections 46-1, Ground Anchors and Soil Nails, General & 46-3, Soil Nails, or Special Provisions for contracts using 2006 SS.
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ix. Prestressing
x. Bonding and Grouting
xi. Concrete Structures (SS Section 51).

xii. Finishing Concrete
xiii. Reinforcement (SS Section 52).
xiv. Shotcrete (SS Section 53).
xv. Materials
xvi. Geosynthetics (SS Section 88)
xvii. Portland Cement Concrete (SS Section 90).

IV. Job Books Setup

A. Contractor’s Submittals (Category 12):
i. Soil Nail/Rock Anchor assembly shop drawings.
ii. Earthwork submittal.
iii. Hydraulic jack calibration chart and date of calibration.
iv. Theoretical elongation calculations.
v. Welding Quality Control Plan - onsite subassembly, stud weld (Category 9).

B. Initial and Acceptance Tests (Category 37).
i. Soil Nail Verification, Proof and Supplemental test results, shotcrete tests and soil compressive test results.

C. Reports of Inspection of Materials (Category 41):
i. High strength bars, Soil Nail assemblies.
ii. Geocomposite drainage materials.
iv. Bar reinforcing steel.

D. Concrete (Category 43):
i. Shotcrete mix proportions and grout mix proportions.
ii. Plant inspection checklist.

V. Preconstruction Discussion with Design & Geotechnical Services

A. Review Log of Test Borings and other geotechnical information for soil conditions.
B. Discuss geotechnical design issues with the Geoprofessional and the Designer.

107 2010 SS, Section 50-1.03B, Prestressing, or 2006 SS, Section 50-1.08, Prestressing.
108 2010 SS, Section 50-1.03B(2)(d), Bonding and Grouting, or 2006 SS, Section 50-1.09, Bonding and Grouting.
109 2010 SS, Section 51-1.03F, Finishing Concrete, or 2006 SS, Section 51-1.18, Surface Finishes.
110 2010 SS, Section 55-1.02, Materials, or 2006 SS, Section 55-2, Materials.
C. Review authorized working drawings for bond length of test nails, drilled hole diameter, centralizer spacing/layout, and grouting procedures.

D. Discuss any concerns developed during the review of project information or as a result of preliminary site reviews.

VI. Preconstruction Meeting with Contractor

A. Discuss proposed drilling methods with Contractor.

B. Remind the Contractor of his responsibility to submit Soil Nail working drawings, Earthwork/Excavation plan, notice of material sources, and grout and shotcrete mix designs in a timely manner.

C. Discuss the locations of survey stakes and reference points to be provided.

D. Discuss with the Contractor in detail how they will perform the verification, proof, and supplemental testing of soil nails.

VII. Submittal Reviews

A. Review Contractor’s earthwork submittal for excavation relative to the limitations of the site and construction safety requirements for earthwork.

B. Review soil nail assembly shop drawing submittal for compliance with project plan requirements.

C. Review Memo to Designers 5-14, Review of Working Drawings for Ground Anchors.

D. Check the calibration dates of proposed test equipment.

E. Verify test loads with contract specification test requirements.

F. Review bonded lengths for verification test assemblies.

G. Check that concrete mix and grout proportions are consistent with the contract specifications.

VIII. Construction

A. Materials:

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111 2010 SS, Section 46-3.01D(2), Reinforced Concrete Crib Wall. or Special Provisions for contracts using 2006 SS.
i. Collect release tags on the soil nail assemblies and bearing plates and verify lot numbers.

ii. Visually check all soil nail tendons and reinforcing steel for damage and defects upon delivery and prior to use.

iii. Visually check epoxy coated or encapsulated tendons for compliance with the contract specifications and for any damage to the corrosion protections.

iv. Measure soil nail assemblies and ensure centralizers are installed as per authorized shop drawings.

v. Verify compliance of geocomposite drainage materials with the contract plans/specifications.

B. Inspection: Excavation and Drilling:

i. Prior to the start of any wall construction, check for any variance between the actual ground surface elevations along the wall line and those shown on the contract plans.

ii. Ensure that the submitted Earthwork/Excavation plan proposes a realistic and detailed construction sequence that includes measures to ensure slope stability during all stages of excavation. The named competent person shall be on site at all times during excavation/earthwork.

iii. Ensure the Contractor’s stability testing has been completed prior to production excavation per the contract specifications\(^\text{112}\).

iv. Ensure that Verification testing has been completed prior to production excavation per the contract specifications\(^\text{113}\).

v. Frequently ensure that stable excavation conditions are being maintained, both for general mass excavation and wall neat finish face excavation. Work with Surveys and the Contractor’s grade setter to ensure the excavation is being performed within the specified tolerances for line and grade, and that over-excavation is not taking place.

vi. Ensure that each preceding segment of the wall is structurally complete prior to performing excavation of the next lift.

vii. Use survey stakes and reference points to verify wall layout line. Work with the Contractor at the beginning of shift to check wall layout and layout holes to be drilled for the day.

viii. Verify drilling method, whether rotary or percussion. Vibration may cause unwanted settlement or raveling of the excavated face.

ix. Ensure the vertical angle and diameter of drilled holes is in accordance with the authorized shop drawings. Drilled hole alignment should be perpendicular to the layout line.

x. If groundwater or soft soil is encountered, the Contractor is required to provide measures to protect holes from caving in, as well conform to the construction dewatering plan to control water flow.

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\(^\text{112}\) 2010 SS, Section 19-3.01A(3)(b), Stability Test for Ground Anchor and Soil Nail Walls, or Special Provisions for contracts using 2006 SS.

\(^\text{113}\) 2010 SS, Section 46-3.01D(2)(b)(ii), Verification Test, or Special Provisions for contracts using 2006 SS.
xi. Ensure the hole is drilled to the specified depth and is clean of debris prior to installation of the soil nail assembly.

xii. Drilling, insertion of soil nails and the grouting should be done in one shift to avoid caving of the soil.

xiii. The Contractor shall complete installation of soil nail assemblies, grouting, placing welded wire mesh and horizontal and/or vertical reinforcement steel, and primary shotcrete lift prior to performing excavation of adjacent area.

xiv. Check that centralizers are placed as per authorized soil nail assembly shop drawings.

xv. Make sure the soil nails are inserted to the correct depth without driving or forcing into the soil.

xvi. Verify grout consistency with flow cone (CA Test 541).

- Grout tube must be inserted at the bottom of hole and gradually pulled up to reduce the formation of air pockets along the soil nail.

C. Inspection: Drainage:

i. Check the geocomposite drain strips and weephole outlet pipes are installed as specified.

ii. Check that drain elements are interconnected and provide continuous drainage paths.

iii. Check the reinforcing steel has been installed at the locations and to the dimensions specified.

D. Testing:

i. Check and calibrate the Caltrans pressure cell equipment using the appropriate calibration information supplied by the Contractor.

ii. Check the soil nail properties necessary to calculate elastic elongation, i.e. steel modulus, grade, cross-sectional area, and unbonded test length.

iii. Check the soil nail length is sufficient to accommodate all testing equipment.

iv. Check the installed test nail’s drilled hole length and measure the unbonded length.

v. Check the Contractor’s testing equipment and methods match the authorized soil nail shop drawings.

vi. Verify the minimum compressive strength of the shotcrete is attained prior to testing.

vii. Check that the displacement gauges are in proper working order and have an appropriate travel length.

viii. Make sure the calibration information for the jack and gauges being used matches the equipment on site. Verify that the jack and gauges have been calibrated within the last year.

ix. Check that the jack bearing pads will not interfere with the soil nail/grout column during testing.

x. Check that the jack is aligned correctly with the soil nail.
xi. Check that the displacement gauge is aligned with the axis of the soil nail and that gauges are mounted independent of the soil nail and testing apparatus.

xii. Check that the jack does not drop onto the soil nail or lie on it. This could cause bending or eccentric loading to the soil nail.

xiii. Check that the minimum alignment load is maintained at all times. Periodically check the jack alignment is maintained.

xiv. Check that constant load is maintained during the creep test.

xv. Check that load increments are applied and held within the specified time limits for the test.

E. Safety:
      • Section XVII, Earthwork and Excavation.
   iii. Hold project-specific Safety Meeting prior to start of testing operation for all employees.

IX. Project Completion / As-Builts

A. On the General Plan sheet, indicate any changed dimensions or obstructions. If any significant differences exist or differ from the contract plans at any particular location, it should be noted on the general plan sheet. Refer to BCM 9-1.0, As-Built Plans.

B. Note any unusual conditions encountered and the corrective methods taken for specific locations for future consideration. Refer to BCM 9-4.0, Report of Completion for Structures.

C. Send all project completion records and As-Builts to the appropriate Office Associate at the following address.

Division of Engineering Services
Structure Construction
1801 30th Street, M.S. 9-2/11H
Sacramento, CA 95816
Email: SC Office Associates@DOT