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Division of Structures

Foundation Manual

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

Signed,

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Preface

The Foundation Manual is intended to provide the field engineer with information that may be of some assistance 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 judgement. What action does the engineer take when pile bearing capacity is not obtained at specified tip or reaches "refusal" ten 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 one's own experience.

There is no substitute for utilizing sound engineering judgment in solving engineering problems. If all problems are solved in this manner, then the engineer can be confident that a good solution was used to solve the problem.

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1 Foundation Investigations

Introduction

The ultimate strength and longevity of any structure depends on the adequacy of its foundation. Engineers administering projects for the Offices of Structure Construction have the responsibility of ensuring that the foundation work performed on their projects is of the quality necessary to allow each and every structure to sustain the design loadings throughout its design life.

It is essential that all personnel working for the Offices of Structure Construction, and Structure Representatives in particular, commit themselves to learning the provisions within the Standard Specifications, Standard Plans, contract plans, special provisions and all relevant documents related to each structure on which they are working. It has been proven time and time again that a thorough understanding of all documents related to a particular project and the effective use of this information leads to the effective administration of structure contracts.

Bridge Construction Memo 2-2.0 states:

"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."

In order to "clear up" problem areas, Structure Representatives must have a thorough understanding of the information contained within the contract documents. They must also know whom to contact for further information or for advice on resolving project problems.

This chapter will give an overview of the foundation investigation process and will also show how the Log of Test Borings and Foundation Report for a structure project are developed. The goal of the chapter is to provide information related to the foundation investigation process so as to assist the reader in the interpretation and effective use of the Log of Test Borings and the Foundation Report during the administration of structure projects.

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 the four Geotechnical Design Sections of Geotechnical Services. Each of the Design Sections is responsible for different areas of the State.

- 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

Personnel from the Geotechnical Design Sections are available to provide support to Offices of Structure Construction employees throughout the life of a construction project. These individuals are referred to as geoprofessionals and they are engineers who specialize in geotechnical engineering and engineering geology. Some are registered as geotechnical engineers and engineering geologists. The Engineer is encouraged to schedule pre-construction meetings with personnel from the appropriate Geotechnical Design Section (Bridge Construction Memo 2-2.0). The primary purpose of the pre-construction meeting would be to forge a good relationship with the engineers/geologists (geoprofessionals) that performed the foundation investigation, wrote the Foundation Report, and developed the Log of Test Borings. At this time there should be discussions that outline potential foundation problem areas and risks in detail. This meeting will prove to be invaluable to Structure Representatives in their efforts to recognize potential problem areas that may need extra attention during the foundation work on the project.

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

At times, consultant engineers design structure projects. Consultant geotechnical companies produce foundation investigations for these projects with Departmental oversight. Issues related to foundations on projects designed by consultants should be discussed with the Department's Oversight Engineer assigned to the project.

Foundation Investigation Overview

Once the Office of Structure Design begins the design of a new structure, widening, strengthening or seismic retrofit, the Project Engineer or Designer sends in a Foundation Investigation Request to the appropriate Geotechnical Design Section. At that point a geoprofessional is assigned to perform the foundation investigation.

The individual assigned to perform a foundation investigation for a structure first collects as much information about the proposed site as possible. They normally accomplish this by reviewing preliminary structure plans, previously written foundation reports, as-built plans, information on the historical seismicity of the area, and historical information on the subsurface conditions in the area of the proposed structure. This planning phase of the investigation gives the geoprofessional an idea of what to look for during fieldwork.

Once all of the preliminary information is collected, a drilling plan is generated that outlines locations for drilling in relation to the structure's proposed foundation locations. The main goal in establishing a plan for a foundation investigation is to collect as much subsurface information at the site as possible while making efficient use of the available drilling equipment and personnel. The geoprofessional then directs a foundation drilling crew during the performance of the subsurface drilling operation (to be described later in this chapter). The purpose of the subsurface drilling operation is to collect soil samples and perform in-situ testing at the site.

The soil samples collected during the subsurface drilling operation, results of in-situ tests, manual field tests, and various observations recorded will provide the necessary information to develop the Log of Test Borings for the project. Once the Log of Test Borings is completed, it is transmitted to the Project Engineer for inclusion in the structure plans.

The information compiled in the Log of Test Borings along with the loads provided by Structure Design is analyzed by the geoprofessionals in Geotechnical Services and foundation recommendations are made. The recommended foundation type as well as other important pieces of information are compiled and included into a Foundation Report for the structure and transmitted to the Project Engineer. These recommendations are used to complete the design of the structure. The Foundation Report is included in the RE Pending File as well as the Materials Handout for the Contractor at time of bid.

Subsurface Drilling Operation

The most important aspect of a foundation investigation is the results obtained from the subsurface drilling operation. Foundation drilling crews conduct one or more drilling operations at the location of a proposed structure. The general purpose of the subsurface investigation is to determine the depth of rock, rock type and quality, soil types, soil strengths, and groundwater levels. The determination of these various parameters enables the development of a soil/rock profile, which is a visual representation of the subsurface conditions interpreted from the subsurface investigations and laboratory testing. The soil/rock profile can be determined by interpolating between like lenses of material in individual borings within the Log of Test Borings.

During the subsurface drilling operation, Geotechnical Services is responsible for the evaluation of the soil and/or rock samples retrieved by the foundation drilling crew. After visual inspections and manual field tests, the soil or rock samples are described and written in the field logs. During the drilling operation, elevations where there are significant changes in material are noted. Soil samples are usually taken from each of the different soil lenses (layers) for laboratory testing.

The appearance and feel of the cuttings, difficulties or changes of the rate of advancement of the drilling tools, and other observations help estimate the mechanical properties or strengths of the soil or rock lenses. These observations are noted within the field logs. Any groundwater encountered during the drilling operation is also noted and special care is taken to accurately determine its elevation and whether or not the groundwater encountered is static or under pressure ("perched" or in an "artesian" condition). These observations along with the tests 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 the 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. As such, 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 on the characteristics of the material. 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 for pile and footing foundations and may also provide valuable information to the Engineer during the course of administering a construction project.

Log of Test Borings

After the subsurface investigation and laboratory testing is complete, the Log of Test Borings is developed for the project. The Log of Test Borings 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 are also described. Each Log of Test Borings includes a standard legend on the left side of the sheet that describes the different symbols and notations used within the Log of Test Borings. Examples of Logs of Test Borings are included in the “Caltrans Soil and Rock Logging, Classification, and Presentation Manual” provided in Appendix A. It can also be found on the Offices of Structure Construction website.

Foundation Report

The foundation report is basically a compilation of all the information retrieved during the foundation investigation and provides the project engineer with a description and an evaluation of the geological formations and soils present at the site of a proposed project. It also describes and evaluates any seismic hazards that may be present at the site such as the amount of ground shaking that can be expected and the probability of liquefaction occurring at the site. The report gives recommendations for the type of foundation that should be used to support the proposed structure and also recommends seismic design criteria such as peak horizontal bedrock acceleration that should be used in the seismic analysis. The report includes the recommendations for bottom of footing elevations, pile type, size and tip elevations.

Most reports include special comments regarding anticipated foundation related constructability concerns 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 special provisions. The Structure Representative should pay particular attention to these comments as advance knowledge of potential problem areas in foundation work allows for more effective problem solving and mitigation. The Foundation Report is normally included in the RE Pending File and is included in the Materials Handout to the Contractor at time of bid. The Engineer should contact the Offices of Structure Construction Headquarters in Sacramento if they do not receive a copy of the Foundation Report for any project assigned to them.

The project plans should be reviewed 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. The Project Engineer and Geotechnical Services representatives should be consulted if there are any discrepancies. Contract change orders will most likely be required to address these discrepancies.

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 how the Contractor makes preparations to deal with the constructability issues discussed in the Foundation Report. Good documentation of all conversations with the Contractor on these issues will help in the evaluation of any potential claims submitted by the Contractor.

Applicability of the Log of Test Borings and Foundation Report to the Contract

It is very important for Structure Representatives, as well as all Structure Construction field staff, to be aware of how the Standard Specifications describe the applicability of the Log of Test Borings, Foundation Report, or any record of subsurface investigation produced by the State. Section 2-1.03 of the Standard Specifications describes the contractors' responsibilities to review these documents prior to performing work for the Department.

In the past, the Log of Test Borings and other information provided to the contractor at time of bid were not considered part of the contract and were provided for information only. The 2006 version of the Standard Specifications has been revised to change this. In particular, Section 2-1.03 Examination of Plans, Specifications, Contract, and Site of Work, has undergone a major revision. While the Contractor is still required to investigate the site and other available information, as before, it is now understood that the information provided by the Department will be used by the Contractor to develop a competitive bid. The accuracy of this information is essential to a claim free contract. It's important to note that while the Department is taking responsibility for the information provided, the Contractor is still required to carefully examine the site and the information provided and are responsible for the conclusions that are drawn from that investigation.

Basic Soil Properties

In order to understand and interpret a Log of Test Borings and Foundation Report, it is important to have a basic understanding of the different types of soils that may be encountered during foundation investigations. Geotechnical Services has recently published the "Caltrans Soil and Rock Logging, Classification, and Presentation Manual". (Appendix A). It contains information on the field and laboratory procedures used in soil classification and descriptions. It will help the Engineer interpret the information presented in the Logs of Test Borings, Foundation Reports and communicating with Geotechnical Services.

The information presented in Chapter 2 of the Caltrans Soil and Rock Logging, Classification, and Presentation Manual (Appendix A) is of particular importance as it outlines the procedure and methodology used to identify and classify rock and soil samples. The 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.

The following is a list of soil particle size definitions used by Geotechnical Services:

CLASSIFICATION	DEFINITION
Boulders	Particles of rock that will not pass a 12-inch square opening.
Cobbles	Particles of rock that will pass a 12-inch square opening but will be retained on a 3-inch sieve.
Course Gravel	Particles of rock that will pass a 3-inch sieve but will be retained on a 3/4-inch sieve.
Fine Gravel	Particles of rock that will pass a 3/4-inch sieve but will be retained on a No. 4 sieve.
Course Sand	Particles of rock that will pass a No. 4 sieve but will be retained on a No. 10 sieve.
Medium Sand	Particles of rock that will pass a No. 10 sieve but will be retained on a No. 40 sieve.
Fine Sand	Particles of rock that will pass a No. 40 sieve but will be retained on a No. 200 sieve.
Silt	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
Clay	Soil passing a No. 200 sieve that can be made to exhibit plasticity (puttylike properties) within a range of water contents, and that exhibits considerable strength when air-dried. A clay is qualified as fat or lean depending an the amount of plasticity
Organic Soil	A soil with sufficient organic content to influence the soil properties.
Peat	A soil composed primarily of vegetable matter in various stages of decomposition. This soil usually has an organic odor, is dark brown to black in color, has a spongy consistency, and a texture ranging from fibrous to amorphous.

Geoprofessionals describe soils by name, group symbol and also provide descriptive components 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 2-3 “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 shown below:

> *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 is generally sufficient to differentiate between the coarse grained soils. However, the distinctions between soil particles such as silts and clays can be difficult. Several simple field exercises utilizing measures of settling, plasticity, dry strength, and permeability characteristics of the soil permit a more accurate classification of these soils. In addition, soil samples can be taken to the laboratory and tested to determine plasticity, unit weight, unconfined compressive strengths and other mechanical properties to refine field classification. In lieu of that, the following can be used in the field to help classify soils in the field:

- 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.

- 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.

- 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.

- 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 clayey soil, this reaction occurs slowly or not at all due to the level of plasticity of the sample.

Geotechnical Drilling and Sampling Equipment

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

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

The following is a brief description of the various pieces of equipment used by Geotechnical Services as well as consultant geotechnical companies.

	DESCRIPTION
2¼-Inch Cone Penetrometer	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 Engineering Geologist records the drilling rate in seconds per foot of penetration. The results of the test are shown graphically to give an indication of the soil's varying densities as the cone penetrates the different layers of soil.
Sample Boring	The 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. The blows per foot are recorded by the Engineering Geologist 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 the site.
Rotary Boring	The 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 the 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.
Auger Borings	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 where the groundwater elevation is. Most equipment can drill to depths of 100 to 200 feet.
Diamond Core Boring	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 Engineering Geologist can inspect the cores to determine the competency of the rock.
Electronic Cone Penetrometer	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, pore water pressures, relative densities, and shear wave velocities.
Bucket Auger	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. It is also a good indicator for the presence of material that is likely to cave in during an excavation.

CHAPTER 2 Type Selection

All structure foundations have one fundamental characteristic in common; that is, they provide a means whereby service and ultimate loads are transmitted from the structure into the supporting geologic medium. The appropriateness of the different types of structure foundations are governed by loading requirements, site-specific geologic conditions, site accessibility, overhead clearance, existing utilities and the proximity of existing facilities such as buildings and railroads as well as site considerations such as vertical clearances and noise restrictions.

The Foundation Report is the primary source for information about the structure foundations on a project. It is prepared by Geotechnical Services in the Division of Engineering Services. The project engineer selects the appropriate foundation type based upon data and recommendations contained in this report. The Foundation Report may include recommendations and engineering data for several foundation types. In this case, field conditions and/or economics will generally determine the foundation type.

Structure foundations can generally be classified in the following categories: (1) footing foundations (frequently referred to as spread footings), (2) pile-supported foundations (driven and non-driven piles), and (3) special case foundation types that would include micro-piles, tie-backs and tie-downs. Pier columns were once considered specialty foundation types but their use has become more prevalent over the years as they are thought to behave well seismically.

Seal courses are frequently 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 against hydrostatic pressure. After the concrete cures, the water is pumped out of the cofferdam and construction of the footing can occur "in the dry."

Generally, footing foundations are more economical than pile supported foundations. Cast-in-Drilled Hole (CIDH) concrete piles that are constructed "in the dry" tend to be the most economical pile-supported foundation with large diameter steel pipe piles generally being the most expensive.

Various geologic and non-geologic features affecting type selection are discussed in the following table. Most of these items will be discussed in more detail elsewhere in this manual.

TYPE SELECTION	USE
Footing Foundations	…are virtually unlimited in use. 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.
Driven Piles	…are used where foundation material will not support a footing foundation or discourages the use of Cast-In-Drilled Hole (CIDH) concrete piles. Pile types are precast concrete, steel structural 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.
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.
Special Case Foundations	…represent special applications and, therefore, have limited use.
Pier Columns	…are an extension of the pier to a planned elevation into rock. They generally used for hillside structures, thus eliminating the extensive excavation that would be required for large spread footings. The location and type of existing structures may restrict excavation limits.
Tiebacks and Soil Piles	…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. Geologic considerations include the soil profile and corrosive soil problems. Non-geologic considerations include adjacent structures, accessibility, and existing utilities.
Tiedowns or Tension Piles	…are used, in general, to address uplift concerns in seismic zones and for seismic retrofitting of existing footing foundations where uplift and overturning must be prevented.
Micro Piles	…are small diameter piles (less than 12 inches) that are drilled and filled with reinforcement and grout.

CHAPTER

3 Contract Administration

The design and construction of structure foundations is one of the most difficult and challenging responsibilities of the Department. A great deal of time and effort is taken in the design phase to adequately describe the existing soils; however the complex and variable geology found in many portions of the State of California tends to complicate these investigations. The investigations and recommendations made by Geotechnical Services are used by the Office of Structure Design to develop a design for a structure. The design should permit the structure to last throughout the years, withstand earthquakes and large storms that may undermine foundations through liquefaction, scour and the like.

Section 5-1.01 Authority of the Engineer of the *Standard Specifications* states that,

"The Engineer shall decide all questions… as to the acceptable fulfillment of the contract on the part of the Contractor; and all questions as to compensation". Contract Administration may be defined as the sum total of all 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: (1) interpretation and enforcement of the plans and specifications, (2) ensuring compliance with applicable Caltrans policies and procedures, (3) objective and subjective decision making (i.e. Engineering Judgment), 4) sampling, testing and inspection of the work, (5) problem solving that may result in changes to the contract to meet design intent, and (6) proper documentation to defend the Department's position regarding the accuracy of the information provided at the time of bid.

A well-administered contract does not always produce a situation where the contract is free from challenges and difficulty but it will provide a foundation that is in the best interest of the structure and therefore the Department. Foundation operations are "high risk" activities for all parties involved as they have the potential to impact construction budgets and schedules. Although it is the Contractor's contractual obligation 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 generally obtained when the Department and the Contractor have an attitude that is one of cooperation; that focuses on identifying issues as early as possible and that promotes working together to resolve them. The Department promotes the formation of a "Partnering" relationship with the Contractor in order to effectively complete the contract to the benefit of both parties. The purpose of this relationship will be to maintain cooperative communication and mutually 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 administrate the contract.

In order for the Engineer to decide the question of acceptable fulfillment of the contract on the part of the Contractor (i.e., successfully administer the contract), the contemplated work must be thoroughly understood. To achieve this, a detailed study of the contract documents must be made. This includes the Standard Specifications, Standard Plans, contract plans, and 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, proximity of utilities, existing structures, highways and railroads, etc. The order of work and construction sequences must be thoroughly understood. 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 in the area. Note any conflicts or potential problems and communicate them to the appropriate parties so that a path to resolution may begin.

In addition to the information described above, other documents to be reviewed are:

DOCUMENT	DESCRIPTION
Log of Test Borings	Prepared by Geotechnical Services and 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.
RE Pending File	Contains all the correspondence relative to a particular project and, therefore, provides not only a historical outline of its development, but information relative to existing or proposed utilities, potential problems and any other special considerations.
Preliminary Report	Prepared by the Preliminary Investigations Unit of the Project Management Branch, Office of Program/Project Management and Support. The 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 Project Designer with the required roadway geometrics, clearances, proposed and existing utilities and/or obstructions, and will discuss any potential problems or other special considerations.

DOCUMENT	DESCRIPTION
Foundation Report	Prepared by Geotechnical Services, it provides detailed information about the foundation investigation done for the structure or project. It is a part of the RE Pending File and included in the Materials Handout to Contractors. This report will contain a description of the area geology, a Log of Test Borings for selected locations and recommendations for foundation types and construction considerations. This report is very informative and should be thoroughly reviewed.
As-Built Drawings	Prepared by the Office of Structure Construction after successful completion of a contract. These documents can be useful when constructing widenings or when constructing new structures near or adjacent to existing structures.

The contract plans and specifications, the documents previously mentioned and a field investigation of the site must all be reviewed for compatibility. It is important that all ambiguities, discrepancies and/or omissions be resolved expeditiously so as to avoid unnecessary delays to the work.

In the past, the Log of Test Borings and other information provided to the Contractor at time of bid were not considered part of the contract and were provided for information only. The 2006 version of the Standard Specifications has been revised to change this. In particular, Section 2-1.03 Examination of Plans, Specifications, Contract, and Site of Work, has undergone a major revision. While the Contractor is still required to investigate the site and other available information, as before, it is now understood that the information provided by the Department will be used by the Contractor to develop a competitive bid. The accuracy of this information is essential to a claim free contract. It is important to note that while the Department is taking responsibility for the information provided, the contractor is still required to carefully examine the site and the information provided and are still responsible for the conclusions that are drawn from these materials.

It is imperative that the Engineer meets with the Project Engineer and the geoprofessional from Geotechnical Services to discuss substructure considerations and foundation details. If an on-site meeting is impractical, the meeting should be held by telephone/teleconference. Clarify and resolve any questions or inconsistencies and get a clear understanding of the foundation material as well as the potential risks or challenges anticipated in constructing the foundations. This would also be the appropriate time 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 technical and contractual requirements for the project, as well as the subsurface conditions that are expected to be encountered at the various foundation locations within the jobsite. 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 pre-construction conference(s) to be held with the Contractor and other interested parties/agencies.

Pre-construction conferences are usually held at about the same time that the Contractor begins mobilizing at the site, but well before work actually starts on the job. Five general subjects are normally covered: (1) safety, (2) labor compliance and affirmative action, (3) utilities, (4) environmental considerations and (5) matters related to the performance of the work itself. Depending on the individual policies of a particular District and the complexity of the project, more than one meeting may be appropriate so as to limit the scope and the number of individuals present. From this meeting should come a common understanding of the proposed work, the risks, challenges and potential solutions that may be expected during the life of the contract.

The pre-construction conference presents an excellent time 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 apply. However, the following list covers some of the areas that must be understood for effective contract administration:

ITEM	REFERENCE
Test Boring Information	*Standard Specifications*, Section 2-1.03
Excavation Safety Plans; Trench Safety	*Standard Specifications*, Sections 5-1.02A & 7-1.01E
Differing Site Condition	*Standard Specifications*, Section 5-1.116
Source of Materials	*Standard Specifications*, Section 6-1.01
Water Pollution	*Standard Specifications*, Section7-1.01G
Sound Control Requirements	*Standard Specifications*, Section7-1.01I
Public Safety	*Standard Specifications*, Section 7-1.09
Preservation of Property	*Standard Specifications*, Section 7-1.11, 19-1.02
Contractor's Responsibility for the Work and Materials	*Standard Specifications*, Section 7-1.16
Protection of Utilities	*Standard Specifications*, Section 8-1.10
Cofferdams	*Standard Specifications*, Section 19-3.03
Water Control & Foundation Treatment	*Standard Specifications*, Section 19-3.04
Foundation Inspection	*Standard Specifications*, Section 19-3.05
Foundation Revisions	*Standard Specifications*, Sections 19-3.07 & 51-1.03
Piling	*Standard Specifications*, Section 49
Seal Course	*Standard Specifications*, Section 51-1.10
Special Concrete Mix Designs	Special Provisions
Applicable Caltrans Policies	Various Manuals
Hazardous Waste Material	special provisions

All utility locations shown on the plans should be verified with the utility representative. Utilities constructed by local municipalities and the Department are not verified by the Utilities Service Alliance (USA) and will require the efforts of the Department and each individual municipality to identify and locate.

The Engineer should request as-built plans from 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 must also be discussed. Problems in this area could result in serious delays. If not solved at the pre-construction conference, these utility issues should be resolved at the earliest possible time.

The Contractor's proposed methods of performing foundation work adjacent to utilities should also be discussed at the pre-construction conference. All those present should be advised of any proposed change orders that may potentially affect their work or property.

All pre-construction conferences should be well documented. When appropriate, minutes of the meeting should be distributed to all attendees. This serves to confirm positions and/or 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.

Footing Foundations

Certain revisions in excavation limits, footing elevations and sizes, and changes to or elimination of seal course concrete are discussed 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 change order will ultimately be needed in order to allow the quantity change for the items affected by this revision (Bridge Construction Memo 2-9.0). Once it is determined that a change is necessary, the Contractor is issued a 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 (Bridge Construction Memo 2-9.0). An example of this letter is provided in Appendix C. The letter should advise the following:

ITEM	REMINDER/STATEMENT
1	A reminder that Section 51-1.03 of the Standard Specifications reserves to the Engineer the right to revise, as may be necessary to secure a satisfactory

ITEM	REMINDER/STATEMENT
	foundation, the footing size and bottom of footing elevations shown on the plans.
2	On projects involving seal courses, a reminder that Section 51-1.22 of the Standard Specifications allows the Engineer to revise or eliminate seal course shown on the plans.
3	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.
4	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.

Pile Foundations

In accordance with Section 49-1.08 "Pile Driving Acceptance Criteria" of the Standard Specifications, driven piles must achieve the required nominal driving resistance <u>and</u> penetrate to the specified tip elevation unless otherwise permitted in writing by the Engineer. Nominal Resistance is usually determined from the equation provided in this Section and is also know as the Gates Formula. Additional information regarding this formula can be found in Chapter 7 of this Manual and in BCM 130-4. The nominal resistance for large diameter piles is determined from non-destructive testing such as the pile driving analyzer (PDA) 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 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 & 8 of this Manual.

During pile driving operations one of the following scenarios will occur: (1) The pile will achieve the required nominal driving resistance but falls short of the specified tip elevation. (2) The pile will achieve the required nominal driving resistance and specified tip elevation. (3) 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 piling of longer lengths 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 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 Section 49-1.08 "Pile Driving Acceptance Criteria" of the Standard Specifications either to allow the 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 pay particular attention to the information provided on the pile data sheets of the contract plans. These sheets contain information on the design requirements/constraints for the piles and may include design tip elevations for compression, tension, lateral, downdrag, liquefaction and scour potential among others. The specified tip elevation is the deepest elevation of the foundation 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 doesn't control the design.

There have been changes made to Section 49-6.01 Measurement in the 2006 Standard Specifications in regard to measurement for piling. The changes are as follows:

> *The length of timber, steel, and precast prestressed concrete piles, and of cast-in-place concrete piles consisting of driven shells filled with concrete, shall be the greater of the following:*
>
> *A. The total length in place in the completed work, measured along the longest side, from the tip of the pile to the plane of pile cut-off.*
>
> *B. The length measured along the longest side, from the tip elevation shown on the plans or the tip elevation ordered by the Engineer, to the plane of pile cut-off.*

Piling that extend beyond the tip elevation shown on the plans as ordered by the Engineer to meet design requirements will be paid under the provisions of part "A" while piling that fails to reach the tip elevations shown on the plans but has been determined to be suitable for the design will be measured in accordance with part "B". (Bridge Construction Memo 130-6)

When steel "H" piles exhibit a trend where the piles need to penetrate beyond the specified tip elevation in order to achieve the required nominal resistance, the Engineer should consider using lugs in order to reduce the additional pile length required. 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 by extra work at force account or agreed price. Bridge Construction Memo 130-5.0 describes this process and shows a detail of a pile lug.

On projects involving Cast-In-Drilled-Hole (CIDH) concrete piles, the Contractor should be notified in writing that CIDH piles must penetrate at least to the specified tip elevation shown on the plans or as ordered by the Engineer and that no additional payment will be made for piles that penetrate below the specified or ordered tip elevation. Any ordered change 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 piling. These revisions shall be made in accordance with Section 49-4.03 of the Standard Specifications. In this instance, the Contractor is paid for 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.

Cast-In-Drilled-Hole (CIDH) concrete piles are sometimes constructed in the presence of groundwater or “in the wet”. This operation uses a drilling slurry to control groundwater and to maintain the stability of the drilled hole. The concrete is placed/poured under tremie and visual inspection is not possible. The Department uses non-destructive testing for these pile types to verify pile integrity. Chapter 9 of this Manual describes this process and outlines the roles and responsibilities of the Engineer to get the piles tested and to address the repair of any anomalous material identified by the testing.

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 discusses and explains the various forms that are to be completed during these activities. The information recorded on the forms is valuable to the Department 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. The information will also be used by Geotechnical Services to refine recommendations for future projects. In addition to the forms, OSC Headquarters keeps a database for various aspects of CIDH Piling constructed using the “wet” specification. (Bridge Construction Memo 130-13.0)

Bridge Construction Memo 9-1.0 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 have resulted 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.

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 Section 5-1.116 of the Standard Specifications. DSC occur when subsurface or latent physical conditions encountered at the site differ materially from those indicated in the contract; or when unknown physical conditions of an unusual nature that differ materially from those ordinarily encountered and are not generally recognized as inherent in the work are found.

According to Section 5-1.116, Differing Site Conditions, of the Standard Specifications, timely notification, documentation, and response is of the utmost importance. Each claim for differing site conditions is handled per project or individually. The Division of Construction has issued Construction Program Directive 01-12 (CPD 01-12) to outline the procedures to be followed should the Engineer receive a notice of a Differing Site Condition. Essentially the Engineer will draft a response and submit it to Management for review and approval prior to the actual response to the Contractor. The timelines for this process are very specific and proactive means will be required to achieve them. Individual Districts may have protocols in place to streamline this process. Consult with the Resident Engineer and the Bridge Construction Engineer immediately upon receipt of a Notice of Differing Site Condition.

There may be a situation where, after Management review, it is decided that the Contractor's Notice of Differing Site Condition has no merit. Should this occur, the Contractor has a timeframe, within which, 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 Section 9-1.04 "Notice of Potential Claim" of the Standard Specifications and applicable sections of the Contract Special Provisions will need to be followed.

CHAPTER

4 Footing Foundations

General

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 while pile foundations are covered in Chapter 5 of this Manual.

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 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.

Since the foundation will be supported only by the supporting soil mass, the quality of the soil is extremely important. The Standard Specifications allow the Engineer to revise elevation of footing foundations to ensure they are founded on quality material. Refer to Chapter 3 “Contract Administration” of this Manual for information on the responsibility of the Engineer as it applies to footing foundations.

Types

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."

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 will also cause an unbalanced load transfer into the soil as shown in Figure 4-1.

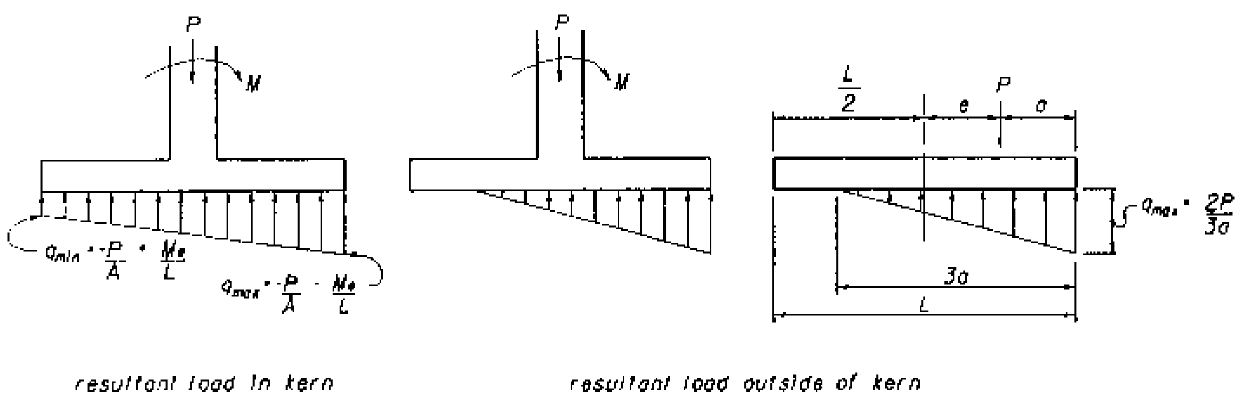


FIGURE 4-1 Loaded footing with moment

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 yet, tension on one side of the footing.

The problems resulting from eccentricities can be addressed by combining two or more columns onto a single footing. This is generally 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 referred to as a cantilever or strap footing.

Combined footings are generally 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 may also 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 supporting numerous columns and/or walls is referred to as a mat footing and is commonly seen in building work.

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

Footing foundations encountered in bridge construction almost always support a single structural member (column, pier or wall) and are invariably 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 have joined adjacent footings together.

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 are based primarily on data obtained empirically and through small, and more recently large, scale testing.

The ultimate strength of the soil 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}).

Failure Modes

The mode of failure for soils with bearing capacity overloads is a shear failure of the soil mass supporting the footing foundation. It will occur in one of three modes: (1) general shear, (2) punching shear, or (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.

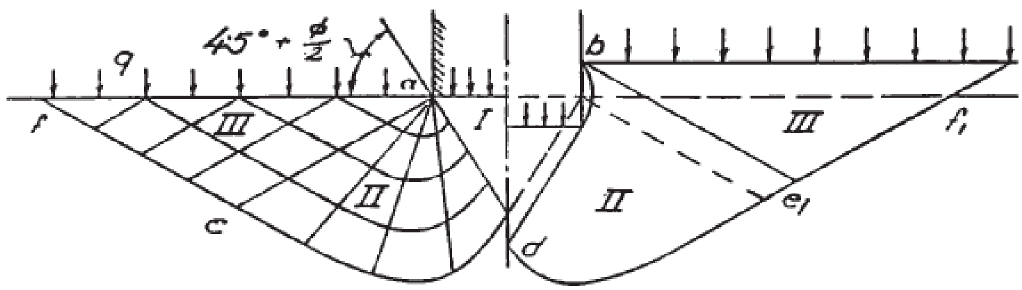


FIGURE 4-2 General shear failure concept

General shear failure is a brittle failure and is for the most part sudden and catastrophic. Although bulging of the ground surface may be observed on both sides of the footing after failure, the failure usually occurs on one side of the footing. For example, (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 is usually 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.

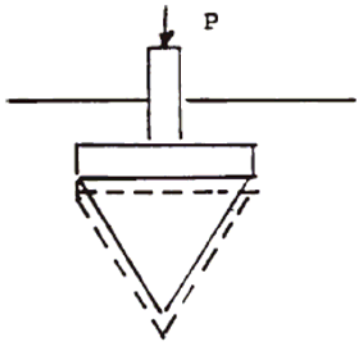


FIGURE 4-3
Punching shear failure

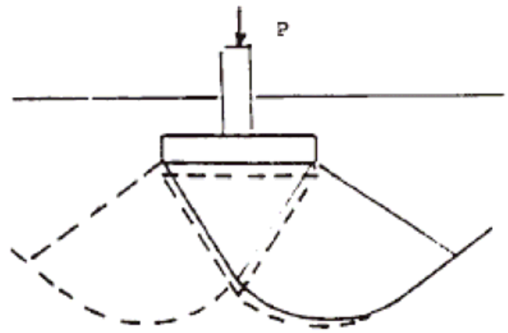


FIGURE 4-4
Local shear failure

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.

Punching shear failure pattern under a rectangular foundation on the surface of loose sand ($D_r = 15\%$). (From De Beer and Vesic, 1958.)

Local shear failure pattern under a rectangular footing on medium dense sand ($D_r = 47\%$). (From De Beer and Vesic, 1958.)

General shear failure pattern under a rectangular footing on dense sand ($D_r = 100\%$). (From De Beer and Vesic, 1958.)

FIGURE 4-5 Failure modes

The mode of failure mode for 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 is usually 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:

$$q_{ult} = \frac{\gamma B}{2} N_\gamma + cN_c + \gamma D_f N_q \quad \text{(Terzaghi)}$$

Where: q_{ult} = ultimate bearing capacity

γ = soil unit weight

B = foundation width

D_f = depth to the bottom of the footing below final grade

c = soil cohesion, which for the undrained 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_γ, N_c, and N_q are dimensionless bearing capacity factors that are functions of the angle of internal friction. The term containing factor N_γ 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.

Factors Affecting Bearing Capacity

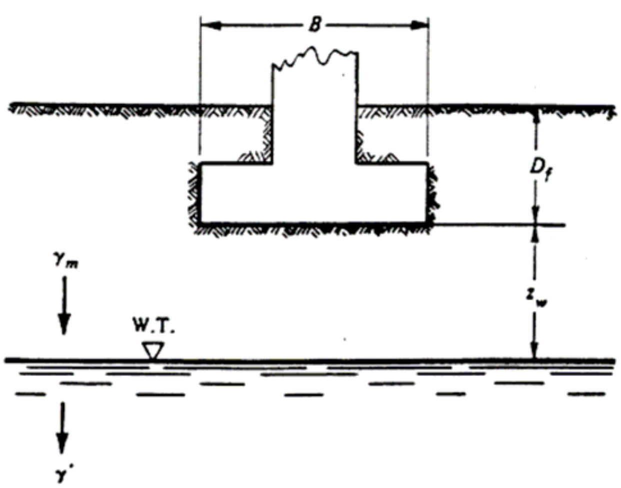
Several factors can affect the bearing capacity of a particular soil. They include soil type, relative density or consolidation, soil saturation and location of the water table and surcharge loads. These factors can act individually or in concert with each other to increase or decrease the bearing capacity 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 $ø$ and γ; 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 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, 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 therefore be neglected. 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. (Refer to Figure 4-6). 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 γ' or γ_{sub} as it is sometime called is determined as follows:

$$\gamma' = \gamma_{sat} - \gamma_w$$

Where: γ' = Submerged unit weight
γ_m = Saturated unit weight (Sometimes shown at γ_{sat})
γ_w = Unit weight of water



for $z_w \geq B$: use $\gamma = \gamma_m$ (no effect)
for $z_w < B$: use $\gamma = \gamma' + (z_w/B)*(\gamma_m - \gamma')$
for $z_w \leq B$: use $\gamma = \gamma'$

FIGURE 4-6 Influence of groundwater table on bearing capacity

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 footing. 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, at any time during construction, must be considered.

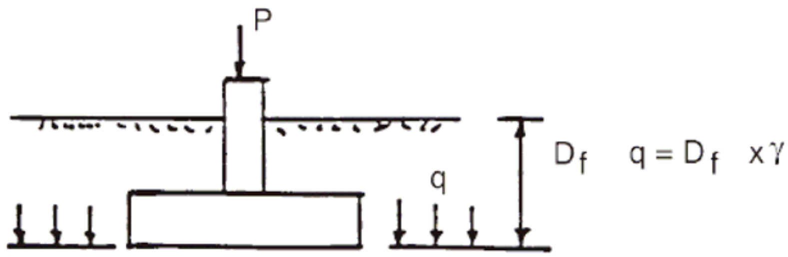


FIGURE 4-7 Surcharge load on soil

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.

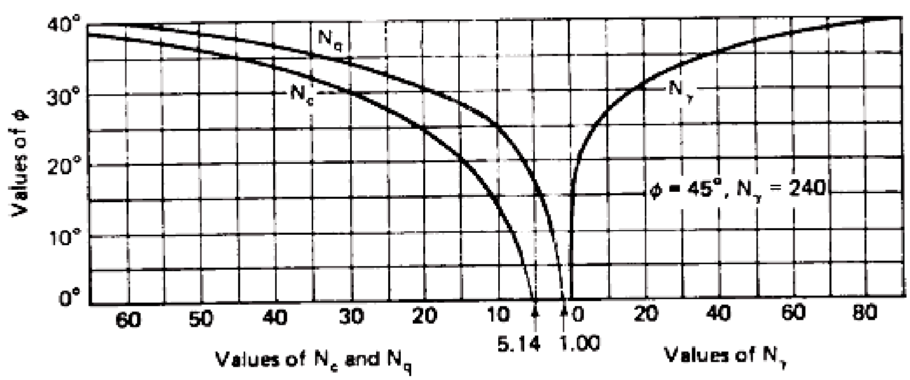


FIGURE 4-8 Relationship between *ø* and bearing capacity factors

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≥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).

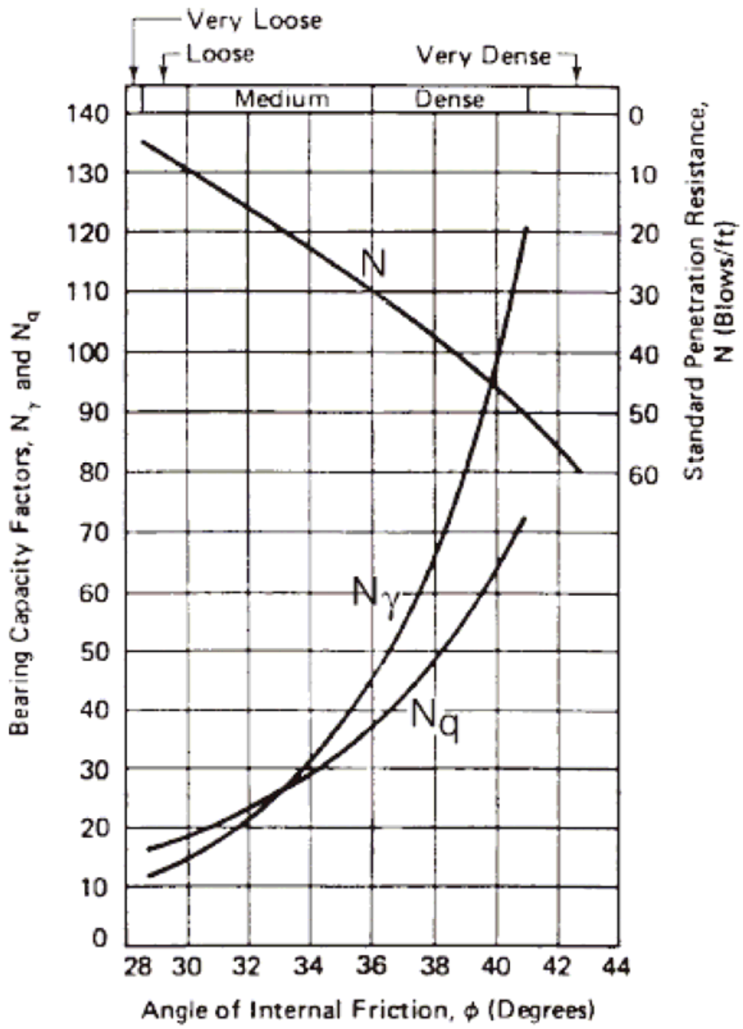


FIGURE 4-9 Relationship of bearing capacity factors to ø and N (standard penetration resistance) for cohesionless soils

The general bearing capacity equation can also 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_γ and N_q, is shown in Figure 4-9. The relationship between N and the angle of internal friction, $ø$, can be also determined from Figure 4-9. When soils are known to have some cohesion, the value of ø determined from Figure 4-9 can then be used in the chart shown in Figure 4-8 to determine the bearing capacity factors, N_γ, N_c, and N_q. Values for $ø$, q_u, N, and γ can be found on the log of test borings or can be approximated by using the tables for granular and cohesive soils shown in Appendix A.

Settlement

Footing foundations will settle over time as the soil densifies from the additional weight it is required to support. The Department's current practice is to limit total permissible settlement for a shallow footing to one inch for multi-span structures with continuous spans or multi-column bents, one inch for single span structures with diaphragm abutments, and two inches for single span structures with seat abutments. To achieve this, allowable bearing pressures are generally 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 in cohesionless materials is for the most part immediate. Cohesive soils, however, consolidate over time as the pressure of the overlying foundation forces water from the soil thereby relieving excess pore water pressures.

Ground Improvement/Soil Modification

Frequently bridges 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 will specify ground modification of the foundation area to "engineer" it for its intended use. Economics, soil type and engineering loads will 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. In general 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 are often time consuming and are often limited to wick drains and settlement periods. As discussed latter on 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 construction of the foundation. The loading typically consists of an embankment constructed to specified limits. Geotechnical Services will determine the need to preload the foundation area, specify the limits of the embankment, and set 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. The hubs should then be monitored (surveyed) and changes to the original elevations recorded. The Engineer is responsible for terminating a settlement period. Data from the hub elevation surveys will be 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 will usually be required when settlement periods greater than 60 days are specified. Geotechnical Services has a Geotechnical Instrumentation Branch that will furnish and provide advice for the installation of the settlement platforms (Refer to BCM 130-13 for additional information and Appendix C for California Test 112 - Method for Installation and Use of Embankment Settlement Devices). Unless this work is outlined in the special provisions, the Engineer will need to write a change order to compensate the Contractor for the initial installation of the settlement platforms.

Construction and Inspection

As discussed in Chapter 3 of this Manual, the Engineer should have a complete understanding of all contract documents as early as practical in the construction process. This will ensure that potential impacts to projects with regard to the foundations are 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 Section 51-1.03 of the Standard Specifications (Refer to Appendix C for sample letter). This reminds the Contractor that footing elevations and seal courses shown on the plans are approximate only and foundation modifications may be required (Bridge Construction Memo 2-9.0).

The Engineer should review and become familiar with the following documents as described in Chapter 3. What follows are particular sections of the Standard Specifications to be considered for footing foundations:

Specification	Issue
Section 19-3.04	Discusses acceptable methods for water control and foundation treatment.
Section 19-3.05	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 approved the foundation.
Section 19-3.07	Discusses measurement of excavation limits and how to address revisions to excavations limits required to meet Design intent.
Section 19-5.03	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.
Section 19-6.01	When bridge footings are constructed in embankment, the embankment shall be constructed to the elevation of the grading plane and the finished slope extended to the grading plane before excavating for the footings.
Section 19-6.025	When a surcharge and settlement period is specified in the Special Provisions, the embankment shall remain in place for the required period before excavating for footings. Also defines the minimum limits of embankment that must be constructed before the settlement period can begin.
Section 51-1.03	Plan footing elevations and seal courses are considered approximate only and the Engineer may order changes in dimensions and/or elevations of footings as may be necessary to obtain a satisfactory foundation. (Bridge Construction Memo 2-9.0).
Section 51-1.04	Pumping of groundwater from foundation enclosures shall be done in such a manner as to prevent removal of any portion of concrete materials. Pumping is not permitted during concrete placement, or for 24 hours thereafter, unless it is done from a suitable sump separated from the concrete work.
Section 51-1.09	After placing, vibrating, and screeding concrete in footings that have both a top mat of rebar and are over 2-1/2 feet deep, the top one foot of concrete shall be reconsolidated as late as the concrete will respond to vibration, but no sooner than 15 minutes after the initial screeding.

Excavations

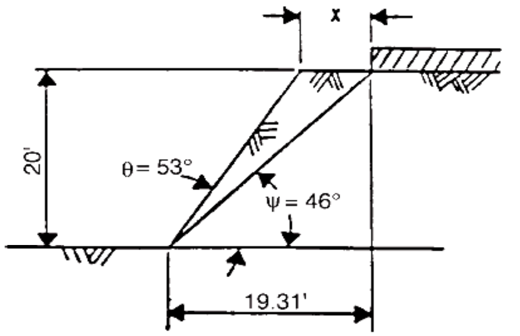
Construction of excavations or trenches is inherent in the construction of foundation elements such as footing foundations. The Caltrans Trenching and Shoring Manual provides information on the complete process for administering, designing and reviewing excavation work and plans. What follows is a brief description of what to consider prior to the start of excavation.

Open Excavations

The open excavation or trench is a potentially dangerous area in 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, or a sloping and benching system. When a sloping and/or 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 website (http://www.dir.ca.gov/samples/search/query.htm) 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.

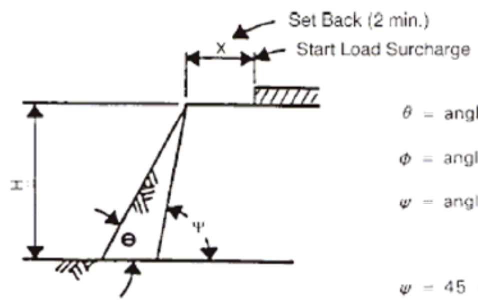
Example:



H = 20'
θ = 53 degrees (3/4:1)
φ = 46 degrees

20'/tan(46) = 19.31'
20'/tan(53) = 15.07'

X = 19.31' – 15.07' = 4.24'

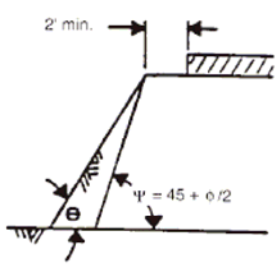


θ = angle of slope

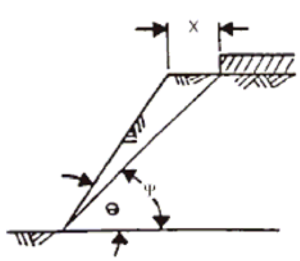
ϕ = angle of internal friction

ψ = angle of the failure plane

$\psi = 45 + (\phi/2)$



If $\theta \leq \psi$ then the surcharge will not affect the stability of the slope, and X may be the OSHA minimum of 2 feet.

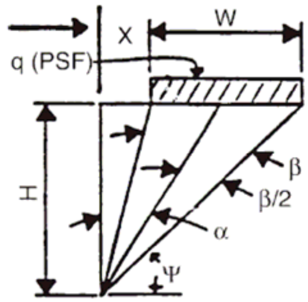


If $\theta > \psi$ then X must be calculated by geometry and $X \geq 2$ feet.

FIGURE 4-10 Slope setback for open excavations/trenches

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 approval. The Trenching and Shoring Manual goes over the procedures for reviewing and approving these plans. An important consideration in shored excavations is the minimum setback for a surcharge when on level ground. In general this setback is equal to the depth of excavation unless specific surcharge loads are considered in the shoring design. The "Bousineaq" 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 Trenching and shoring Manual has several examples how this formula is used and the OSC Website has a spreadsheet that can be used to calculate the pressures.



To calculate lateral pressures due to surcharge, the "Bousineaq" strip load formula is recommended.

At Depth "H",

$$\sigma_h(\mathrm{PSF}) = \frac{2q}{\pi}(\beta_r - \sin\beta\cos 2\alpha)$$

where β_r is in radians

At full height H,

$$\alpha + \frac{\beta}{2} \leq \psi$$

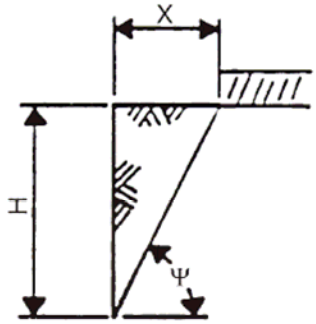
$$\alpha = \arctan\frac{X}{H} + \frac{\beta}{2}$$

$$\beta = \arctan\frac{X+W}{H} - \arctan\frac{X}{H}$$

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

FIGURE 4-11 Effect of surcharge loads for shored excavations

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 approved shoring plans and clearly designated in the field. Refer to the Caltrans Trenching and Shoring Manual for information regarding shoring design and construction.



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, ψ is about 55°

FIGURE 4-12 Setback calculation for shored excavations when surcharges are not considered in the shoring design

Wet Excavations

Section 19-3.04 "Water Control and Foundation Treatment" describes methods to be utilized when water is encountered in excavations and seal courses are not shown on the plans. The means and methods used to control groundwater are at the option of the contractor. These means and methods need to be clearly understood, as there are environmental considerations when dealing with the control of groundwater. The special provisions have sections that address the control and disposal of ground water. All employees of the Office of Structure Construction 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 prior to commencement of work.

Sump pumps are frequently used to remove surface water that enters an excavation and minor infiltrations of groundwater. The sumps and any connecting interceptor ditches should be located well outside the footing area and below the bottom of footing so that the groundwater will not disturb the bearing surface of the foundation.

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 may also lead to settlement of existing structures adjacent to the operations. The amount of soil particles carried away can be determined by periodically collecting discharge 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 are generally 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).

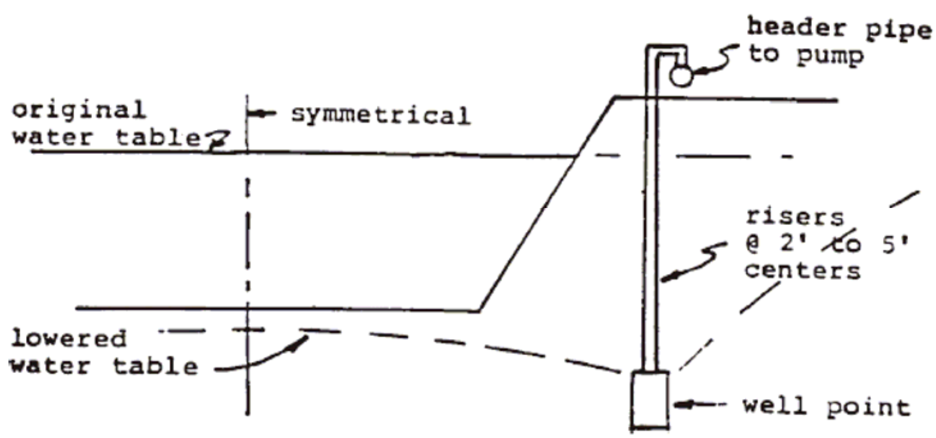


FIGURE 4-13 Single stage well point system

A well point is a section of perforated pipe about 2 to 3 inches in diameter and 2 to 4 feet in length. The 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 perimeter of the excavation, 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. The 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, it is cautioned that lowering the water table by pumping from underlying layers of sand may not be a preferred option as it will cause large progressive settlement of the clay strata in the surrounding area. By lowering the watertable in the sand lens the condition in the clay lens switches from an undrained 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 watertable 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).

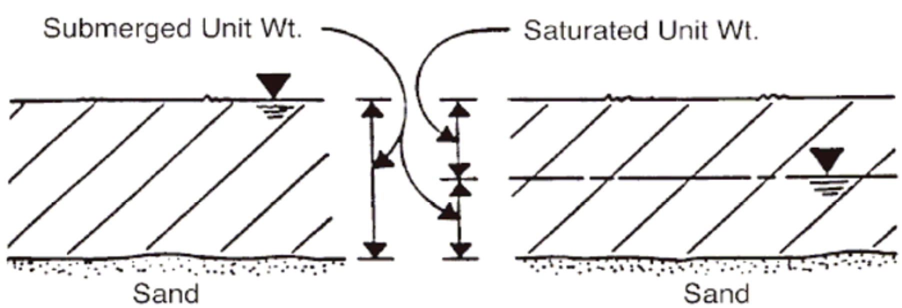


FIGURE 4-14 Saturated vs. submerged unit weight

Bottom of Excavation Stability

The control of groundwater can be 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 must also consider soil heave and piping as they relate to the stability of the bottom of the excavation.

Heave is the phenomena 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 a 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, γz (Figure 4-15).

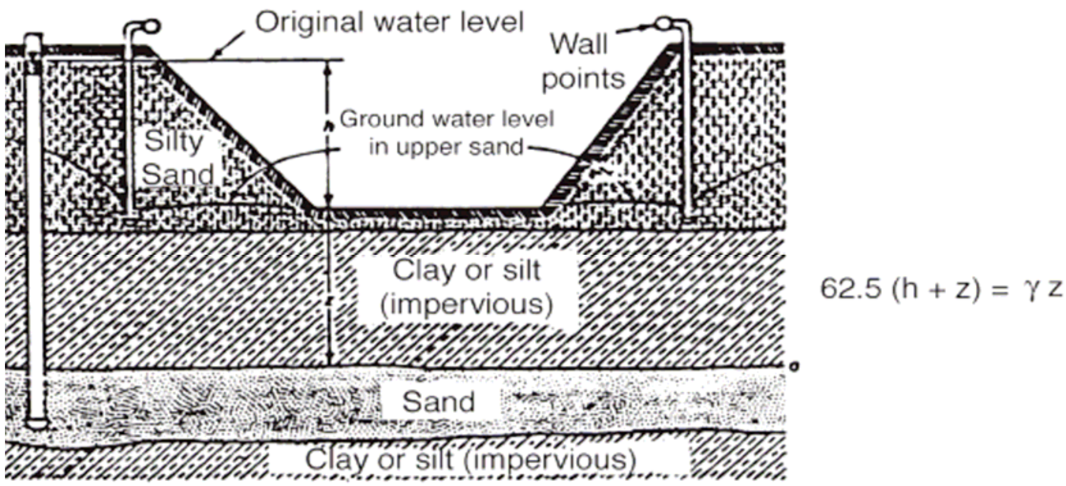


FIGURE 4-15 Bottom of excavation stability problems due to excess hydrostatic head against an impervious layer

Piping is generally 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.

Foundation Inspection & Construction Considerations

Inspection should include determination of the following:

1) Stability of slopes and sides of excavations conform to Cal-OSHA requirements.

2) Verification that the foundation material conforms to the information shown on the Log of Test Borings (allowance should be made for some non-uniformity such as small pockets and lenses of material having somewhat different properties).

3) Condition of the foundation-bearing surface (undisturbed by excavation operations and uncontaminated by sloughing and/or entrance of water).

4) Proximity of structures, highways, railroads, and other facilities that may require shoring or underpinning. (Should be done prior to excavation)

5) Foundation element forms conform to layout, depth, dimensions, and construction grade shown on the plans. Forms are mortar tight.

6) Reinforcing steel is firmly and securely tied in place, shear steel hooked to both top and bottom rebar mats and securely tied. Proper concrete cover over top rebar mat.

7) During concrete placing operations ensure that the concrete has the proper mix number, truck revolutions, concrete temperature and back-up alarm. Wet down rebar and forms, do not allow concrete to drop over 8 feet. Reconsolidate and finish top one foot of concrete no sooner than 15 minutes after initial screeding, then cure.

8) A bench of sufficient width to prevent sloughing or cave-in should be provided around the excavation for access and work area.

The footing forms are either built out of timber or consist of prefabricated panels. The forms are generally 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 footing elevation, a pour strip or similar device must be attached to the forms to designate the top of footing elevation.

The footings for shored excavations are often excavated and placed/poured “neat” which means that the excavation limits are essentially the footing limits. The concrete is placed against the sides of the excavation thereby 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. Ensure that “neat” excavations conform to the planned footing dimensions. If they vary, then place the exact, as-constructed footing dimensions 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 betaken to make sure that the footing concrete isn’t damaged during shoring removal operations.

Whether footings are formed or excavated “neat”, a template should be constructed to ensure that the positioning 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 should not be hung or suspended from the formwork or templates as the weight of the suspended rebar can cause settlement in the form panels affecting pour grades and displace during concrete placement. Top reinforcing steel mats supported should be blocked to the forms or sides of the excavation. The bottom reinforcing steel mat that supports the vertical column steel should be adequately blocked to prevent any settlement. In addition, reinforcing steel dowels are required to be tied in place prior to concrete placement and not “stabbed in” during or after concrete placement.

The effective depth of reinforcing steel is critical and must always 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 should 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 desirable 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. Verify that settlement of the rebar cage hasn’t occurred by re-inspecting minimum clearances between the bottom of the excavation and the bottom reinforcing steel mat. 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.

Foundation Problems and Solutions

Inspection of the excavated surface at the planned footing elevation after the excavation is completed is mandatory (Section 19-3.05 of the Standard Specifications requires the Contractor to notify the Engineer after the excavation is completed). A thorough physical inspection of the foundation material by the Engineer is required 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 the Department. The phrase “contaminated material” as used here should not be confused with materials contaminated with lead, hydrocarbons, heavy metals, etc.

Information on environmentally contaminated materials will be addressed in the contract plans and special provisions.

Disturbed and/or Contaminated Material

Disturbed 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 is usually caused by the excavation means and methods. It may include excavating below the footing elevation or disturbing the grade with the teeth on the excavator bucket. Contamination is usually 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 shall 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 in order to avoid or minimize the disturbance and/or contamination of the foundation surface:

1) Under-excavate with mechanical equipment and excavate to bottom of footing by hand or by using a cleanup bucket.

2) Divert surface water away from the excavation.

3) Minimize exposure of the foundation material to the elements by constructing footings as soon as possible after excavation.

Unsuitable Foundation Material

The importance of suitable foundation material cannot be overstated. The Engineer is responsible for determining the suitability of the foundation 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:

1. Penetration tests - granular soils

2. Finger tests - cohesive soils

3. Pocket penetrometer - cohesive soils

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

The Log of Test Borings 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. Contact Geotechnical Services and the project engineer and discuss the questionable material, related concerns and possible resolutions.

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. Corrective action required to address disturbed or contaminated material is the responsibility of the Contractor and addressing unsuitable material is the responsibility of the Engineer. The corrective actions are similar in either situation. They fall into two categories: replacement of the original foundation material to achieve the original bottom of footing elevation and revisions to the structure to address a different bottom of footing elevation. There are engineering/design considerations in either of these paths and it is important to discuss considerations and consequences with the project engineer, Geotechnical Services and the Contractor and work toward a solution that fulfills the design intent and keeps the job moving.

Options that can be used to restore the foundation material at the bottom of footing elevation to its specified elevation after removal of unsuitable or contaminated material are as follows:

1. 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.

2. 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 the structure to address a different bottom of footing elevation or a lower than anticipated bearing capacity should be discussed with the project engineer. The revisions may require a redesign of the structure and are not minor in nature. These options, while possible, may not be the best alternatives in real construction situations. They are as follows:

1. Maintain top of footing as planned and overform footing depth. The rebar cage will remain at the theoretical elevation shown on the 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.

2. 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 project engineer.

3. 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 must also be considered, as it cannot exceed tolerable limits. This decision should be discussed with the project engineer 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 must also 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 the specified method of payment for the following revisions (Standard Specifications - Section 19-3.07):

1. Raising the bottom of a spread footing above the elevation shown on the plans.

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

For other revisions, agreed price or force account methods should be used when the Engineer determines that the above method is unsatisfactory or doesn’t address changes to the character of the work as a result of the changes.

Safety

As stated previously excavations are a potentially dangerous construction activity. Cal-OSHA has requirements that must be followed prior to the start of 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 Cal-OSHA excavation permit.
- Identify a “competent person” responsible for the excavations.
- Provide an excavation plan to the Engineer for review and approval prior to starting excavation. (Section 5-1.02A “Excavation Safety Plans” of the Standard Specifications)
- Provide an engineered system stamped by an engineer registered in the State of California for any engineered shoring system.
- An engineer registered in California must stamp any sloping or benching system that is greater than twenty feet.

Once approved, the excavation needs to be inspected to ensure compliance with the approved plan and Cal-OSHA requirements. Daily inspections (prior to start of shift and after any hazard-increasing occurrence such as rain) of excavations or protective systems shall 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 shall 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 away from the excavation lip usually provides more stable ground to anchor the rail posts. Spoil piles must be located more than 2 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.

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 less than 5 feet in depth.

Excavations can be considered confined spaces, as they are prone to hazardous atmospheres with limited access and egress. Cal-OSHA 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 shall 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:

1) 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 Cal-OSHA approved protective covers, troughs, or caps. Approved manufactured covers, troughs, or caps will have the manufacturer's name, model number, and the Cal-OSHA approval number embossed or stenciled on the cover, trough, or cap. Any manufactured protective device not so identified is not legal.

2) When work is proceeding above any surface of protruding rebar, impalement protection shall be provided by the use of: (1) guardrails, (2) an approved fall protection system, or (3) approved protective covers or troughs. Caps are prohibited for use as impalement protection for workers working above a level of 7 ½ feet above the protruding rebar.

Protective covers used for the protection of employees working above grade shall have a minimum 4 x 4 inch square surface area or 4 ½ inches in diameter if round. Protective covers or troughs may be job-built, provided they are designed to Cal-OSHA 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 approved design is on file in the job records prior to their use.

CHAPTER

5 Pile Foundations - General

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 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. Whereas a replacement pile is a pile that is placed or constructed within a previously drilled borehole and replaces the excavated soil. Displacement, or driven, piles are discussed in Chapter 7 of this Manual while Chapter 6 discusses replacement, or cast-in-place, piles.

Driven piles are braced, structural columns that are driven, pushed or otherwise forced into 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.

Specifications

The specifications for piling are contained in Section 49 of the Standard Specifications. Project specific requirements and revisions to the Standard Specifications are included in the contract special provisions. The project 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 special provisions provide 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 project engineer may provide the option of using either steel "H" piling or precast concrete piles. When this option is written into the contract, other conditional clauses are usually provided (no additional compensation for piling driven below specified tip, etc.) and 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 project engineer.

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 Project Engineer 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 project plans. (Note: The names of the different classes of Standard Plan piles were revised in the 2006 version of the Standard Plans)

The Standard Specifications contain the general information for pile work. This includes specifics for types of materials to be used, methods of construction, measurement, payment, etc. It is important to 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.

Cast-in-Place Piles

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

- Steel shells driven permanently to the required nominal resistance and penetration and filled with concrete.
- Steel casings installed permanently to the required penetration and filled with concrete.

- Drilled holes filled with concrete.
- 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 assumed to have no structural value and are only used to facilitate construction. The third type is typically known as a Cast-in-Drilled-Hole (CIDH) Pile. The last type is essentially a CIDH pile drilled in rock. Sometimes combinations of two or more type of cast-in-place piles are used to construct a single pile. This can happen when soft materials such as clays overlay rock formations.

Cast-In-Drilled-Hole (CIDH) piles are made of reinforced concrete that is 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 a pile driving operations might damage adjacent structures such as pipelines, buildings, etc. 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 ground water should be present.

If there are concerns about the presence of ground water, the slurry displacement method specifications may need to be incorporated into the contract. CIDH piles are discussed in more detail in Chapters 6 and 9 of this manual. Special consideration piles such as those for changeable message signs (CMS) are discussed in Chapter 13.

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 on the following page. Driven piles are discussed in more detail in Chapter 7 of this manual.

TYPE OF PILE	DESCRIPTION
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. They can be either conventionally reinforced or prestressed (most common). They can also be either precast (most common) or they can be cast in driven steel shells. The types of steel shells vary 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.

	Caltrans has standard details for splicing precast concrete piles but it is a difficult, time consuming, 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 “H” piles and pipe piles (empty or concrete filled). The pipe section is a standard alternate for the Class 45 and 70 piling, but is seldom used. Although steel piling is relatively expensive on a “per foot” furnish basis, it has a number of advantages. The steel piles come in sizes varying from HP 8x36 to HP 14x117 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. Splice details are shown on the Standard Plans or project plans for contracts that permit the use of steel piling. Pile welding work requires the submittal and approval of a Welding Quality Control Plan. The requirements for the Welding Quality Control Plan are addressed in the contract special provisions Sometimes “H” piles must be driven below the specified tip elevation before minimum bearing is attained. This can present an administrative problem (cost) if the length driven below the specified tip elevation is significant. Steel lugs welded to the piles are commonly used to solve this problem. This subject is covered in detail in Bridge Construction Memo 130-5.0.
Driven Piles – Wood	Untreated timber piles may be used for temporary construction, revetments, fenders and similar work; and in permanent construction where the cutoff elevation of the pile is below the permanent ground water table and where the piles are not exposed to marine borers. 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 45 Tons, whereas most structure piles are designed for at least 70 Tons.

Alternative Piles

Currently there are several alternative piles that have been approved for use by the Department. They are used on a site-specific basis. There are three (3) types of Micro-piles (DBM, Malcolm and Nicholson). The Tubex Grout Injection Pile is another alternative pile system. These systems have generalized drawings and have been successfully system tested by the Department. The GeoGrout Foundation System has pile system load test results on file with the Department but no generalized drawings. Refer to Chapter 13 and Appendix D for information, drawings and schematics of the various alternative piles.

CHAPTER

6 Cast-In-Place Piles

Description

Few terms are as self-descriptive as the one given 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 their 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 in two categories: the first is CIDH piling without inspection pipes (dry method), and the second is CIDH piles with inspection pipes (wet method). This chapter is applicable for both the dry and wet method of CIDH pile construction. Chapter 9 of this manual provides supplemental information on the wet method of CIDH pile construction. Note that piling dewatered with the help of a temporary casing requires inspection pipes even if the piling is poured dry.

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 when concrete is placed. Because of cave-in 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 unless 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 of this manual.

A dry hole, by definition, typically contains no standing groundwater within the drilled hole, although the material at the bottom of the drilled hole may be damp or wet. However, the dry method of construction may still be used when a small amount of groundwater is present in the drilled hole. Refer to Bridge Construction Memo 130-7.0 for information on the specific definition of a "dry" drilled hole.

Specifications

The Standard Specifications describe four different types of cast-in-place pile. The first type is the cast-in-drilled-hole pile, which is described further in this chapter. The second type is the cast-in-driven-steel-shell pile. For this type of pile, a steel shell is driven to a specified tip elevation and bearing value. The ground material within the steel shell is then removed and the steel shell is filled with reinforced or non-reinforced concrete. Refer to Chapter 7 of this Manual for additional information on driven piles. The third type of pile is concrete cast within a permanently installed steel casing. For this type of pile, a steel casing is installed to a specified tip elevation using any approved means; the soil inside the casing is removed by drilling and then filled with reinforced concrete. The fourth type is a rock socket filled with concrete; which is similar to a cast-in-drilled-hole pile, but placed in rock and usually below a permanently installed steel casing that has had the rock removed and ultimately filled with reinforced concrete.

The Standard Specifications contain much of the information necessary to administer the construction of CIDH piles. Section 49-4 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 job-specific requirements and revised 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 any revised specifications noted should be discussed with the Contractor.

Drilling Equipment

The drilling auger is the most commonly used drilling tool for drilling holes for CIDH piles. Augers may be used in granular and cohesive materials.

There are two basic varieties of augers—the standard short section (Figure 6-1) and continuous flight. Both have flights of varying diameter and pitch. 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 predrilling driven piles. They may also be used where overhead clearance is not a problem.

FIGURE 6-1
Auger – short section

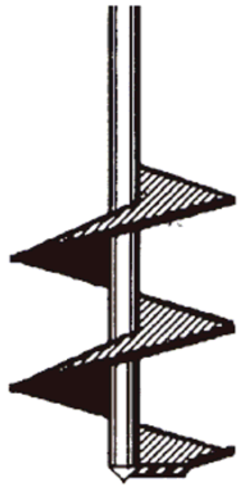


FIGURE 6-2
Auger - single flight

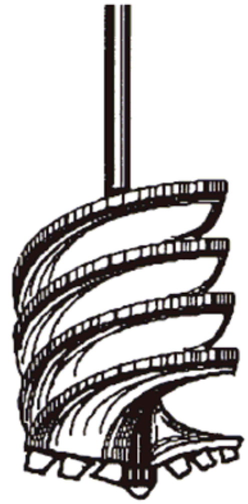


FIGURE 6-3
Auger – double flight

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.

There are a variety of different auger shapes/styles that may be used in different situations. Augers may be single flight (Figure 6-2) or double flight (Figure 6-3). 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.

Rock augers 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.

FIGURE 6-4
Drilling bucket

FIGURE 6-5
Drilling bucket/cleanout bucket comparison

Drilling buckets (Figure 6-4) 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.

Cleanout buckets 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. Figure 6-5 shows the difference between the cleanout bucket and the drilling 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 Core barrel

Core barrels (Figure 6-6) 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.

Down-hole hammers (Figure 6-7) 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 6-7 Down-hole hammer

Rotators (Figure 6-8) and oscillators (Figure 6-9) 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.

FIGURE 6-8 Rotator

FIGURE 6-9 Oscillator

Reverse circulation drilling equipment (Figure 6-10) is used to advance a drilled hole through difficult wet ground formations. 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 recirculated back into the drilled hole to repeat the process.

FIGURE 6-10 Reverse circulation drilling equipment

Temporary steel casings (Figure 6-11) 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.

FIGURE 6-11 Steel casing

Drilling is performed almost exclusively with portable drilling rigs. These units can be self-propelled (Figure 6-12), truck-mounted (Figure 6-13), or crane-mounted (Figure 6-14).

FIGURE 6-12 Drill rig – crawler mounted

FIGURE 6-14
Drill rig – crane mounted

FIGURE 6-13
Drill rig – truck mounted

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.

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.

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

ITEM	ACTION
1	Placement of a low cement/sand mix and redrilling the area of the cave-in.
2	If permitted by the contract special provisions, use a drilling slurry (refer to Chapter 9 of this manual).
3	Use of a casing, which is pulled when placing concrete.

In the case of groundwater, the following action or combination thereof may be required:

ITEM	ACTION
1	Placement of a low cement/sand mix and redrilling the hole.
2	Drilling to tip elevation, using a pump to remove the water and cleaning out the bottom of the pile. (See BCM 130-7.0 for information on the specific definition of a "dry" drilled hole)
3	If permitted by the contract special provisions, use drilling slurry (refer to Chapter 9 of this manual).
4	Placement of a casing, again using a pump to remove the water, and pulling casing during concrete placement (keeping bottom of casing below the concrete surface). See BCM 130-7.0 for information on the specific definition of a "dry" drilled hole
5	Dewatering the entire area using well points, deep wells, etc. This should be thoroughly discussed with the Bridge Construction Engineer and the project geoprofessional.
6	By contract change order, substitute an alternative type of piling. Again, this should be discussed with the project designer, the project geoprofessional, and the Bridge Construction Engineer.

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 Project engineer, Geoprofessional, Resident Engineer and the Bridge Construction Engineer.

Under certain conditions, the Standard 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 project designer and project 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.

Inspection and Contract Administration

Before drilling begins, the Engineer should have a pre-construction meeting with the Contractor and any subcontractors that will be involved in the work. 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.

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 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 project engineer should be contacted for clarification.

CIDH piles are designed to resist compressive loads, tensile loads, and lateral loads. Most CIDH piles are designed to resist these loads using skin friction, with minimal or no contribution from end bearing. The project engineer should be contacted to determine the manner in which the pile was designed to transfer load.

The specifications require the Contractor to submit a Pile Placement Plan to the Engineer for review and approval. The Pile Placement 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 placement. Typical requirements for all CIDH piling include the following:

ITEM	PILE PLACEMENT PLAN REQUIREMENT & REASONING
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	Plan view drawings of pile showing reinforcement and inspection pipes, if required. ***Reasoning:*** *This gives the Engineer advance knowledge of how the Contractor plans to install the inspection pipes within*

ITEM	PILE PLACEMENT PLAN REQUIREMENT & REASONING
	the pile bar reinforcement cage and whether the proposed method of installation is appropriate. Inspection pipes are required when the Contractor proposes to use casing primarily to control ground water.
5	Methods for placing, positioning, and supporting bar reinforcement. ***Reasoning:*** *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.*
6	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.*
7	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.*
8	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.*

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 project engineer 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 means as described in the Pile Placement 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. The drilled hole should be checked 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 project 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 shall be met before anyone enters a drilled hole. Note that 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.

Pile bar reinforcement cage clearances and blocking should be checked 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 Placement Plan and some means must be devised to ensure concrete placement to the proper pile cutoff elevation.

Immediately before placing concrete, the bottom of the drilled hole should be checked 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. Its 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. Refer to BCM 130-7.0 for information on the specific definition of a "dry" drilled hole.

Concrete placement warrants continuous inspection. Those involved in the work should thoroughly review Standard Specifications Sections 49-4 and the contract special provisions. Applicable portions of Section 90 should also be reviewed with respect to concrete mix design, consistency of the concrete mix, and concrete curing requirements.

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.

The following drilling problems can cause pile defects:

ITEM	DRILLING PROBLEM/RESULTING PILE DEFECT
1	A cleanout bucket is not used to clean up the bottom of the drilled hole ***Result:*** *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 6-15.*
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 6-15.*
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 6-16.*

These problems are preventable. Adherence to the contract specifications and timely inspection will ensure the best quality pile and mitigate most of these problems.

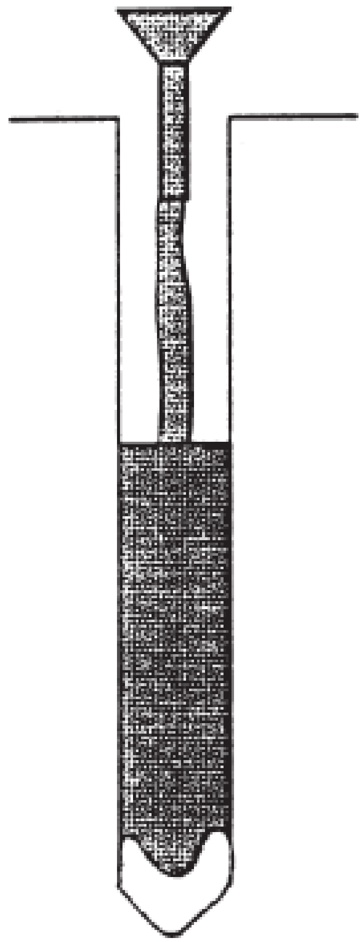


FIGURE 6-15
Pile defects - no cleanout, tapered bottom of hole

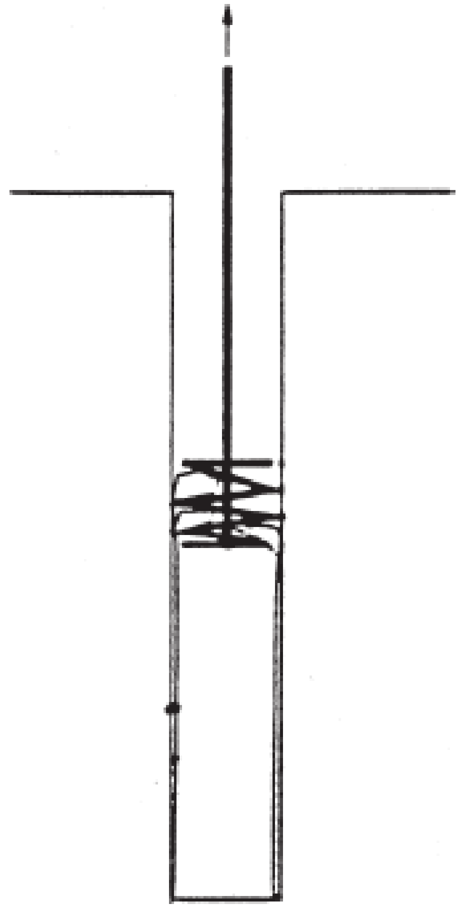


FIGURE 6-16
Pile defects - smeared drill cuttings

The following concrete placement problems can cause pile defects:

ITEM	PLACEMENT PROBLEM/RESULTINGPILE DEFECT
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 6-17.*
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*

ITEM	PLACEMENT PROBLEM/RESULTINGPILE DEFECT
	pile. This defect is shown in Figure 6-18.
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 6-19.*
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 6-20.*

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.

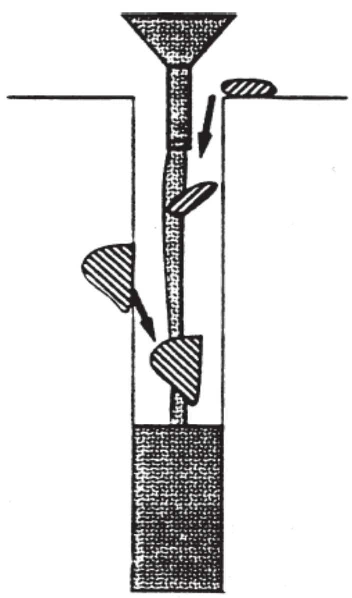


FIGURE 6-17
Pile defects - cave in

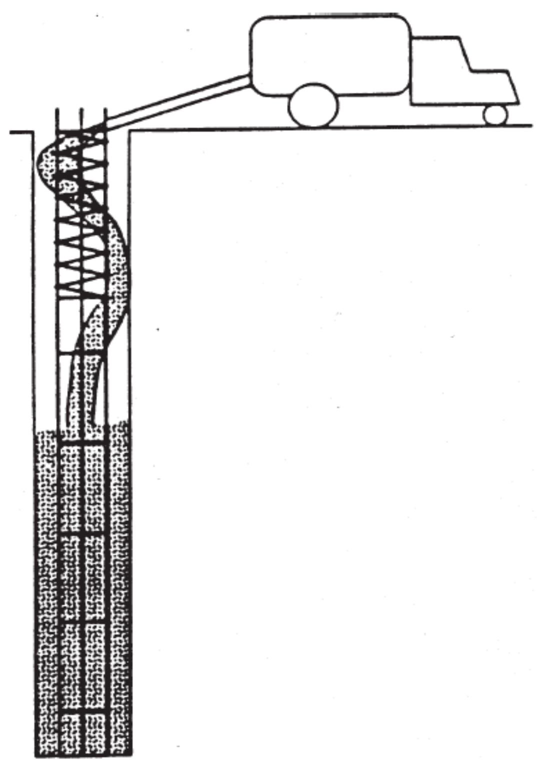


FIGURE 6-18
Pile defects - concrete segregation

The following casing removal problems can cause pile defects:

ITEM	CASING REMOVAL PROBLEM/PILE DEFECT
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 6-21(a), (2) the concrete sets and the casing cannot be removed as shown in Figure 6-21(b), and (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 6-21(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.

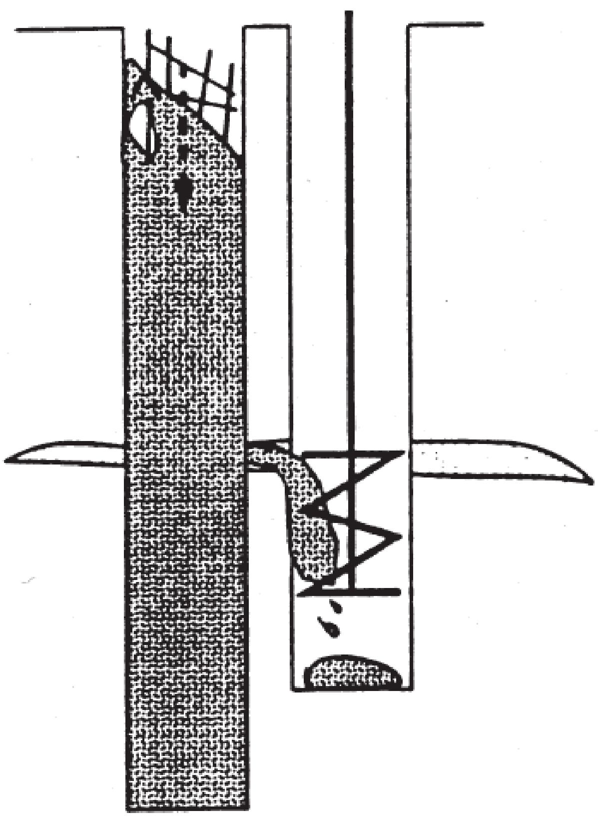


FIGURE 6-19
Pile defects - adjacent hole blowout

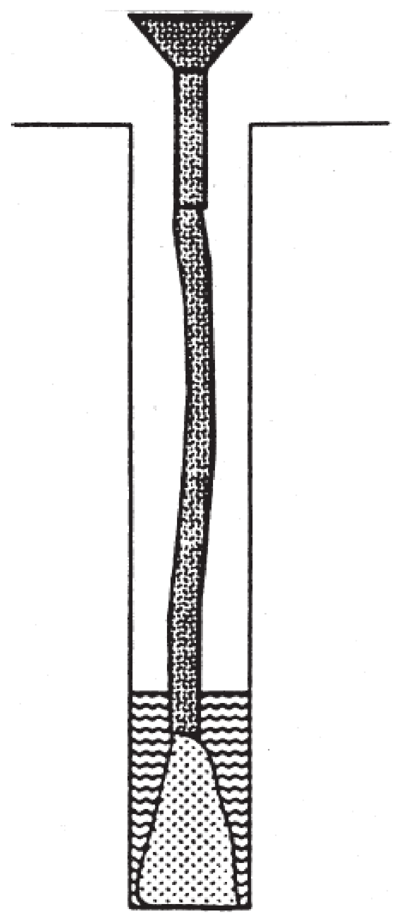
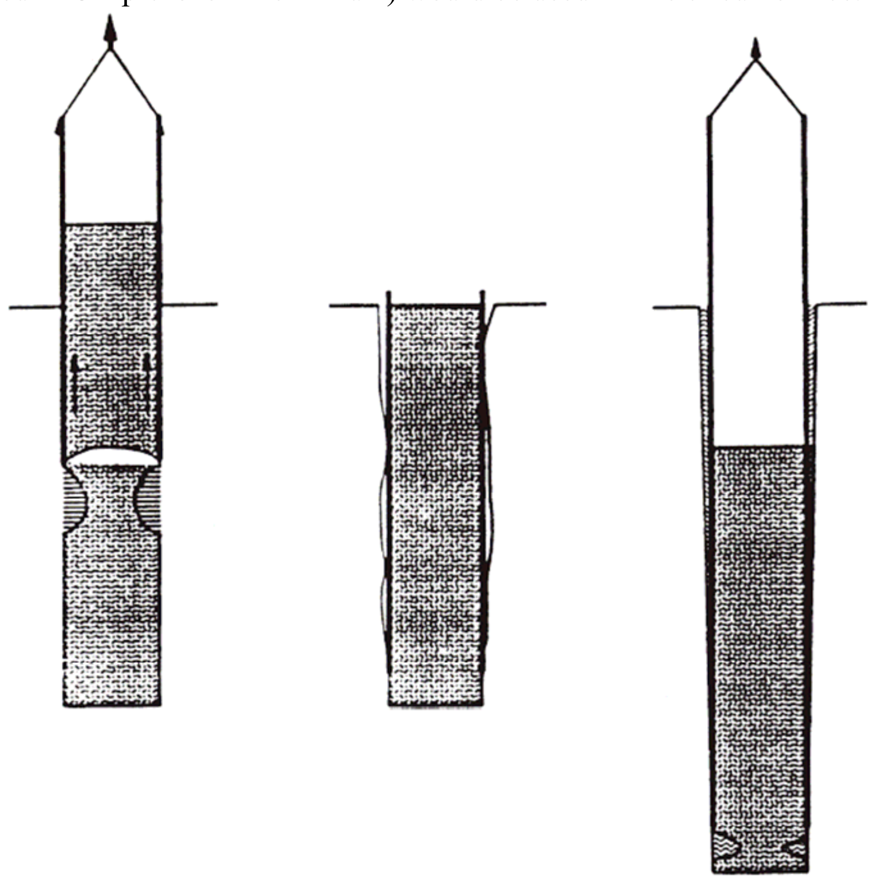


FIGURE 6-20
Pile defects - water in the hole

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. It is recommended the penetration value 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 of this manual) would be used in this circumstance.



(a) **(b)** **(c)**

FIGURE 6-21 Pile defects casing problems

Safety

As with all construction activities, the Engineer should be aware of safety considerations associated with the operation. As a minimum, the Engineer shall 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, 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, common sense safety practices should be used 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, the crane certificate should be checked.

In addition, footing excavations should be properly sloped or shored as discussed in Chapter 4 of this Manual. 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 Trenching and Shoring Manual

Worker and public safety must be enforced during drilling and excavating operations. A full body harness should 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.

CHAPTER

7 Driven Piles

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. Steel H-Piles and pipe piles are also used. These piles are 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 a hammer.

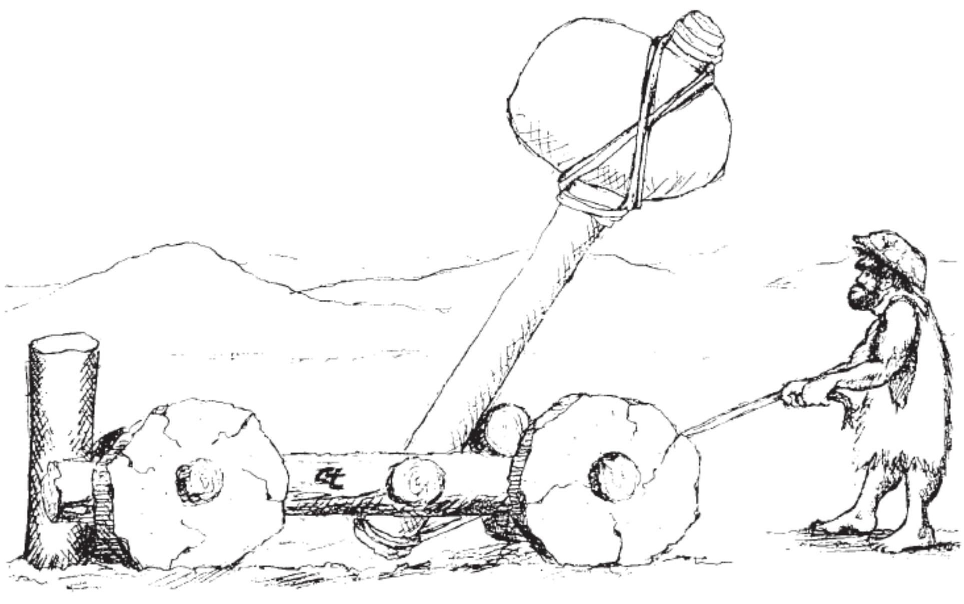


FIGURE 7-1 Early pile hammer

The first hammers were drop hammers and they were used exclusively until the invention of the steam engine, which eventually resulted in steam hammers. Subsequent technological advances have lead to the development of air, diesel and 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.

The remainder of 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.

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 to be reviewed are as follows:

• Section 19: Earthwork

• Section 49: Piling

• Section 58: Preservative Treatment of Lumber, Timber and Piling

The following are taken from the May 2006 Standard Specifications and should be reviewed as applicable:

Section 19-6.01:

- Rocks, broken concrete or other solid materials larger than 0.33 foot are not allowed in fill where piles are to be driven.

- When bridge footings are to be constructed in embankment, the embankment shall be constructed to the elevations of the grading plane and the finished slope extended to the grading plane before driving piles or excavating for the footing.

Section 19-6.025:

- Where an embankment settlement period is provided for in the special provisions, the embankment shall remain in place for the required settlement period before excavating for the abutment, wingwall, or retaining wall footings or driving foundation piles at each location.

Section 49-1.03:

- Foundation piles of any material shall be of such length as is required to obtain the specified penetration, and to extend into the cap or footing block as shown on the plans or specified in the special provisions.

- For driven piling, the Contractor shall furnish piling of sufficient length to obtain the specified tip elevation shown on the plans or specified in the special provisions.

Section 49-1.05:

- Driven piles shall be installed with impact hammers that are approved in writing by the Engineer. Impact hammers shall be steam, hydraulic, air or diesel hammers. Impact hammers shall develop sufficient energy to drive the piles at a penetration rate of not less than 1/8 inch per blow at the specified nominal resistance.

- Steam or air hammers shall be furnished with a boiler or air capacity that is at least equal to that specified by the manufacturer of the hammer to be used. The boiler or compressor shall be equipped with an accurate pressure gauge at all times. The valve mechanism and other parts of steam, air, or diesel hammers shall be maintained in first class condition so that the length of stroke and number of blows per minute for which the hammer is designed will be obtained. Inefficient steam, air, or diesel hammers shall not be used.

Section 49-1.06:

- Piles, to be driven through embankments constructed by the Contractor, shall be driven in holes drilled or spudded through the embankment when the depth of new embankment is in excess of 5 feet. The hole shall have a diameter of not less than the greatest dimension of the pile cross-section plus 6 inches. After driving the pile, the space around the pile shall be filled to ground surface with dry sand or pea gravel. (This is to prevent frictional down drag on the piles due to differential settlement between the new embankment and original ground and to ensure that the pile path is free from large diameter embankment material obstructions).

Section 49-1.08

- Except for piles to be load tested, driven piles shall be driven a value of not less than the nominal resistance shown on the plans unless otherwise specified in the special provisions or otherwise permitted in writing by the Engineer. In addition, when a pile tip elevation is specified, driven piles shall penetrate at least to the specified tip elevation unless otherwise permitted in writing by the Engineer.

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.

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 location of the defined terms.

TERM	DEFINITION
Anvil	The bottom part of a hammer that receives the impact of the ram and transmits the energy to the pile.
Butt of Pile	The term commonly used in conjunction with the timber piles—the upper or larger end of the pile, the end closest to the hammer.
Cushion Blocks	Usually plywood pads placed on top of precast concrete piles to eliminate spalling.
Cushion Pad	A pad of resilient material or hardwood placed between the drive cap insert, or helmet, and drive cap adapter.
Drive Cap Adapter	A steel unit designed to connect specific type of pile to a specific hammer. It is usually connected to the hammer by steel cables.
Drive Cap Insert	The unit that fits over the top of pile, holding it in line and connecting it to the adapter.
Drive Cap System	The assembled components used to connect and transfer the energy from the hammer to the pile.
Follower	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 percentage of the energy of the hammer, the Standard Specifications (Section 49.1.05) require the first pile in any location be driven without the use of a follower so as to be able to make comparisons with operations utilizing a follower. In water, the first pile to be driven should be one sufficiently long to negate the need for the follower. The information from the first pile can be used as base information when using the follower on the rest of the piling. Beware of soil strata that may change throughout the length of a footing. Underwater hammers and extensions to the leads can be used as alternatives to driving with a follower

TERM	DEFINITION
Hammer Energy	The amount of energy available to be transmitted from the hammer to the pile. Usually measured in foot-pounds.
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.
Mandrel	A full-length steel core set inside a thin-shell casing. It increases the capacity of the casing so that it can be driven. It helps maintain pile alignment and prevents the casing from collapsing. It is removed after driving is completed and prior to placing reinforced concrete.
Moonbeam	A device attached to the end of a lead brace that allows a pile to be driven with a side batter.
Penetration	The downward movement of the pile per blow.
Pile Butt	A member of the pile crew other than the operator and oiler.
Pile Gate	A hinged section attached to the pile leads, at the lower end, which acts to keep the pile within the framework of the pile leads.
Pile Hammer	The unit that develops the energy used to drive piles, the two main parts of which are the ram and the anvil.
Pile Rig	The crane used to support the leads and pile driving assembly during the driving operation.
Ram	The moving parts of the pile hammer, consisting of a piston and a driving head, or driving head only.
Rated Speed	The number of blows per minute of the hammer when operating at a particular maximum efficiency.
Spudding	Spudding is the driving of a short and stout section of pile-like material into the ground to punch through or break up hard ground strata to permit pile driving. Used extensively in the driving of timber piles.
Striker Plate	A steel plate placed immediately below the anvil. Also known as an anvil.
Stroke	The length of fall of the ram.
Tip of Pile	The first part of the pile to enter the ground.

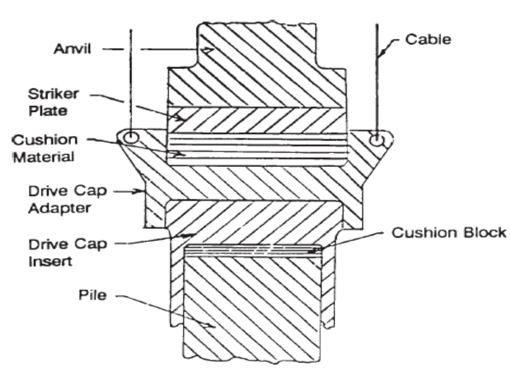


FIGURE 7-2 Drive cap system

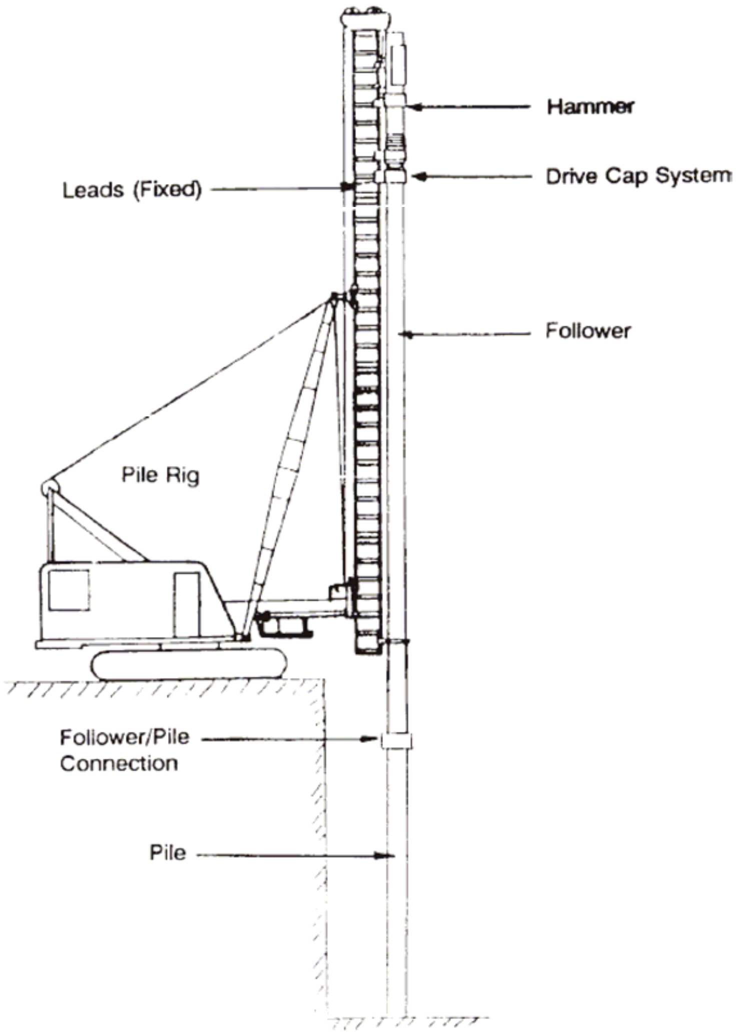
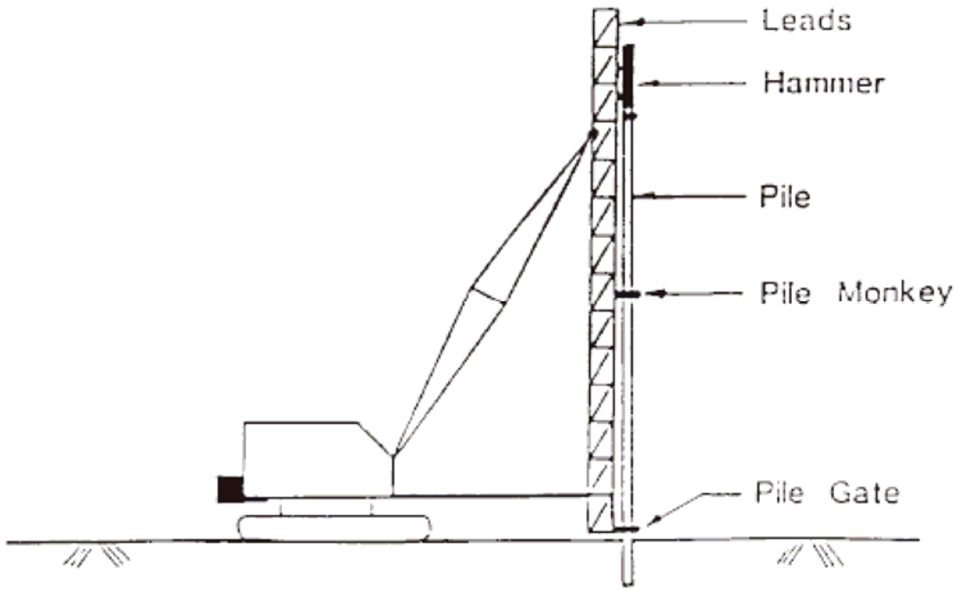


FIGURE 7-3 Typical pile rig configuration



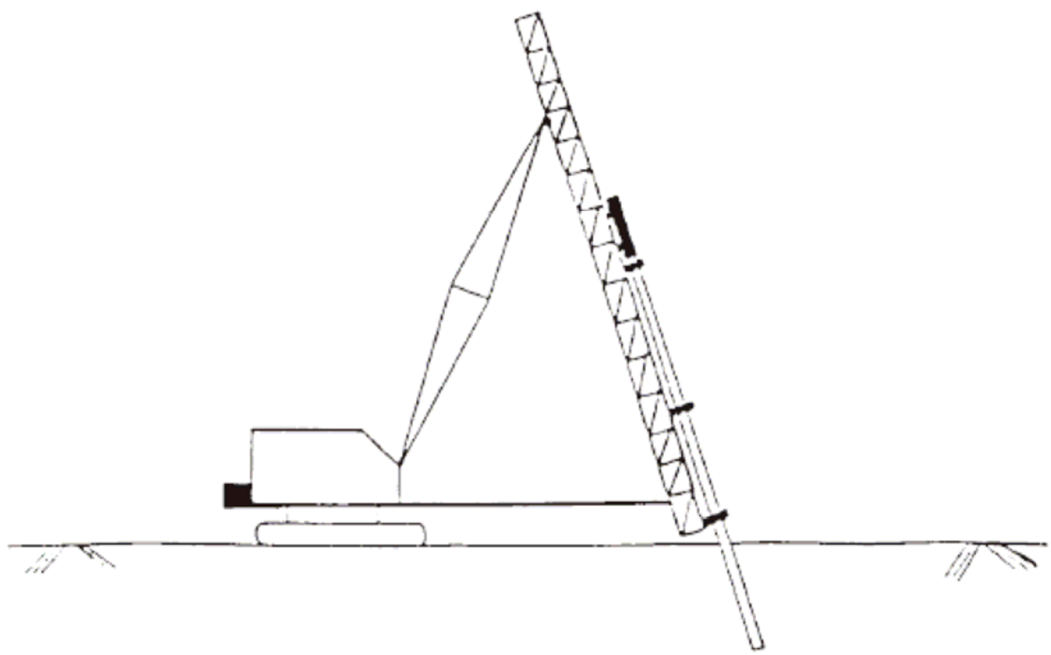


FIGURE 7-4 Fixed lead system

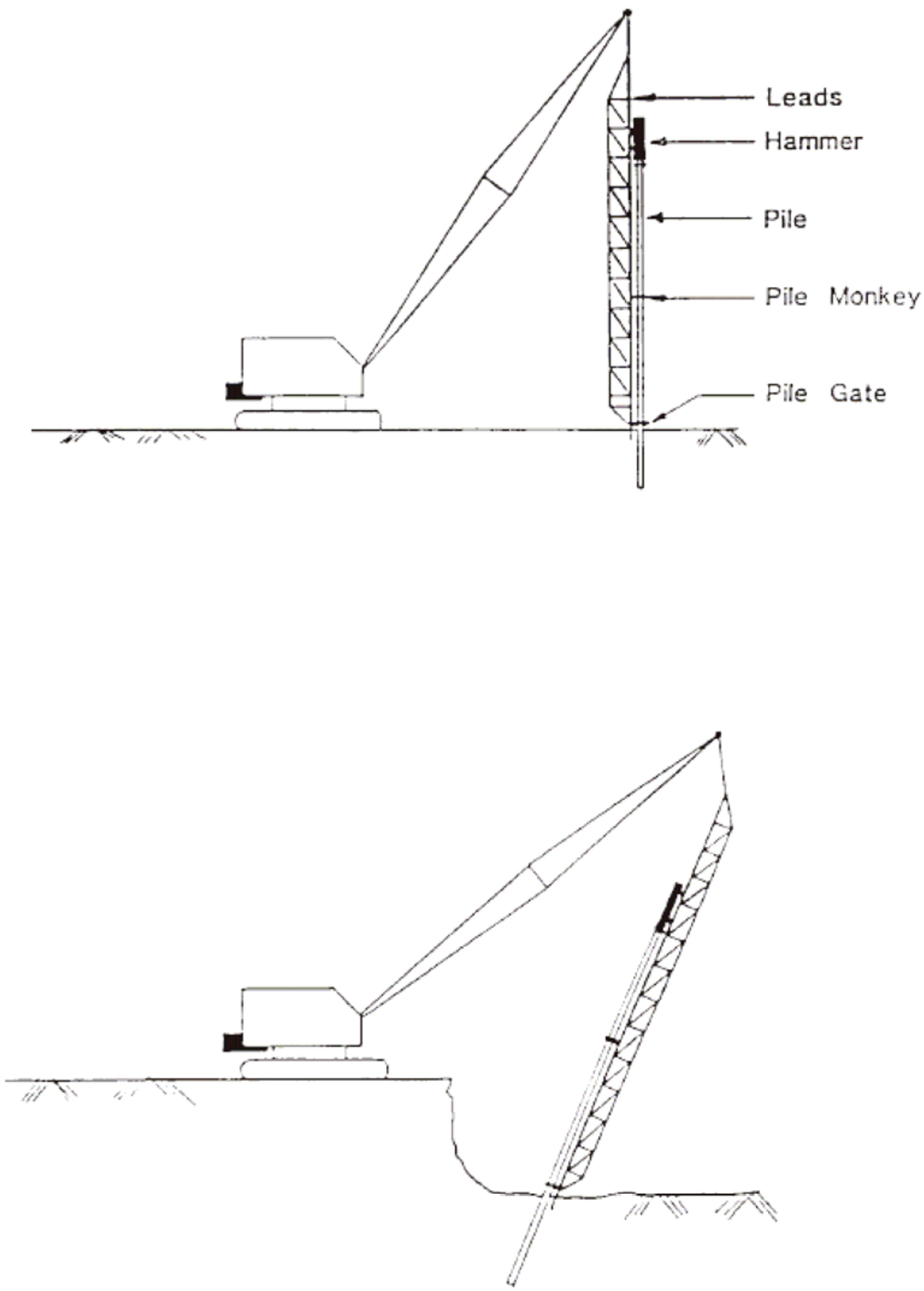


FIGURE 7-5 Swinging lead system

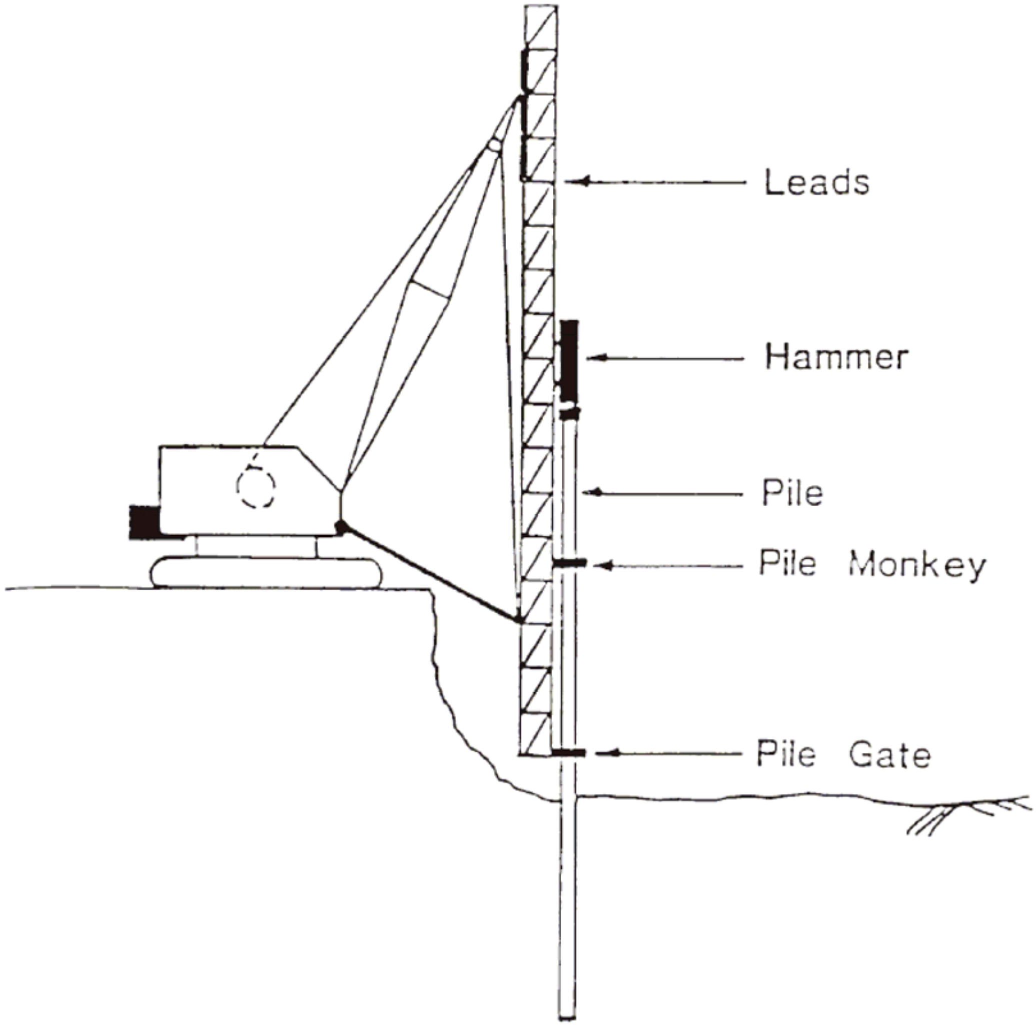


FIGURE 7-6 Semi-fixed lead system

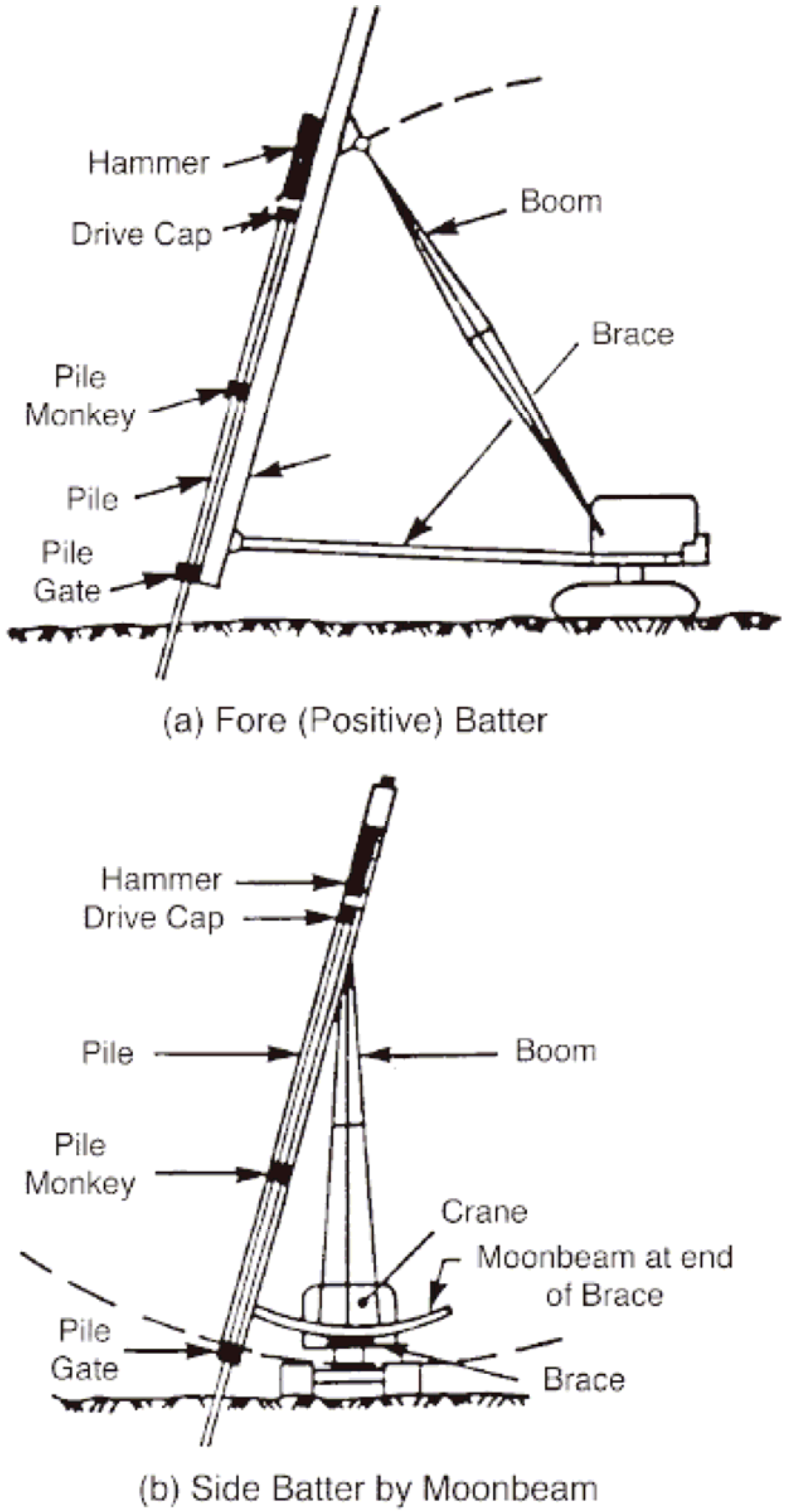


FIGURE 7-7 Lead configurations for battered piles

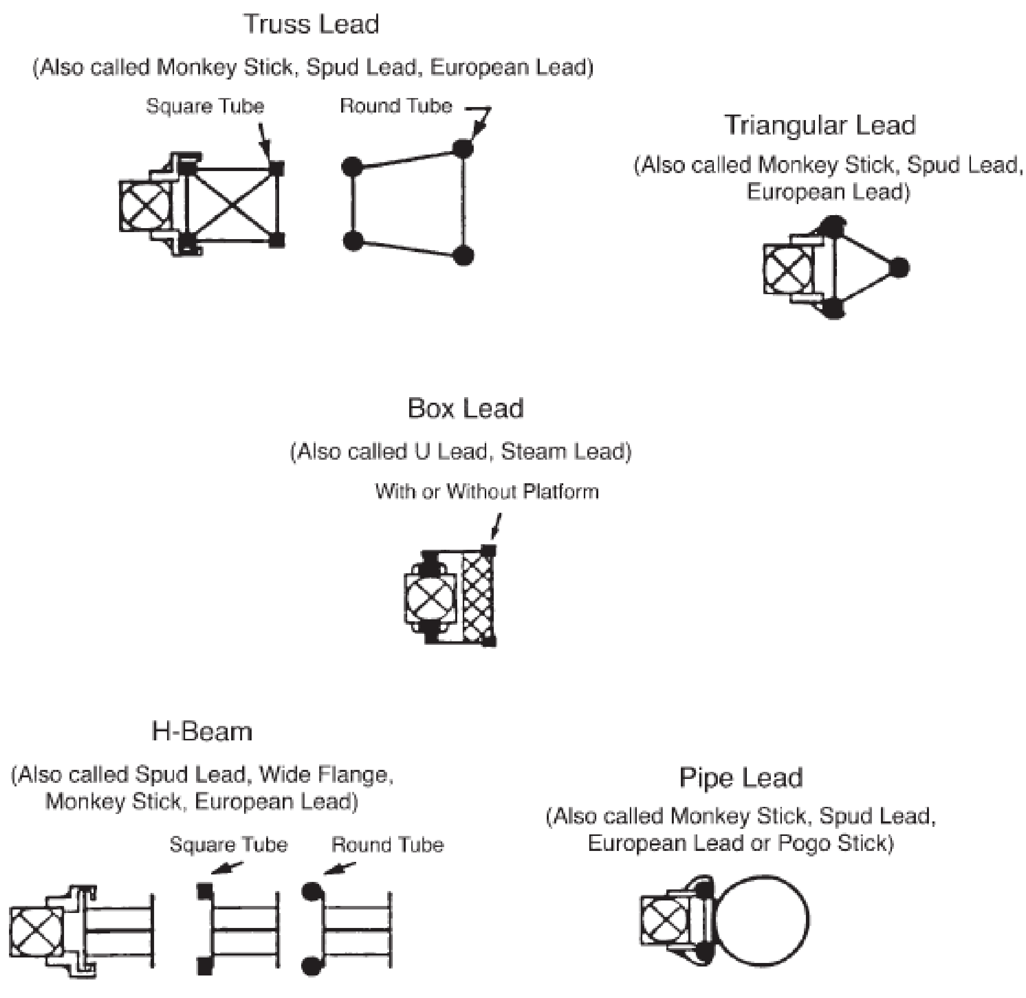


FIGURE 7-8 Lead types

Hammer Types

Many different types of pile driving hammers are used in the pile industry today. In 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.

The Drop Hammer

Invented centuries ago, the drop hammer 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 specifically allowed by the special provisions. Hammer weight and stroke restrictions will be found in Section 49-1.05 of the Standard Specifications.

When using a drop hammer the Engineer should:

NO.	ITEM DESCRIPTION
1	Ensure that you have the correct weight for the hammer being used. If in doubt, have it weighed.
2	Ensure the drop hammer lead sections are properly aligned and that all lead connections are properly tightened.
3	Ensure, while in use, that the hoist line is paying out freely.

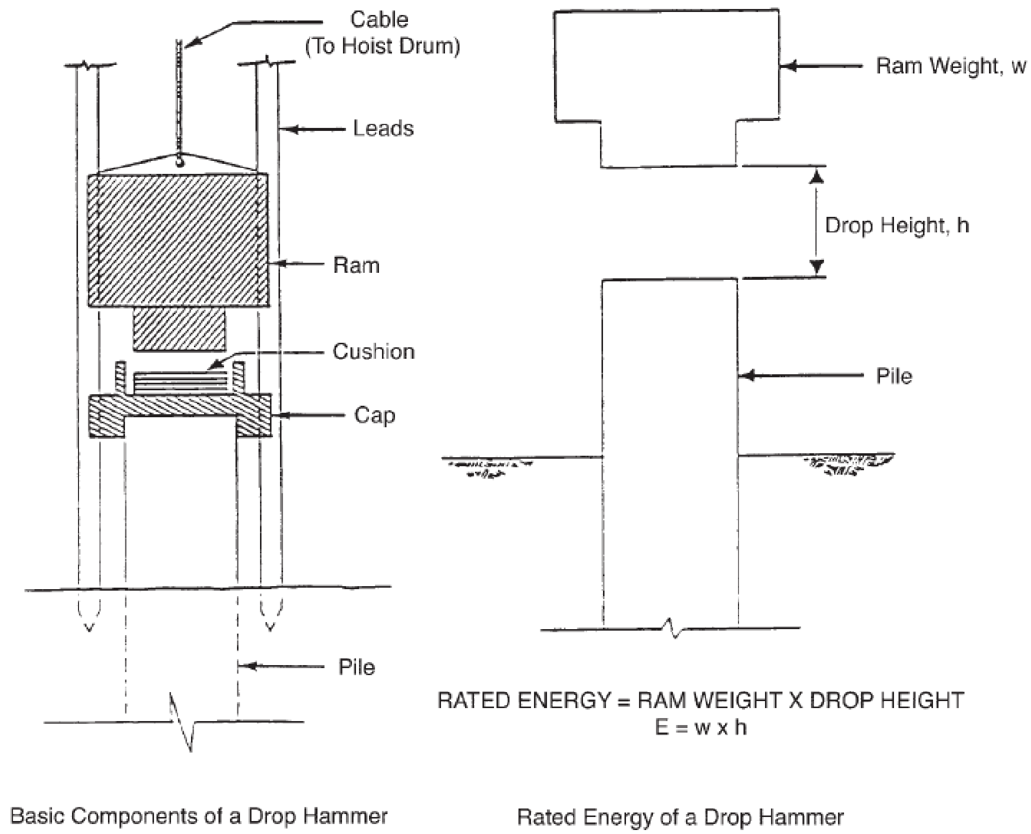


FIGURE 7-9 Drop hammer

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:

NO.	ITEM DESCRIPTION
1	Have the manufacturer's current specifications for the type and model of hammer being used.
2	Ensure all required parts of the hammer are intact and in good operating condition.

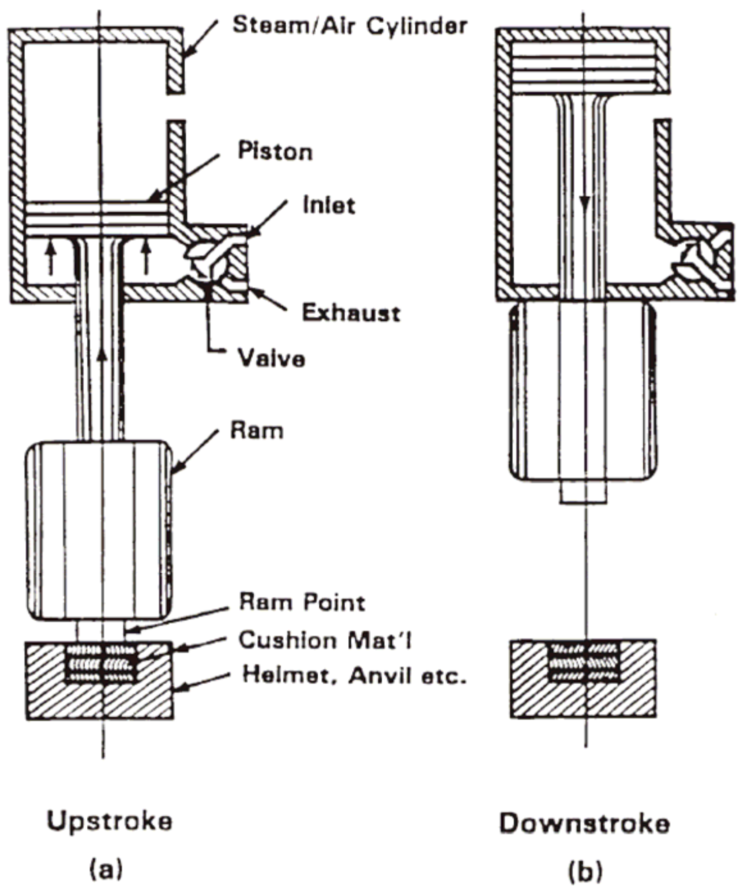


FIGURE 7-10 Single acting steam/air hammer

Double Acting Steam/Air Hammers

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 down stroke.

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:

NO.	ITEM DESCRIPTION
1	Have the manufacturer's current specifications for the type and model of hammer being used.
2	Ensure all required parts of the hammer are intact and in good operating condition.
3	Have chart available declaring rated energy vs. operating speed of hammer.

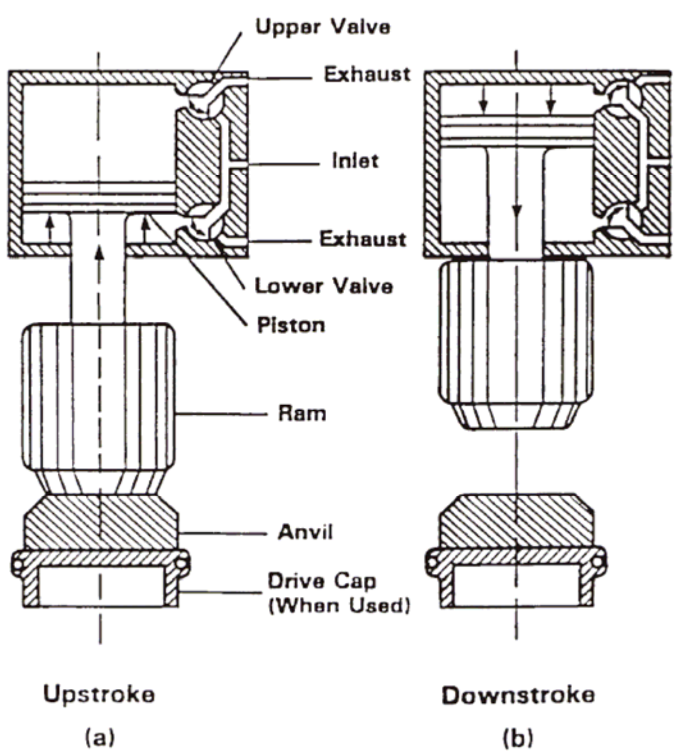


FIGURE 7-11 Double acting steam/air hammer

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 adding the differential force due to the motive fluid pressure acting over the large piston head to the weight of the striking parts and 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.

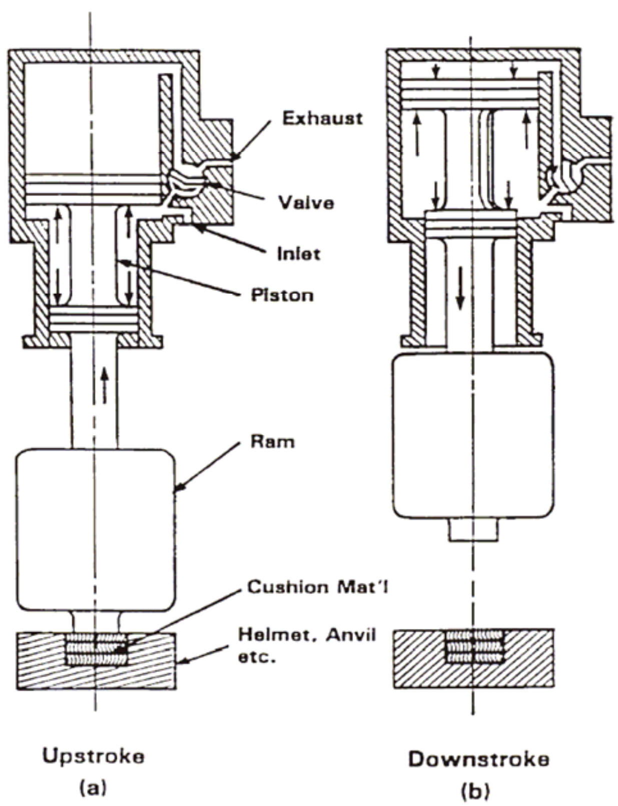


FIGURE 7-12 Differential acting steam/air hammer

When using a differential acting steam/air hammer the Engineer should:

NO.	ITEM DESCRIPTION
1	Have the manufacturer's current specifications for the type and model of hammer being used.
2	Ensure all required parts of the hammer are intact and in good operating condition.
3	Have chart available declaring rated energy vs. operating speed of hammer.

Diesel Pile Hammers

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 by a pump 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.

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 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 containing 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 noisy and they tend to spew oil and grease throughout. They can also emit unsightly exhaust, although newer models have been designed to be somewhat more environmentally friendly.

When using a diesel hammer the Engineer should:

NO.	ITEM DESCRIPTION
1	Have the manufacturer's current specifications for the type and model of hammer being used.
2	Ensure all required parts of the hammer are intact and in good operating condition.
3	Have chart available declaring rated energy vs. operating speed of hammer.
4	Be aware of the actual stroke of the hammer during driving and that it will vary depending on soil resistance.

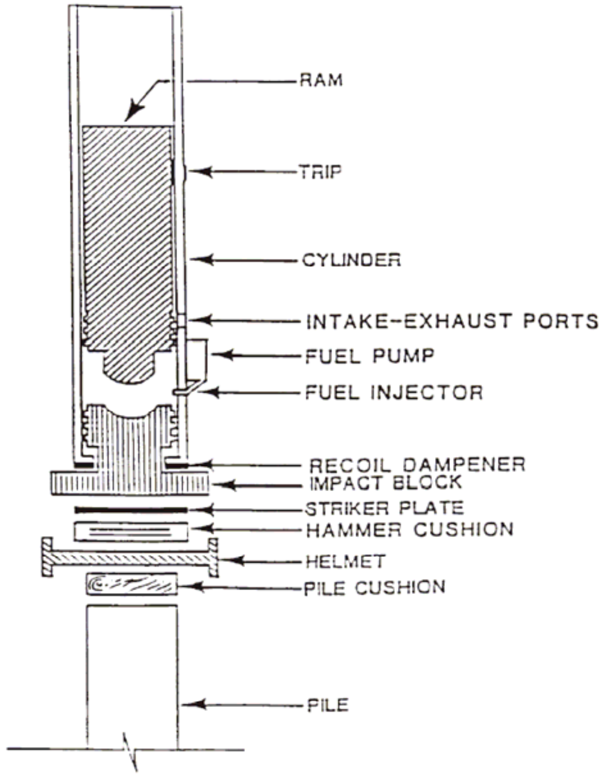


FIGURE 7-13 Single acting diesel hammer

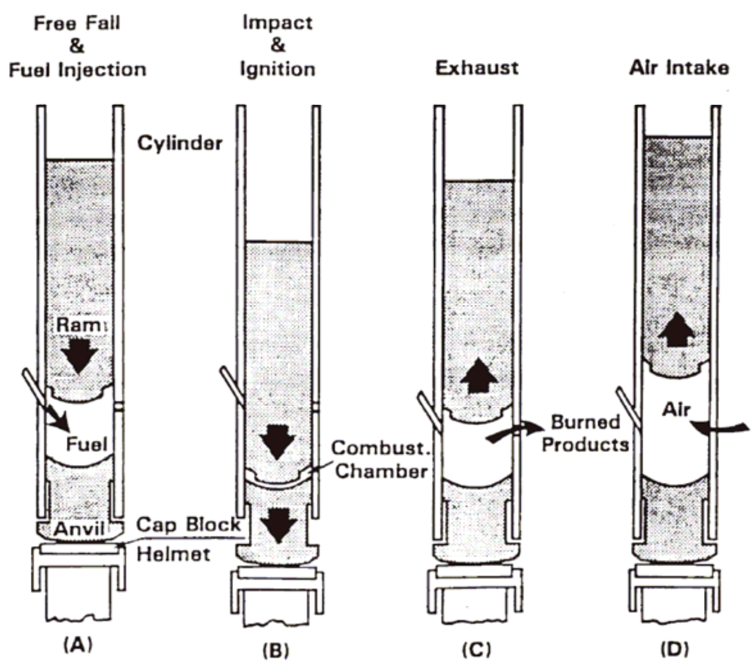


FIGURE 7-14 Operational cycle for single acting diesel hammer

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 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.

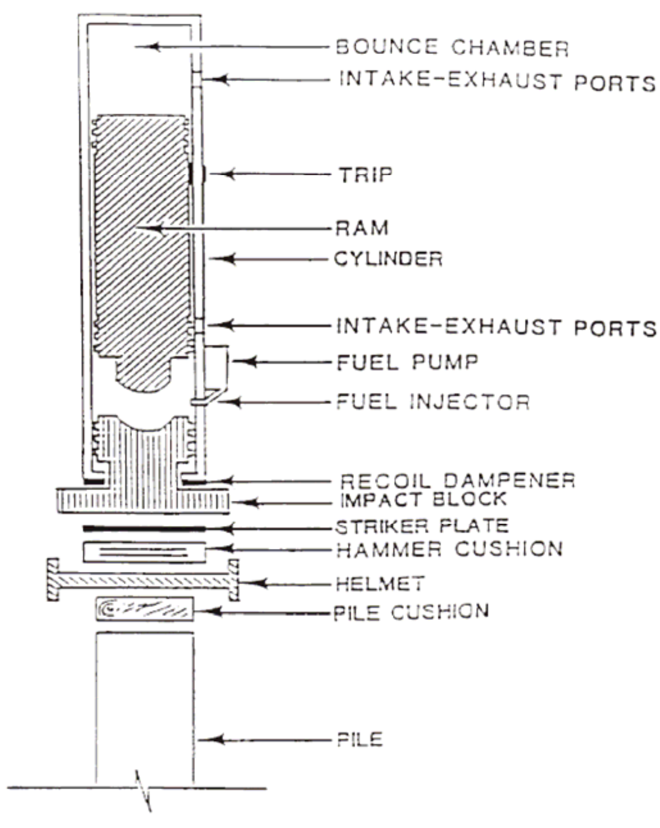


FIGURE 7-15 Double acting diesel hammer

When using a double acting diesel hammer the Engineer should:

NO.	ITEM DESCRIPTION
1	Have the manufacturer's current specifications for the type and model of hammer being used.
2	Ensure all required parts of the hammer are intact and in good operating condition.
3	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.

Vibratory Driver/Extractor

Vibratory pile drivers/extractors could be likened to 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 as much large amplitude ground vibration as the pile driving equipment discussed above. This makes the vibratory hammer desirable in areas where excessive ground motions could possibly cause damage to adjacent structures.

Section 49-1.05 of the Standard Specifications prohibits 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 are to install 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 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 approved 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 approved 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:

NO.	ITEM DESCRIPTION
1	Be aware of specific pile requirements and limitations stated in the special provisions and the Standard Specifications.
2	Discuss the proposal with the Bridge Construction Engineer, the project designer, and the geoprofessional.

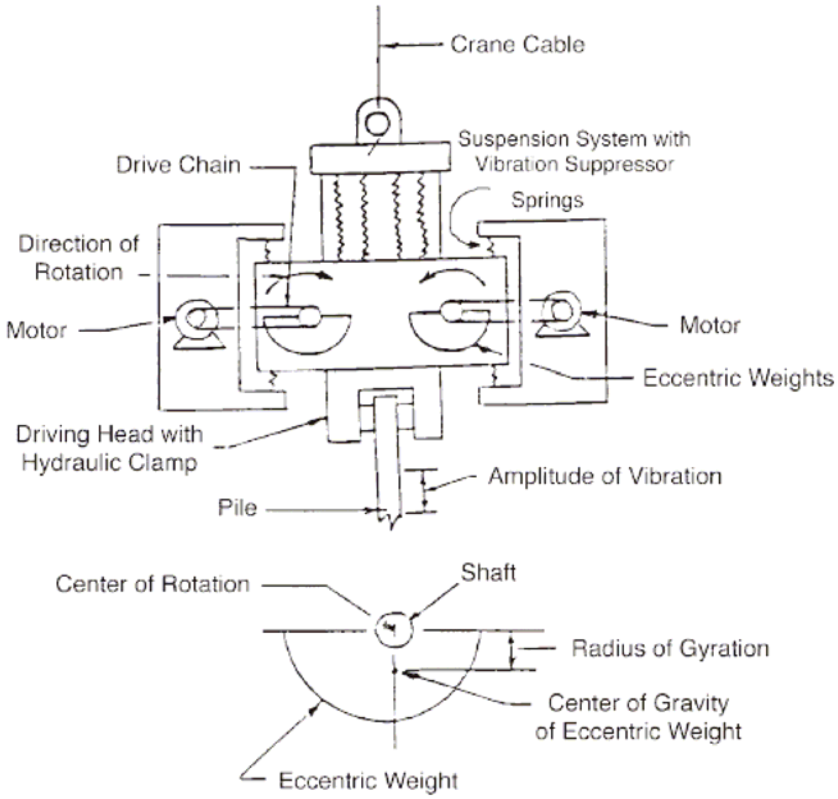


FIGURE 7-16 Vibratory driver/extractor

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 light weight hammer with a longer stroke).

The hydraulic hammer has a variable stroke, which is readily controlled from a control box located in the cab with the crane operator or in a separate cab, as is the case for larger hammers. 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:

NO.	ITEM DESCRIPTION
1	Hammer efficiencies in the range of 80% to 98%, while saying that diesel hammers have efficiency in the range of 30% to 40%.
2	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.
3	They claim the operation to be quieter than the typical diesel hammer.
4	The typical exhaust of the diesel hammer is eliminated, since only the motor driving the hydraulics is the source of exhaust.
5	Avoids diesel hammer problems of soft ground starting and operating in extreme climates.

General Hammer Information

Section 49-1.05 of the Standard Specifications requires that the Contractor 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. 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 my 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 predrilling. 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.

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:

(HAMMER ENERGY) – (ENERGY LOSSES) = (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.

Section 49-1.08 of the Standard Specifications requires that the bearing value of driven piles be determined using the Gates formula as follows (Refer to Appendix E for examples):

$$R_u = (1.83 \times (E_r)^{1/2} \times \log_{10}(0.83 \times N)) - 124$$

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 100)

This formula is appropriate for most piles and Standard Plan piles in particular. Acceptance criteria that require larger capacities than 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. The processes are explained in detail in the next chapter but a brief description of each one follows.

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.

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 the Department is one of the most widely known. 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 shown on the Log of Test

Borings. 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. To date wave equation analysis has been used for driveability 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.

Driveability Study. The wave equation analysis can be used as a driveability study during the design phase to validate design assumptions for things like 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, the 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 will be difficult.

Hammer Acceptance Study. The Hammer Acceptance Studies are done after the contract is bid or awarded. Current contracts require the contractor to submit information on the actual driving system proposed for the project. This information is used by the Foundation Testing Branch to perform a wave equation analysis. Some of the more current contracts require the contractor to perform their own wave equation analysis. Essentially a driveability study is performed using the actual hammer information instead of assumed values. From this information, the Engineer can decide if the proposed hammer 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 hammer is not efficient. Either way the results of the driveability study are used as a basis for accepting or rejecting the hammer submittal.

Acceptance Curve Study. The studies outlined above use theoretical or empirical information to develop a model that gives a pretty accurate indication of what will 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 Equation 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 Standard Specifications (Section 49-1.08). 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 for samples of acceptance curves.

Another situation where acceptance curves are useful is in situations where the ground conditions during driving are not what control the design. Examples of this 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 or in the case of re-moldable clays where skin friction is lost during the driving operation and 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 pour water pressures are redistributed. The driven pile will actually achieve greater a 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.

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 plans. When this is done during construction the special provisions will outline 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.

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 hammer 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.

Engineers and inspectors use the manufacturer's maximum rated energy as an indication of the driving capability of the hammer. It is used in the Gates formula as required by the Standard 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 and from job to job. 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 and type of cushion, improper adjustment of valve gear, poor lubrication, 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.

Battered Piles

When battered piles are driven, an adjustment to the hammer energy needs to be made since the path of the ram is not plumb. The hammer path will follow the slope of the battered pile so the stroke used to compute delivered energy must be adjusted to reflect the change in vertical fall of the ram. This is simple to determine for single acting air, steam or diesel hammers. For example, a 140 Ton pile driven with a Delmag 30 hammer will require 28 blows per foot using the Gates Formula. If the pile were driven on a 1:3 batter the minimum blow count would be increased to 30 blows per foot ((3.162/3) X 28 = 30). Refer to Appendix E for an example of this.

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 as energy delivered by differential action or pressure imparted on the downward stroke should remain constant.

Preparing to Drive Piles

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, so that as many of them as possible can be eliminated or resolved before they occur.

The following material is essentially a checklist of what the Engineer should look for both before pile driving begins and while pile driving is underway. This list is 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:

NO.	ITEM DESCRIPTION
1	Review the Plans, Special provisions, Standard Specifications and Foundation Report for requirements on pile type, required bearing and penetration, predrilling depths (critical with tension piles as well as compression piles), tip protection or pile lugs and limitations on hammer types or other specific limitations or requirements.
2	Check for Form TL-29, “Release of Materials.”
3	Check Welding Quality Control Plan (WQCP) and welder certification requirements.
4	Prepare the pile layout sheet. Form DH-OS C80 in the CR&P Manual
5	Prepare the pile log forms. Form DH-OS C79 in the CR&P Manual
6	Advance preparation of a chart, table or graph that correlates the blow count, stroke, blow rate, etc., to the bearing value is suggested for each hammer. An example is included in Appendix E. Verify the hammer is an approved hammer in accordance with the requirements of Bridge Construction Memo 130-2.0 and is able to develop sufficient energy to drive the piles at a penetration rate of not less than 1/8-inch per blow at the required bearing value. Refer to the “Verification of Hammer Energy” section later in this chapter.
7	Review the mechanics of the hammer type to be used for further verification of components in the field.
8	Obtain the necessary safety equipment (Refer to the “Safety” section later in this chapter) and inspection tools (tape measure, paint, stop watch, etc.)

Once out in the field, prior to start up of pile driving:

NO.	ITEM DESCRIPTION
1	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 layout piles for the Contractor.
2	Confirm pile materials, tips and lugs. Refer to the “Materials Checklist” later in this chapter.
3	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.
4	Verify the reference elevation.
5	Layout and mark piles for logging. Mark additional reference points near the anticipated tip elevations so that monitoring can take place at smaller increments.
6	Locate a good place to inspect operations. Notify the pile foreman of location and signals to be used.

When pile driving starts:

NO.	ITEM DESCRIPTION
1	Verify the pile location at the start of driving.
2	Verify plumbness or batter of the pile at the start of and during driving.
3	Monitor and log the blow count, stroke and penetration (Refer to the “Logging of Piles” section later in this chapter).
4	Stop driving at proper bearing and penetration.

After completion of driving piles:

NO.	ITEM DESCRIPTION
1	Verify proper pile cutoff.
2	Prepare copies of pile logs to be sent to the Office of Structure Construction in Sacramento in accordance with Bridge Construction Memo 3-7.0.

Verification of Hammer Energy

Several verification methods are available to field staff 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 is given in Bridge Construction Memo 130-3.0.

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. An example is also 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 Standard 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).

Materials Checklist

Precast Concrete Piles

CHECK ITEM	CHECK DESCRIPTION
1	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. Section 49-1.07 of the Standard Specifications requires that piles be at least 14 days old before driving.
2	Lifting anchors for Class C piles are to be removed to a depth of one inch and the hole filled with epoxy. Piles without Class C designation shall have the anchors removed along the portion of pile above the final ground line. Section 49-3.01 of the Standard Specifications covers this subject..

Discuss with the Contractor the type and method of rigging planned to lift the 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 (Refer to Section 49-3.03 of the Standard Specifications).

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

Allowable Stress = $5\sqrt{fc'}$ 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 cover 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.

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" and "Piles" sections of the Special provisions, usually in accordance with a Welding Quality Control Plan and the AWS D1.1, Structural Welding Code. Assistance may be obtained by calling the Office of Materials Engineering and Testing Services (METS).

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 of the Bridge Construction Records and Procedures Manual.

CHECK ITEM	CHECK DESCRIPTION
1	Check for proper diameter and shell thickness. Paint one-foot marks and lengths on the piles. The Contractor may assist in this.
2	Check welded joints for any sign of improper welding. When piles are to be spliced, a Welding Quality Control Plan will be required. Refer to the Special provisions for information pertaining to this plan. Refer to Section 49-5.02 of the Standard Specifications for additional information on types of welds allowed in splices.

Timber Piles

Check the butt and tip diameters to ensure compliance with Section 49-2.01 of the Standard Specifications. Treated timber piles shall be driven within 6 months after treatment.

Piles shall 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 0.3 inch in nominal thickness per Section 49-2.03 of the Standard Specifications.

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.

Logging of Piles

It is Office of Structure Construction policy to log at least one pile, in it's 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 retap them later. A good log of the piles within the footing will help the Engineer to determine how many piles might require a restrike/retap to prove bearing. If all the piles drove in a similar manner, it might be possible to restrike/retap as few as 10% of the piles that did not originally achieve bearing. If the piles all drove differently, a restrike/retap 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), or (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 pile is undergoing hard driving and suddenly 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 project Engineer 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.

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.

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 Log of Test Borings 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, Section 49-1.05 of the Standard Specifications states: "When necessary to obtain the specified penetration and when authorized by the Engineer, the Contractor may supply and operate one or more water jets and pumps, or furnish the necessary drilling apparatus and drill holes not greater than the least dimension of the piles to the proper depth and drive the piles therein." For driven piles, shells or casings, the Standard Specifications also require the use of special driving tips, heavier pile sections, or other measures as approved by the Engineer, to assist in driving or prevent damage to a pile through a hard layer of material.

The special provision should address the job specific requirements or limitations for jetting or predrilling. If not, the Engineer should consult with Geotechnical Services and Structure Design if hard driving is anticipated and the Contractor is considering jetting or predrilling to address it. While these methods may be used, there is the potential for these methods to impact the capacity of the pile. Therefore, there may need to be limitations, such as depth or diameter of predrilling, on the use of these procedures.

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) 10 or more blows per inch, or (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 predrilling or jetting.

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 given in the Standard Specifications. 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/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 predrilling and jetting. These methods are typically used when economics dictate this to be the best solution or when larger hammers cannot be utilized because they will overstress the pile.

"Jetting" uses water pressure to remove soils and has the potential to impact the capacity or alignment of a pile; as such care must be exercised when used. Two methods are generally employed: (1) pre-jetting, and (2) side jetting. In terms of controlling pile alignment pre-jetting is best. A pilot hole is simply jetted to the desired depth. After the jet pipe is withdrawn the pile is immediately inserted in the hole and driven. With side jetting the jet pipe is inserted into the ground adjacent to the pile and the jetting and driving take place concurrently. Care must be taken when this is done with a single jet, as the pile tip will tend to move off line in the direction of the jetted side. Larger piles are frequently side jetted with multiple pipe systems. These systems can be located outside the pile or within the annular space of hollow piles. In addition, the pipe arrangement of multiple pipe systems is usually symmetrical, thus enabling better control of pile alignment. Jetting uses water to facilitate driving and the end result is a volume of muddy water that must be addressed in the Storm Water Pollution Prevention Plan or Water Pollution Control Program.

Drilling a "starter hole" to facilitate the advancement of a driven pile is known as predrilling. As per Section 49-1.05 of the Standard Specifications, the hole drilled shall 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 predrilling is limited to address this. There should be information in the Contract Plans, the Foundation Report or the Special Provisions that outlines these restrictions.

Driving tips strengthen the tip/toe of the pile so that it can penetrate through obstructions and dense lenses. Cutting shoes are another form of driving tip that allows piles with thinner wall thicknesses to be driven through dense lenses. Closed ended steel pile 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 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 define 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.

Section 49-1.07 outlines 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 resistance, additional means are required to achieve the specified tip elevation. These may include predrilling, jetting 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 the Office of Structure Construction.

Piles typically are designed to meet several different design criteria (Tension, Compression, Lateral, etc.) 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. Chapter 3 of this Manual discusses this issue in detail.

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.

Soft Piles and Re-Drive

The Standard Specifications require the Contractor to satisfy requirements for minimum nominal 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 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

- The pile can be “re-driven” several days after initial driving 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” or “re-strike” of the pile. To do this, pile driving is stopped when the pile is a certain distance above the specified tip elevation (a few to several inches). The pile is then driven the remaining distance at a later date. This allows the soil the 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.

The Gates formula is still used for pile acceptance during re-strikes. 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 pile resistance. Since the distance driven in a re-strike is less than one foot, the number of blows per foot will need to be extrapolated from the field results based on the length of re-drive. The extrapolated value will be used to determine nominal resistance in the Gates formula.

Following are some ground conditions and the expected outcome after re-driving to address soft piles:

CONDITION	DESCRIPTION
1	Loose submerged fine uniform sand. Driving temporarily produces a quick condition. Re-drive will probably not indicate any change in capacity.
2	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.
3	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.

On contracts where soft driving in clay materials is anticipated, specific re-drive guidelines are frequently given in the Special provisions. The 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.

Re-driving is a tool that the Engineer can use in an attempt to obtain an acceptable pile even though the Standard 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 penetration 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 covers this in detail.

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 during driving so that the pile can be pulled and re-driven correctly.

While the Standard Specifications state "piles materially out of line will be rejected", there's no tolerance provided in the specification that define when a pile truly is or isn't "materially out of line". Some contracts have specific tolerances outlined in the Special provisions that defines 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.

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.

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:

ITEM NO.	DESCRIPTION
1	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.
2	Stand as far away from the operation as practical while still inspecting the work.
3	Keep clear of any steam, air or hydraulic lines.
4	Watch the swing of the rig so as not to be hit by the counterweight.
5	Wear safety glasses. There is a high incident of flying debris during the driving operation (dirt from piles, concrete from piles and steel chips).
6	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.
7	Hearing protection is required due to high noise levels.
8	Have a planned route for rapid escape. If required to move quickly there will not be time to look around first.
9	Wear old clothes. Park your car and stand upwind when possible. Diesel oil does not wash out of clothes!
10	Look where you are walking. The protective covers may not be securely in place over the predrilled holes.
11	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.

CHAPTER

8 Static Pile Load Testing and Pile Dynamic Analysis

Introduction

Chapter 1 of this Manual explained how Geotechnical Services performs a foundation investigation for all new structures, widenings, strengthenings 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, a 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 the Foundation Testing Branch, a subgroup of Geotechnical Support in Geotechnical Services performs 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.

Reasons For Static Load Testing and Pile Dynamic Analysis (PDA)

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 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 Geoprofessionals the 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; unlike driven piles. 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 "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)
- Determining the length of pre-cast friction piles so as 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. PDA is performed on all contracts that have piles that require capacities larger than those of the piles in the Standard Plans.

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.

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 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) Loads are applied in increments; typically equal to 10% of the design load. Each increment of load is held for a predetermined time interval. The load increments are applied until the pile starts to "plunge", or up to the point where the capacity of the testing system is reached. The "plunge" point is where little or no additional load is needed to cause the pile to displace. In general, a pile is considered to have failed when the total displacement exceeds 1/2 inch under load. An acceptable pile is one that reaches double the design load without exceeding this displacement.

FIGURE 8-1 Static pile load test (five-pile array)

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 rock 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 set in the "Quick Load Method" of ASTM D1143 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:

- 4.5 Meganewton (1,000,000-pound) Load Test System
- 9 Meganewton (2,000,000-pound) Load Test System
- 17.5 Meganewton (4,000,000-pound) Load Test System
- 35 Meganewton (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 and is available for download at:
http://www.dot.ca.gov/hq/esc/geotech/requests/plt.pdf

Pile Dynamic Analysis (PDA)

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

The PDA 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 PDA 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 PDA is able to measure pile stresses generated during driving. During installation, damage to a pile can often be detected by the PDA. 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 PDA very 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 very 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 PDA 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 PDA 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 PDA Operator to ensure that the instrumentation is attached correctly and that the data is transmitted to the PDA computer. Afterward, driving continues until the specified tip elevation is reached. Is 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 restrike. The timeframe for "set-up" is usually overnight but can be longer. Before the restrike, PDA 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 the 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 of this Manual.

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 (PDA) Test Request Form. A copy of this form is included in Appendix F and is available for download at:
http://www.dot.ca.gov/hq/esc/geotech/requests/pda.pdf

Contract Administration of Static Pile Load Testing and 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 of review the project plans, Special Provisions, Standard Specifications, and Bridge Construction Memo 130-2.0 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.

Section 49-1.04 of the Standard Specifications states that 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 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 Contract 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 State 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.

Section 49-1.04 of the Standard Specifications states that no piles may be drilled, cast, cut to length or driven for a structure 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 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 pre-cast 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. The Standard Specifications state that the Engineer will not require more than 5 working days to perform each static load test unless otherwise provided in the Special Provisions. This is important, in that the Department 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.

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 specifications. Since the Foundations Testing Branch has several new testing devices, the Engineer should discuss and confirm the load test pile array set-up 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 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 Project Engineer 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 Project 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 plans will warrant a contract change order.

If a construction project includes Pile Dynamic Analysis, the Special Provisions will state when the piles to be analyzed are to be made available for State personnel so that the necessary preparations before 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 or 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, the State will be responsible for additional costs incurred by the Contractor. Substantial pile revisions (as a result of poor test results for example) 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 Engineers set up a good line of communication between themselves and 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.

CHAPTER

9 Slurry Displacement Piles

Introduction

A slurry displacement pile is a Cast-In-Drilled-Hole (CIDH) pile whose method of construction differs from the usual CIDH pile in that 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 of this manual 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.

History

The use of drilling slurry is commonly associated with methods used by the oil well drilling industry over the last 100 years, which naturally 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, by 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 done to date has given conflicting results; however most indicate that pile capacities may be less than that of CIDH piles constructed without the use of drilling slurry. However, the design method used by Caltrans for determining the pile capacity adequately accounts for the potential 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 has lead to the development of contract specifications for use of the slurry displacement method of CIDH pile construction.

Processed clay mineral slurries are considered to be 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.

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 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 or in the Foundation Report. Sometimes experience on adjacent projects may also give an indication of the site conditions.

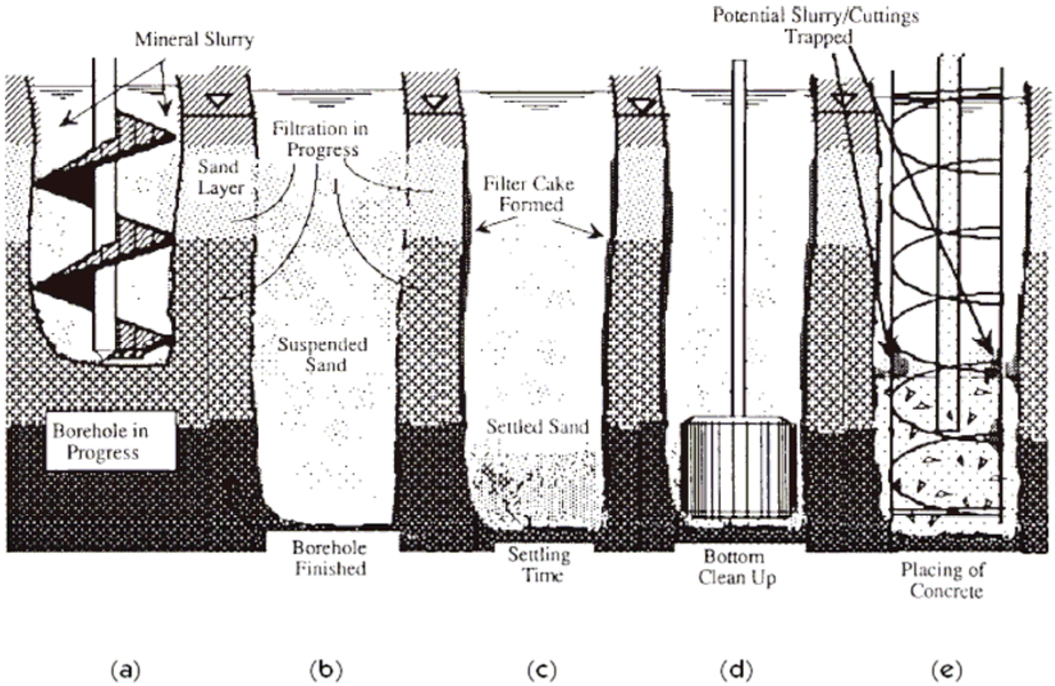


FIGURE 9-1 Slurry displacement method

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 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 recontaminate 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. Once the slurry is within the specified limits, the 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)). The tip of the rigid 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.

Principles of Slurry Usage

All slurries of whatever kind 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(a) & (b)). 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.

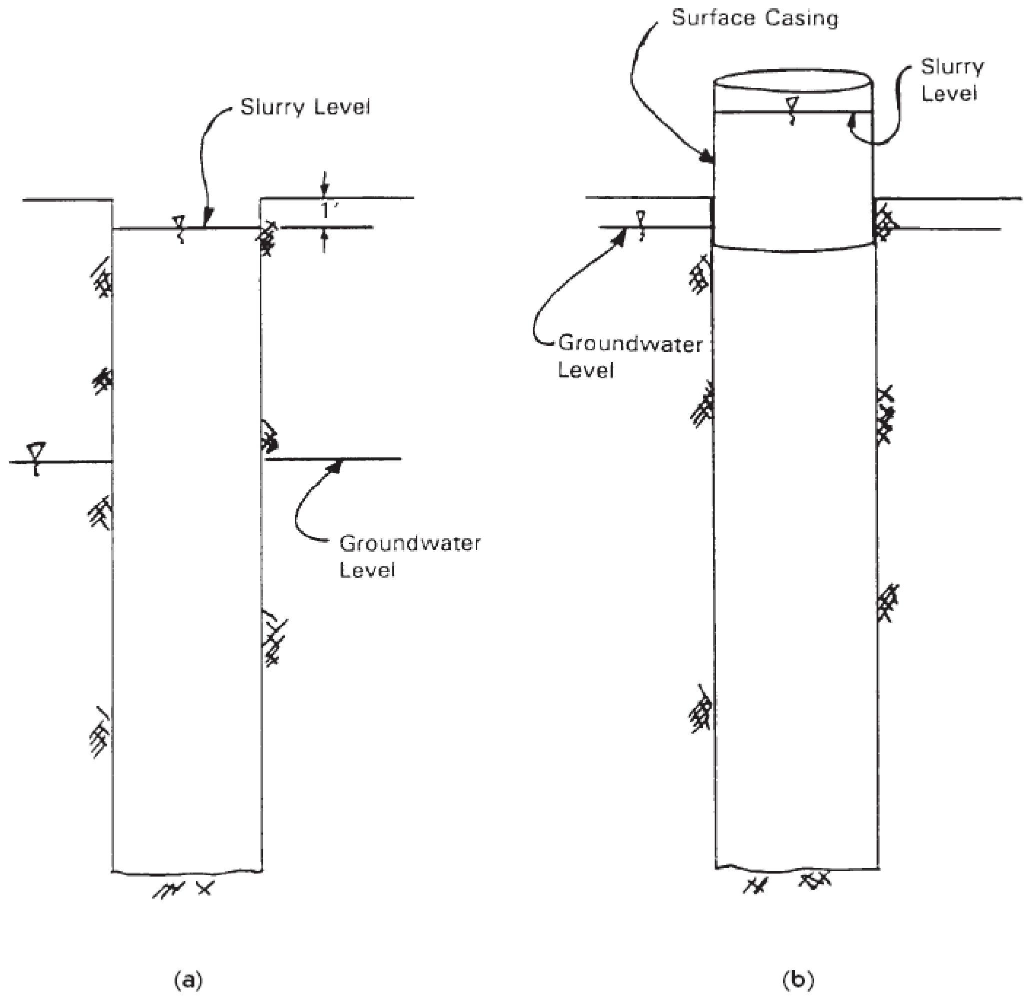


FIGURE 9-2(a)(b) Positive hydrostatic pressure

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 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 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(a)). 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 9-3(b)). 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.

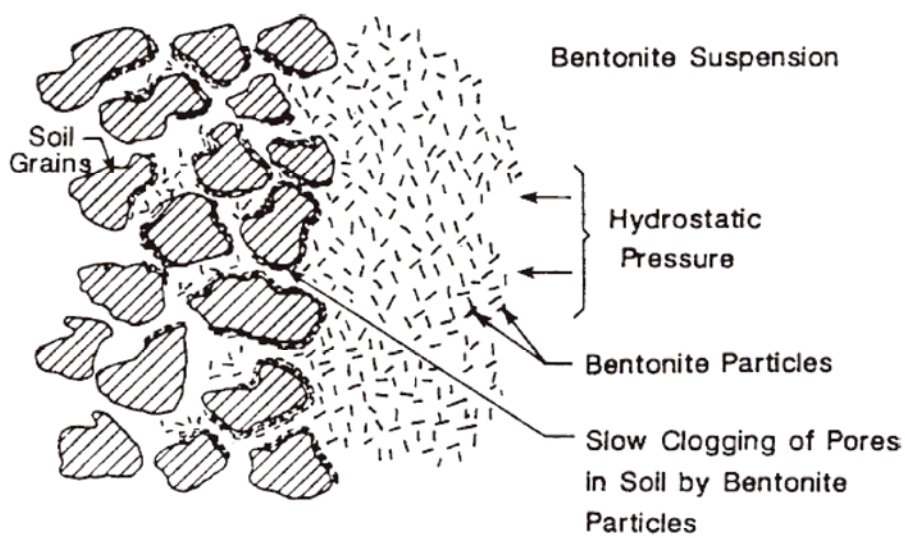


FIGURE 9-3(a) Filtration – loose ground formation

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.

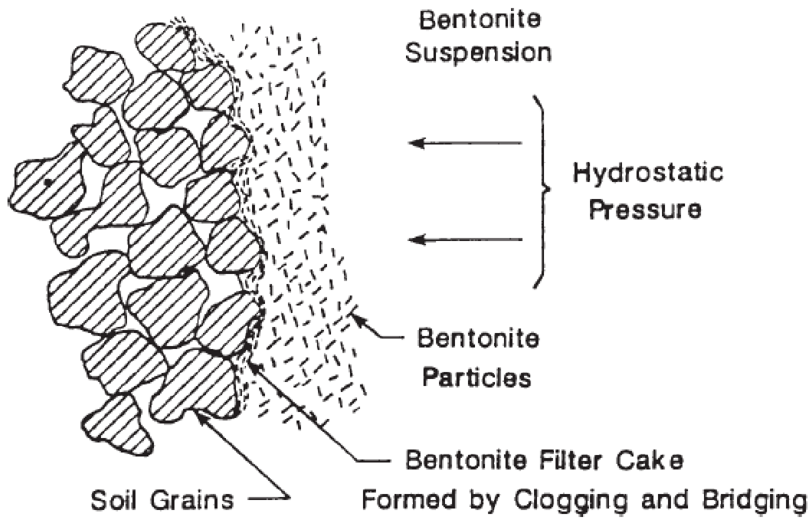


FIGURE 9-3(b) Filtration – tight ground formation

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.

In regards to 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. These synthetic slurries that consist of very long, chain-like hydrocarbon molecules (polymers) 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 9-3(c)). 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.

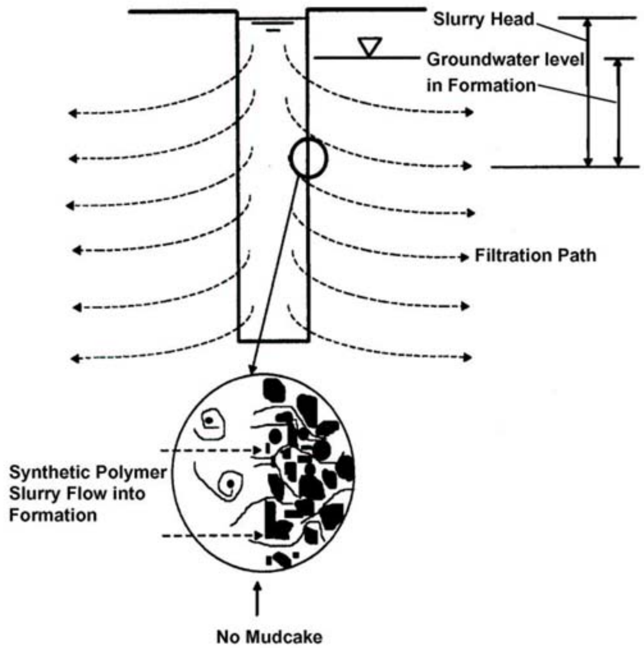


FIGURE 9-3(c) Stabilization with synthetic polymer slurry

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.

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 9-4).

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.

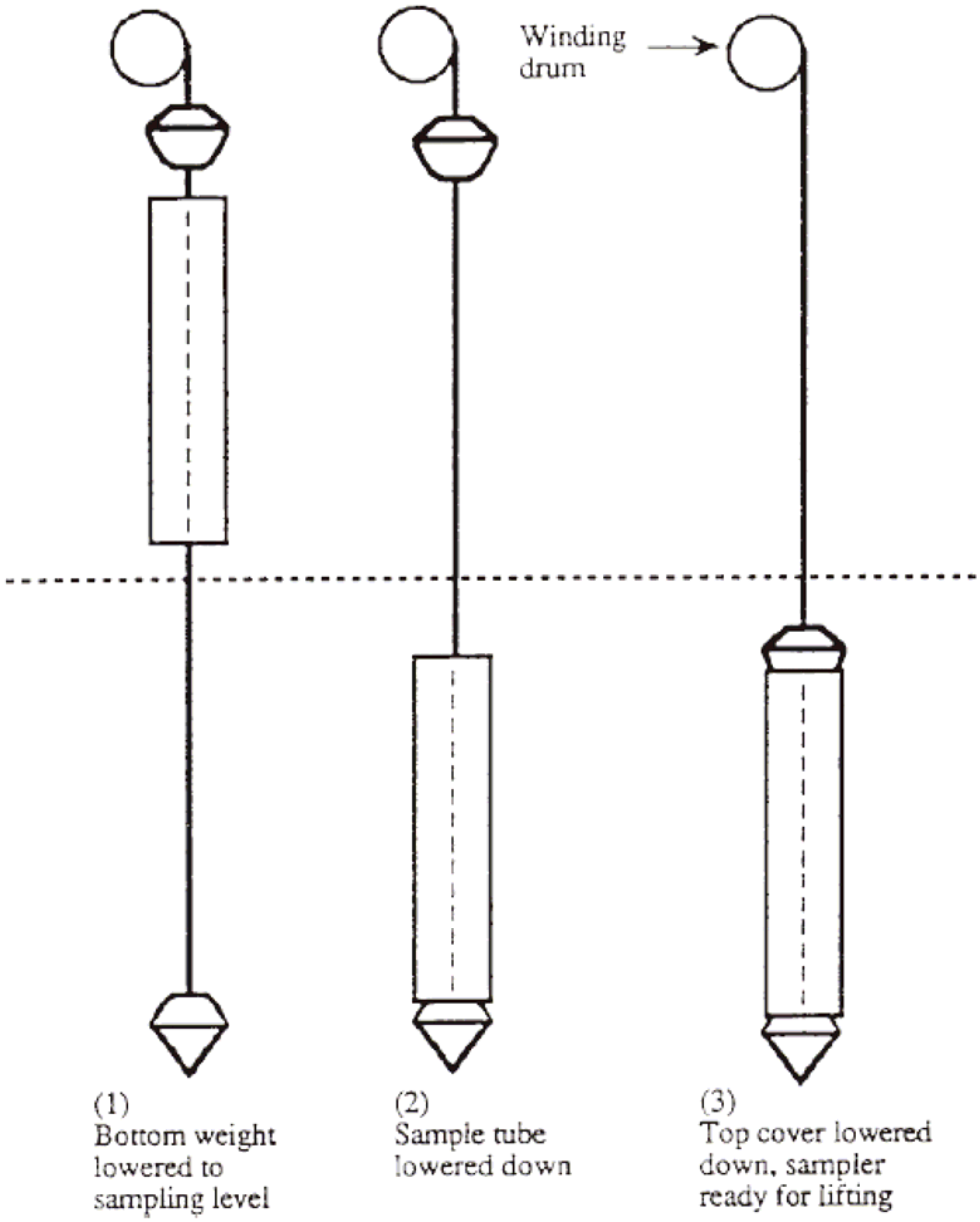


FIGURE 9-4 Slurry sampler schematic

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.

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 9-5 Density test kit

Density is tested using the test kit shown in Figure 9-5 in conformance with the test method described in American Petroleum Institute (API) Recommended Practice 13B-1, Section 1. This test method can be obtained by contacting the Offices of Structure Construction in Sacramento or accessing its intranet website at http://onramp.dot.ca.gov/hq/oscnet/.

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 9-6 Sand content test kit

Sand content is tested using the test kit shown in Figure 9-6 in conformance with the test method described in API Recommended Practice 13B-1, Section 5. This test method can be obtained by contacting the Offices of Structure Construction in

Sacramento or accessing its intranet website at http://onramp.dot.ca.gov/hq/oscnet/.

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.

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 9-7 Marsh funnel viscosity test kit

The viscosity of drilling slurry is tested using the test kit shown in Figure 9-7 in conformance with the test method described in API Recommended Practice 13B-1, Section 2.2. This test method can be obtained by contacting the Offices of Structure Construction in Sacramento or accessing its intranet website at http://onramp.dot.ca.gov/hq/oscnet/.

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, 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 contract.

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 desanding 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.

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 contract specifications state that water may only be used as drilling slurry when a temporary 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. This resulted in many defective piles that required repair.

The question that may arise from this limitation 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 since the drilling casing acts as a temporary casing.

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 a 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 on site after settlement of all suspended materials has occurred unless hazardous materials have contaminated the water.

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 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.

FIGURE 9-8 Bentonite slurry

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 and 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 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. Because the amount of filter cake that forms on the sides and bottom of the drilled hole depends on so many variables and because research of the effect of filter cake on the ability of the pile to transfer load through skin friction has not been completed, the Offices of 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 unagitated 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 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. This will prevent excessive suspended materials and to 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 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 9-9). The slurry plant contains screens, shakers, desanding centrifuges (Figure 9-10), and agitators, and is capable of mixing, storing, and cleaning the mineral slurry. Figure 9-11 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 9-9 Mineral slurry plant

FIGURE 9-10 Desanding centrifuges

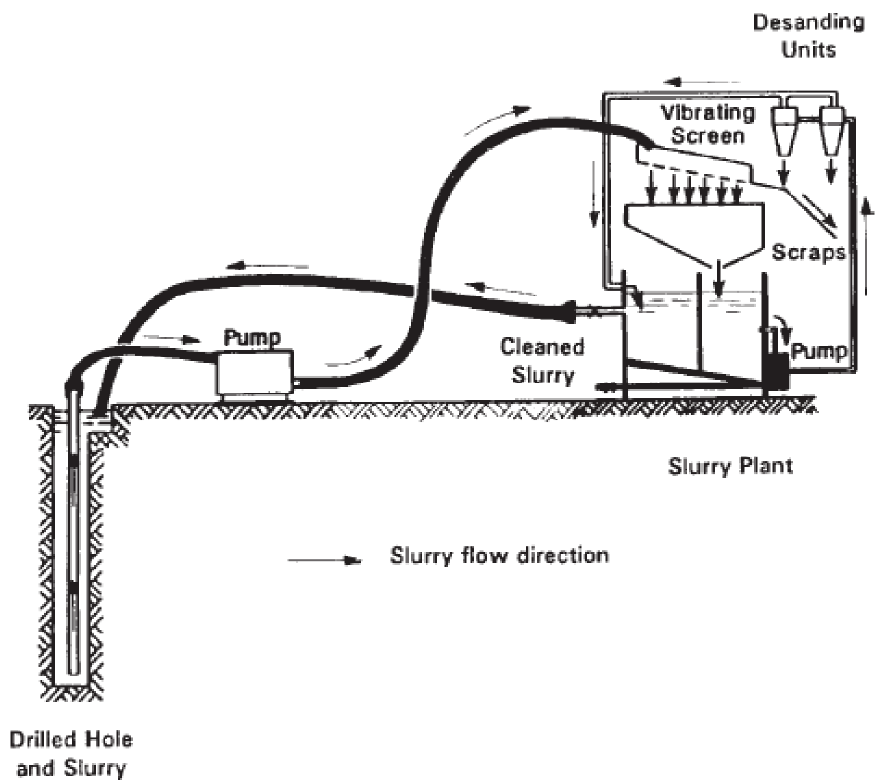


FIGURE 9-11 Recirculation and cleaning schematic

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.

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 the Offices of Structure Construction in Sacramento before approval 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 in accordance with Section 7-1.13 of the Standard Specifications. 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.

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.

The synthetic 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 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 CDP™, manufactured by KB International LLC; Shore Pac®, manufactured by CETCO Construction Drilling Products; and Novagel™, manufactured by Geo-Tech Services, LLC.

Super Mud is an emulsified (water-in-oil, liquid form) synthetic polymer product. A liquid form of SuperMud is currently approved for use on Caltrans projects. No other form is approved. (Figure 9-12)

FIGURE 9-12 SuperMud container

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 9-13)

FIGURE 9-13 SlurryPro CDP container

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 9-14)

FIGURE 9-14 ShorePac container

Novagel™ is a dry form synthetic polymer slurry product. A dry granular form of Novagel™ is currently approved for use on Caltrans projects. No other form is approved. (Figure 9-15)

FIGURE 9-15 Novagel container

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 much 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 Novagel™ due to the extensive research that had been done by the 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®, Novagel™ 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 Novagel™ 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, Novagel™, with a very high viscosity value up to 110 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 approve the manufacturer's representative. Assistance on approval of a manufacturer's representative may be obtained from the Offices of 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.

The manufacturer's representative may also recommend the use of chemical additives to adjust the synthetic slurry to the existing ground conditions. Caltrans has little experience with chemical additives and their use should be discussed with the Office of Structure Construction in Sacramento before approval is given for their use.

The contract specifications also require the manufacturer 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 shall not 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, shall be disposed of outside of the highway right-of-way as described in Section 7-1.13 of the Standard 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 California Regional Water Quality Control Board or the local sanitation district, for this type of disposal. The dried solids can be disposed of as mentioned above.

Equipment

The equipment used to construct CIDH piles by the slurry displacement method are 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 9-16). 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, because 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.

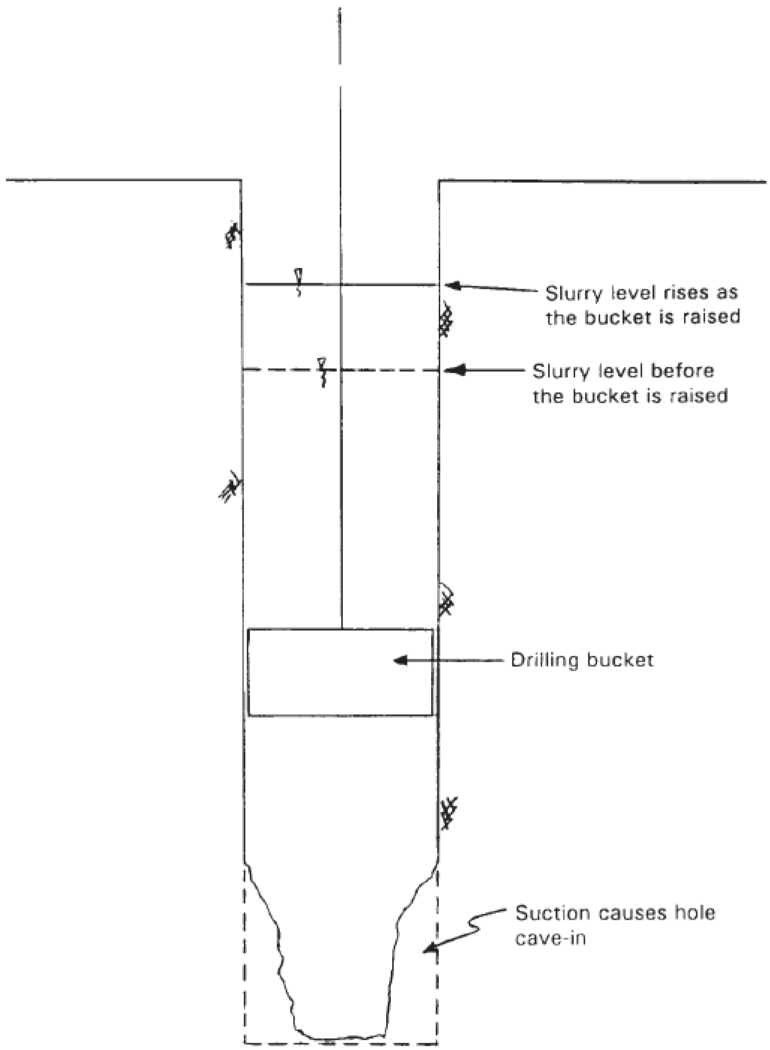


FIGURE 9-16 Hole collapse induced by pressure changes

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 9-17). 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 9-18). 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.

FIGURE 9-17 Cleanout bucket

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.

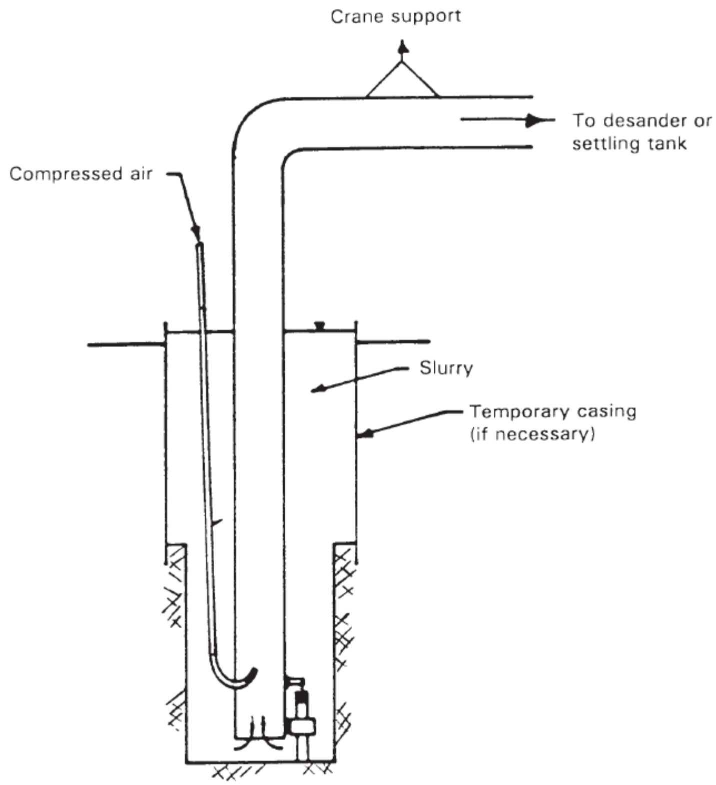


FIGURE 9-18 Airlift schematic

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 9-19). 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 – full-length – a temporary casing in the drilled hole and use the water as the drilling slurry in order to avoid a quick condition at the bottom of the drilled hole.

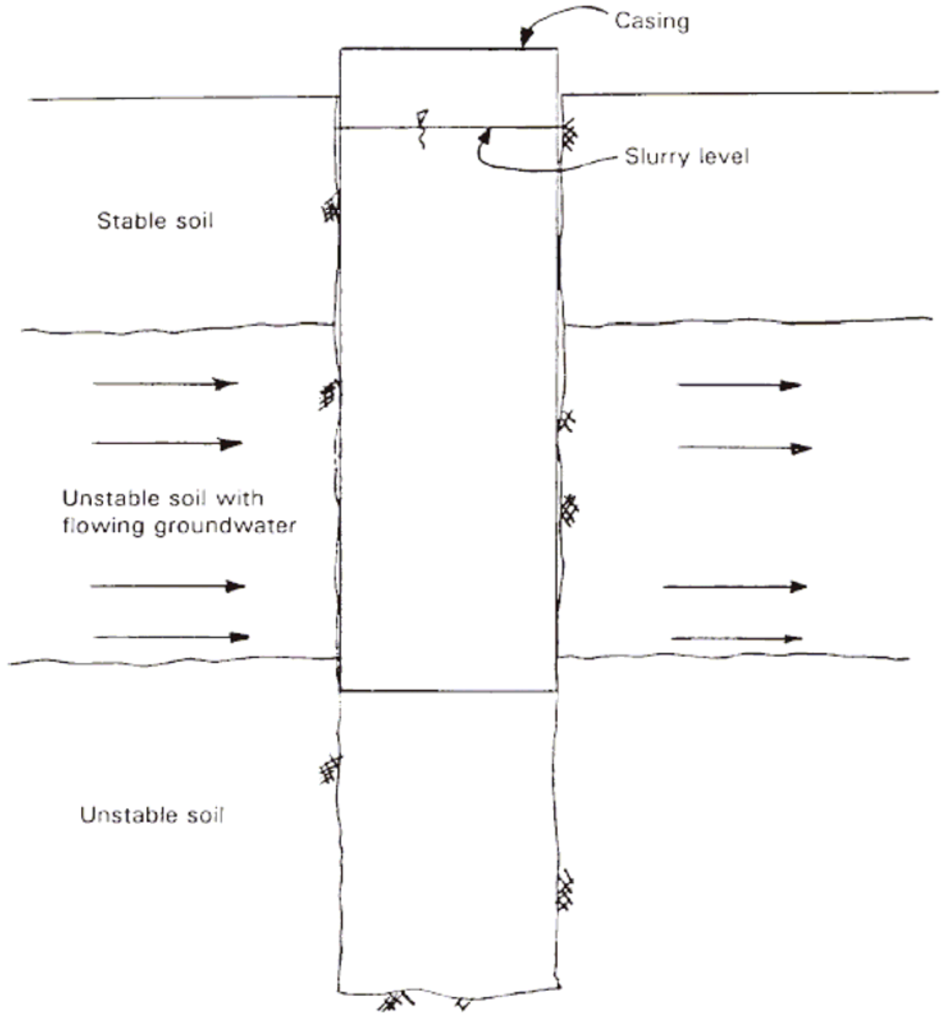


FIGURE 9-19 Use of casing

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.

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 tubes, 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 Section 49-4.03 of the Standard Specifications 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.

Concrete Compressive Strength and Consistency Requirements

Before any pile construction work using the slurry displacement method can begin, the Contractor shall demonstrate the concrete mix design can meet the required compressive strength requirements and consistency requirements. This is accomplished by producing a concrete test batch. The concrete test batch must demonstrate the proposed concrete mix design achieves the specified nominal penetration at the time of placement. For piles where the concrete placement operation is expected to be 2 hours or less, the test batch shall demonstrate that the proposed concrete mix design achieves either a penetration of at least 2 inches or a slump of at least 5 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 shall demonstrate that the proposed concrete mix design achieves either a penetration of at least 2 inches or a slump of at least 5 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, 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 give the Engineer and the Contractor the opportunity to observe how the concrete mix will behave before it is used.

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.

Mineral

For mineral slurries, samples shall 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 undergo either recirculation or continuous agitation in the drilled hole. The Contractor must address which method of agitation will be used in the pile placement 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 shall be taken from the slurry plant storage tank and the bottom of the drilled hole. As the mineral slurry is recirculated and cleaned, samples shall 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 shall be taken at the mid-height and at the bottom of the drilled hole. As the mineral slurry is continuously agitated, samples shall be taken every two hours for testing. If the samples at the two locations do not have consistent test values, the mineral slurry shall 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 is required to 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 recleaned 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 shall be 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 shall be 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.

Synthetic

For synthetic slurries, sampling for testing shall be conducted before, during, and after the drilling operation, and as necessary to verify and control the physical properties of the slurry. Samples shall be taken at the mid-height and at the bottom of the drilled hole. Once the drilling operation has been completed, additional samples for testing shall 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 unagitated 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 recleaned 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 shall be 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 shall be 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.

Pile Acceptance Testing Access Requirements

During pile construction work, the contract specifications require the installation of inspection tubes at specific intervals around the perimeter of the pile bar reinforcement cage. This is necessary to provide access for acceptance testing.

Pile Concrete Placement Requirements

During pile construction work, the contract specifications require that concrete shall 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 require the concrete mix design to meet the trial batch requirements for compressive strength concrete. These requirements are described in the contract special provisions. The concrete mix must contain at least 675 pounds of cementitious material per cubic yard. It is also important to compare the maximum aggregate size in the concrete mix design to the bar reinforcement spacing. The bar spacing should be no less than five (5) times the maximum aggregate size and preferably larger than five (5) inches. The Project 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 shall 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 excess bleed water due to the high fluidity of the concrete.

The method of concrete placement should not permit 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 shall 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 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.

When concrete placement begins, the tip of the concrete placement tube shall 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 shall 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 should be cleaned, capped, and pushed at least 10 feet into the concrete head before restarting concrete placement. 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.

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.

Inspection and Contract Administration

The reader is advised to review this section in Chapter 6 of this manual. 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 CIDH piles constructed using the slurry displacement method. This section outlines the additional requirements for CIDH piles constructed using the slurry displacement method.

The specifications require the Contractor to submit to the Engineer a Pile Placement Plan for review and approval. The Pile Placement Plan should provide sufficient detail for the Engineer to grasp the means, methods and materials the Contractor plans to use to successfully complete pile placement. Typical requirements include those listed in Chapter 6 of this manual, as well as additional requirements including the following:

ITEM	PILE PLACEMENT PLAN REQUIREMENT & REASONING
1	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. ***Reasoning:*** 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.
2	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. ***Reasoning:*** 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.
3	Suppliers' test reports on the physical and chemical properties of the slurry and any proposed slurry chemical additives, including Material Safety Data Sheet. ***Reasoning:*** 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.
4	Slurry testing equipment and procedures. ***Reasoning:*** 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 specifications.
5	Methods of removal and disposal of excavation, slurry, and contaminated concrete, including removal rates. ***Reasoning:*** 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

ITEM	PILE PLACEMENT PLAN REQUIREMENT & REASONING
	with Section 7-1.13 of the Standard Specifications.
6	Methods and equipment for slurry agitating, recirculating, and cleaning. ***Reasoning:*** This gives the Engineer advance knowledge of the means the Contractor proposes to use for mixing, circulating, cleaning and reusing the slurry. This is especially important if the Contractor proposes to use mineral slurry.

In order to facilitate pile testing, the contract specifications require the installation of inspection tubes (Figure 9-20). Before the cage is placed in the drilled hole, the Engineer should verify that these tubes 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 9-21 shows a typical inspection tube layout and spacing pattern within the pile bar reinforcement cage. These tubes must be placed in a straight alignment, securely fastened in place, and be watertight. These tubes 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 tubes are not placed in a straight alignment or are not securely fastened, the test probe will not fit in the inspection tube. One way of testing the tube 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 tube prior to concrete placement to ensure that the inspection tubes remain straight during and after concrete placement. . Inspection tubes need to be filled with water prior to the start of concrete placement. The reason for this is to prevent the inspection tube from separating from the pile concrete (debonding) or overheating during the curing process. This helps keep the inspection tube intact so that it can be used for crosshole sonic logging at a later point if necessary. Once the inspection tube 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 specifications also require the Contractor to log the locations of any inspection tube couplers and submit the log to the Engineer. This is necessary because inspection tube 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.

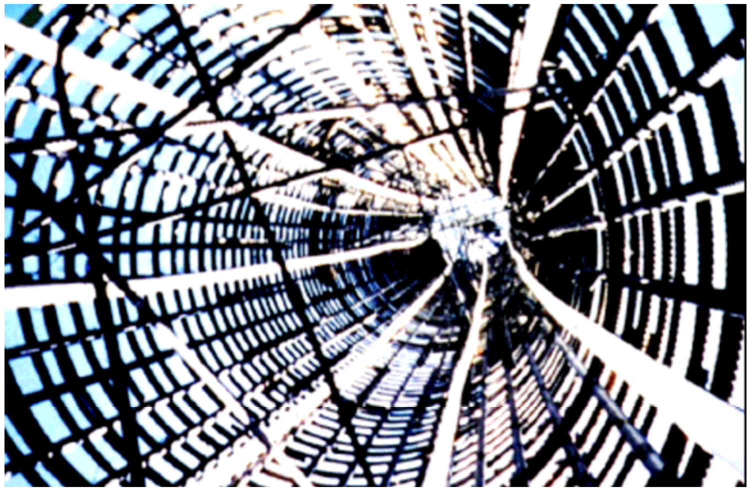


FIGURE 9-20 Inspection tubes

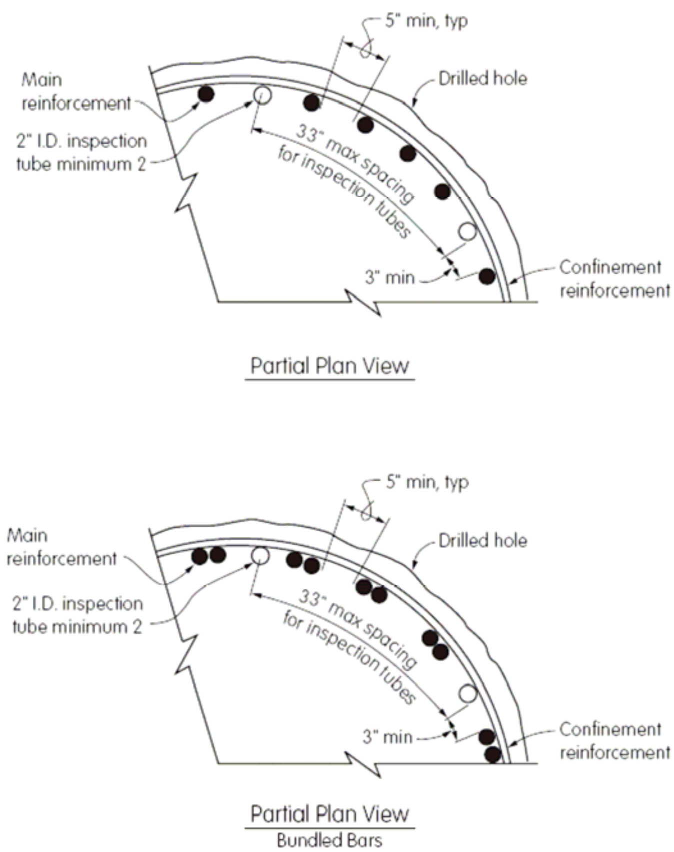


FIGURE 9-21 Location of inspection tubes within the pile

The Engineer shall notify the Foundation Testing Branch, Office of Geotechnical Support and Geotechnical Services as soon as the proposed pile concrete placement date is known, in accordance with the provisions of Bridge

Construction Memo 130-1.0. This places the Foundation Testing Branch 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 approved Pile Placement 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 job. 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 approved Pile Placement 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. Engineers 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 Placement 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.

Pile Acceptance Testing

After concrete placement and before acceptance testing is performed, the inspection tubes shall be checked by the Contractor for blockages and straightness with a dummy probe that is the same size and shape as the gamma ray scattering test probe in accordance with the contract specifications. The contract specifications allow the Contractor to use two different sized dummy probes to test the inspection tubes for blockages and straightness because there are currently two different sized Gamma-Gamma Logging (GGL) test probes in use. Inspection tubes that cannot accept either of the dummy probes shall 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 DES CIDH Pile Committee before any coring is performed. The reason the inspection tubes must be refilled with water is as discussed in the previous Section it is essential to keep the PVC tubes filled with water during the entire curing process to reduce debonding problems.

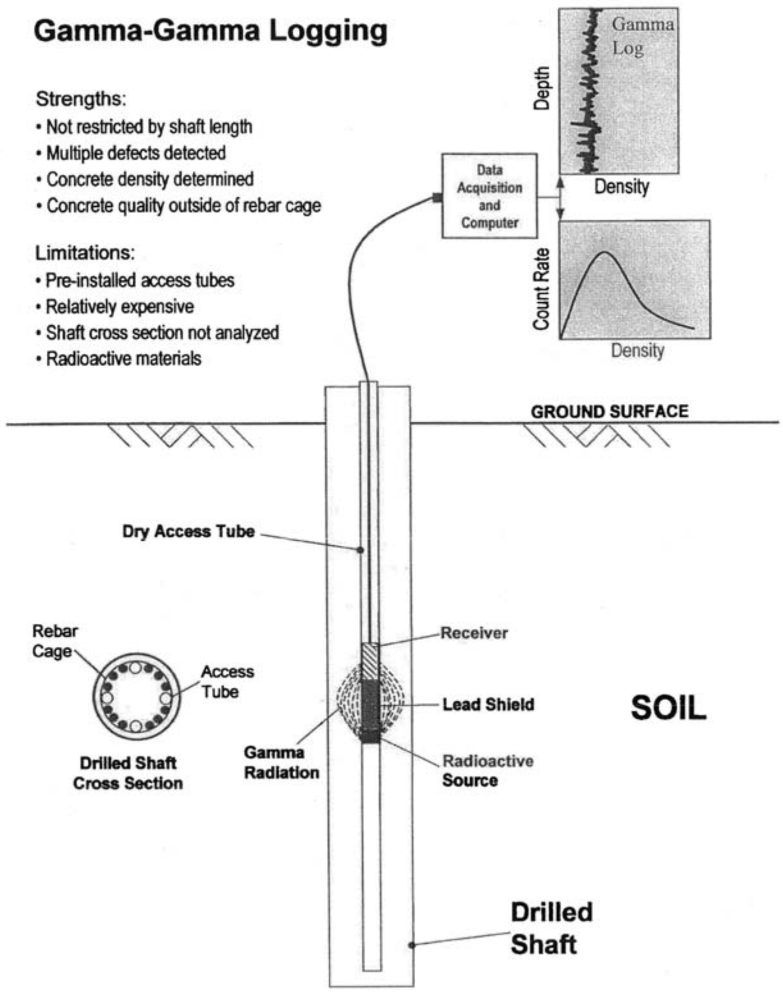


FIGURE 9-22 Gamma-gamma logging test schematic

Determining the soundness of slurry displacement piles is of understandable concern. 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 tubes 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 9-22).

The contract specifications state that the Gamma-Gamma Logging (GGL) method of testing piles constructed using the slurry displacement method will be used to determine acceptance of the pile, in accordance with the provisions of California Test Method 233.

In Gamma-Gamma Logging (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 more dense the material. The nuclear probe used in these tests contains a source which is relatively weak - a plus, considering the precautions that would otherwise have to be taken - and its effective range of sensitivity is limited to a 3 inch radius of concrete around the inspection tube. 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 tubes for a 100-foot length pile.

The contract specifications also state that crosshole sonic logging or other means of inspection may be used to perform acceptance testing. Typically, crosshole sonic logging or other means of inspection are used to complement the results of the Gamma-Gamma Logging (GGL) test and are only performed after gamma ray scattering 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 the Foundation Testing Branch, Office of Geotechnical Support, Geotechnical Services, or by consultant personnel under the auspices of the Foundation Testing Branch. 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.

Further information on pile acceptance testing may be found at the Foundation Testing Branch web page, located at
http://www.dot.ca.gov/hq/esc/geotech/ft/gamma.htm

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

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 drilling 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 9-23(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 be displaced by the fluid concrete as shown on Figure 9-23(b). These materials can also fall out of suspension and settle onto the head of concrete during concrete placement, become enveloped by the concrete, and attach to the pile bar reinforcement cage or the sides of the drilled hole as previously described. These deposits will register on the pile testing results 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.

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 9-24. 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 9-25. 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.

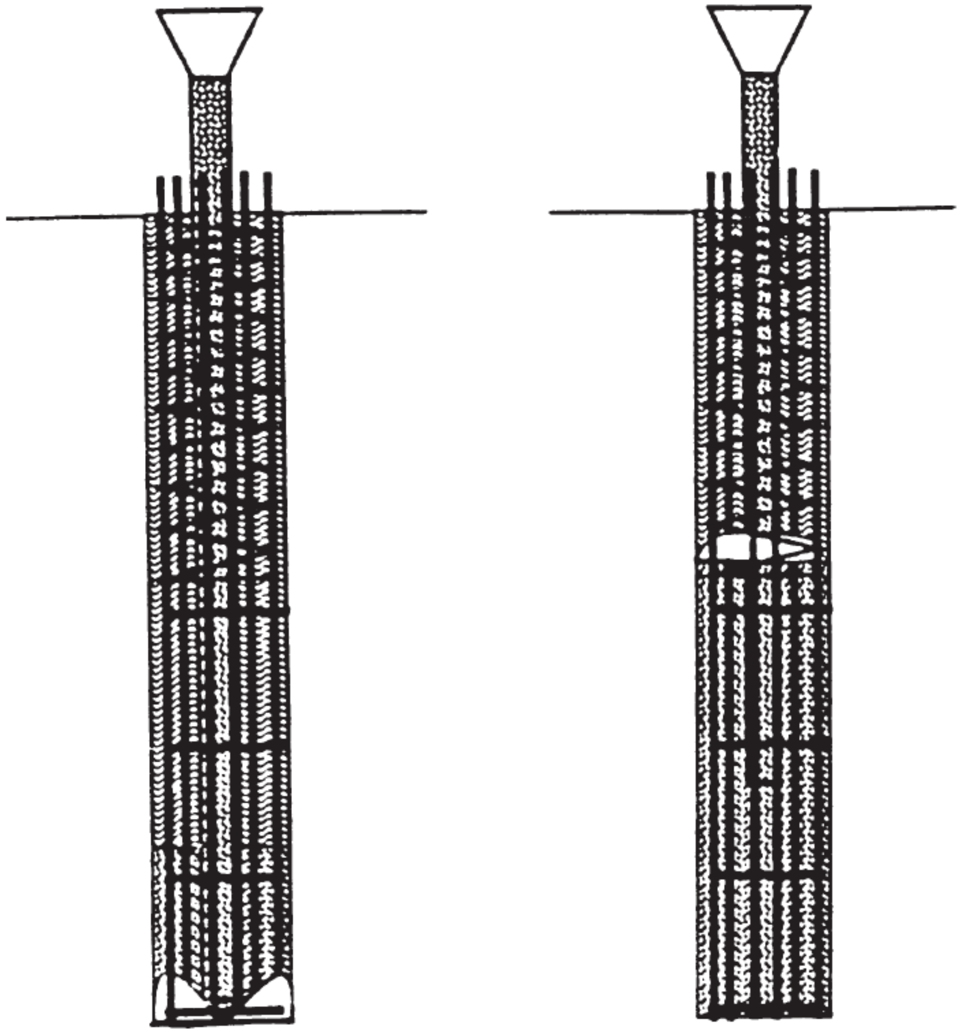


FIGURE 9-23 (a) (b) Defects from settled materials

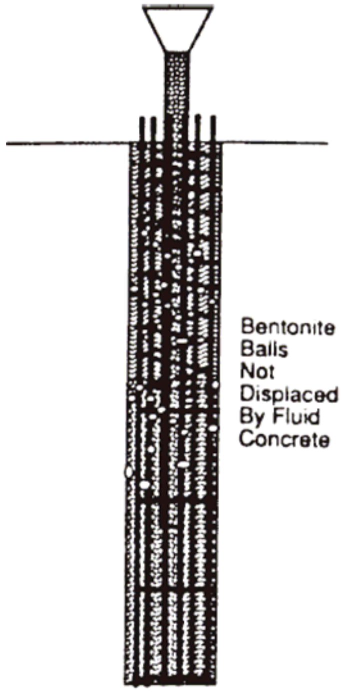


FIGURE 9-24
Defect from improperly mixed mineral slurry

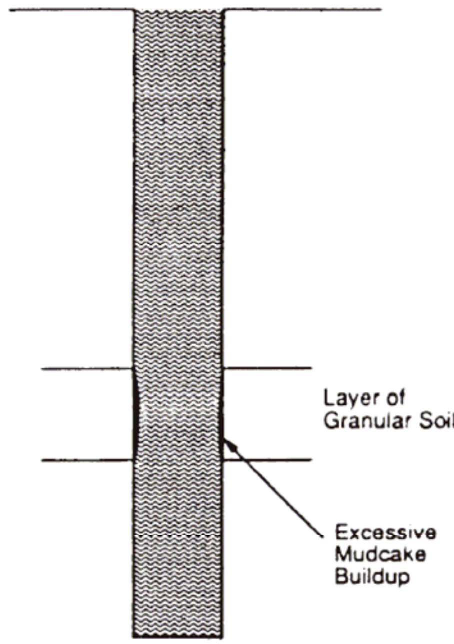


FIGURE 9-25
Defect from excess filter cake buildup

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 9-26). 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.

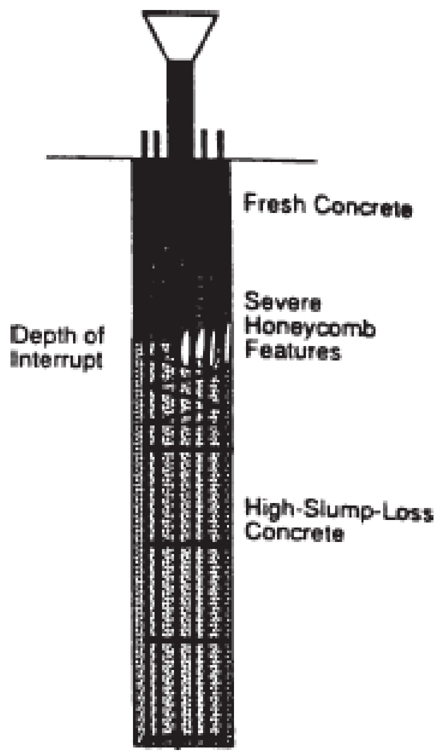


FIGURE 9-26 Defect from concrete placement problems

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 Foundation Testing Branch recommends rejection of the pile and the Engineer rejects the pile, the Contractor shall be informed in writing that the pile is rejected and given a copy of the test results. The contract specifications also require that the placement of concrete under drilling slurry be suspended until written modifications to the method of pile construction are submitted to and approved by the Engineer. This is to prevent additional failures due to the method of pile construction.

Pile Mitigation and Acceptance

What Happens When a Pile is Rejected

Once a pile has been rejected, 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. For whatever solution the Contractor proposes, additional investigation will be necessary to determine the nature and extent of the defect.

When a pile has been rejected, the Engineer should confer with the Foundation Testing Branch and decide if the Foundation Testing Branch will perform crosshole sonic logging on the rejected pile. 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.

Pile Mitigation Methods – Repairs, Replace, Supplement

There are several ways to mitigate a pile once it has been determined to have anomalies and been rejected. The pile can be repaired, replaced or supplemented in some way. The following sub-sections address how to take corrective action on a rejected pile.

REPAIRS

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. 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 shall consult with the Project Geotechnical Professional to assess the effect of accessing the defect upon the skin friction capacity of the pile.

Grouting Repair

Grouting repairs are used to mitigate defective concrete within the pile. These repairs 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 tubes. Generally, the inspection tube is removed at the defective area using a high-pressure water jet, which cuts the inspection tube into small pieces that are then flushed out through the top of the inspection tube. Once the inspection tube 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 tubes may be observed, meaning the water injected into one inspection tube may return to the ground surface through adjacent inspection tubes. 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.

Up to this point, these operations are occasionally performed before submittal of a Pile Mitigation Plan to give the Contractor information on the assessment of the nature of the defective area and which repair methods are appropriate. However, if the Pile Mitigation Plan is broad enough in scope to handle multiple grouting repair scenarios, these operations can be performed during the actual pile mitigation work.

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 tube 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 defective area. High-pressure grout injection is performed, usually through one of the inspection tubes, 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 tube. For this reason, permeation grouting is not recommended for repair of defects completely within the pile.

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 tubes. First, the inspection tube 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 <u>not</u> 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.

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 tubes. Generally, only the bottom of the inspection tube is removed and the area below the inspection tube 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 tube.

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

Supplemental and Replacement Piles

Occasionally, piles can be so riddled with defects that repair of the pile is not feasible. In this case, supplemental piles or pile replacement may be required. 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.

Pile Mitigation Plan Development and Approval Procedures

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

Development and review of the Pile Mitigation Plan is a shared responsibility between the Engineer, the Contractor, and the Division of Engineering Services (DES) Pile Mitigation Plan Review Committee.

Responsibilities of the Engineer

The Engineer is responsible for the following:

ITEM	RESPONSIBILITY
1	Arranging for acceptance testing with the Foundation Testing Branch. Based on the results of acceptance testing, accept or reject the pile and notify the Contractor in

ITEM	RESPONSIBILITY
	writing and supply the Contractor with a copy of the test results.
2	Once the pile has been rejected, determine in consultation with the Foundation Testing Branch whether additional acceptance testing will be performed. If additional acceptance testing is performed, notify the Contractor in writing and supply the Contractor with a copy of the test results.
3a	Discuss whether the pile requires mitigation for structural, geotechnical, or corrosion reasons with the Project Designer, the Project Geotechnical Professional, or the Corrosion Engineer. If the pile requires mitigation, use the Pile Design Data form provided with the test results to collect design information on what the capacity of the repaired pile needs to be. If the pile requires mitigation, discuss with the Project Designer, the Project Geotechnical Professional, and the Corrosion Engineer and come to a consensus on whether the pile can be repaired or must be supplemented or replaced.
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 mitigation, schedule and conduct a Repair Feasibility meeting with the Contractor as described in the contract specifications.
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 DES Pile Mitigation Plan Review Committee for technical review.
6	Upon the recommendation of the DES Pile Mitigation Plan Review Committee, either return the Pile Mitigation Plan to the Contractor for resubmittal or approve the Pile Mitigation Plan.

Responsibilities of the Contractor

The Contractor is responsible for developing and submitting the Pile Mitigation Plan to the Engineer for review and approval. 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 Repair Feasibility Meeting. Pile Mitigation Plans must contain the following information:

ITEM	REQUIREMENT & REASONING
1	The designation and location of the pile addressed by the mitigation plan. ***Reason:*** Self-explanatory.
2	A 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 descriptions of the mitigation work to be performed, including

ITEM	REQUIREMENT & REASONING
	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.
4	An 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	A list of affected facilities, if any, with methods and equipment for 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	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	A list of materials, with quantity estimates, and personnel, with qualifications, to be used to perform 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 licensed as a Civil Engineer by the State of California. ***Reason:*** This ensures that the Pile Mitigation Plan is developed or at least reviewed by an engineer. This is necessary to ensure that the structural, geotechnical, and corrosion design requirements of the rejected pile are met.
10	For 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.

ITEM	REQUIREMENT & REASONING
11	For piles to be repaired, provisions for access for additional pile testing if required by the Engineer. ***Reason:*** Generally, pile mitigation methods utilize the existing inspection tubes for accessing the defective zone. This is almost always the case for grout repairs. The repair generally renders the inspection tube unusable for the purposes of additional acceptance testing. Should the pile require additional acceptance testing after mitigation work has been performed; the Contractor must address how to preserve the existing inspection tubes or provide new access, usually new-cored holes.
12	For piles to be replaced or supplemented, the proposed location and size of additional piling. ***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 piles to be replaced or supplemented, structural details and calculations for any modification to the structure to accommodate the replacement or supplemental piling. ***Reason:*** See Item 12 above.

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 approved these plans for statewide use. Engineers should expect to receive a Standard CIDH Pile Anomaly Mitigation plan if the drilling contractor is an ADSC member. The plan will require a cover letter 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 approve the Pile Mitigation Plan.

To aid the Engineer, a copy of these Caltrans approved, standard mitigation plans can be obtained by contacting the Offices of Structure Construction in Sacramento or accessing its intranet website at http://onramp.dot.ca.gov/hq/oscnet/.

Responsibilities of the DES Pile Mitigation Plan Review Committee

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

ITEM	RESPONSIBILITY
1	Provide advice to the Engineer, Project Designer, Project Geotechnical Professional, and the Corrosion Engineer regarding pile mitigation procedures and methods.
2	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 approved or needs to be returned to the Contractor for correction and resubmittal.

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

Pile Mitigation Field Procedures and Pile Acceptance

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

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 approved Pile Mitigation Plan. Evaluation of the acceptability of the pile mitigation work is dependent upon whether the procedures described in the approved 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 approval.

For grouting repairs, the Contractor should monitor and record observations of inspection tube 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 tube 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 approved 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 approved Pile Mitigation Plan.

Procedures For Approving the Pile Mitigation Work Performed in the Field and Pile Acceptance

The approved 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 approved Pile Mitigation Plan. However, there are circumstances when the pile must be retested. Procedures for access for retesting are provided in the approved Pile Mitigation Plan.

For all types of pile mitigation, once the mitigation work is complete, the Contractor is required to submit a Mitigation Report to the Engineer for review and approval. 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 approved 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 tubes are grouted and the pile can be accepted.

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 of this manual. However, there is one additional item that requires further attention; and that is the drilling slurry itself.

Some of the components of drilling slurries, especially chemical additives, are considered to be 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 for the pile placement 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.

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.

CHAPTER 10 Pier Columns

Description

Pier columns are an extension of the pier to a planned elevation in bedrock material and are usually the same size, or slightly larger, than the 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, December 2000, Section 3-1 Deep Foundations*, "Pier Columns" on page 6:

> "Pier columns are utilized when the presence of rock precludes conventional drilling equipment. Excavation by hand, blasting, and mechanical/chemical splitting are some methods used in hard rock."

Pier column excavation is considerably more expensive than conventional auger drilling and the pay limits must be clearly defined. The pier column cutoff elevation and tip elevation (upper and lower limits of the hard material) should be shown in the Pile Data Table. The pay limits for Structure Excavation (Pier Column) and Structure Concrete (Pier Column) shall be shown on the plans. See Appendix D.

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)*, April 2005, "Pile Shaft Design"* Chapter 12. Also, Federal Highway Administration (FHWA) has useful information on drilled shafts.

Specifications

The special provisions will contain a great deal of information regarding pier columns and should be reviewed along with the contract plans and Standard Specifications prior to the start of work. 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 shall 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 columns(s) so that these excavations do not extend below the top of the neat line areas. The contract plans will 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.

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 will 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 are somewhat new to the Department and may also be used to perform this work. For additional information on these tools refer to Chapter 6 of this Manual.

Excavation

One of the first orders of work, after access roads are constructed to the pier 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 also be used for pier construction.

After establishing survey control points, excavation operations begin. Usually, soft material is excavated by conventional methods, such as a Gradall, flight auger, clambucket 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 dimensions). 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. is 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. Handwork to some degree is required at the bottom of all pier columns.

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.

ITEM	POTENTIAL PROBLEM
Alignment	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.
Surveying	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.
Access	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.
Blasting	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 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

ITEM	POTENTIAL PROBLEM
	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 not our responsibility, it falls under the umbrella of Geotechnical Services who should be consulted whenever blasting is contemplated. If you have any questions on the responsibility of Caltrans in regards to blasting, contact the Caltrans Headquarters Construction Safety Officer. See Appendix D – 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 Section 5-1.02A of the Standard 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.

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 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. Guy wire plans will be required for supporting column forms and column 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.

CHAPTER

11 Tiebacks, Tiedowns, & Soil Nails

Introduction

Chapter 2 “Type Selection” classifies tiebacks and tiedowns as special case foundations. They 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. Tiedowns, sometimes referred to as Tension Piles, are used generally for seismic retrofitting or existing footings where uplift and overturning must be prevented.

Tiebacks

Tiebacks are used in both temporary and permanent structures. The use of tiebacks 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 tiebacks.

Components

Tiebacks are constructed by drilling holes at a slight angle (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.

The following list describes various tieback components:

COMPONENT	DESCRIPTION
Prestressing Steel – Support Member	This transfers load from the wall reaction to the anchor zone and is generally a prestress rod or strand.
Bond Length	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.
Unbonded Length	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.
Anchorage	This consists of a plate and anchor head or threaded nut and permits stressing and lock-off of the prestressing steel.
Grout	This provides corrosion protection as well as the medium to transfer load from the prestressing steel to the soil or rock.

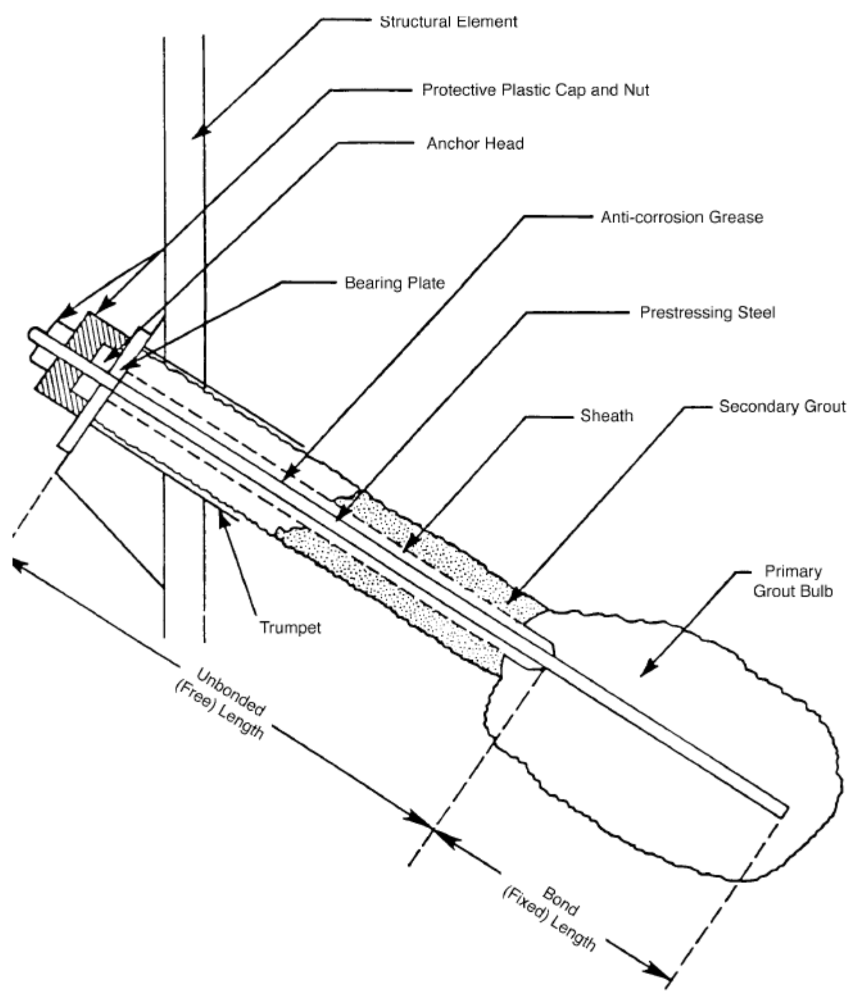


FIGURE 11-1 Tieback detail

In addition to enabling the construction of higher/taller walls and deeper excavations, tiebacks 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 tieback system that has been pre-approved by the Department and conforms to the design requirements shown on the plans and meets/exceeds the testing requirements specified in the contract documents. The Contractor has the option of choosing which system will be installed. Tieback shoring designs are often proprietary and require sophisticated engineering techniques and the calculations submitted by Contractors and Consultants. The designed bonded length is based onsite specific soil parameters/mechanical properties. The tieback shop drawings and design calculations are submitted to the Office of Structure Design, Documents Unit, in Sacramento for distribution, review, and approval. The project engineer in Structure Design, geoprofessional in Geotechnical Services, the staff specialist for Earth Retaining Systems in Structure Design Services and Earthquake Engineering in Sacramento, the DES Prestressing Committee and the Structure Construction field personnel all review the shop drawings. The project engineer approves 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 Office of Structure Construction Substructure Committee is also available to provide assistance.

Specifications for tieback anchors are generally found in the contract special provisions. Tieback anchors shall be installed in accordance with the manufacturer's recommendations. In case of a conflict between the manufacturer's recommendations and the special provisions, the special provisions shall prevail.

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 shall witness all performance and proof testing of the tiebacks.

Sequence of Construction

Sequence of tieback construction is as follows:

SEQUENCE	DESCRIPTION
1	Drill the holes to the required length and diameter.
2	Install the prestressing steel unit. (Strands or Bar)
3	Place primary grout.
4	Complete Performance and Proof Tests (refer to section on testing later in this chapter).
5	Lock-off and stress.
6	Place secondary grout.

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

Safety

Check the Contractor's construction sequence against the approved plans. As excavation proceeds from the top down, look for signs of failure in the lagging or changes in the soil strata.

Tieback systems use powerful hydraulic rams to prestress or post tension the system. The premise is the same as what is done to in prestressed bridges. Structure Construction employees should not stand behind the hydraulic ram or cross it while stressing is taking place. The Prestressing Manual and the OSC Code of Safe Practices should be consulted for additional safety considerations.

Tiedowns

Tiedown anchors, or tiedowns, are similar to tiebacks 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. Tiedowns 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.

An example of a prestressing bar tiedown anchor is shown in Figure 11-2.

The Contractor is responsible for providing the tiedown anchor system that conforms to the design requirements shown on the plans and the testing requirements specified in the contract documents. The option of choosing which system to be installed is left to the Contractor. After selecting a tiedown system, the Contractor sends the shop drawings and calculations to the Office of Structure Design, Documents Unit, in Sacramento for distribution, review, and approval similar to the process outlined above for tiebacks.

The record of readings from the Performance and Proof tests shall be documented by the Contractor and provided to the Engineer. Structure Construction field personnel shall witness all performance and proof testing of the tiebacks.

Specifications for tiedown anchors are generally found in the contract Special Provisions. Tiedown anchors shall be installed in accordance with the manufacturer's recommendations. In the case of a conflict between the manufacturer's recommendations and the special provisions, the special provisions shall prevail.

Sequence of Construction

Sequence of tiedown construction is as follows:

SEQUENCE	DESCRIPTION
1	Drill the hole the required depth and diameter.
2	Install the prestressing strands or bar.
3	Place primary grout.
4	Complete Performance and Proof Tests (refer to section on testing later in this chapter).
5	Lock-off and stress.
6	Place secondary grout.

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

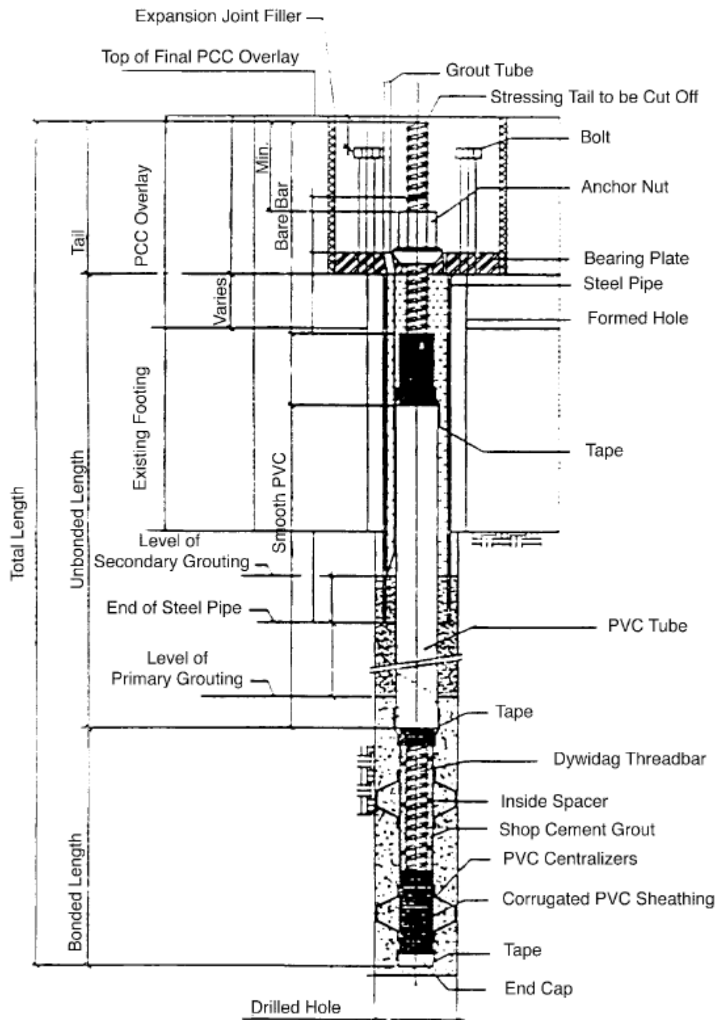


FIGURE 11-2 Tiedown anchor

Testing of Tieback and Tiedown Anchors

Both tiedowns and tiebacks 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 anchors. If the test results indicate that the anchors are not achieving capacity, additional monitoring and testing, as outlined in the contract special provisions, will be 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 project engineer. The specific requirements for testing will be provided in the contract special provisions, the following is a general explanation of the required tests.

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 that the residual movement of the anchor after stressing is within tolerable limits. As a minimum, the first two production anchors installed should be Performance tested. Do not wait until many anchors have been installed before testing the first two anchors 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 is installed. Each load increment or decrement shall be held constant for at least one minute or until measured deflection is negligible. The maximum load should generally be held for one hour to determine long-term creep susceptibility. As stated earlier, the contract plans and/or special provisions will specify the testing and acceptance criteria for each test and the number of Performance tests required at each location.

Proof Tests

A proof test involves incrementally loading a production anchor to verify that the design capacity can be safely carried and that the free length is as specified. The proof test is a single cycle test where the load is applied in increments until the specified maximum load value (150% of the design load) is reached. Each load shall be applied in less than one minute and held constant for at least one minute but not more than two minutes.

General Acceptance Criteria – Proof & Performance Tests

CRITERIA	PERMORMANCE TESTS
1	Achieve test results that indicate that the anchor is capable of supporting 150% of the design force for the anchor shown on the plans.
2	The measured elastic movement exceeds 0.80 of the theoretical elongation of the unbonded length plus the jacking length at the maximum test load.
3	The creep movement between one and 10 minutes is less than 0.04—inch.

CRITERIA	PROOF TESTS
1	Achieve test results that indicate that the anchor is capable of supporting 150% of the design force for the anchor shown on the plans.
2	The pattern of movements is similar to that of adjacent performance tested tiebacks.
3	The creep movement between one and 10 minutes is less than 0.04—inch.

The special provisions outline an acceptance criteria for these tests, however a performance tested or proof tested tieback which fails to meet the second 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.

General Construction Control

ITEM	DESCRIPTION
1	Mill certs should be provided for the steel tendons. a) Check the steel for damage. b) Ensure that grease completely fills the free length plastic tube. c) Securely tape the bottom of the free length. d) Compare the actual free length dimensions versus the dimension specified.
2	Double corrosion protection anchors should be completely fabricated before being delivered to the job site. Bar anchors are installed full-length into the hole. Record the actual free and bond length for each installed anchor.
3	Tendons shall 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 hole where incomplete grout cover will cause loss of capacity and future corrosion.
4	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.
5	Testing – check to ensure the tendon is concentrically located in the center hole 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.

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 tiebacks, 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 tieback 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:

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

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 steeply 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.

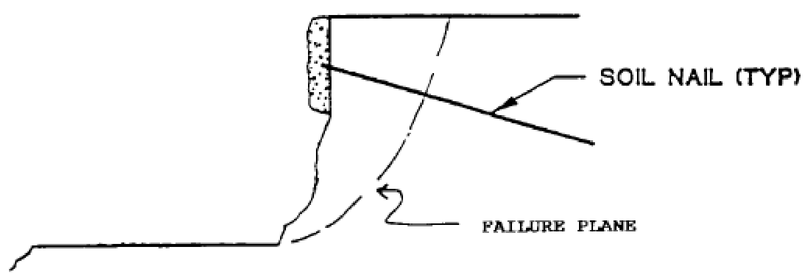


Figure 11-3 Soil nail schematic

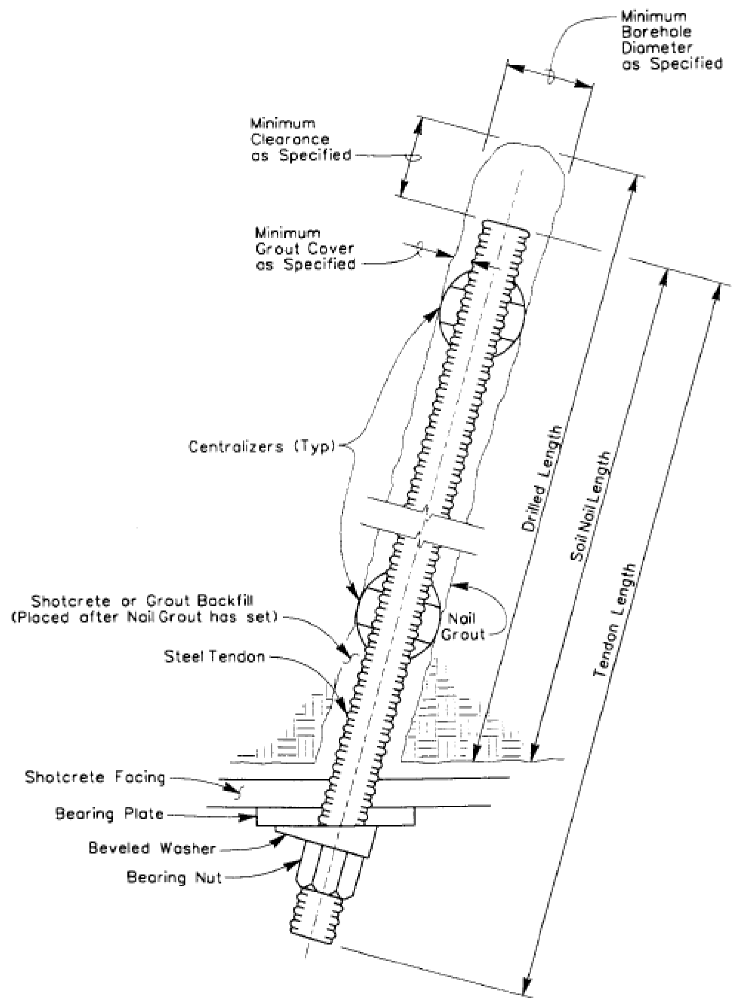


FIGURE 11-4 Soil nail details

Sequence of Construction

Soil Nail Wall Construction Sequence is as follows:

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

Engineer's Responsibility

The Structure Representative shall ensure that the soil nail wall is being built in accordance with the contract documents. The Department is responsible for reviewing and approving the shop drawings and construction details. The review process is similar to that of tiebacks and tiedowns. One important difference between tieback designs and those of soil nails is that of design responsibility. Tiebacks 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 shall 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 of all field activities.

A good reference for field inspectors is the Soil Nailing Field Inspectors Manual - Soil Nail Walls – Demonstration Project 103, Publication No. FHWA-SA-93-068, Federal Highway Adminstration, U. S. Department of Transportation, 1994, by James A. Porterfield, David M. Cotton, R. John Byrne.

Contractor's Responsibility

The Contractor is responsible for constructing the soil nail wall in accordance with the contract documents. 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 shall furnish a complete test result to the Engineer for each soil nail assembly tested.

Testing of Soil Nail Walls – Verification, Proof & Supplemental

The contract documents 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.

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 Project Engineer and shown on the plans. Refer to Figure 11-5 for a test nail detail.

Verification testing has two criteria the first is a creep test and the second is a maximum load test. They involve incrementally loading the test soil nail assembly to its design load, holding it for an hour and loading the nail to 150% of the design load. Movement of the soil nail end shall be 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 Special Provisions will outline acceptance criteria for the verification 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 any case, additional performance 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 performance test nails. This information should be provided to the Project Engineer and Geoprofessional to help resolve this issue.

Proof Testing

Proof testing is performed on production nails that are shown on the plans. In addition the Special Provision will indicate a specific number of proof tests to be performed at locations identified by the Engineer in the field. The testing means and methods as well as the acceptance criteria for these tests are different than those for performance tests and are outlined in the Special Provisions.

Supplemental Testing

Supplemental testing is done on a specified number of nails (up to one-half of the production nails) and is completed immediately after the completion of creep testing. The testing and acceptance criteria will be specified in the Special Provisions.

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 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.

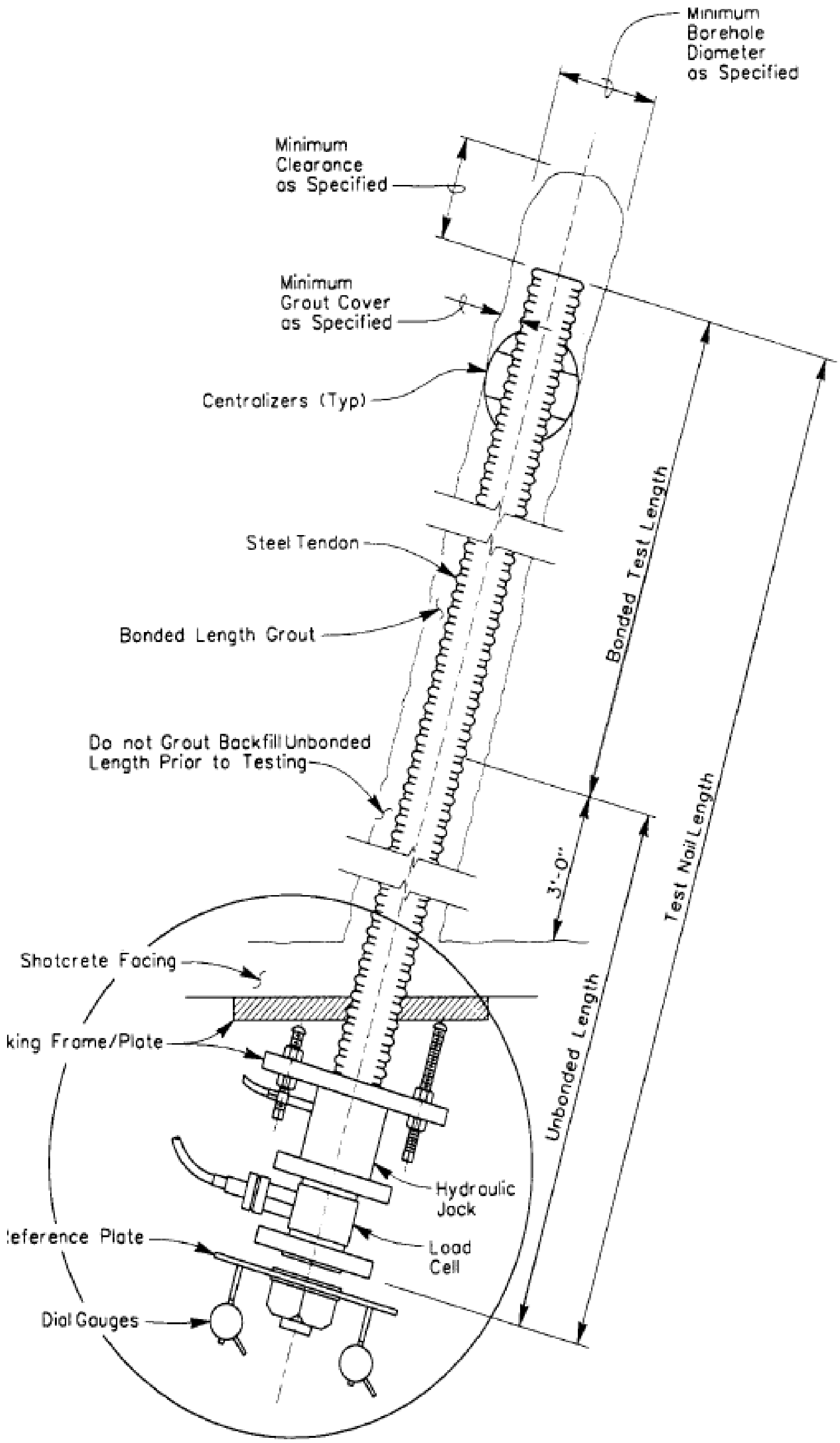


FIGURE 11-5 Verification/test nail detail

CHAPTER

12 Cofferdams and Seal Courses

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.

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" but it is suitable for only comparatively small excavations where there is no serious ground water problem.

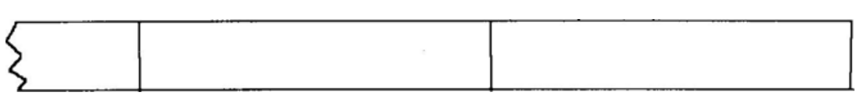


FIGURE 12-1 Single sheet piling

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.

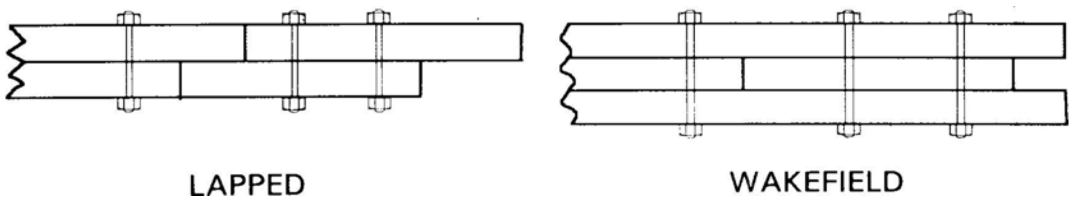


FIGURE 12-2 Lapped and wakefield sheet piling

"Tongue and groove" sheet piling is also used. This is made from a single piece of timber that is cut at the mill with a tongue and groove shape.

FIGURE 12-3 Tongue and groove wood sheet piling

Precast concrete sheet piles 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. The Department does not normally encounter pre-cast 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.

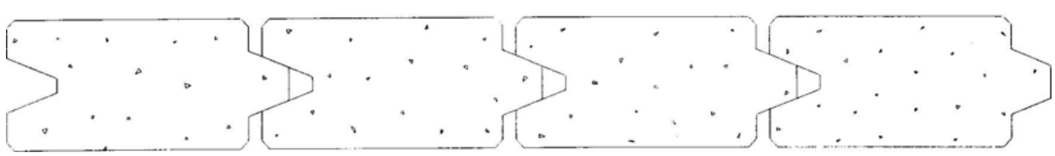


FIGURE 12-4 Concrete sheet piling

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.

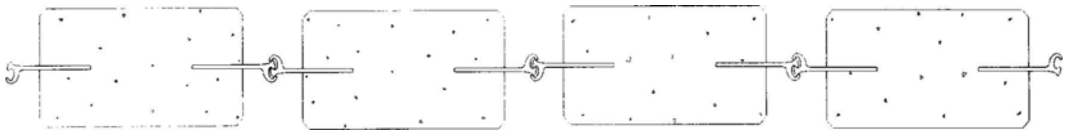


FIGURE 12-5 Concrete sheet piling with steel interlocks

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". 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 section steel sheet piling

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, where the center of the sheet is offset to provide a greater moment of inertia in the cross section. A "deep-arch" section 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 has a stiffness considerably greater than that of the "deep-arch" and is used in deeper excavations.

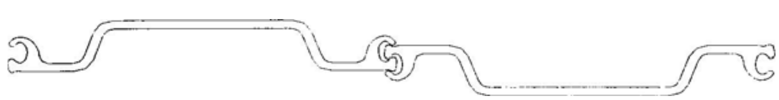


FIGURE 12-7 Arch-web steel sheet piling

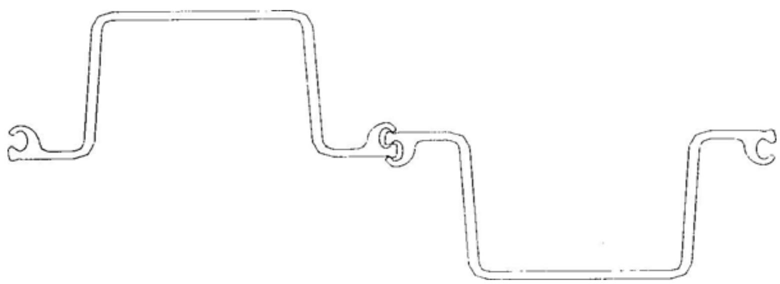


FIGURE 12-8 Deep-arch steel sheet piling

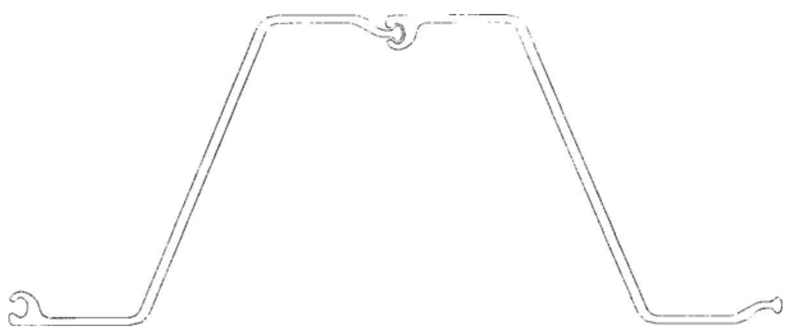


FIGURE 12-9 Z-section steel sheet piling

The choice of the type of steel sheet pile to be used on a given job 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 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. Tiebacks, sometimes prestressed, can be used in large land cofferdams where a system of cross bracing is impractical.

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;

and closed-end (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 will 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 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 may not be shown on the 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 shall be done in accordance with the provisions of Section 19-3.04 of the Standard Specifications.

Seal Course

Section 51-1.10 "Concrete Deposited Under Water" states that a seal course should be used when the Engineer determines that 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 and 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 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 Standard 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 130-17.0. Bottom of footing elevations should not be revised as a result of eliminating or revising seal courses unless shown on the plans or addressed in the special provisions.

Concrete Deposited Underwater (Tremie Placement Method)

The Tremie Placement Method is a name given to the method of placing concrete under water 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.

Briefly described, 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 assure success for this operation are:

FACTOR	DESCRIPTION
1	Tremie concrete shall have a penetration of between 3 and 4 inches.
2	Concrete shall contain a minimum of 675 pounds of cementitious material per cubic yard. (Standard Specifications - Section 90-1.01)
3	Concrete placement and the maneuvering of the tremie pipe must be done smoothly and deliberately.
4	Concrete delivery must be adequate and timely.
5	The concrete mix design should be geared to good flow characteristics.

Seal Course Inspection

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 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.

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 130-17.0 and Bridge Design Aid “Seal Course” included in Appendix I.

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 working drawings and calculations to the Engineer for approval in accordance with Sections 5-1.02 and 19-3.03 of the Standard

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. In addition, Section 19-3.04 of the Standard Specifications states the following: “If the contractor elects to use a concrete seal course … the provisions of the fourth paragraph and the first 2 sentences of the fifth paragraph of Section 51-1.10, "Concrete Deposited Under Water," shall not apply for spread footings and the entire Section 51-1.10 shall not apply to pile footings. The successful performance of the seals, if used, shall be solely the responsibility of the Contractor.”

Engineer’s Responsibility

The Engineer is responsible for performing an independent analysis, or check, of the contractor’s cofferdam and for approving the Contractor’s drawings. In situations where a seal course is shown on the 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 information in the following sections of the Standard Specifications: 5-1.02, 19-3.03, 19-3.04, 19-3.07, 19-3.08, 51-1.10, 51-1.22; and the following Bridge Construction Memos: 2-9.0 and 130-17.0.

Dewatering

Section 51-1.10 of the Standard Specifications requires 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 provisions 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 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.

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 job specific Code of Safe Practices should be consulted for specific requirements.

CHAPTER

13 Alternative Piles and Special Considerations

Introduction

Micropiles

The primary reference for this chapter is from Micropile Design and Construction Guidelines Implementation Manual, Publication No. FHWA-SA-97-070, Federal Highway Administration, U.S. Department of Transportation, June 2000, by Tom Armour, Paul Groneck, James Keeley, and Sunil Sharma.

Micropile Definition and Description

A micropile is a small-diameter (typically less than 300mm), drilled and grouted *replacement pile* that is typically reinforced. A micropile is constructed by drilling a borehole, placing reinforcement, and grouting the hole. Micropile construction uses similar equipment and techniques as tiebacks, tiedowns, and soil nails (Chapter 11). Many contractors who specialize in drilling and grouting, tiebacks, tiedowns, 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 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 there is little lateral resistance provided by these types of piles their use 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 and 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 all of the 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.

Applications

Micropiles are currently used in two general applications, (1) structural support and (2) in-situ reinforcement.

In-situ Reinforcement includes:
- Slope Stabilization and Earth Retention
- Ground Strengthening and Protection
- Settlement Reduction
- Structural Stability

Structural Support includes:
- Earth Retention
- Foundations for New Structures
- Seismic Retrofitting
- Underpinning of Existing Foundations

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/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.

Caltrans Applications

AASHTO will be adding a section on micropiles in the future. But while the rest of the country sees the value, Caltrans will limit the use of micropiles due to the lateral demand requirements. The lateral load capacity of micropiles is small as 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.

Caltrans is currently using micropiles for seismic retrofits, earth retention, and foundations for new structures (retaining/sound walls).

Seismic Retrofit

Caltrans has used micropiles for seismic retrofitting of existing highway bridge structures. The existing bridge foundations are retrofitted to increase the capacity so as to resist tension/uplift forces resulting from a seismic event.

A somewhat 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. See Appendix x.

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/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 micropiles may ultimately yield an improved earthquake mitigation foundation system.

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. 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 two #36 epoxy coated bundled rebar. See Appendix x.

Foundations for New Structures (Retaining Walls)

In 2007, construction started on a retaining wall on Rte 74 in District 12, Orange County. Micropiles support the retaining wall, concrete barrier slab, and concrete barrier. Tiebacks are also used to support the retaining wall.
See Appendix x.

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. 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.
See Appendix x

Construction and Contract Administration

The Contract Special Provisions will outline all the submittal requirements and construction requirements for micropiles. Depending on the project location, the design, and the contractor, different drilling and grouting techniques may be used. Per the special provisions, the contractor is required to submit to Caltrans for review and approval all micropile working drawings and a step-by-step procedure describing all aspects of pile installation. The Caltrans Structure Representative will coordinate with the Foundation Testing Branch (FTB) for any Caltrans required load tests. The special provisions may require performance tests to be performed and recorded by the contractor. 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.

Measurement and Payment

Per the Contract Special Provisions, micropiles will be measured and paid for by the meter. The contract price paid per meter for micropile shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in constructing micropiles, including protecting and monitoring existing culverts, drilling, providing temporary casings, double extra strong steel pipe, grout, grout socks, cutting tips, drill bits, pile anchorage, and disposing of materials resulting from pile installation, complete in place, as shown on the plans, as specified in the Standard Specifications and special provisions, and as directed by the Engineer.

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.

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 pile reinforcement and steel pipe is hoisted into place.

Changeable Message Signs

Changeable message signs (CMS) are typically large diameter Cast-In-Drilled-Hole (CIDH) pile foundations. Figure xx shows a 5-ft diameter pile with a minimum depth of 22-ft for CMS Model 500. Construction of CMS foundations is made difficult when groundwater is encountered. If there is groundwater, then the slurry displacement method is usually required (Chapter 9). The contract special provisions will outline all the requirements. Small-sized, inexperienced contractors may have difficulty meeting “wet method spec” submittal requirements and construction requirements. Structure Representatives need to thoroughly communicate all the requirements. The preconstruction meeting is a good forum to initially discuss slurry displacement requirements.

A Log of Test Borings (LOTB) might not be included in small CMS projects making it difficult to anticipate the presence of groundwater. A proactive Structure Representative can obtain LOTB as-builts from the nearest bridge structure location. The proactive Structure Representative should review the LOTB as-builts and share the information with the contractor. As-builts are available at District Headquarters and on-line on the intranet (Bridge Inspection Records Information System (BIRIS) and Document Retrieving System (DRS)).

Personnel safety must be enforced during drilling and excavating operations. Full body harness should 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.

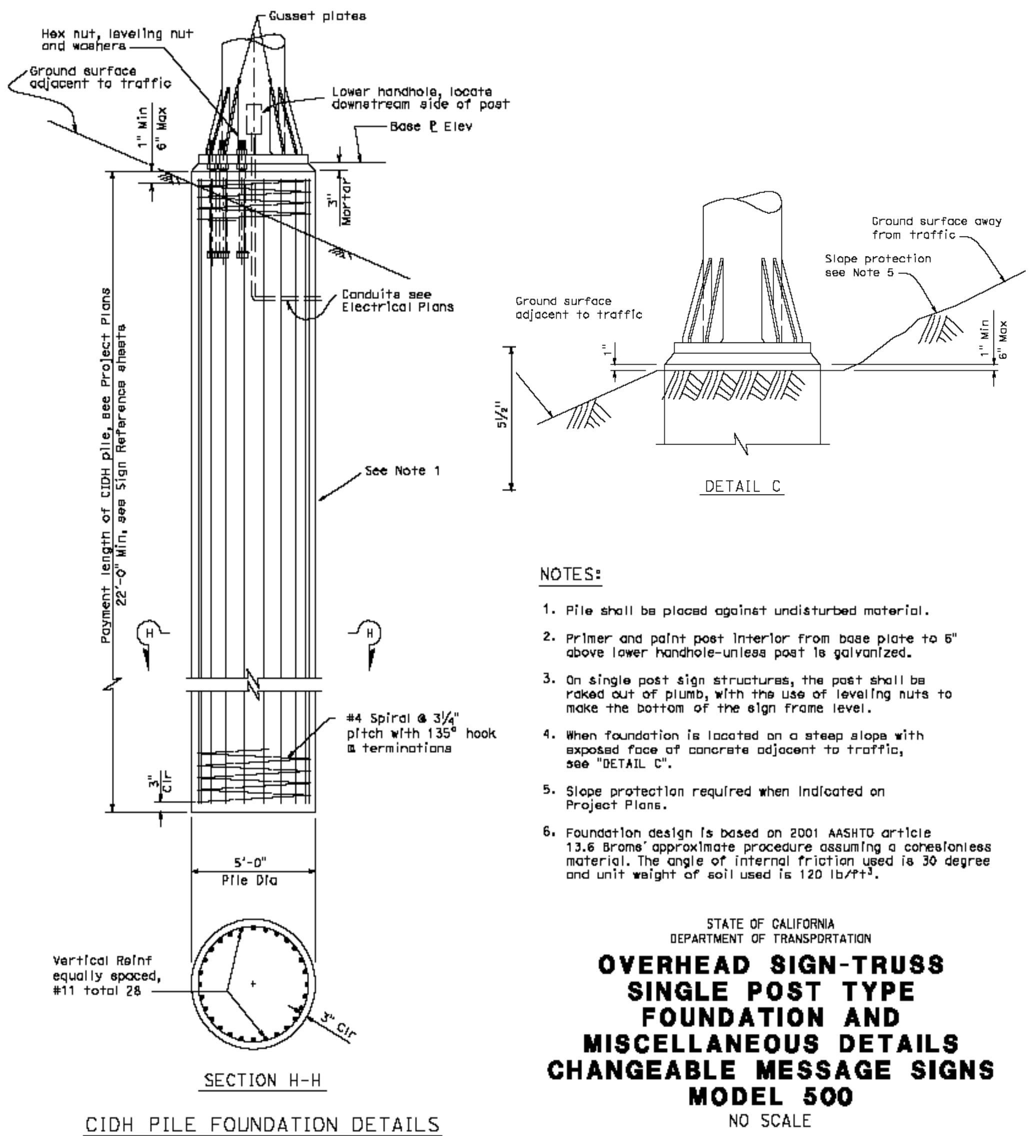


FIGURE 13-1 CMS details from 2006 Standard Plan S116

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