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Division of Structures

Foundation Manual

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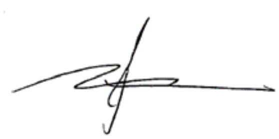
Special thanks to the Caltrans engineers who drafted the original 1984 Foundation Manual and to the Caltrans engineers who drafted the 1996 revision. Their vision, dedication, and research, produced a manual that has been used throughout the Department.

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.

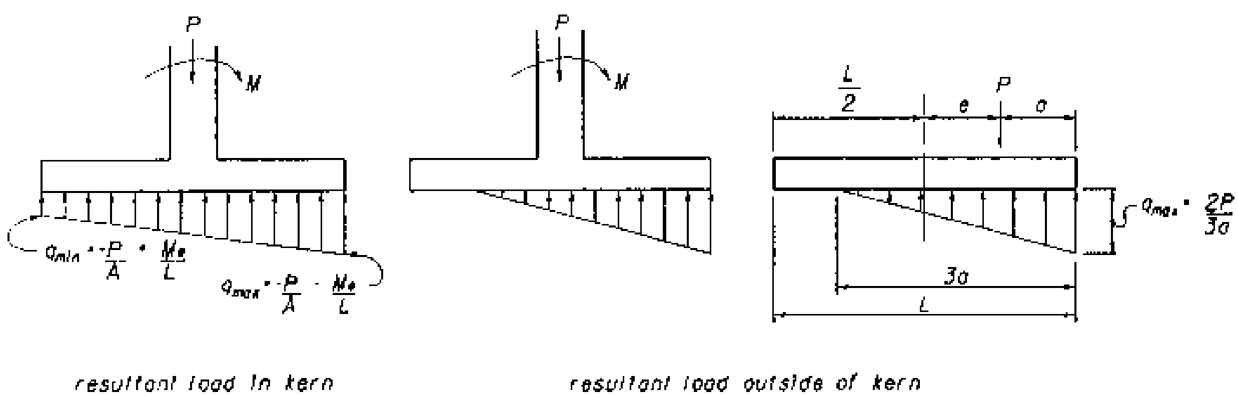


FIGURE 4-1 Loaded footing with moment

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.

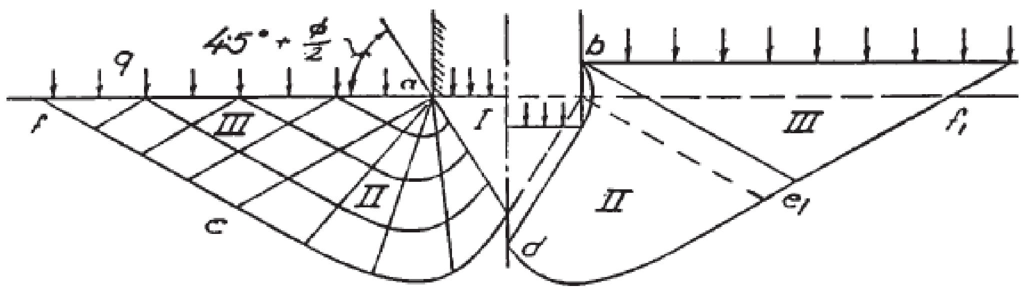


FIGURE 4-2 General shear failure concept

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.

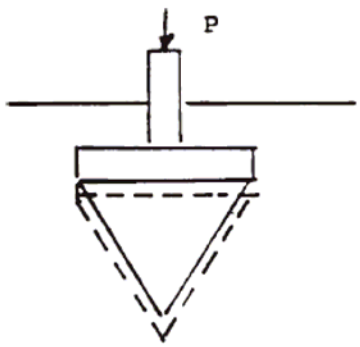


FIGURE 4-3
Punching shear failure

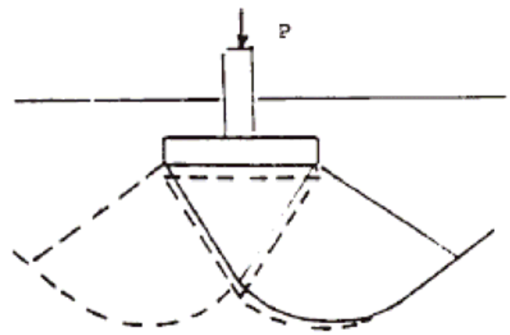


FIGURE 4-4
Local shear failure

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

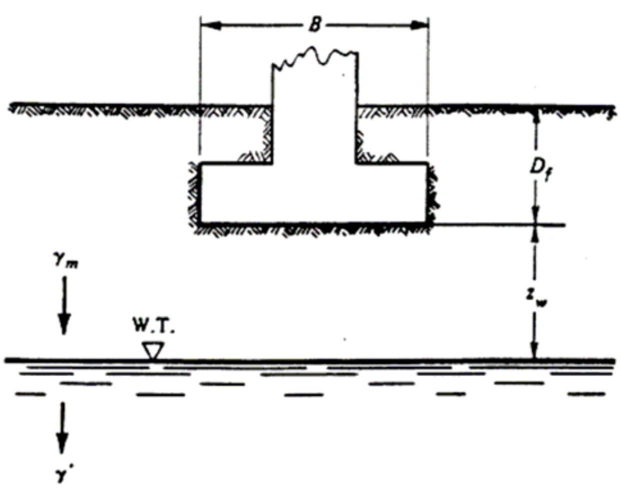
Several factors can affect the bearing capacity of a particular soil. They include soil type, relative density or consolidation, soil saturation and location of the water table and surcharge loads. These factors can act individually or in concert with each other to increase or decrease the bearing capacity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

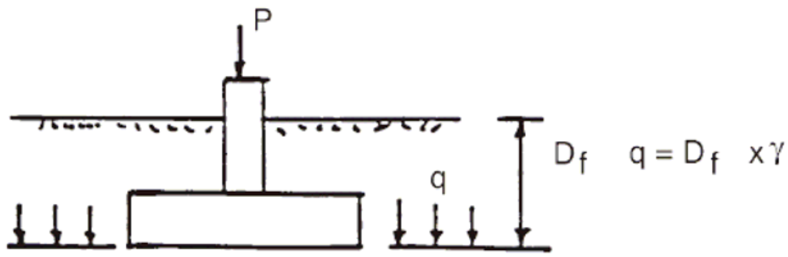


FIGURE 4-7 Surcharge load on soil

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.

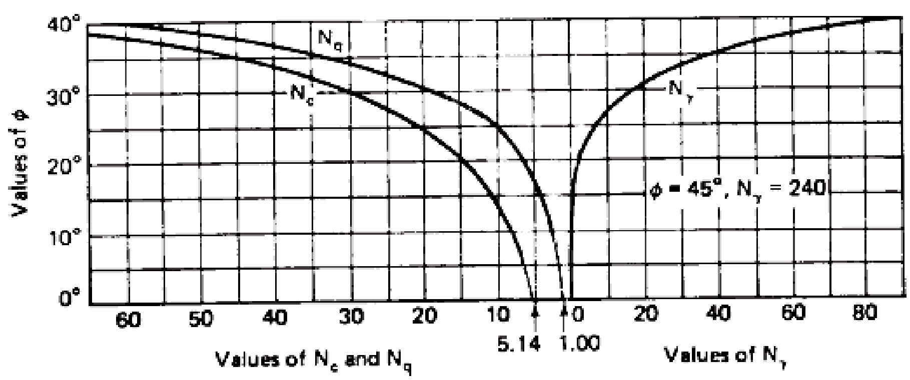


FIGURE 4-8 Relationship between ø and bearing capacity factors

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).

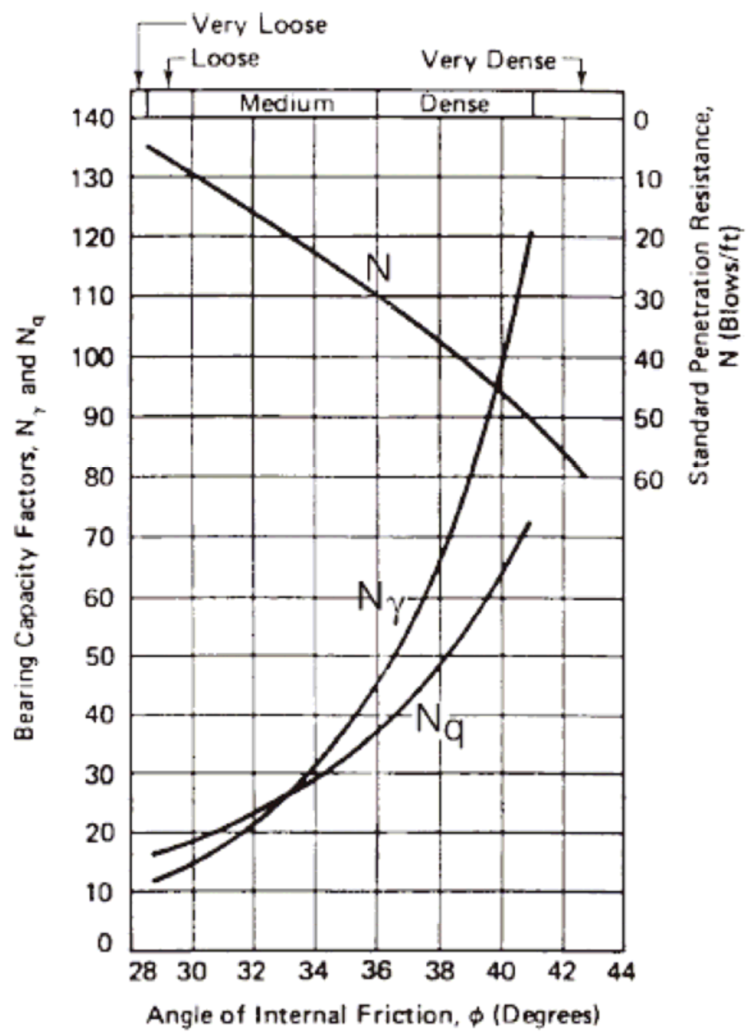


FIGURE 4-9 Relationship of bearing capacity factors to ø and N (standard penetration resistance) for cohesionless soils

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

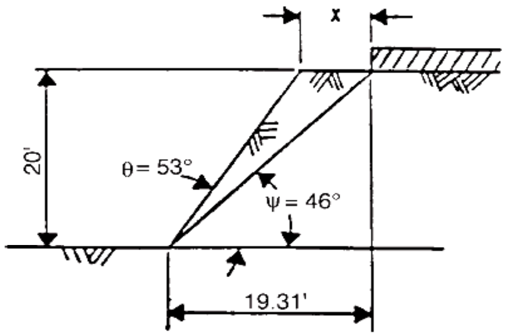
Construction of excavations or trenches is inherent in the construction of foundation elements such as footing foundations. The Caltrans Trenching and Shoring Manual provides information on the complete process for administering, designing and reviewing excavation work and plans. What follows is a brief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

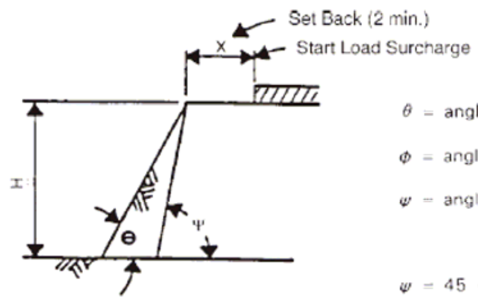
Example:



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’

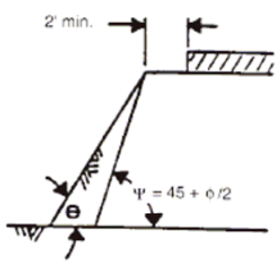


θ = angle of slope

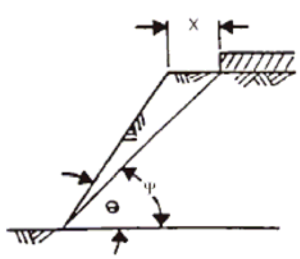
ϕ = angle of internal friction

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.

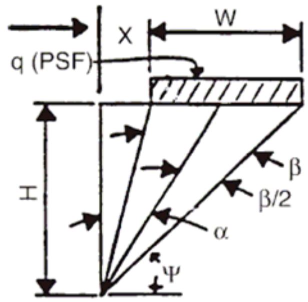


If $\theta > \psi$ then X must be calculated by geometry and X $\geq$ 2 feet.

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",

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

where β_r is in radians

At full height H,

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

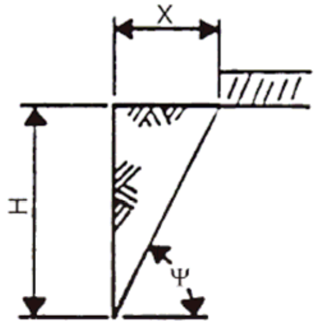
$$\alpha = \arctan\frac{X}{H} + \frac{\beta}{2}$$

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

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

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).

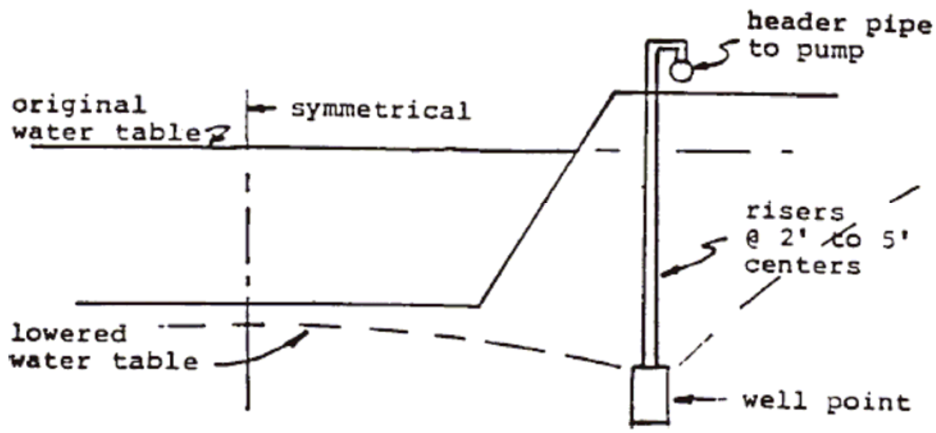


FIGURE 4-13 Single stage well point system

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).

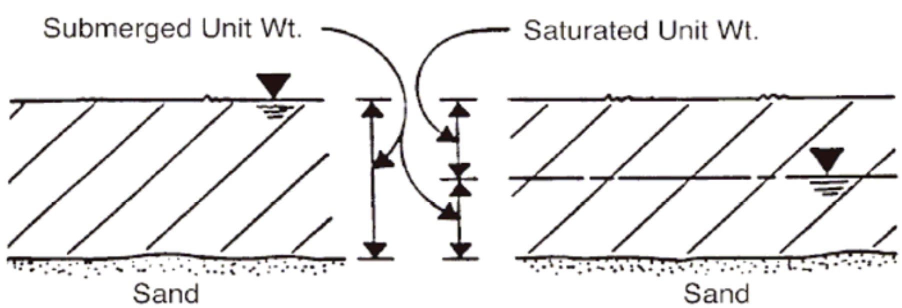


FIGURE 4-14 Saturated vs. submerged unit weight

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).

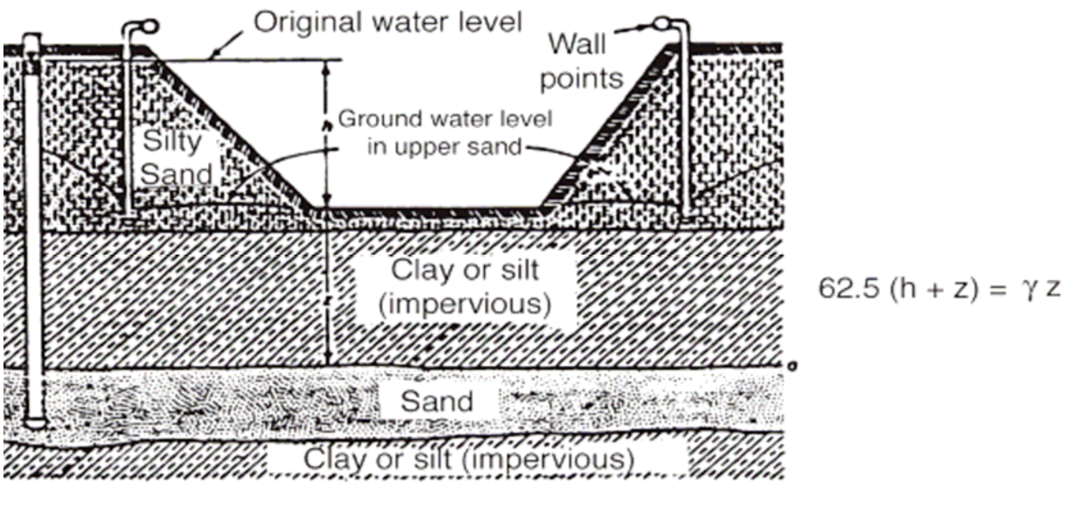


FIGURE 4-15 Bottom of excavation stability problems due to excess hydrostatic head against an impervious layer

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

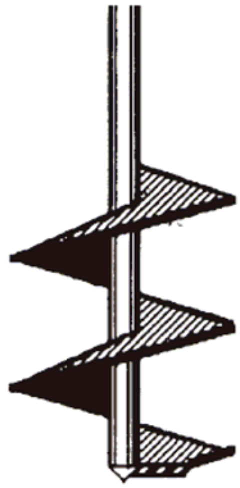


FIGURE 6-2
Auger - single flight

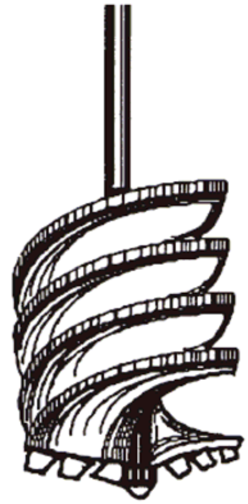


FIGURE 6-3
Auger – double flight

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.

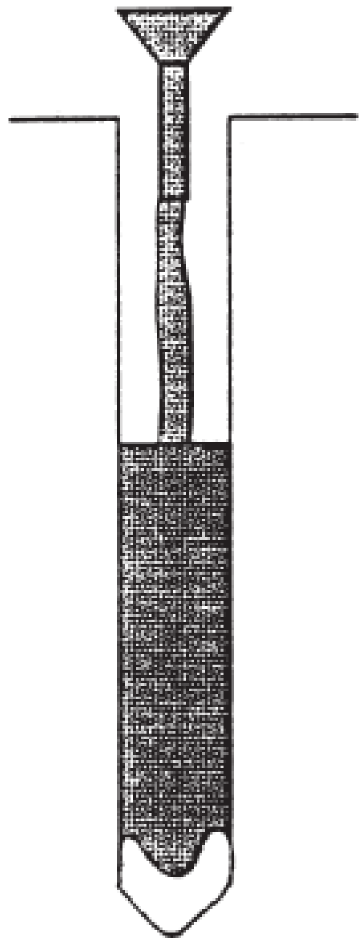


FIGURE 6-15
Pile defects - no cleanout, tapered bottom of hole

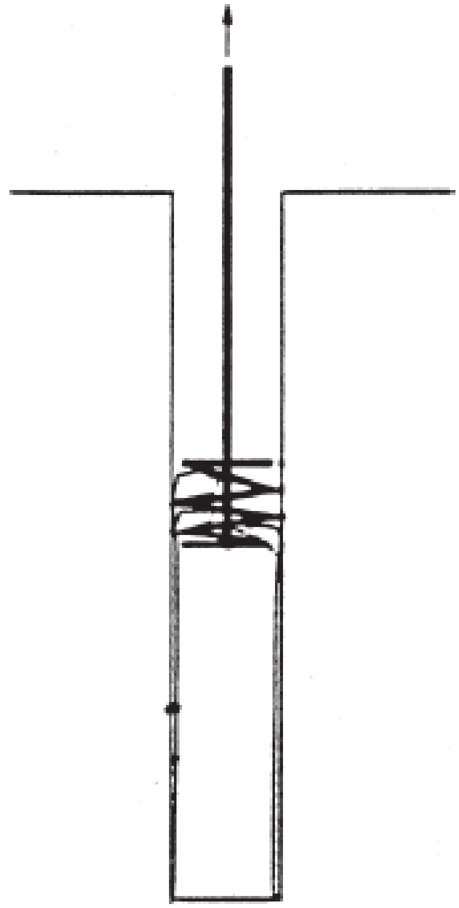


FIGURE 6-16
Pile defects - smeared drill cuttings

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.

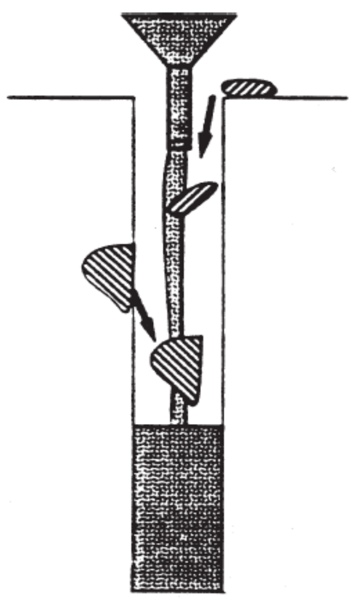


FIGURE 6-17
Pile defects - cave in

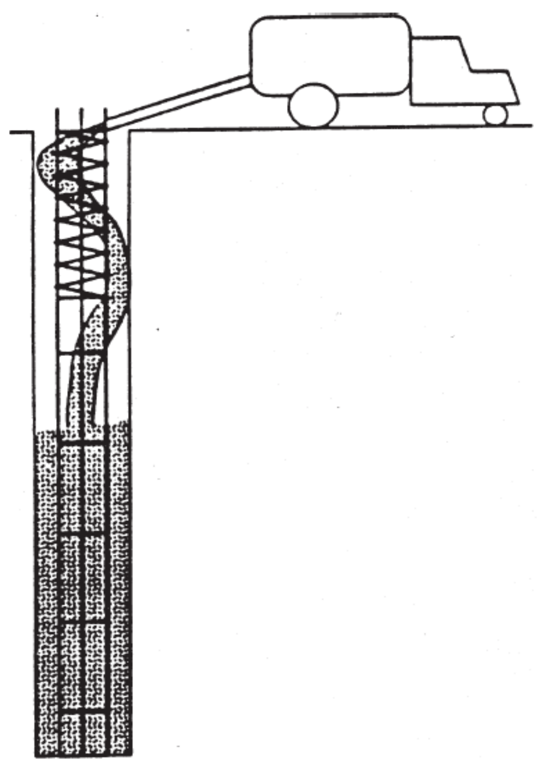


FIGURE 6-18
Pile defects - concrete segregation

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.

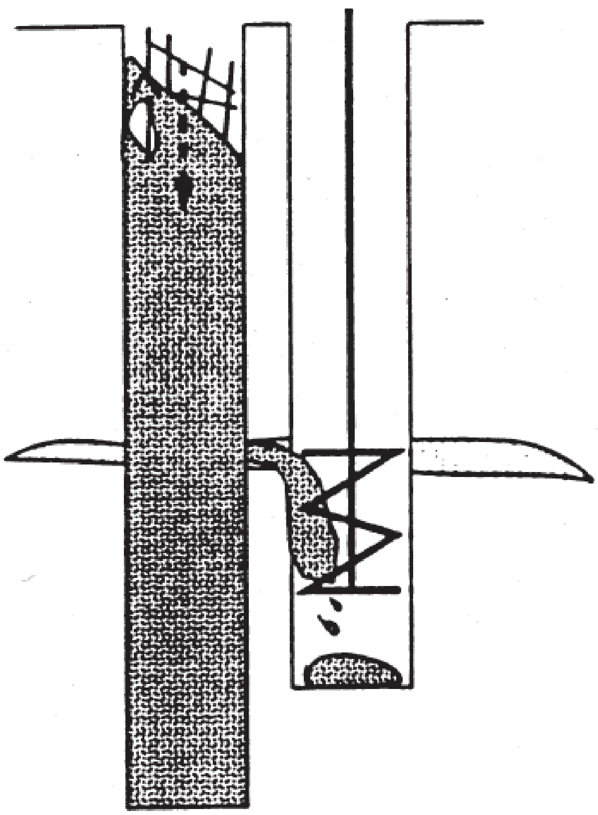


FIGURE 6-19
Pile defects - adjacent hole blowout

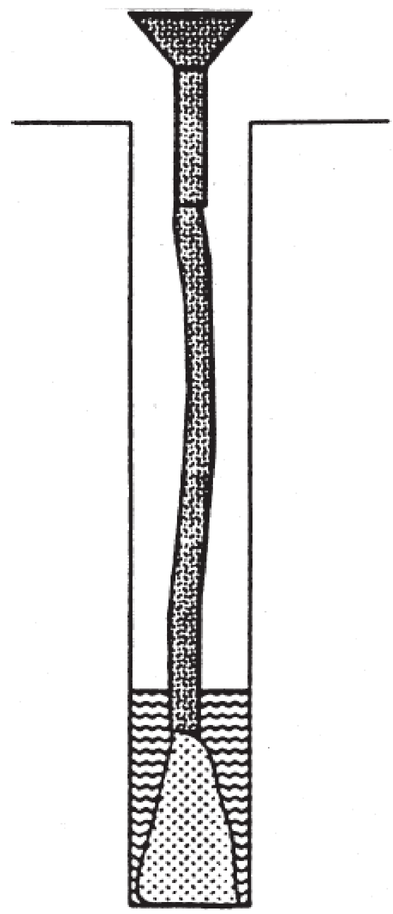
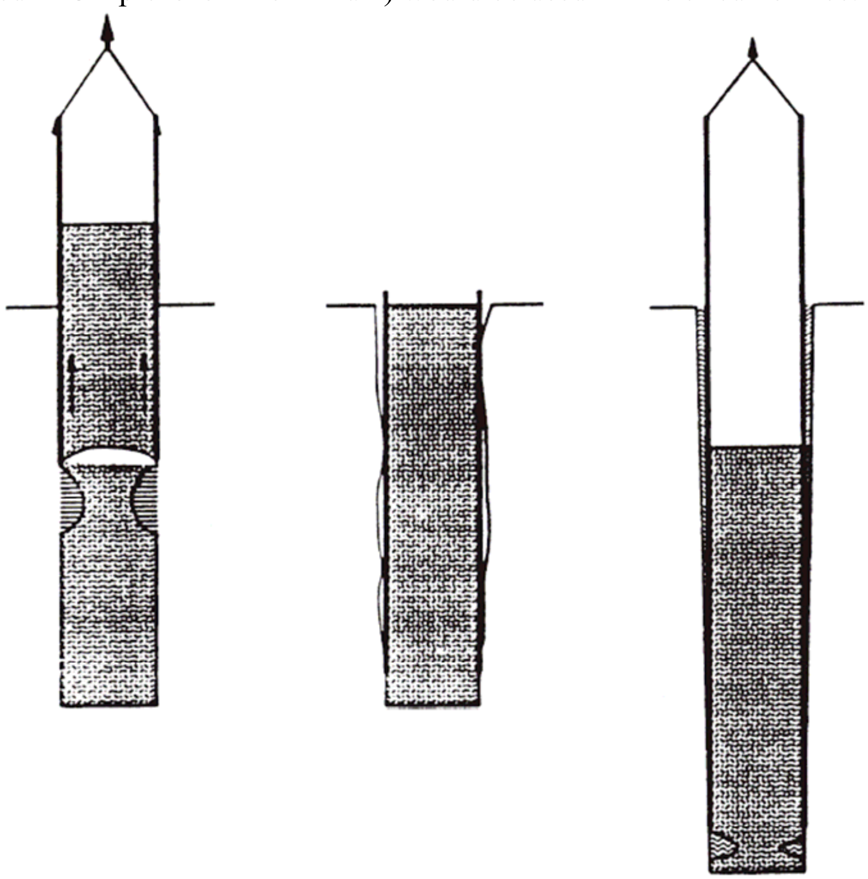


FIGURE 6-20
Pile defects - water in the hole

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.

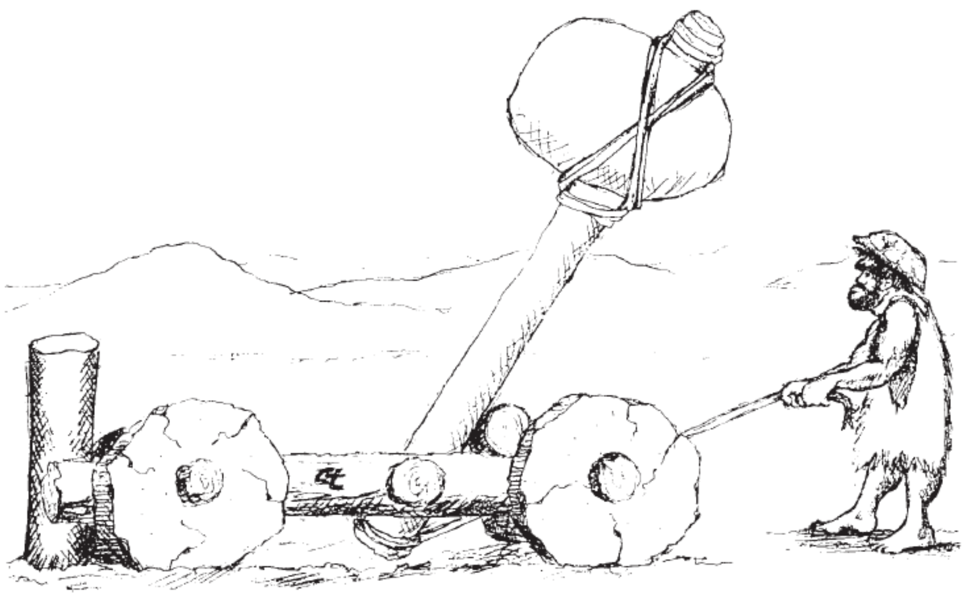


FIGURE 7-1 Early pile hammer

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.

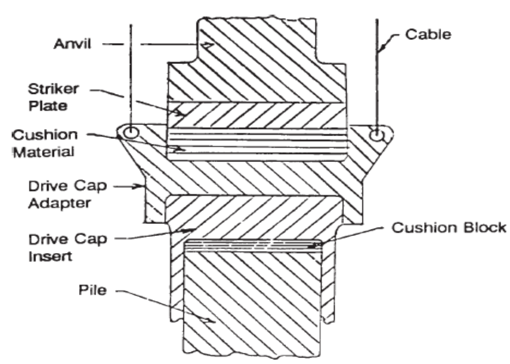


FIGURE 7-2 Drive cap system

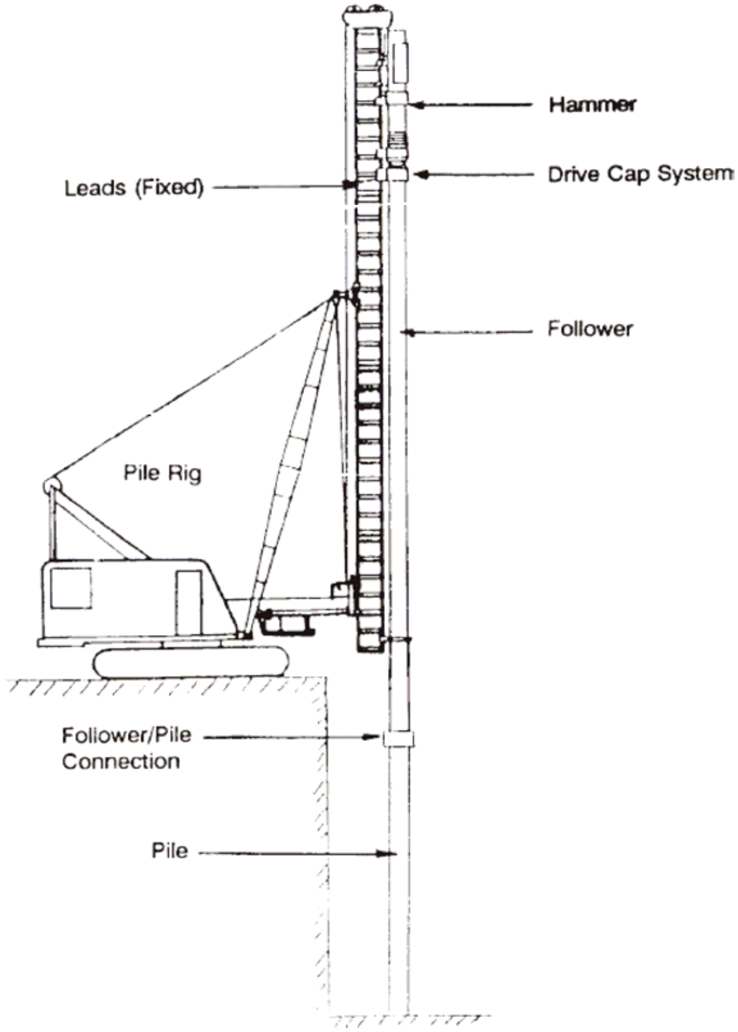


FIGURE 7-3 Typical pile rig configuration

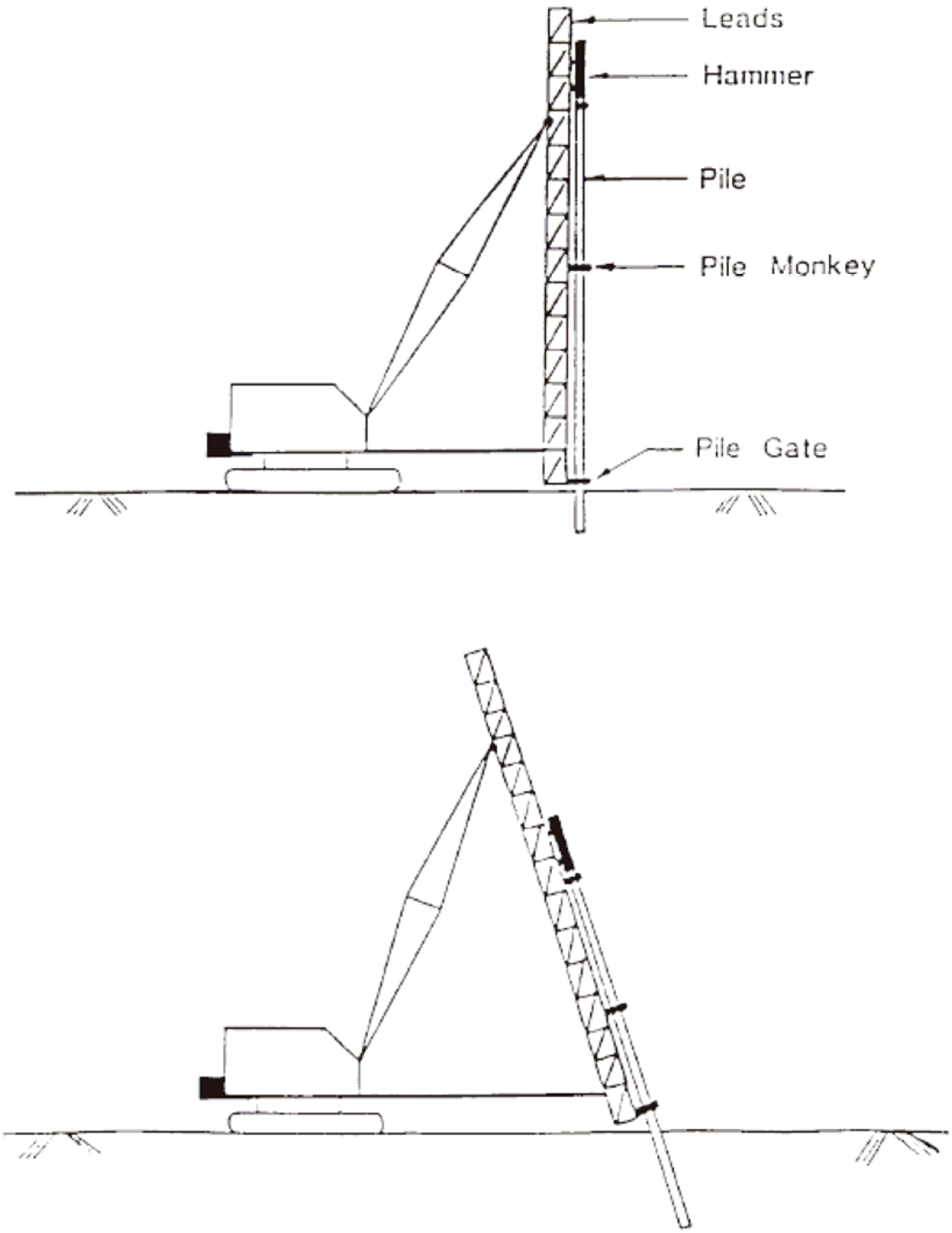
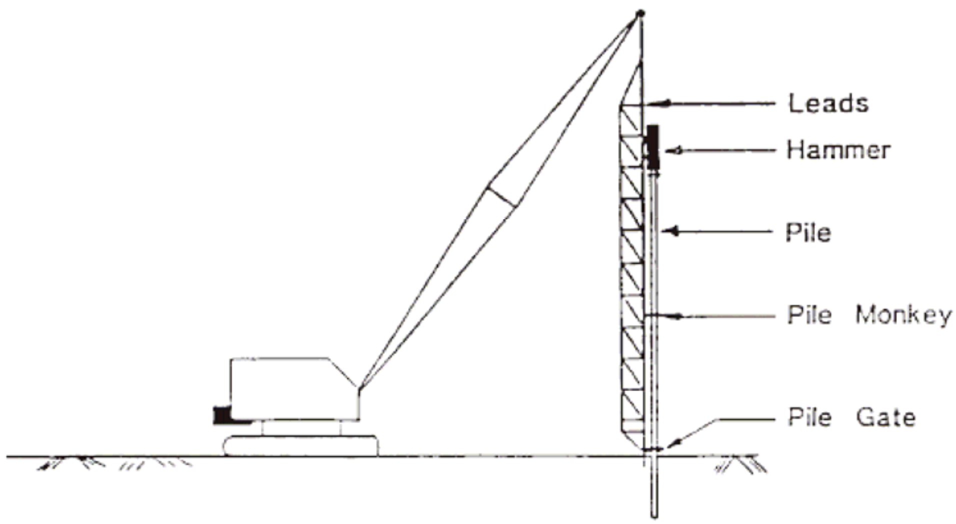


FIGURE 7-4 Fixed lead system



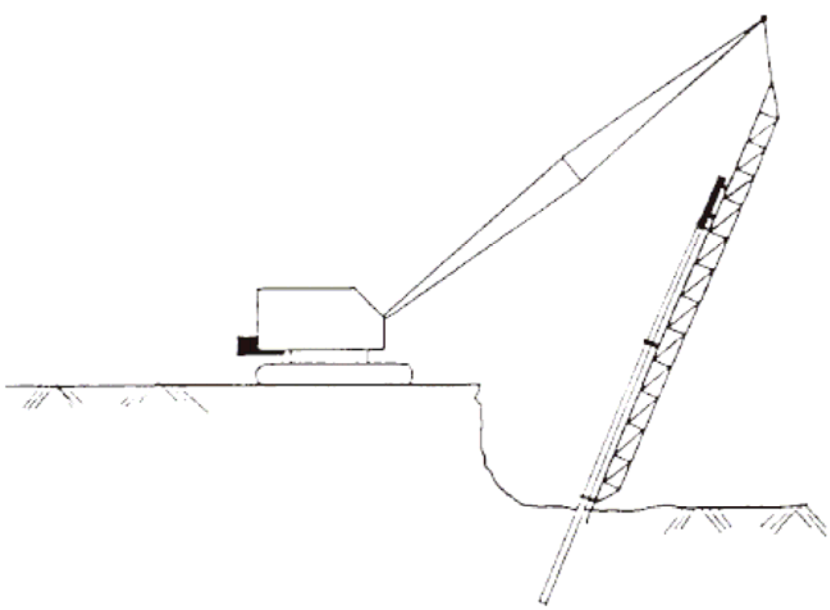


FIGURE 7-5 Swinging lead system

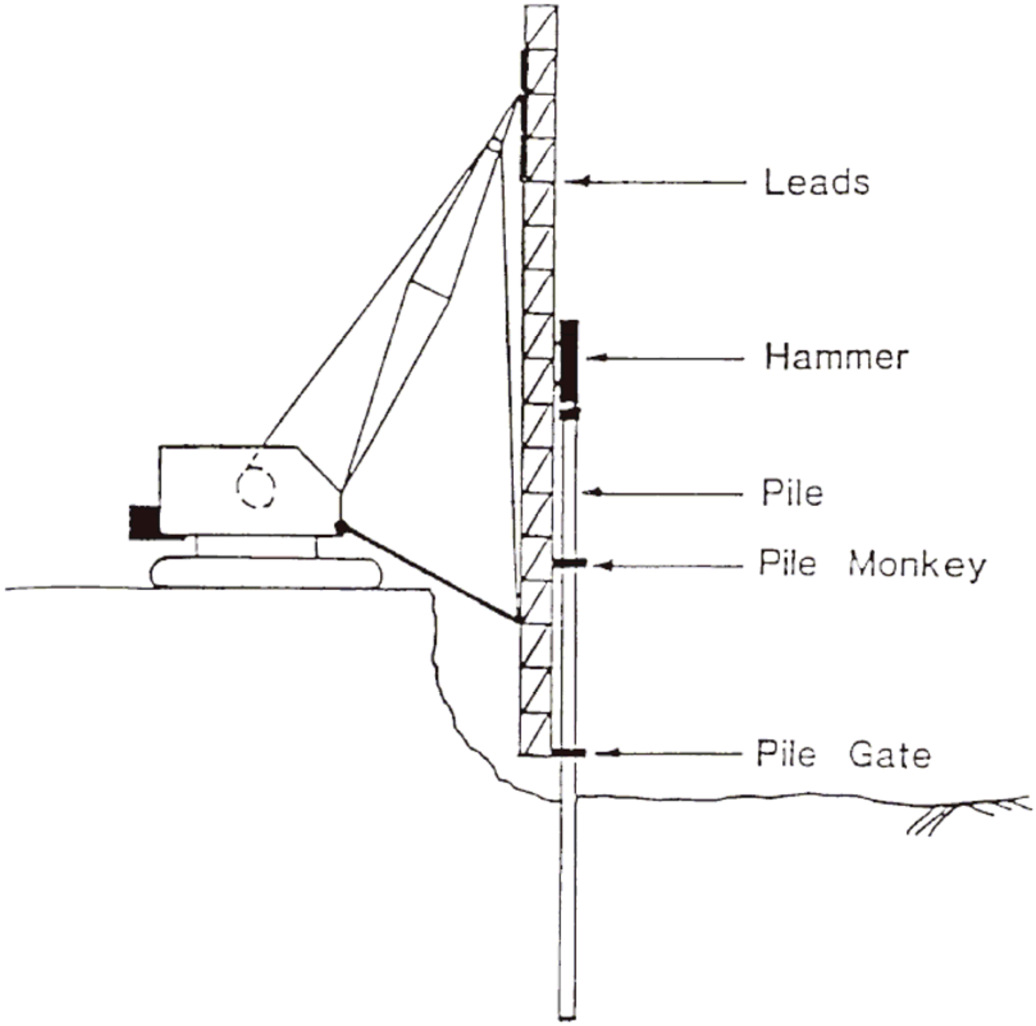


FIGURE 7-6 Semi-fixed lead system

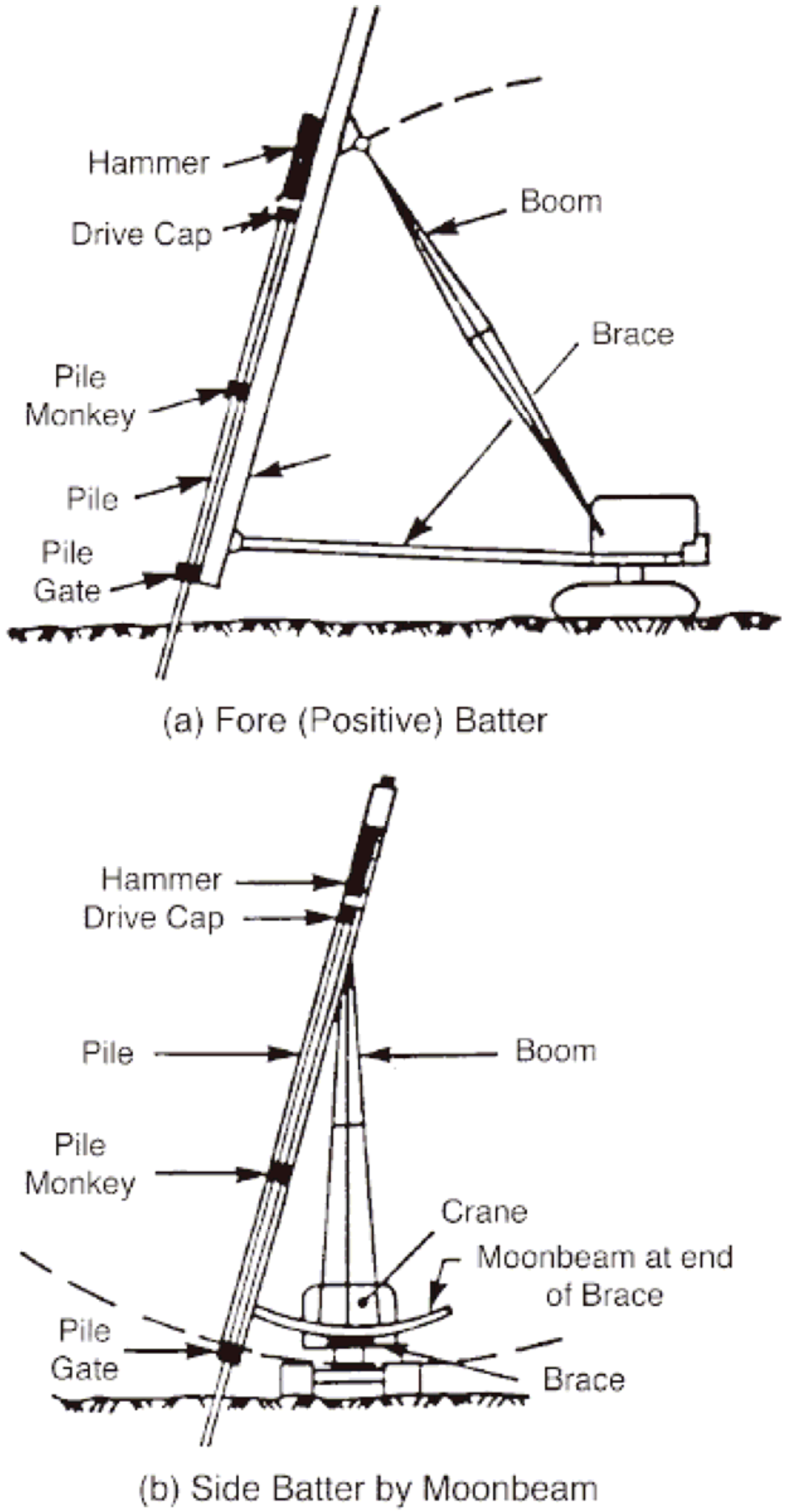


FIGURE 7-7 Lead configurations for battered piles

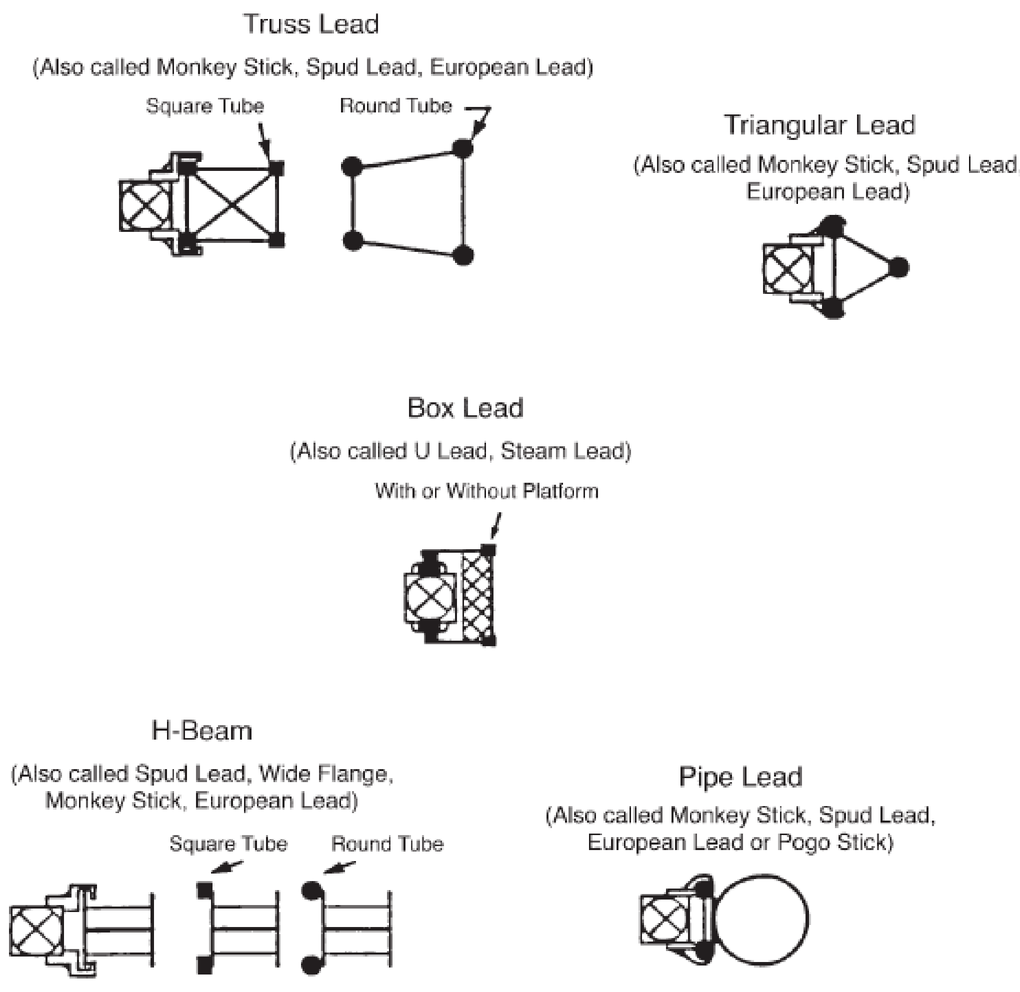


FIGURE 7-8 Lead types

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.

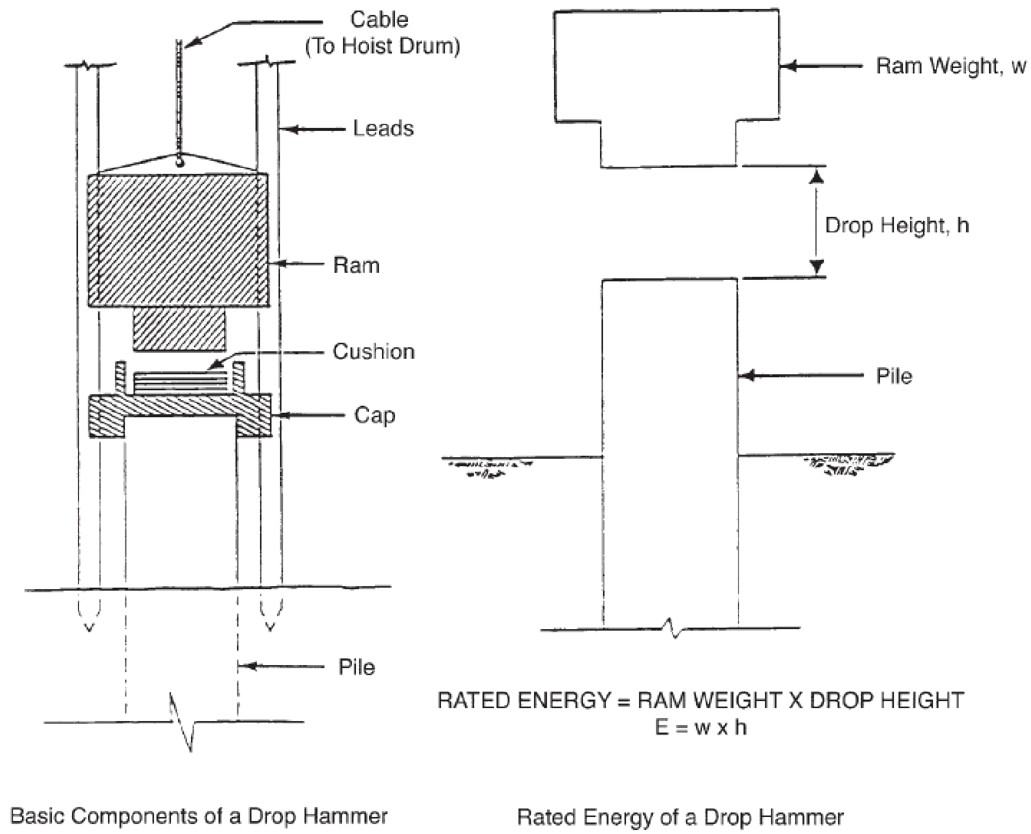


FIGURE 7-9 Drop hammer

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.

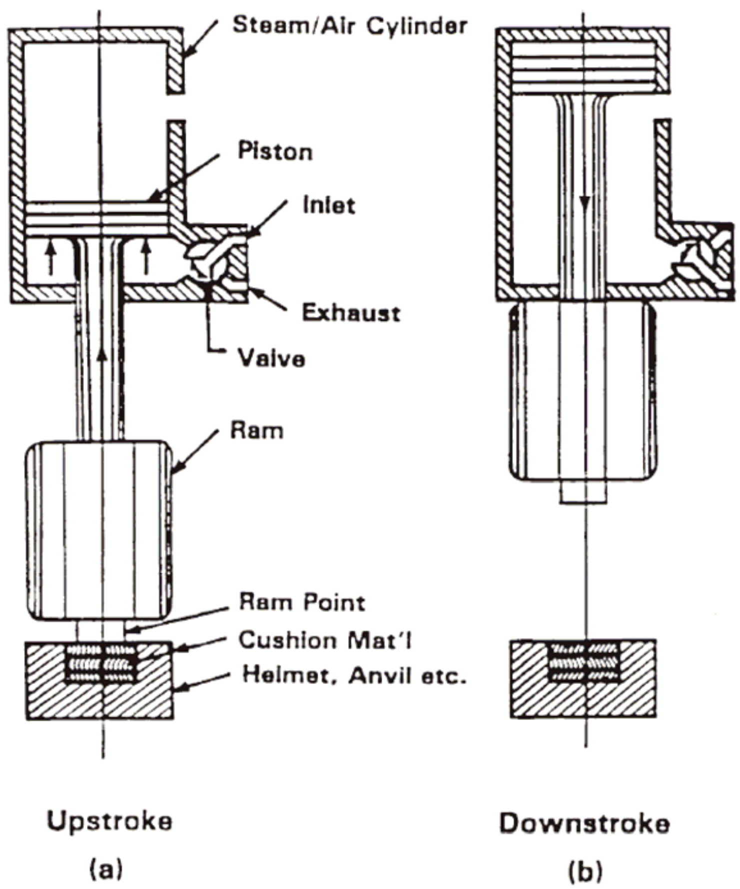


FIGURE 7-10 Single acting steam/air hammer

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.

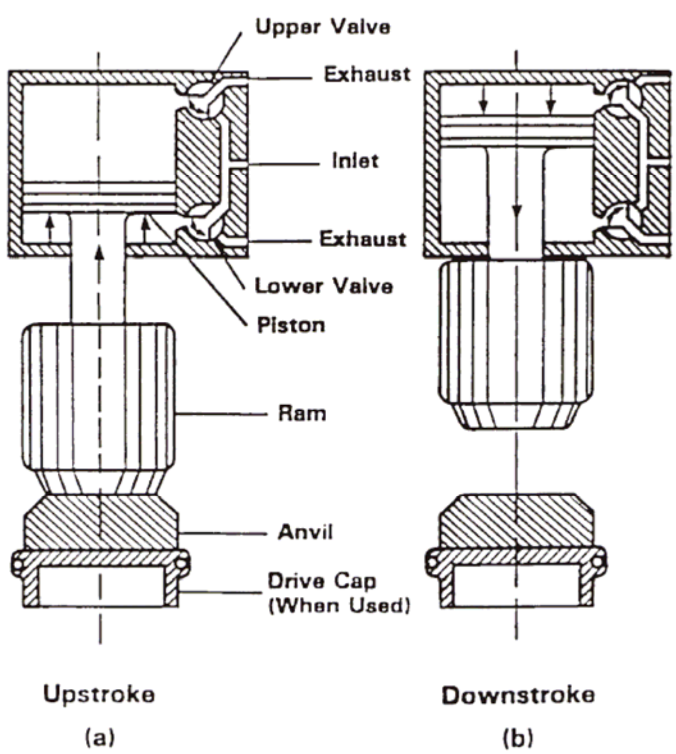


FIGURE 7-11 Double acting steam/air hammer

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.

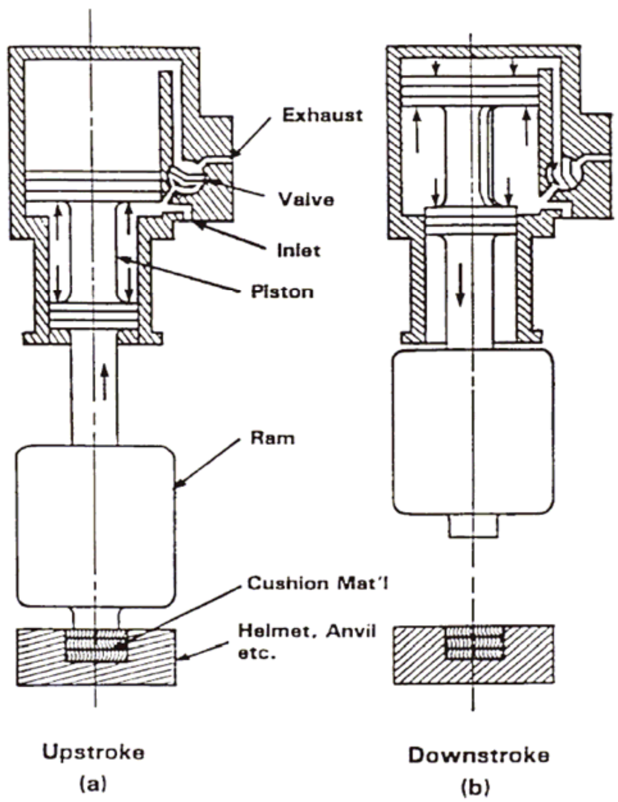


FIGURE 7-12 Differential acting steam/air hammer

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.

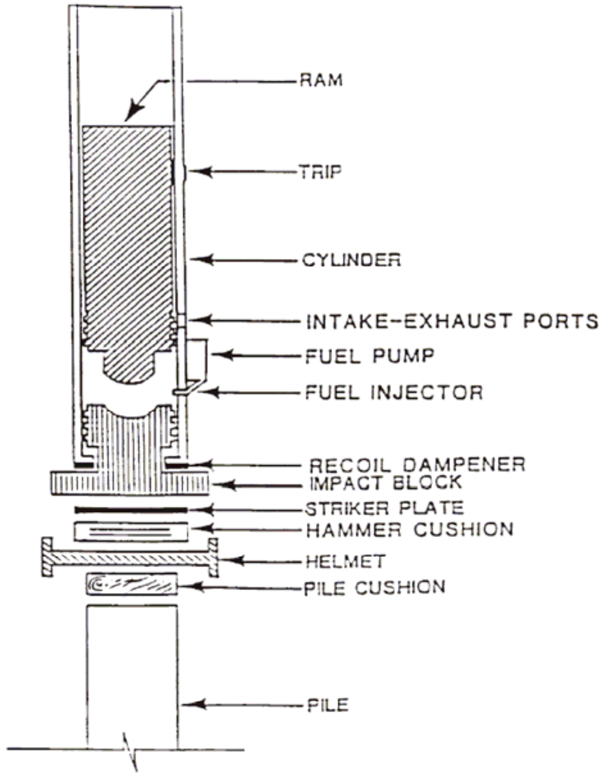


FIGURE 7-13 Single acting diesel hammer

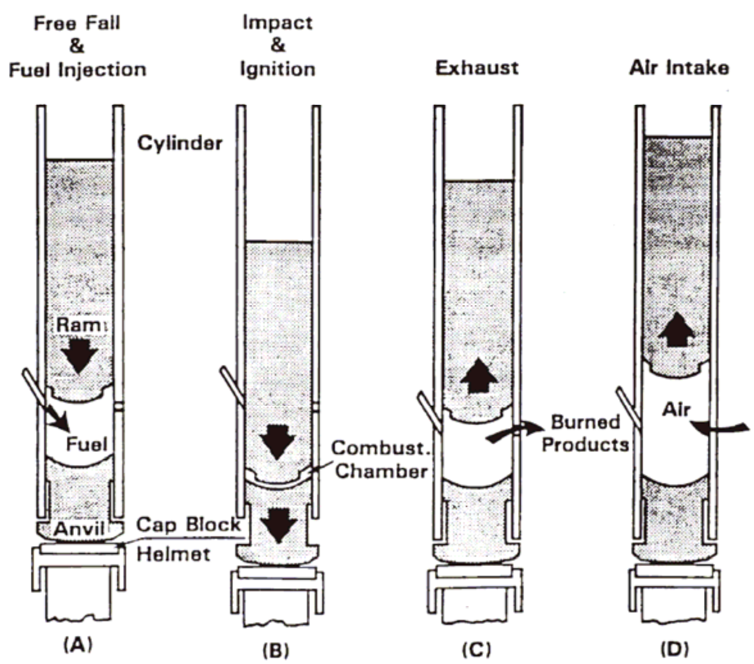


FIGURE 7-14 Operational cycle for single acting diesel hammer

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.

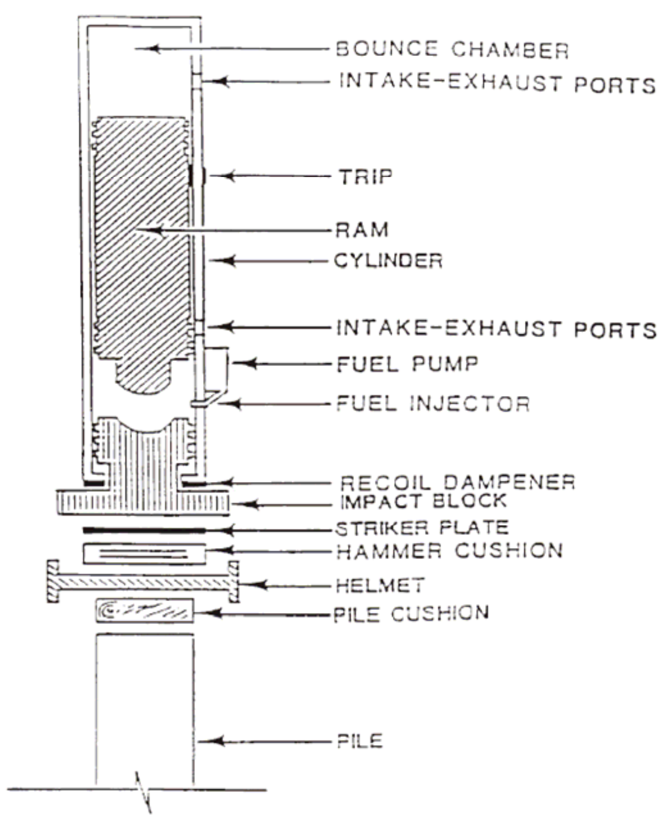


FIGURE 7-15 Double acting diesel hammer

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.

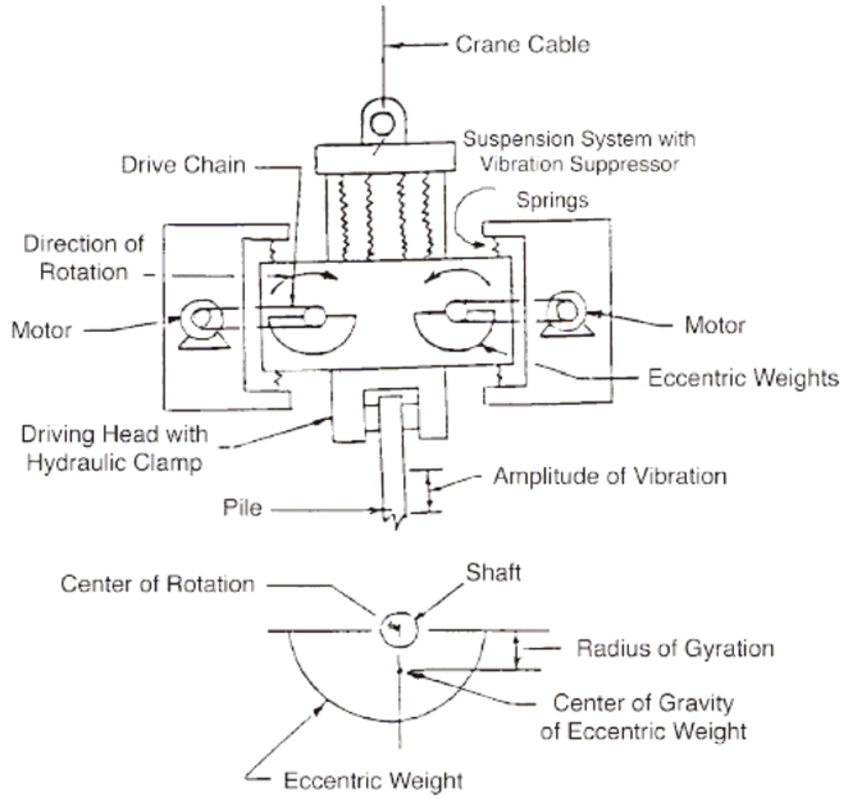


FIGURE 7-16 Vibratory driver/extractor

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.

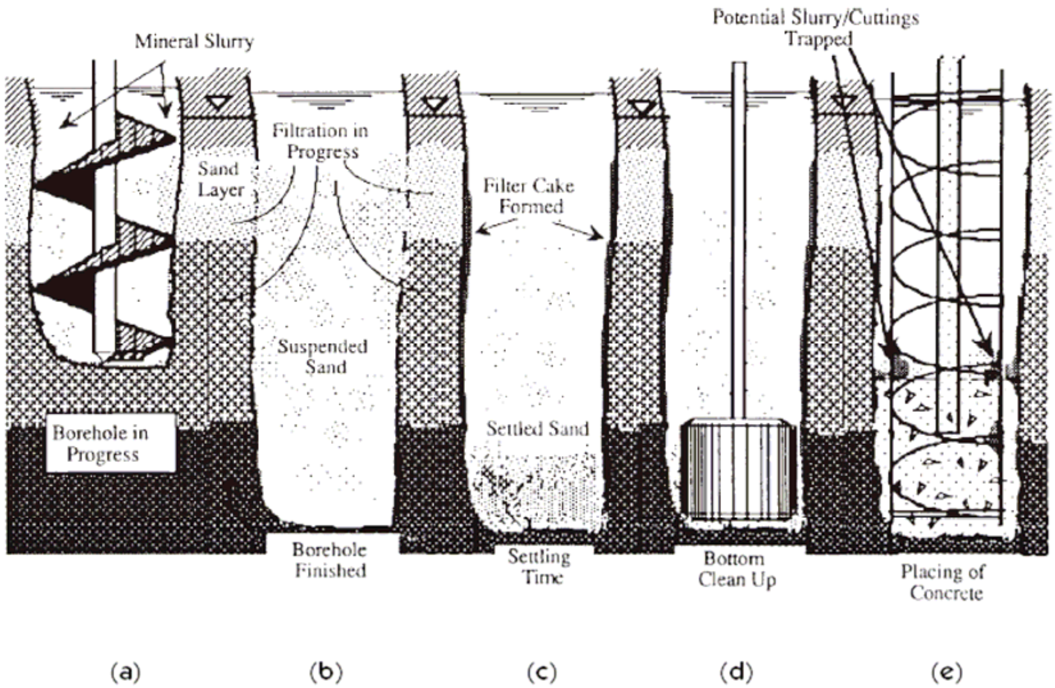


FIGURE 9-1 Slurry displacement method

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.

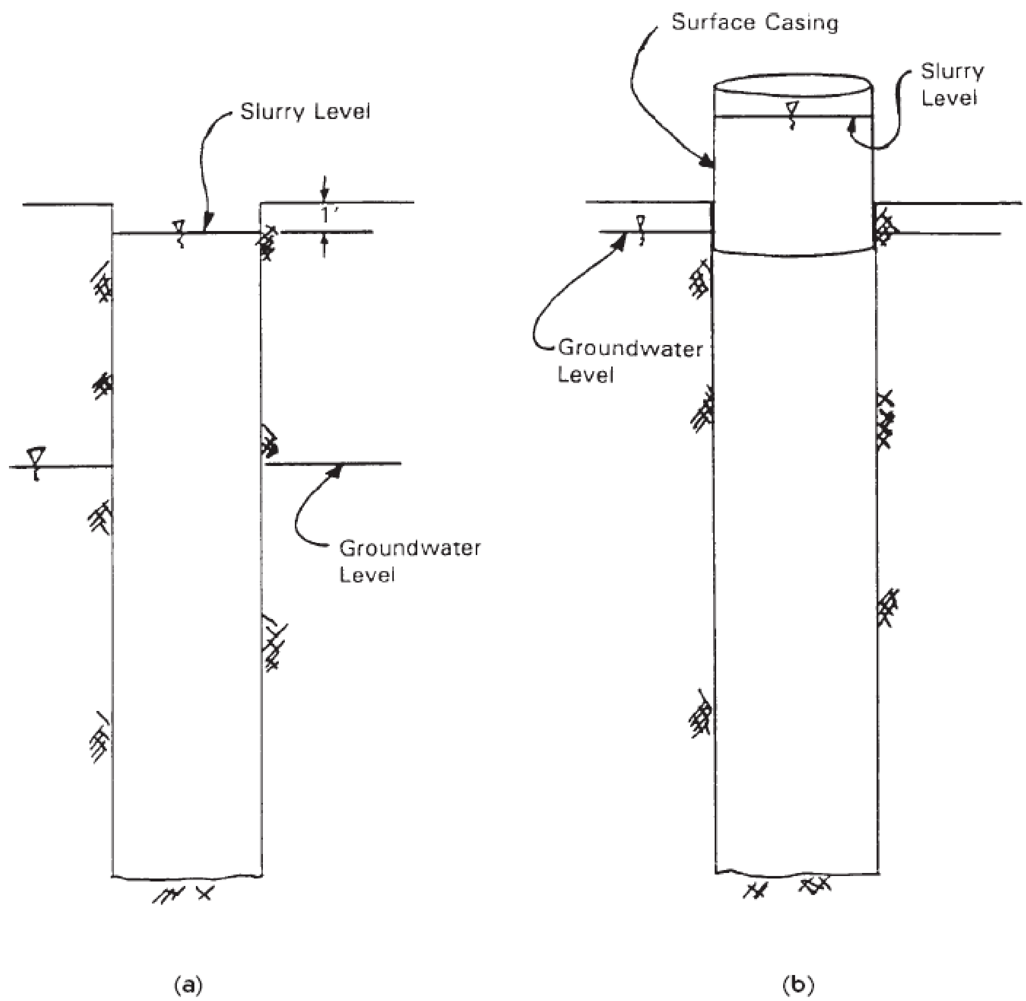


FIGURE 9-2(a)(b) Positive hydrostatic pressure

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.

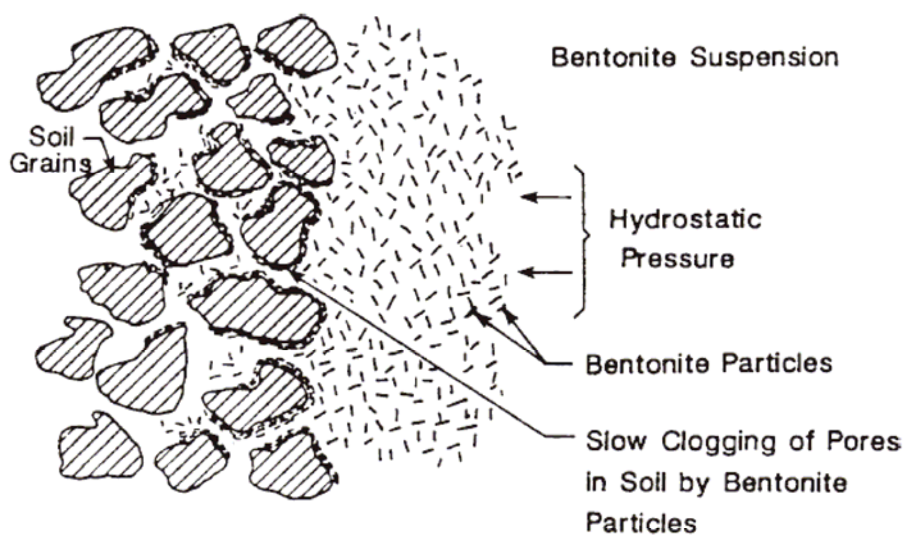


FIGURE 9-3(a) Filtration – loose ground formation

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.

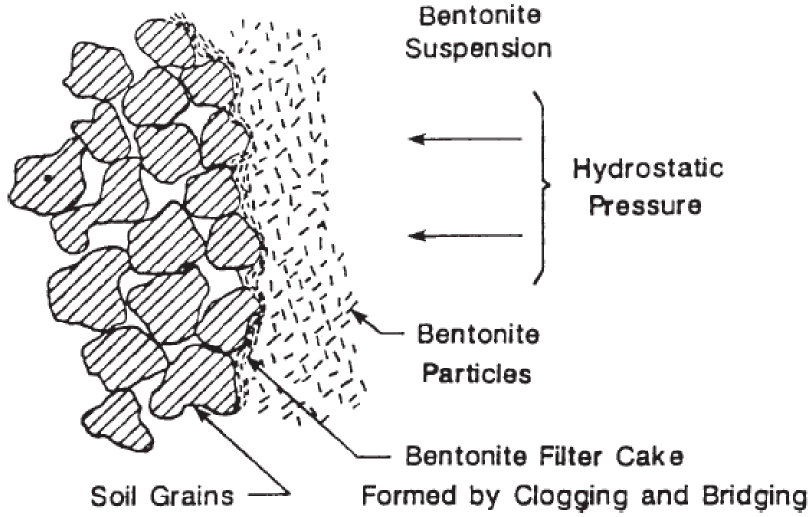


FIGURE 9-3(b) Filtration – tight ground formation

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.

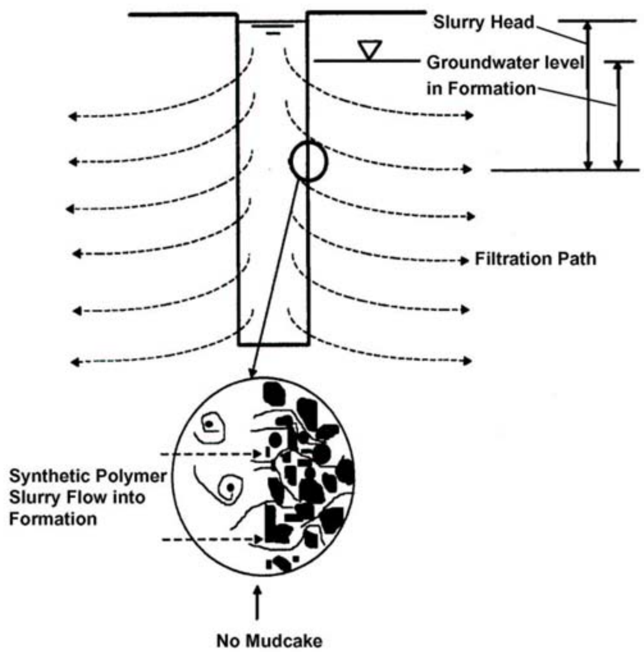


FIGURE 9-3(c) Stabilization with synthetic polymer slurry

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.

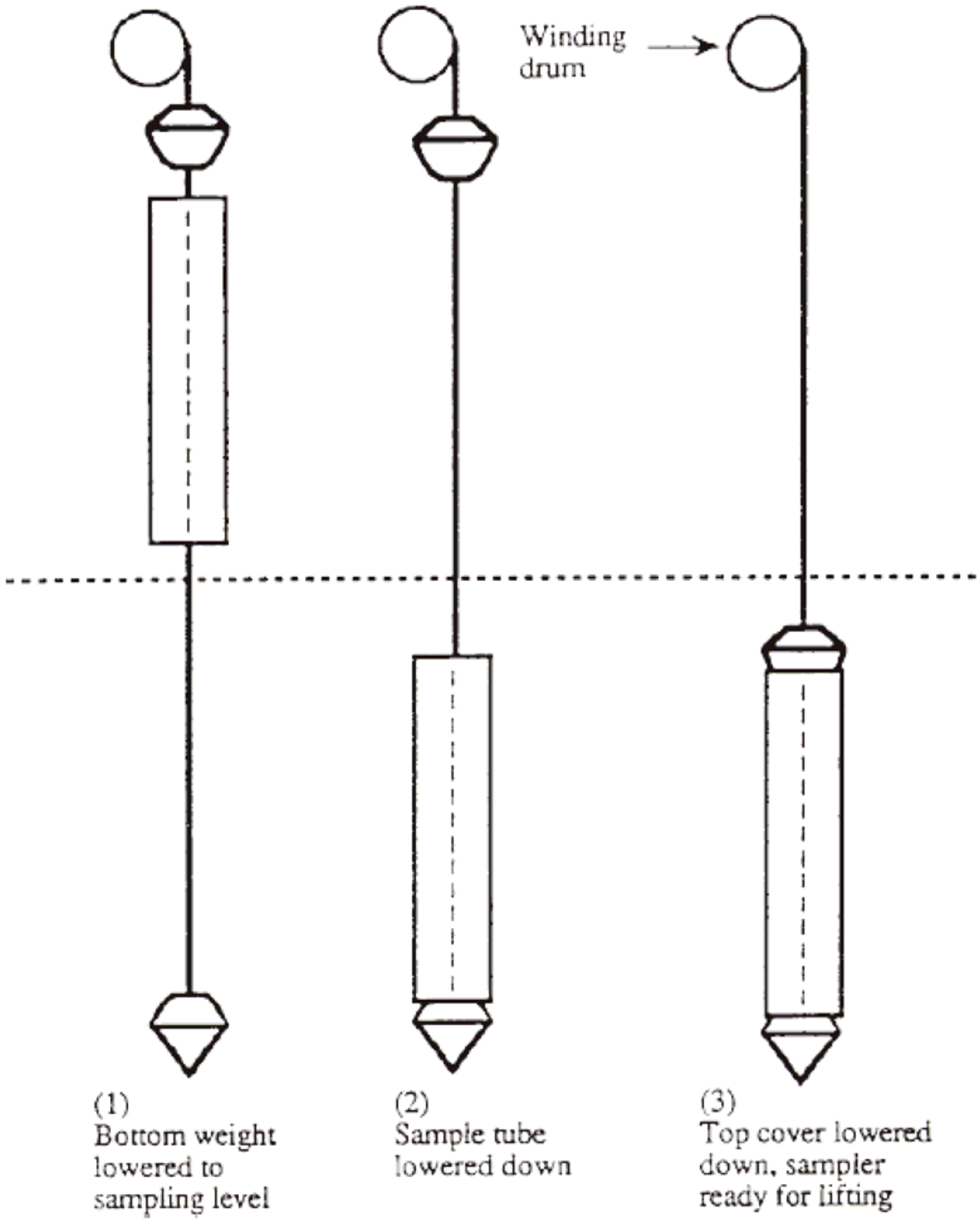


FIGURE 9-4 Slurry sampler schematic

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

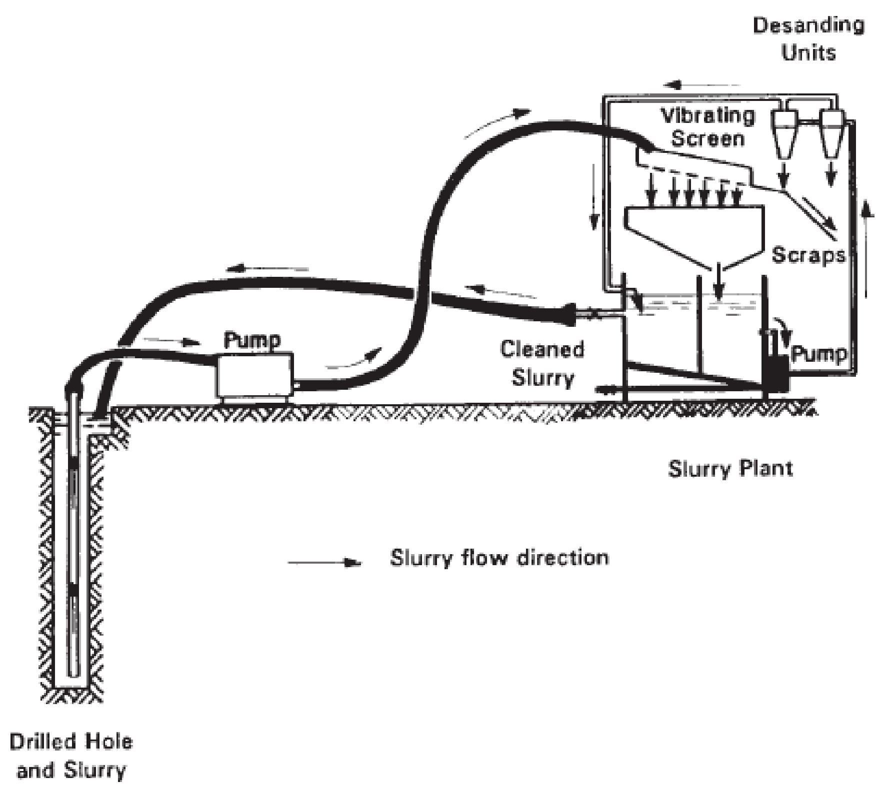


FIGURE 9-11 Recirculation and cleaning schematic

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.

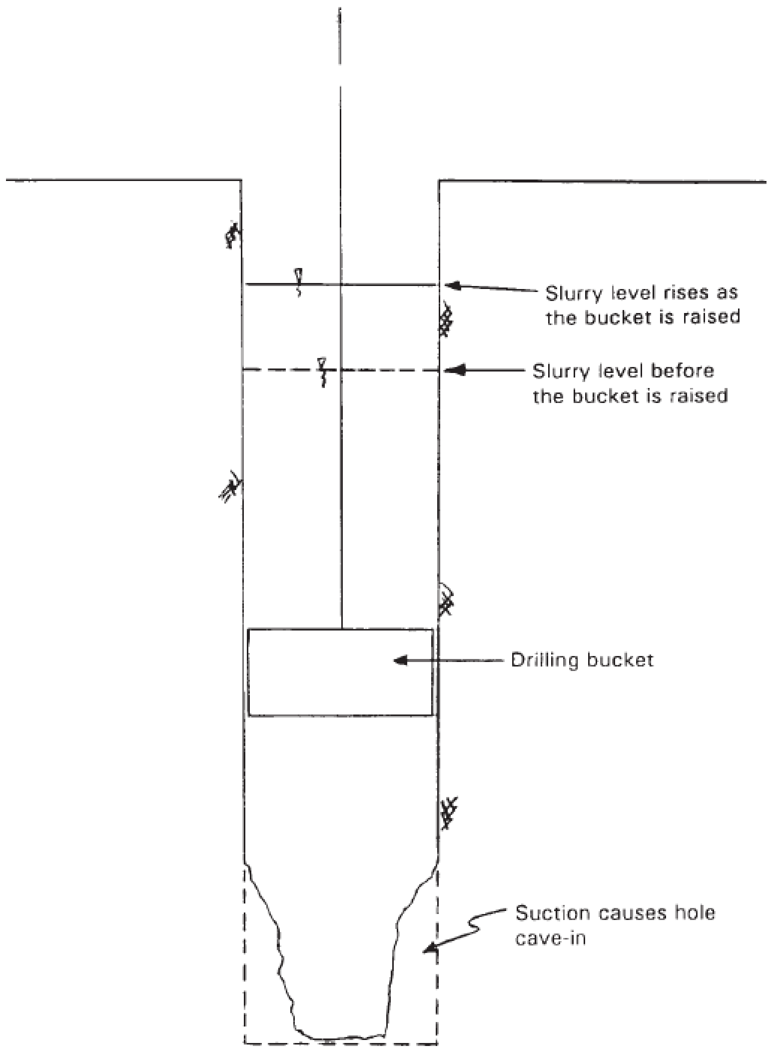


FIGURE 9-16 Hole collapse induced by pressure changes

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.

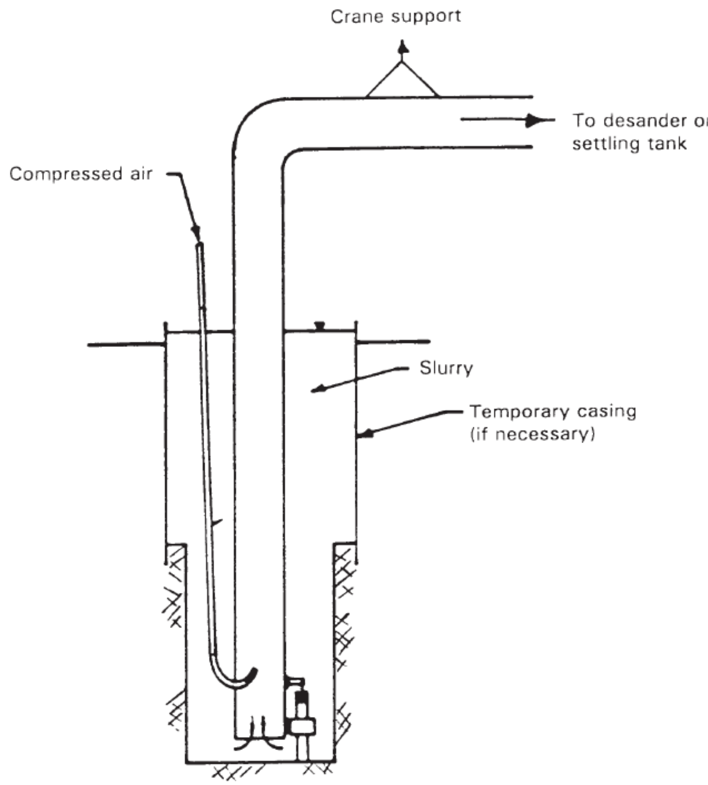


FIGURE 9-18 Airlift schematic

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.

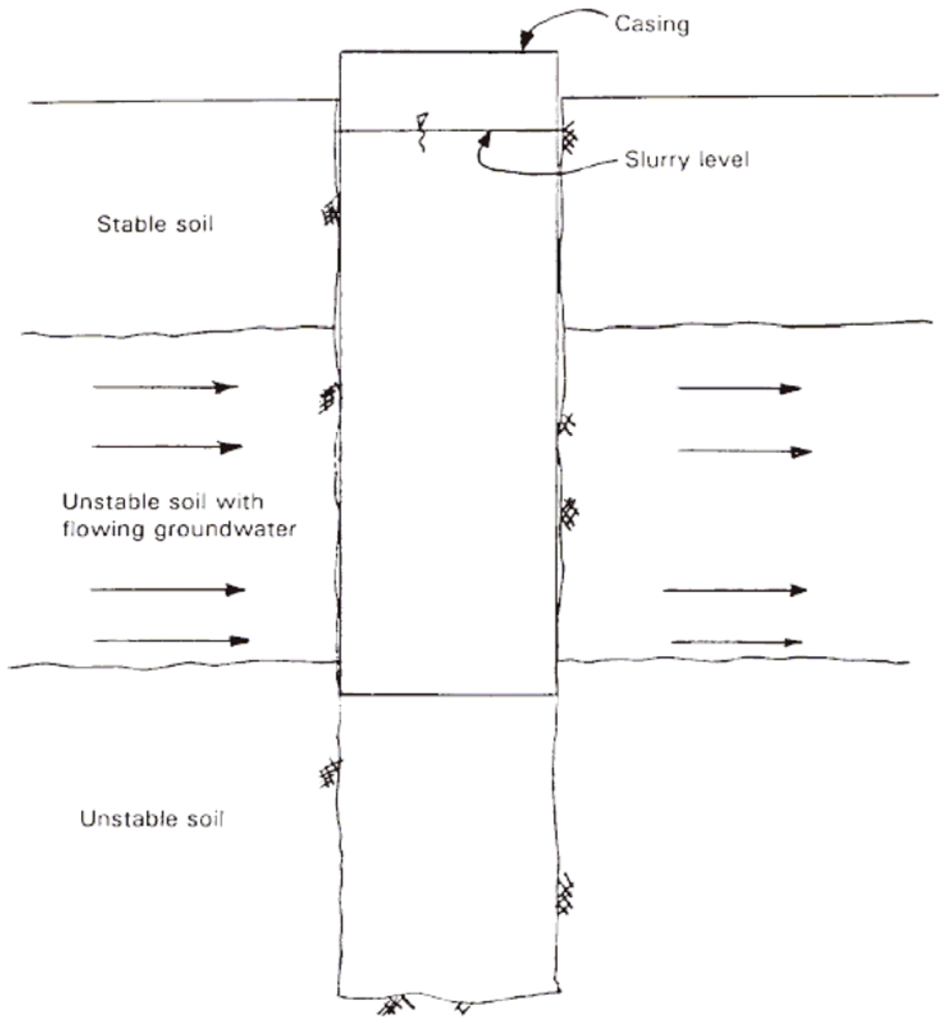


FIGURE 9-19 Use of casing

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.

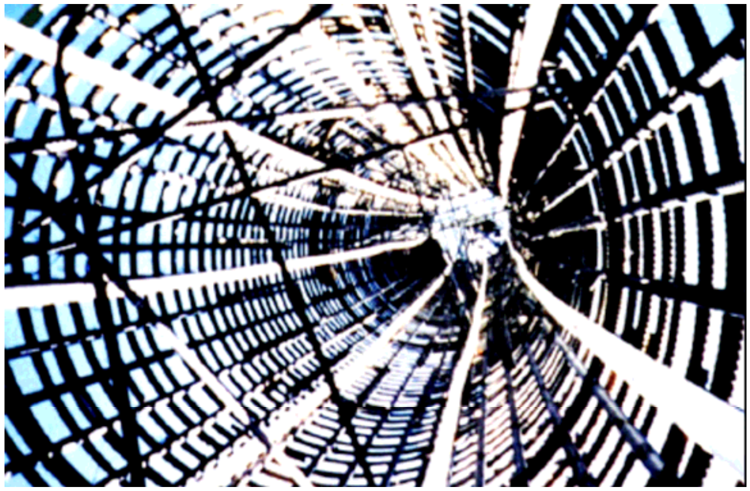
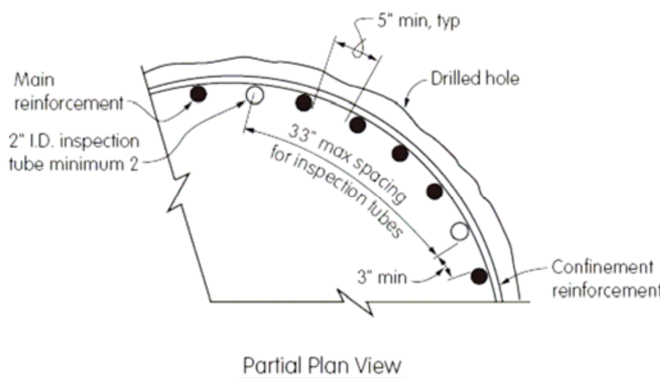


FIGURE 9-20 Inspection tubes



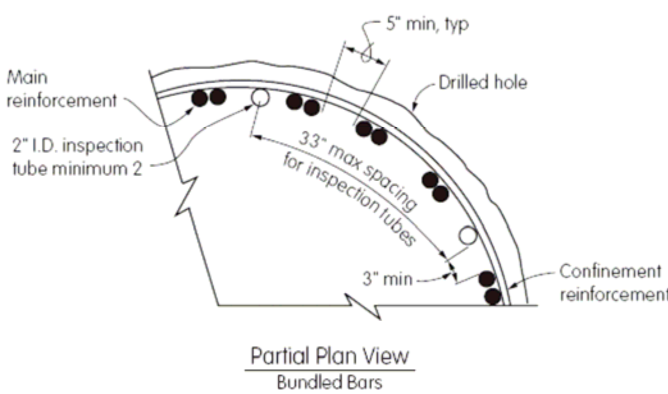


FIGURE 9-21 Location of inspection tubes within the pile

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.

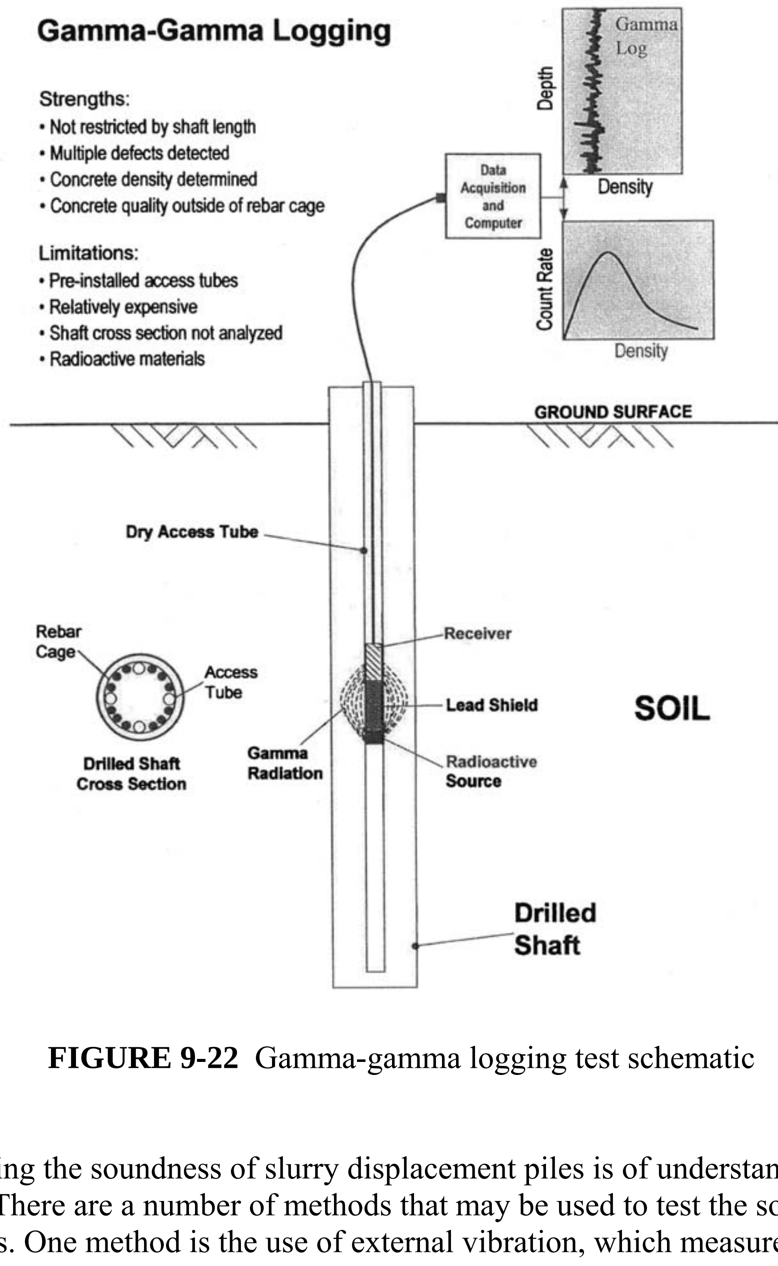


FIGURE 9-22 Gamma-gamma logging test schematic

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.

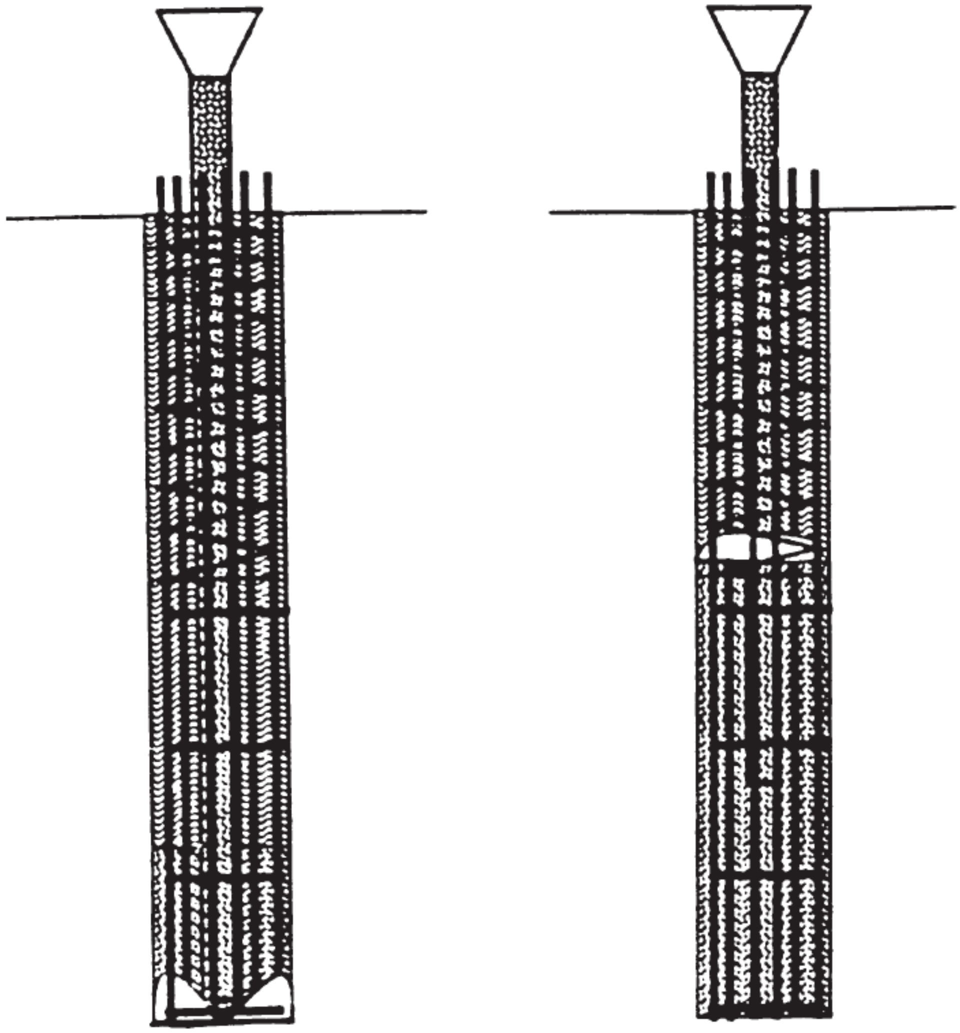


FIGURE 9-23 (a) (b) Defects from settled materials

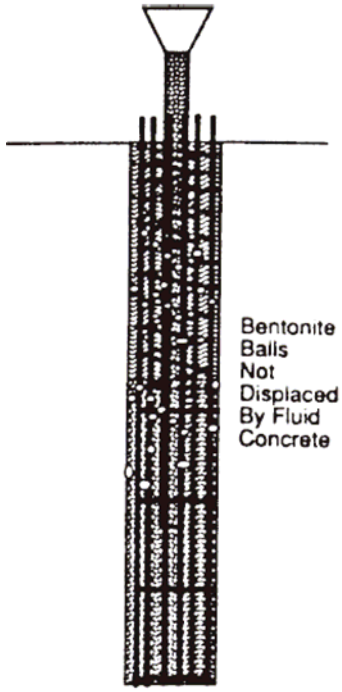


FIGURE 9-24
Defect from improperly mixed mineral slurry

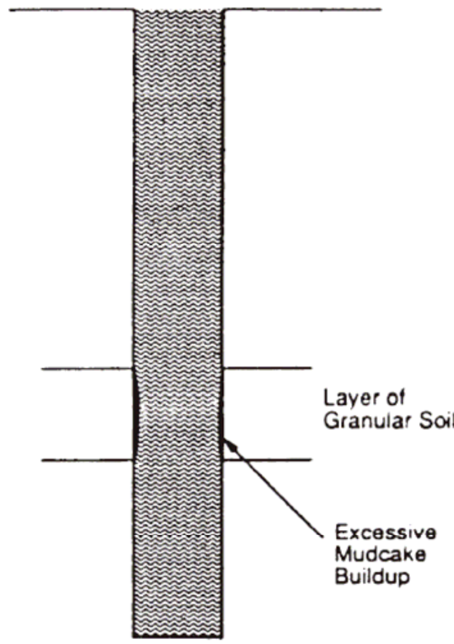


FIGURE 9-25
Defect from excess filter cake buildup

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.

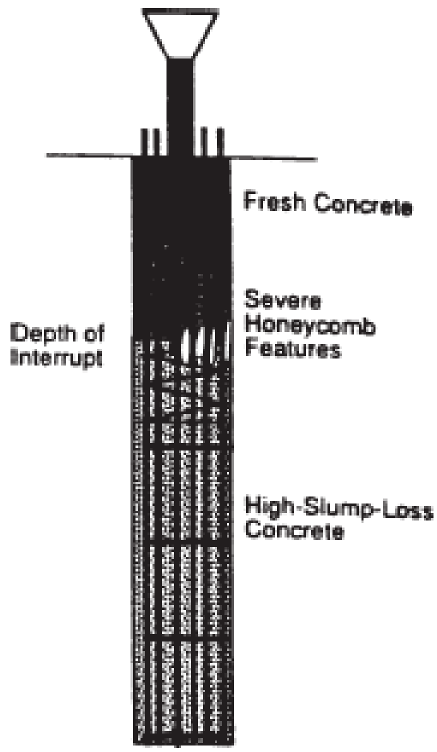


FIGURE 9-26 Defect from concrete placement problems

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 not present in the discharge or water flow testing indicates the defective area is impermeable. All water resulting from the washing operation must be removed from the void prior to placement of grout. This is typically done with compressed air. Grout is then pumped into the void, in effect, “replacing” the voided area with grout. Replacement grouting cannot be used if the grout has a means of escaping the void area. If a side of the void area includes the side or bottom of the drilled hole, replacement grouting generally cannot be used to repair the pile defect.

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.

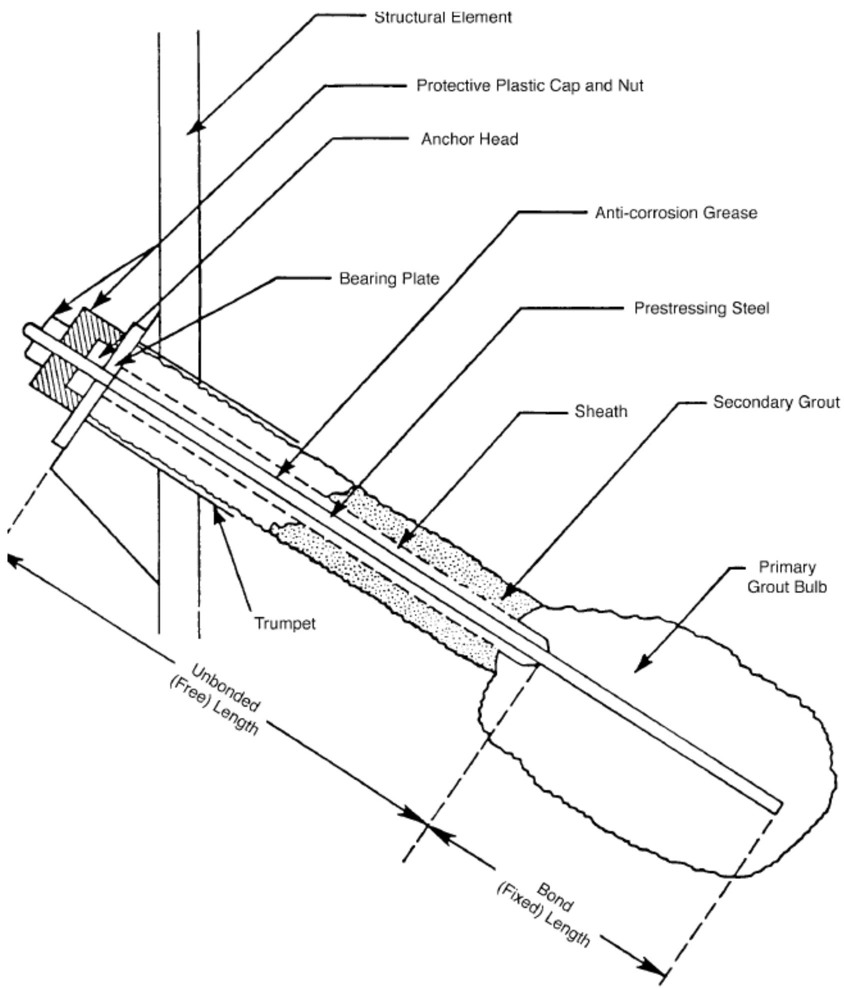


FIGURE 11-1 Tieback detail

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.

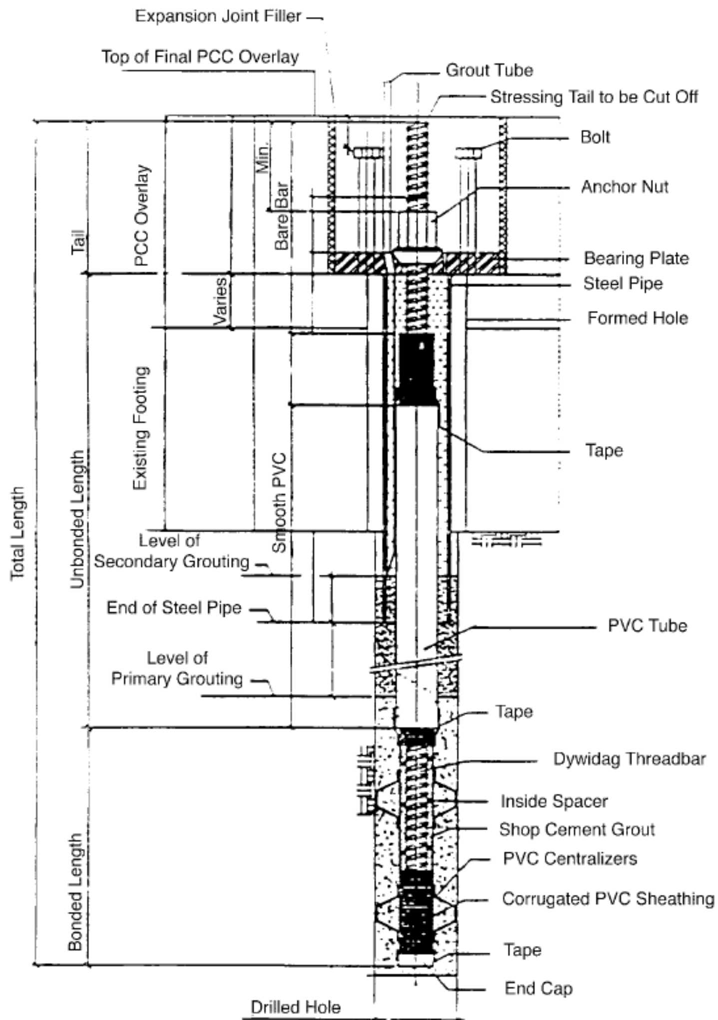


FIGURE 11-2 Tiedown anchor

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.

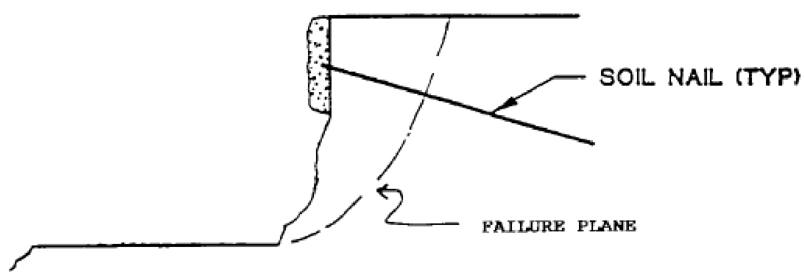


Figure 11-3 Soil nail schematic

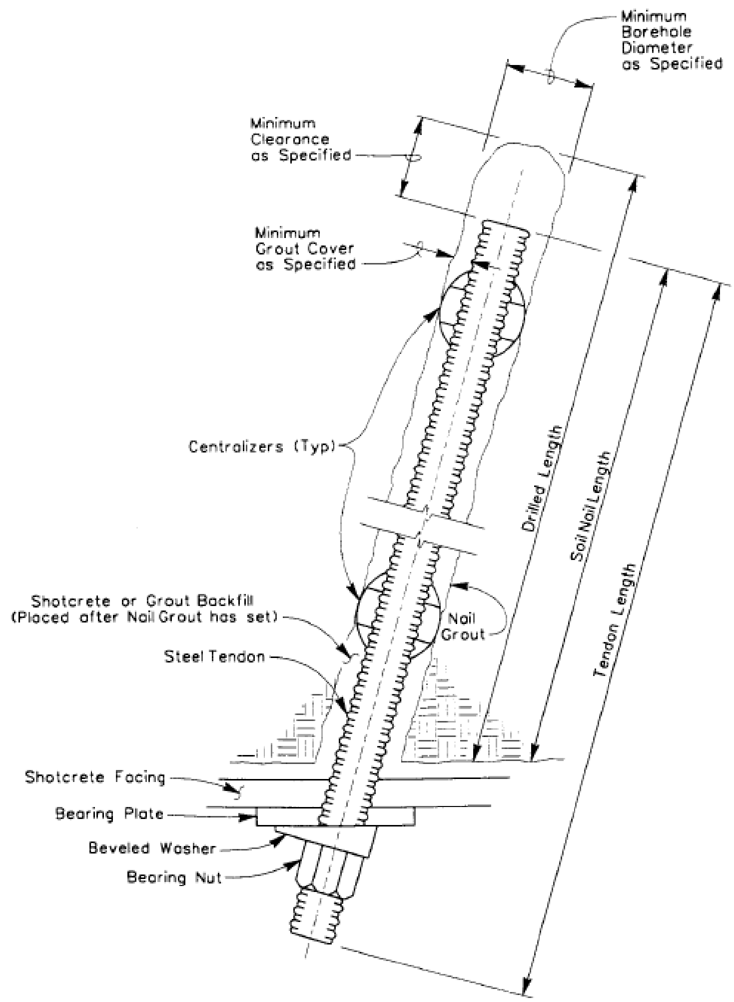


FIGURE 11-4 Soil nail details

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.

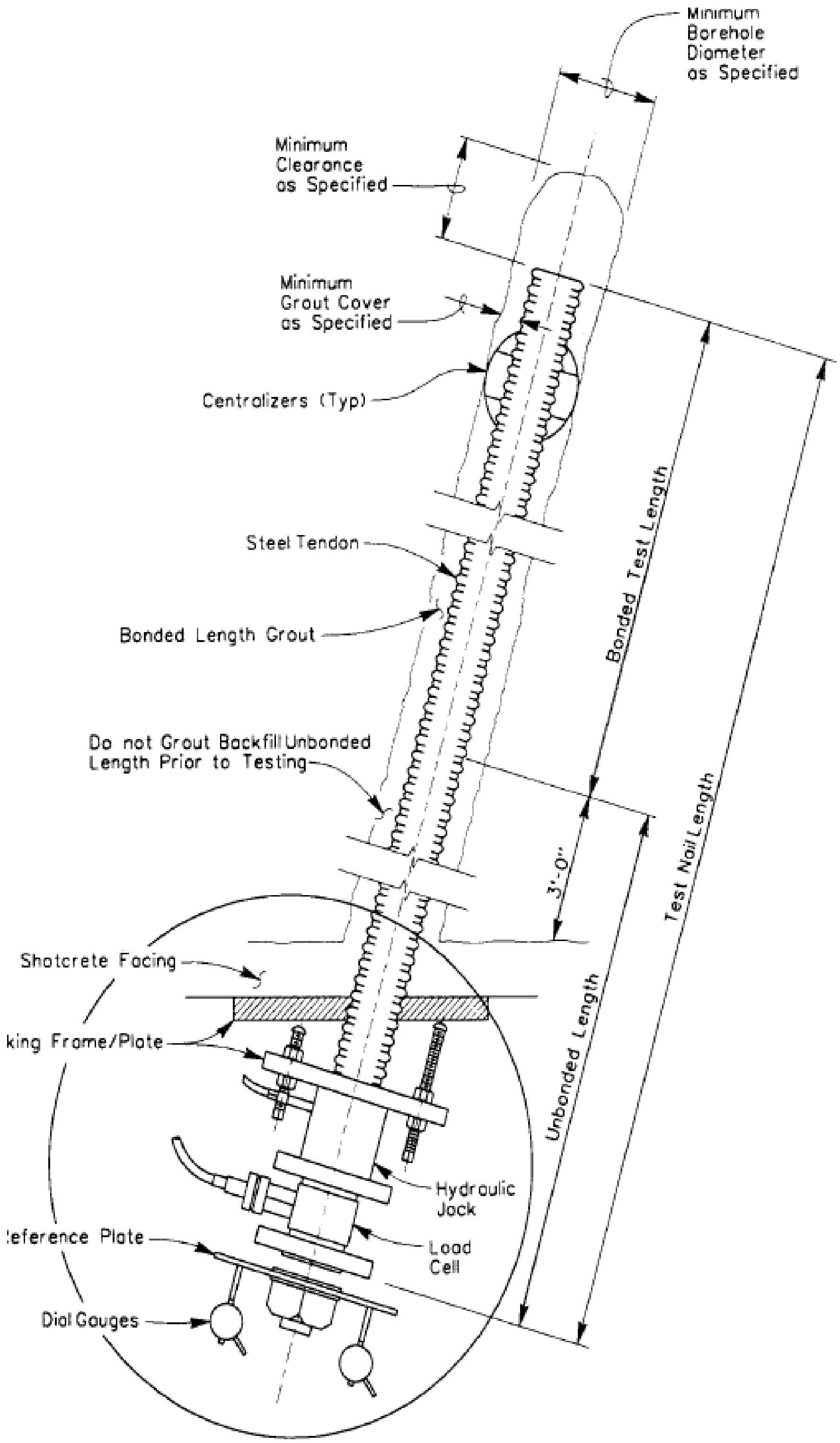


FIGURE 11-5 Verification/test nail detail

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.

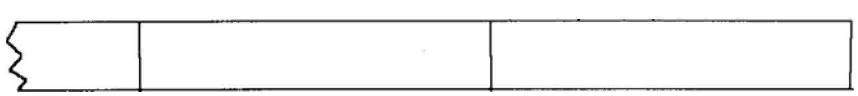


FIGURE 12-1 Single sheet piling

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.

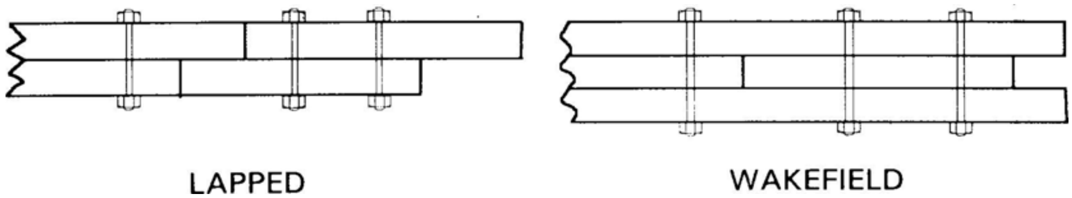


FIGURE 12-2 Lapped and wakefield sheet piling

"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.

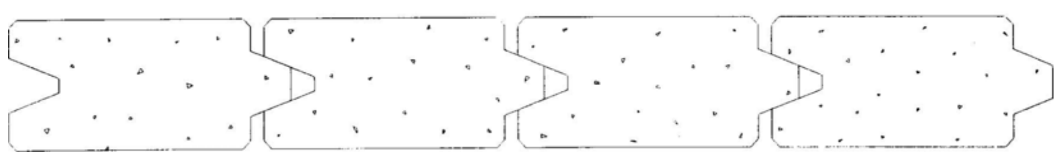


FIGURE 12-4 Concrete sheet piling

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.

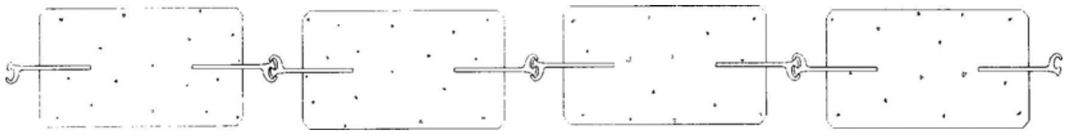


FIGURE 12-5 Concrete sheet piling with steel interlocks

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.

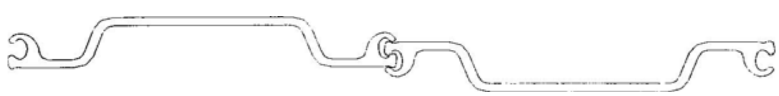


FIGURE 12-7 Arch-web steel sheet piling

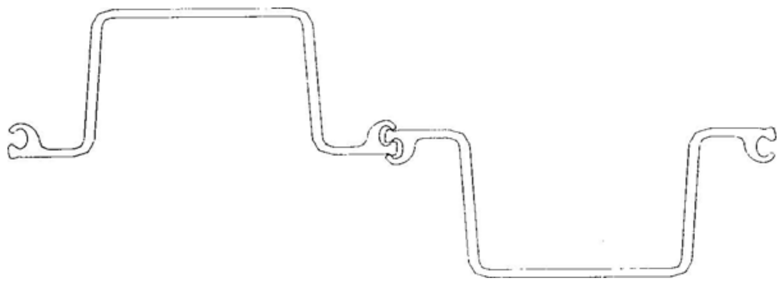


FIGURE 12-8 Deep-arch steel sheet piling

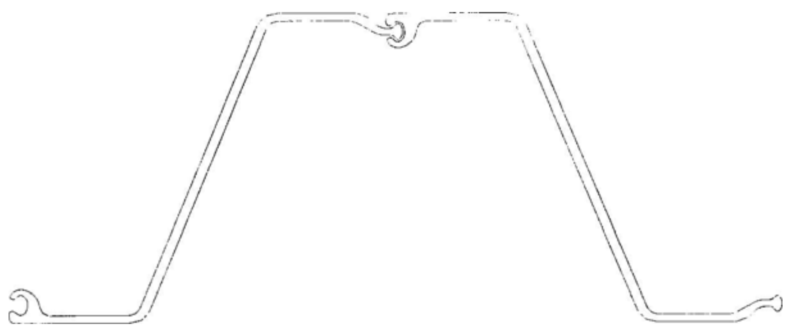


FIGURE 12-9 Z-section steel sheet piling

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.

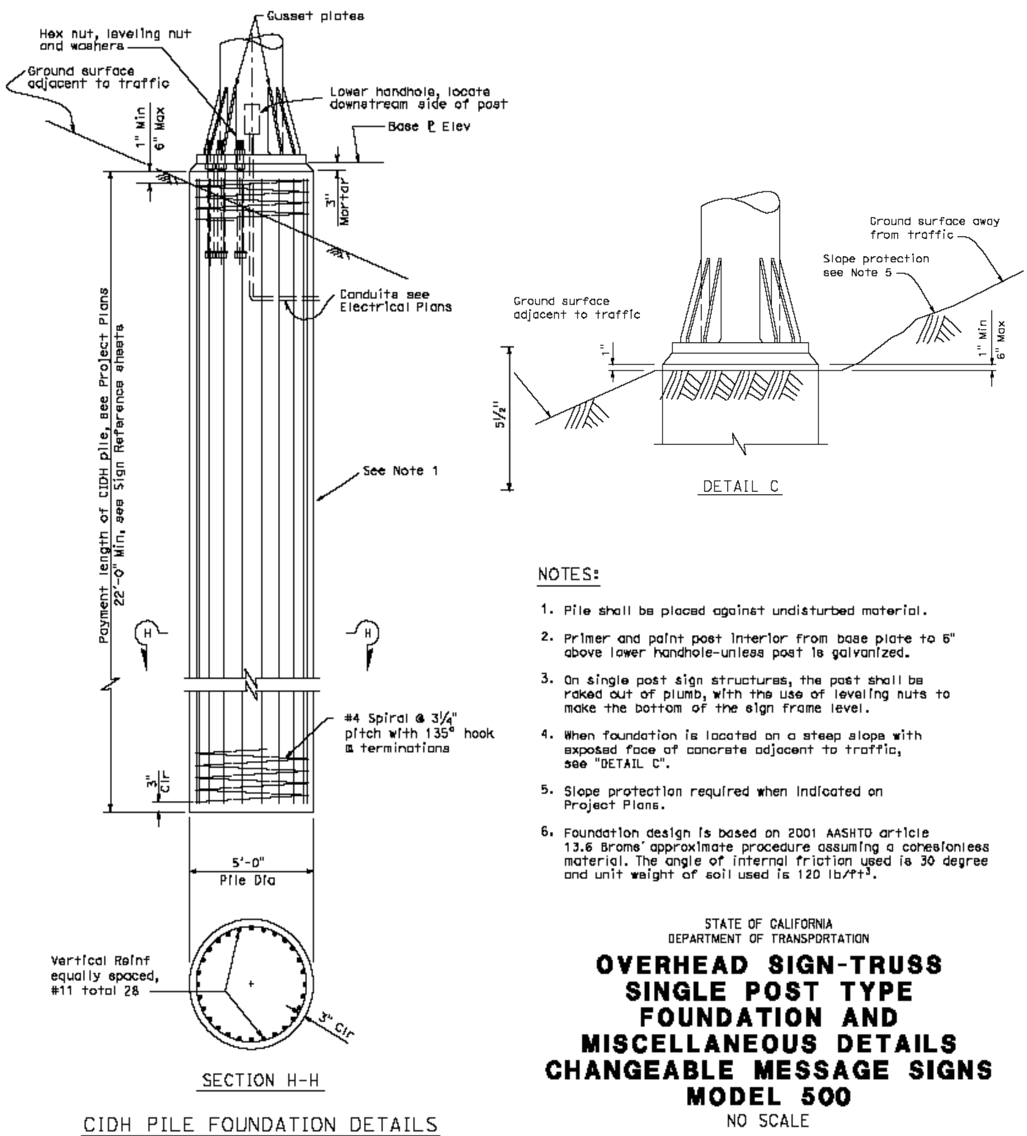


FIGURE 13-1 CMS details from 2006 Standard Plan S116

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APPENDIX

A Foundation Investigations

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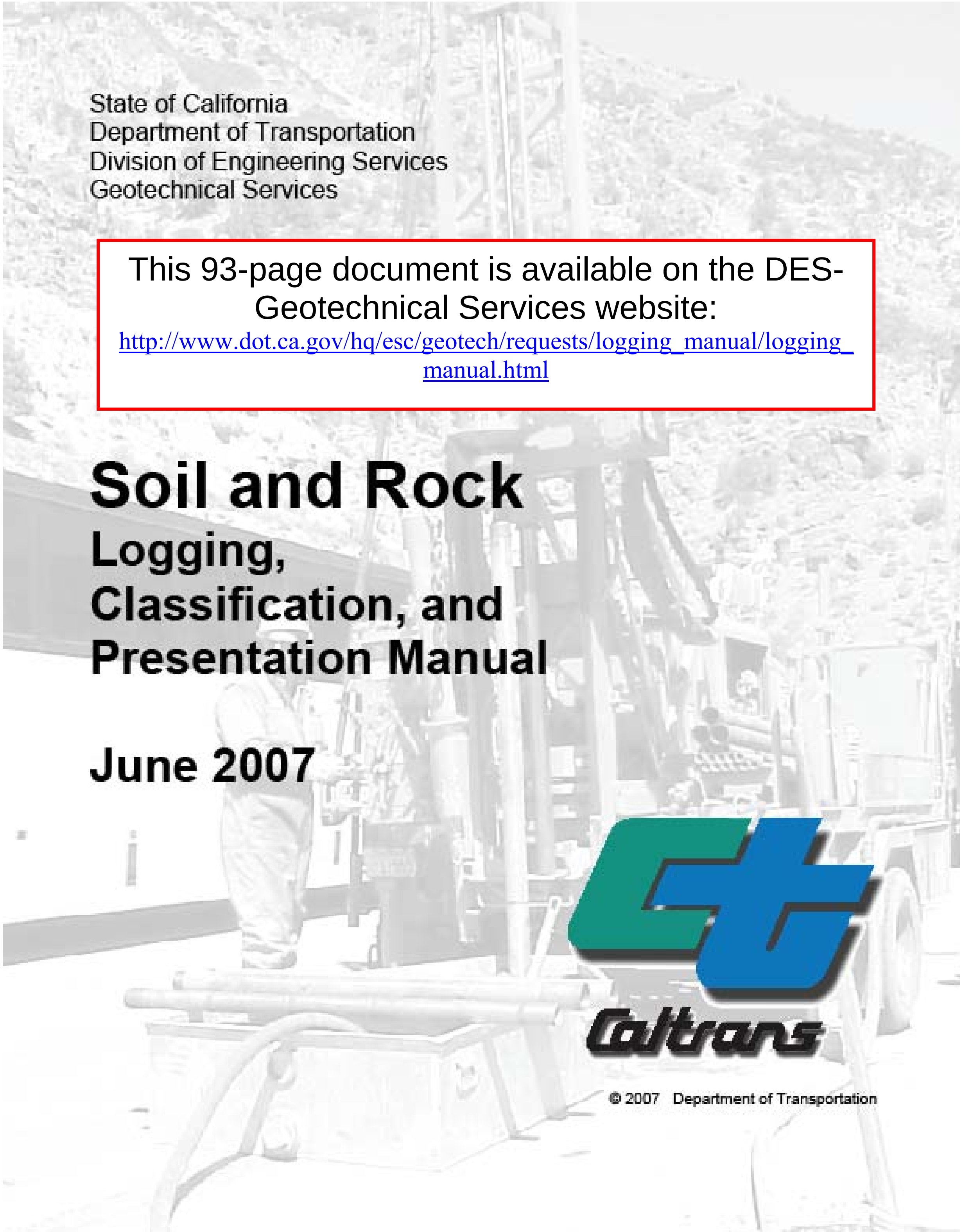


State of California
Department of Transportation
Division of Engineering Services
Geotechnical Services

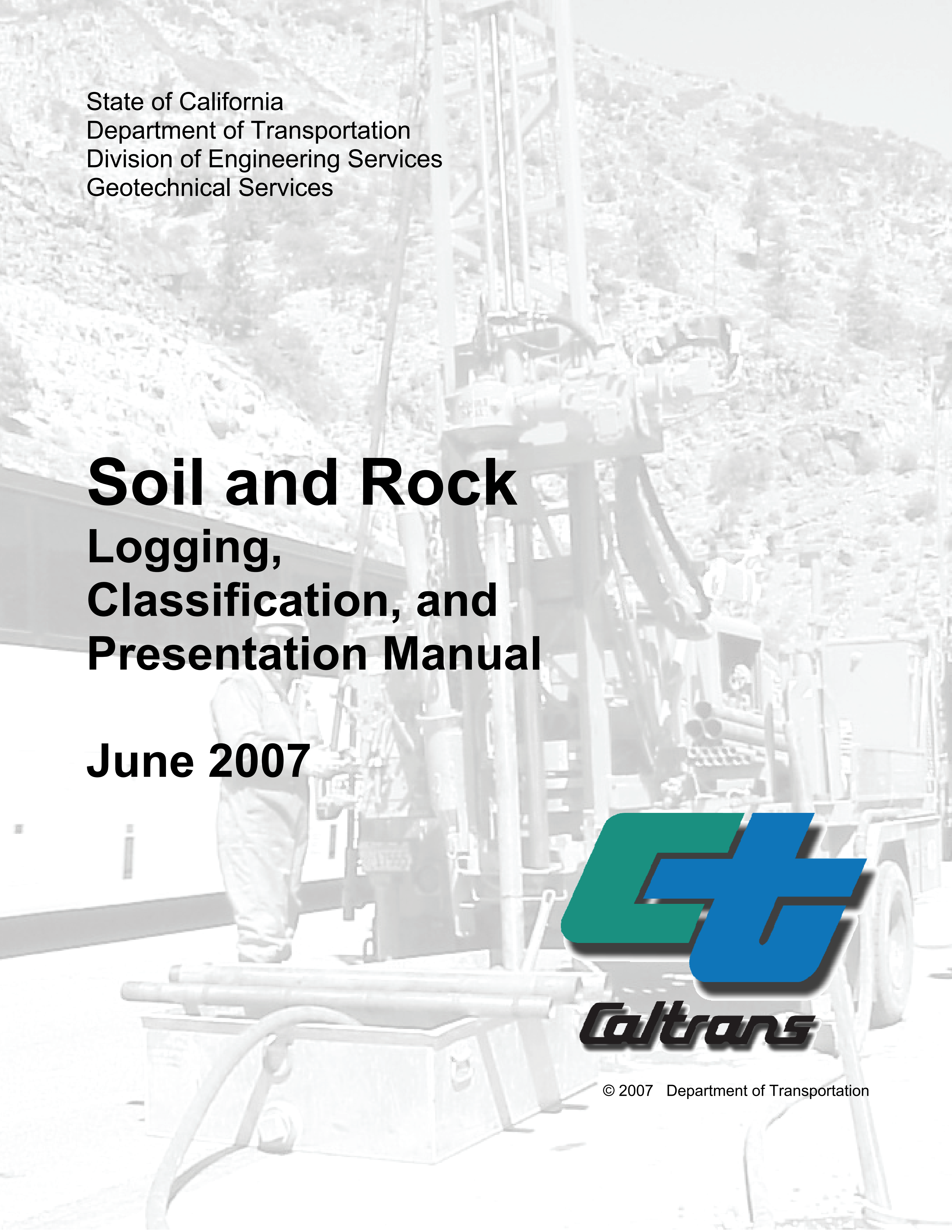
This 93-page document is available on the DES-Geotechnical Services website:
http://www.dot.ca.gov/hq/esc/geotech/requests/logging_manual/logging_manual.html

Soil and Rock

Logging, Classification, and Presentation Manual

June 2007

Caltrans



State of California
Department of Transportation
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Soil and Rock

Logging, Classification, and Presentation Manual

June 2007

Caltrans

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Preface

Detailed soil and rock descriptions and classifications are an essential part of the information developed to support Caltrans' design and construction processes. Subsurface information for any given area is, and can be, generated and accumulated over a prolonged period of time by various geotechnical practitioners for different projects and purposes. It is imperative that geotechnical practitioners working on Caltrans projects use standardized terminology and procedures to maintain consistency in borehole logging and reporting practices. Geotechnical Services in the Division of Engineering Services, has published this Manual to ensure the Department's investment in maintaining consistent logging practices.

This Manual, "*Soil and Rock Logging, Classification, and Presentation Manual*", improves upon the original version of the manual, "*Soil and Rock Logging Classification Manual (Field Guide)*", published in 1996, by addressing the following:

- Serves as a comprehensive reference for Departmental staff, consultants, and contractors
- Provides standardized soil *description* and *identification* procedures utilizing field data
- Provides standardized soil *classification* procedures utilizing laboratory data
- Provides standardized rock *description* and *identification* procedures utilizing field and laboratory data
- Serves as a basis for Departmental products and tools, such as:
  - Boring Log presentation formats,
  - Log of Test Boring (LOTB) legend sheets,
  - Descriptive terminology presented in geotechnical reports, and
  - Geotechnical Data Management System

The information presented in this Manual is based predominantly on American Standards for Testing Materials (ASTM) and other publications. These references provide standardized methods for identifying, describing, or classifying soil and rock; however, they do not provide adequate descriptive terminology and criteria for identifying soil and rock for engineering purposes. Consequently, this manual extends, and in some cases modifies these standards to include additional descriptive terms and criteria.

In addition to soil and rock identification, description, or classification, this Manual contains instructions that present Departmental standards for borehole and sample identification, minimum material requirements for various laboratory tests, and boring log presentation formats.

Geotechnical Services staff and any other organization providing geotechnical reports or records of geotechnical investigations for the Department shall use the procedures presented in this Manual.

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(Names have been listed alphabetically.)

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Section 1: Introduction

1.1 Intent of this Manual

The intent of this Manual is to define the Department's practices and procedures for soil and rock description, identification, classification, and preparation of boring logs.

Standardized terminology and consistent presentation procedures for projects statewide benefit the Department's staff, engineering consultants, bidders, and contractors. Geotechnical Services staff as well as any other organization providing geotechnical reports or records of geotechnical investigations to the Department shall follow the procedures presented in this Manual.

The following terms, as defined below, are used throughout this Manual to convey the Department's policy:

Term	Definition
Shall, Required	*Mandatory Standard*. The associated provisions **must** be used. There is no acceptable alternative.
Should	*Advisory Standard*. The associated provisions are preferred practices.
May, Optional	*Permissive Standard.* Use or application of the associated provisions is left to the discretion of the Geoprofessional.

1.2 Limitations

Although this manual may be used to train new employees, this is not its primary intent.

This manual does not replace education or experience and shall be used in conjunction with professional judgment. Not all aspects of this manual may be applicable in all circumstances and should be applied with consideration of a project's many unique aspects.

This manual does not purport to address all of the safety problems, if any, associated with its use. It is the responsibility of the user of this standard to establish, or adhere to, appropriate safety and health practices and determine the applicability of regulatory limitations prior to use. The reader shall follow at a minimum, the *Caltrans Code of Safe Drilling Practices*.

1.3 Exceptions to Policy

Exceptions to the policy and procedures set forth in this Manual require prior approval by the Geotechnical Services Deputy Division Chief. Staff shall use the procedure for obtaining approval for an exception, as documented in a memorandum to all staff dated June 15, 2007, included in Appendix C.

1.4 Revisions to the Manual

Staff who wish to propose changes to the Manual shall do so in accordance with the *Soil and Rock Logging, Classification, and Presentation Manual Committee Charter and Standard Procedures,* included in Appendix C.

1.5 Organization of this Manual

The Manual is divided into five sections, as described below:

Section 1

- Explains the intent and organization of this Manual and the process for requesting exceptions and proposing changes to the Manual
- Presents an overview of the logging process and acceptable presentation formats

Section 2

- Presents the Department's field description and indentification procedures for soil and rock, without the benefit of laboratory testing
- Explains procedures for handling and labeling of samples
- Explains how to perform a quality check of borehole logs and soil and rock samples

Section 3

- Describes the Department's classification procedures for soil and rock samples for which the data was refined by appropriate laboratory tests

Section 4

- Presents the process for developing and presenting geotechnical information on a *Log of Test Boring (LOTB)* or a *Boring Record (BR)*.

Section 5

- Specifies presentation content and formats for *Log of Test Boring (LOTB)* and *Boring Record (BR)*.

1.6 Overview of the Logging Process and Presentation Formats

The Department uses the following formats to present subsurface information:

- Log of Test Boring (LOTB), and/or
- Boring Record (BR).

An LOTB is typically associated with a structure facility and is attached to Project Plans. A BR is typically associated with an earthwork facility and is attached to a Geotechnical Report.

The process of creating boring logs, i.e., Log of Test Boring (LOTB) and Boring Record (BR) can be summarized in four steps:

- Field sampling and descriptions *(Section 2)*
- Quality check of field descriptions *(Section 2)*
- Refinement of descriptions, and classification of soil, based on laboratory test results, if performed *(Section 3)*
- Preparation of the boring logs *(Sections 4 and 5)*

(See Figure 1-1.)

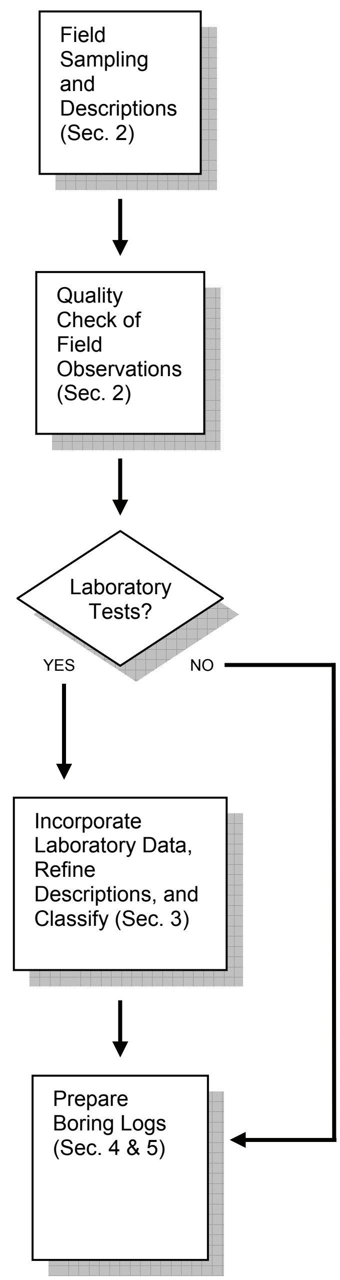
Prior to the field investigation, the geoprofessional should have general understanding of the local soils and geologic information, and know the parameters and the basic descriptors required for the planned analyses. Specific laboratory tests, such as strength, consolidation, or permeability may govern the type of drilling and sampling used.

Recovering and labeling, and accurately describing and classifying samples is a detailed process that typically necessitates a thorough check of field notes and samples in the office before requesting laboratory tests. In some cases, the geoprofessional may use only field observations. *(See Section 2.)*

In other cases, it may be the judgment of the geoprofessional that a combination of field observations and laboratory test results are needed to describe or classify the soil or rock samples, and generate appropriate layer descriptions for LOTB or BR. *(See Sections 2 and 3.)*

If the results of laboratory tests change the description of the sample generated by field observation, the classification and/or description resulting from the laboratory tests shall be used on the LOTB and/or the BR, and in the geotechnical report. Disclosure of the tests on the LOTB and/or the BR makes it clear whether the sample or layer descriptor was based on visual observation or on laboratory test results. *(See Sections 4 and 5.)*

Figure 1-1
Logging and Presentation Process



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Section 2: Field Procedures for Soil and Rock Logging, Description, and Identification

2.1 Introduction

This section presents the procedures for logging, describing, and identifying soil and rock samples in the field based on visual and manual procedures.

The information presented in this section is predominantly based on:

- American Society for Testing and Materials (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.

Although ASTM D 2488-06 provides a standardized method for indentification of soils, it does not provide adequate descriptive terminology and criteria for identifying soils for engineering purposes. Section 17 of ASTM D 2488-06 states, "*this practice provides qualitative information only,*" and Note 4 adds, "*The ability to describe and identify soils correctly … may also be acquired systematically by comparing numerical laboratory test results for typical soils of each type with their visual and manual characteristics*."

This Manual extends, and in some cases modifies, the ASTM standard to include additional descriptive terms and criteria. It is not our intent to replace the ASTM standards but to build on them, and make them better understood.

The identifications and descriptions in the field logs may be corrected, calibrated, or verified later based on laboratory test results of selected soil samples to develop the final boring logs, as described in Section 3.

The process of correction, calibration, and verification in developing the updated logs based on laboratory test results can effectively serve the purpose of self-training and self-calibration for future field identification and description of soil samples.

In addition to soil and rock identification and description, this section contains instructions that describe proper hole and sample identification practices, and minimum material requirements for various laboratory tests.

2.2 General Project and Hole Information

One of the most important aspects of field work is properly identifying the location of the project site, drilling tools and methods used, and the personnel involved in the field work. Figure 2-1 presents the information that is required to be recorded for every hole.

Figure 2-1
Information Required for Borehole

Item	Description
1	**Date(s) of work**
2	**Hole Identification**
3	**Project and Site Information:** • Project Name • Structure/Bridge Name and Number (if available) • Project Number (Charge District - Expenditure Authorization, 8-digits) • District • County • Route • Postmile, range and prefix
4	**Borehole Location and Elevation:** • Location (at least one of the following): ○ Station and offset ○ Latitude and longitude, horizontal datum ○ Northing and Easting, local coordinate reference system *Note:* In the absence of accurate coordinate data, a suitable and verifiable field description may be temporarily used. (e.g., postmile and centerline offset, distance to fixed object or benchmark, etc.) • Elevation, vertical datum, benchmark description • Survey method(s) used, approximate accuracy
5	**Personnel:** • Logger/Geoprofessional • Drillers
6	**Drilling and Sampling Equipment (verify with Driller):** • Drill rig (manufacturer and model, and Caltrans Equipment Identification number) • Drilling method (mud rotary, air rotary, solid auger, hollow stem auger. etc.) • Drill rod description (type, diameter) • Drill bit description • Casing (type, diameter) and installation depth • SPT Hammer Type: Safety/Automatic Hammer, etc. ○ Lifting mechanism (for safety hammer) ○ Manufacturer & model ○ Caltrans Equipment Identification number ○ Measured SPT energy efficiency ratio (if available) • Type of sampler(s) and size(s) ○ Undisturbed Shelby tube ○ Undisturbed Piston ○ Split spoon (e.g. SPT, Cal Mod, etc.) ○ Core (both rock and soil) ○ Disturbed (include auger cuttings) ○ Other
7	**Groundwater** • Method (observed while drilling, measured in hole, etc.) • Date, time, and depth of each reading
8	**Hole Completion** • Cause of termination (e.g., drilled to depth, refusal, early termination of traffic control, etc.) • Abandonment (e.g., grout, soil cuttings, dry bentonite chips, piezometers installed, slope inclinometer installed, TDR, instrumentation, etc.)

2.3 Assignment of Hole Identification

Holes shall be identified using the following convention:

HHH – YY – NNN

Where:

HHH: The Hole Type or Sounding Codes defined in Figure 2-2, which generally follow ASTM D 6453-99

YY: 2-digit year

NNN: 3-digit number (001-199)

The numbers 001–099 are reserved for holes used to produce a foundation report; numbers 101–199 are reserved for holes used to produce a geotechnical design report.

The *YY-NNN* component of the hole identification is unique and matched to a Caltrans project expenditure authorization number (EA), not to a site, structure, or bridge number. If two drilling methods are used, such as auger boring followed by rotary drilled boring, the prominent tool governs the selection of Hole Type Code (HHH).

Figure 2-2
Hole Type Code and Description

Hole Type Code	Description
A	Auger boring (hollow or solid stem, bucket)
R	Rotary drilled boring (both conventional and wire-line)
P	Rotary percussion boring (Air)
HD	Hand driven (1-inch soil tube)
HA	Hand auger
D	Driven (dynamic cone penetrometer)
CPT	Cone Penetration Test
O	Other

2.4 Soil Description and Identification Procedures

This section presents the method for identification and description of soil based on ASTM D 2488-06 and USBR (2001). The detail of description provided for a particular soil should be dictated by the complexity and objectives of the project. Optional descriptors should be considered by the geoprofessional on a project by project basis.

2.4.1 Soil Description and Identification

When describing and identifying soil in the field, the geoprofessional shall record the field data following the sequence presented in Figure 2-3 below. Items marked "required" shall be used, when applicable, to describe the soil sample to ensure complete descriptive coverage. For example, percent cobbles and/or boulders is only required if cobbles and/or boulders are encountered.

Figure 2-3

Identification and Description Sequence

Sequence	Identification Components	Refer to Section	Required	Optional
1	Group Name	2.4.2	●	
2	Group Symbol	2.4.2	●	
	Description Components			
3	Consistency (for cohesive soils)	2.4.3	●	
4	Apparent Density (for cohesionless soils)	2.4.4	●	
5	Color (in moist condition)	2.4.5	●	
6	Moisture	2.4.6	●	
7	Percent of cobbles or boulders	2.4.7	●	
8	Percent or proportion of soils	2.4.8	●	
9	Particle Size Range	2.4.9	●	
10	Particle Angularity	2.4.10		○
11	Particle Shape	2.4.11		○
12	Plasticity (for fine-grained soils)	2.4.12	●	
13	Dry Strength (for fine-grained soils)	2.4.13		○
14	Dilatency (for fine-grained soils)	2.4.14		○
15	Toughness (for fine-grained soils)	2.4.15		○
16	Structure	2.4.16		○
17	Cementation	2.4.17	●	
18	Description of Cobbles and Boulders	2.4.18	●	
19	Additional Comments	2.4.19		○

Below are some examples that illustrate the application of the descriptive sequence based on field procedures.

Example of a complete descriptive sequence for a sample using required and optional components:

> *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.*

Example of a complete descriptive sequence for a soil sample using only required components:

> *Well-graded SAND with GRAVEL (SW), medium dense, brown to light gray, wet, little coarse GRAVEL, mostly coarse to fine SAND, few fines, weak cementation.*

Example of a complete descriptive sequence that omits the percent or proportion of the primary soil constituent, which may be used when the percentage or proportion of the primary soil constituent can be clearly inferred:

> *Well-graded SAND with GRAVEL (SW), medium dense, brown to light gray, wet, little coarse GRAVEL, few fines, weak cementation.*

2.4.1.1 Soil Description for Intensely Weathered or Decomposed Rock

Intensely weathered or decomposed rock that is friable and that can be reduced to gravel size or smaller by normal hand pressure shall be identified and described as rock followed by the soil identification or classification, and description in parenthesis (per Section 2.5).

2.4.2 Group Name and Group Symbol

Using visual examination and simple manual tests, this section provides standardized criteria and procedures for describing and identifying soil in the field per ASTM D 2488-06. The soil is to be identified by assigning a group name and symbol. The Figures in this section are to be used for the identification of both fine and coarse-grained soil and to determine the appropriate group symbol(s) and name(s) to be used.

The ASTM procedure for identifying and describing fine-grained and coarse-grained soils is only applicable to material passing the 3-inch sieve. If the presence of cobbles or boulders or both is identified during the site exploration, the percentage of cobbles and boulders shall be estimated and reported per Section 2.4.7.

Borderline Symbol – Because ASTM D 2488-06 is based on estimates of particle size distribution and plasticity characteristics, it may be difficult to clearly identify the soil as belonging to one category. To indicate that the soil may fall into one of two possible basic groups, a borderline symbol shall be used with the two symbols separated by a slash. For example: SC/CL or CL/CH.

A borderline symbol shall be used when:

- The percentage of fines is estimated to be between 45 and 55%. One symbol shall be for a coarse-grained soil with fines; the other for a fine-grained soil, e.g., GM/ML or CL/SC.
- The percentage of sand and the percentage of gravel are estimated to be about the same, e.g., GP/SP, SC/GC, GM/SM.
- The soil could be well graded or poorly graded, e.g., GW/GP, SW/SP.
- The soil could either be a silt or a clay, e.g., CL/ML, CH/MH, SC/SM.
- A fine-grained soil has properties that indicate that it is at the boundary between a soil of low plasticity and a soil of high plasticity, e.g., CL/CH, MH/ML.

The order of the borderline symbols shall reflect similarity to surrounding or adjacent soils. For example, soils in a borrow area have been identified as CH, and one sample is considered to have a borderline symbol of CL and CH. To show similarity, the borderline symbol shall be CH/CL.

The group name for a soil with a borderline symbol shall be the group name for the first symbol, except for:

- CL/CH lean to fat clay,
- ML/CL clayey silt, and
- CL/ML silty clay

Borderline symbols should not be used indiscriminately. Use of a single group symbol is preferable.

Dual Symbol – A dual symbol is two symbols separated by a hyphen, e.g., GP-GM, SW-SC, CL-ML. They are used to indicate that the soil has been identified as having the properties of a classification in accordance with ASTM Test Method D 2487-06 requiring dual symbols, i.e., when the soil has between 5 and 12% fines, or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.

2.4.2.1 Fine Grained Soils

A soil is considered to be fine-grained if it contains 50% or more fines. Particles that pass through a Number 200 sieve are defined as fine-grained. Fine-grained soils shall be assigned the Group Name and Symbol according to Figure 2-4, below.

Figure 2-4

Flow chart for fine-grained soils *(from ASTM D 2488-06)*

Group Symbol	Fines	Coarseness	Sand or Gravel	Group Name
CL	<30% plus No.200	<15% plus No.200		Lean CLAY
		15-25% plus No.200	% sand ≥ % gravel	Lean CLAY with SAND
			% sand < % gravel	Lean CLAY with GRAVEL
	≥30% plus No.200	% sand ≥ % gravel	< 15% gravel	SANDY lean CLAY
			≥ 15% gravel	SANDY lean CLAY with GRAVEL
		% sand < % gravel	< 15% sand	GRAVELLY lean CLAY
			≥ 15% sand	GRAVELLY lean CLAY with SAND
ML	<30% plus No.200	<15% plus No.200		SILT
		15-25% plus No.200	% sand ≥ % gravel	SILT with SAND
			% sand < % gravel	SILT with GRAVEL
	≥30% plus No.200	% sand ≥ % gravel	< 15% gravel	SANDY SILT
			≥ 15% gravel	SANDY SILT with GRAVEL
		% sand < % gravel	< 15% sand	GRAVELLY SILT
			≥ 15% sand	GRAVELLY SILT with SAND
CH	<30% plus No.200	<15% plus No.200		Fat CLAY
		15-25% plus No.200	% sand ≥ % gravel	Fat CLAY with SAND
			% sand < % gravel	Fat CLAY with GRAVEL
	≥30% plus No.200	% sand ≥ % gravel	< 15% gravel	SANDY fat CLAY
			≥ 15% gravel	SANDY fat CLAY with GRAVEL
		% sand < % gravel	< 15% sand	GRAVELLY fat CLAY
			≥ 15% sand	GRAVELLY fat CLAY with SAND
MH	<30% plus No.200	<15% plus No.200		Elastic SILT
		15-25% plus No.200	% sand ≥ % gravel	Elastic SILT with SAND
			% sand < % gravel	Elastic SILT with GRAVEL
	≥30% plus No.200	% sand ≥ % gravel	< 15% gravel	SANDY elastic SILT
			≥ 15% gravel	SANDY elastic SILT with GRAVEL
		% sand < % gravel	< 15% sand	GRAVELLY elastic SILT
			≥ 15% sand	GRAVELLY elastic SILT with SAND
OL/ OH	<30% plus No.200	<15% plus No.200		ORGANIC SOIL
		15-25% plus No.200	% sand ≥ % gravel	ORGANIC SOIL with SAND
			% sand < % gravel	ORGANIC SOIL with GRAVEL
	≥30% plus No.200	% sand ≥ % gravel	< 15% gravel	SANDY ORGANIC SOIL
			≥ 15% gravel	SANDY ORGANIC SOIL with GRAVEL
		% sand < % gravel	< 15% sand	GRAVELLY ORGANIC SOIL
			≥ 15% sand	GRAVELLY ORGANIC SOIL with SAND

Clay and Silt – Identify the soil as a Lean CLAY (CL), a Fat CLAY (CH), a SILT (ML), or an Elastic SILT (MH), using the criteria in Figure 2-5:

Figure 2-5

Identification of clayey and silty soils

Group Symbol	Dry Strength	Dilatancy	Toughness
ML	None to low	Slow to rapid	Low or thread cannot be formed
CL	Medium to high	None to slow	Medium
MH	Low to medium	None to slow	Low to medium
CH	High to very high	None	High

Organic Soil – Identify the soil as organic, OL/OH, if the soil contains enough organic particles to influence the soil properties. Organic soils usually have a dark brown to black color and may have an organic odor. Often, organic soils will change color, for example, black to brown, when exposed to the air. Some organic soils will lighten in color significantly when air-dried. Organic soils normally will not have a high toughness or plasticity. The thread for the toughness test will be spongy.

Identification of Peat – A sample composed primarily of vegetable tissue in various stages of decomposition that has a fibrous to amorphous texture, usually a dark brown to black color, and an organic odor, shall be designated as a highly organic soil and shall be identified with the Group Name and Symbol, PEAT (PT), and not subjected to the identification procedures described hereafter.

2.4.2.2 Coarse-Grained Soil

A soil is considered coarse-grained if it contains fewer than 50% fines. (Coarse-grain particles will not pass through a Number 200 sieve.) Soil is identified as gravel if the percentage of gravel is estimated to be greater than the percentage of sand. Soil is identified as sand if the percentage of gravel is estimated to be equal to or less than the percentage of sand.

Figure 2-7
Flow chart for coarse-grained soils *(from ASTM D-2488-06)*

	Fines	Grade	Type of Fines	Group Symbol	Sand/Gravel	Group Name
Gravel	≤ 5%	Well		GW	< 15% sand	Well-graded GRAVEL
					≥ 15% sand	Well-graded GRAVEL with SAND
		Poorly		GP	< 15% sand	Poorly graded GRAVEL
					≥ 15% sand	Poorly graded GRAVEL with SAND
	10%	Well	ML or MH	GW-GM	< 15% sand	Well-graded GRAVEL with SILT
					≥ 15% sand	Well-graded GRAVEL with SILT and SAND
			CL or CH	GW-GC	< 15% sand	Well-graded GRAVEL with CLAY
					≥ 15% sand	Well-graded GRAVEL with CLAY and SAND
		Poorly	ML or MH	GP-GM	< 15% sand	Poorly graded GRAVEL with SILT
					≥ 15% sand	Poorly graded GRAVEL with SILT and SAND
			CL or CH	GP-GC	< 15% sand	Poorly graded GRAVEL with CLAY
					≥ 15% sand	Poorly graded GRAVEL with CLAY and SAND
	≥ 15%		ML or MH	GM	< 15% sand	SILTY GRAVEL
					≥ 15% sand	SILTY GRAVEL with SAND
			CL or CH	GC	< 15% sand	CLAYEY GRAVEL
					≥ 15% sand	CLAYEY GRAVEL with SAND
Sand	≤ 5%	Well		SW	< 15% gravel	Well-graded SAND
					≥ 15% gravel	Well-graded SAND with GRAVEL
		Poorly		SP	< 15% gravel	Poorly graded SAND
					≥ 15% gravel	Poorly graded SAND with GRAVEL
	10%	Well	ML or MH	SW-SM	< 15% gravel	Well-graded SAND with SILT
					≥ 15% gravel	Well-graded SAND with SILT and GRAVEL
			CL or CH	SW-SC	< 15% gravel	Well-graded SAND with CLAY
					≥ 15% gravel	Well-graded SAND with CLAY and GRAVEL
		Poorly	ML or MH	SP-SM	< 15% gravel	Poorly graded SAND with SILT
					≥ 15% gravel	Poorly graded SAND with SILT and GRAVEL
			CL or CH	SP-SC	< 15% gravel	Poorly graded SAND with CLAY
					≥ 15% gravel	Poorly graded SAND with CLAY and GRAVEL
	≥ 15%		ML or MH	SM	< 15% gravel	SILTY SAND
					≥ 15% gravel	SILTY SAND with GRAVEL
			CL or CH	SC	< 15% gravel	CLAYEY SAND
					≥ 15% gravel	CLAYEY SAND with GRAVEL

2.4.3 Consistency (Cohesive Soils)

The preferred procedure for the determination of consistency of cohesive soils is to obtain relatively undisturbed samples and perform field tests with a pocket penetrometer or torvane. *(See Appendix A for details on the test procedures.)*

Use the terms and criteria indicated in Figure 2-8 below to describe the consistency of cohesive soils. These terms generally follow, with some modifications, AASHTO (1988) and Bureau of Reclamation (2001) standards.

Figure 2-8
Descriptors for Consistency of Cohesive Soils

Description	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.25 to 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 2.0	Indented by thumbnail with difficulty

2.4.4 Apparent Density (Cohesionless Soils)

Use the AASHTO (1988) standards to describe the apparent density of cohesionless soils, as indicated in Figure 2-9 below.

Figure 2-9
Descriptors for Apparent Density of Cohesionless Soils

Description	SPT N_{60} (blows/ft)
Very loose	0 – 4
Loose	5 – 10
Medium dense	11 – 30
Dense	31 – 50
Very dense	>50

Apparent density of a coarse-grained (cohesionless) soil is based on a corrected Standard Penetration Test (SPT) N_{60} value as described in Appendix A and provided here:

$$N_{60} = N_{measured}\ X\ (ER_i / 60)$$

where,

$$ER_i = \text{Hammer energy ratio}$$

N values are highly dependent on the energy efficiency of the SPT method. Inconsistency in the N values across a site may be attributed to variations in energy efficiency between different drill rigs and crews.

2.4.5 Color

Color is an important property in identifying organic soils, and it may also be useful in identifying materials of similar geologic origin within a given locality. Use the color name from the *Munsell Color System* to describe the color of a <u>moist</u> soil sample at the time of drilling and sampling. If the sample contains layers or patches of varying colors, record this information and describe all observed colors. For example:

Brown to light yellowish brown

For additional information, see ASTM D 1535-06, *Standard Practice for Specifying Color by the Munsell System*.

2.4.6 Moisture

Use the ASTM D 2488-06 standard to describe the moisture condition, as indicated in Figure 2-10 below.

Figure 2-10
Descriptors for Moisture

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

2.4.7 Percent of Cobbles or Boulders

When particles greater than 3 inches in diameter are encountered, they shall be identified and described as "COBBLES," or "BOULDERS," or "COBBLES and BOULDERS" as defined in Section 2.4.9. Cobbles and boulders reported as present within a matrix shall be estimated, by volume, and reported by percentage of total volume.

Estimation of volume of cobbles and/or boulders is based upon recovered intersected lengths, drilling chatter, and observations and experience of the driller and/or geoprofessional.

A subset of rock descriptors shall be used to describe cobbles and boulders as explained in Section 2.4.18. Isolated boulders may be treated as individual units and described as such.

For example, if it is estimated that 40% by volume of the material is cobbles, describe the sample in this way:

> *Well-graded SAND with GRAVEL and COBBLES (SW), medium dense, brown to light gray, wet, about 40% COBBLES, about 20% coarse subrounded to rounded flat and elongated GRAVEL, about 75% coarse to fine rounded SAND, about 5% fines, weak cementation; COBBLES consist of sandstone, fresh, hard, intersecting lengths from 8 to 10 inches.*

Note, the percentages of constituents in the example do not add up to 100%., as cobbles are estimated by total volume, whereas gravel, sand,

and fines, are estimated by weight of the total sample excluding the cobbles and boulders, per Section 2.4.8

If the sample or layer is estimated to be more than 50% cobbles and/or boulders by volume, the layer shall be described as "COBBLES" or "BOULDERS" or "COBBLES and BOULDERS" with the soil matrix description following. Note, this is a departure from the descriptive sequence in Section 2.4.1. For example, if it is estimated that 60% by volume of the material was cobbles, describe the layer as:

> *COBBLES with some well-graded SAND with GRAVEL, about 60% COBBLES (sandstone, fresh, hard, intersecting lengths from 8 to 10 inches), matrix consists of 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.*

Note that the Group Symbol is not used in the last example, because the cobbles and boulders were the predominant material.

2.4.8 Percent or Proportion of Soils

Use the ASTM D 2488-06 standard to describe the estimated percentage (to the nearest 5%) or proportion of gravel, sand, and fines, by weight of the total sample excluding the cobbles and boulders, as shown in Figure 2-11, below.

Figure 2-11
Descriptors for percent or proportion of soils

Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

The percentages of gravel, sand, and fines must add up to 100 %. The term "about" shall be used if the percentage or proportion of constituents is estimated in the field. (The word "about" shall be removed if the percentage was revised based on laboratory particle size analysis results.)

2.4.9 Particle Size

Use the ASTM D 2488-06 standard to describe the size of particles, as shown in Figure 2-12, below.

Figure 2-12
Descriptors for Particle Size

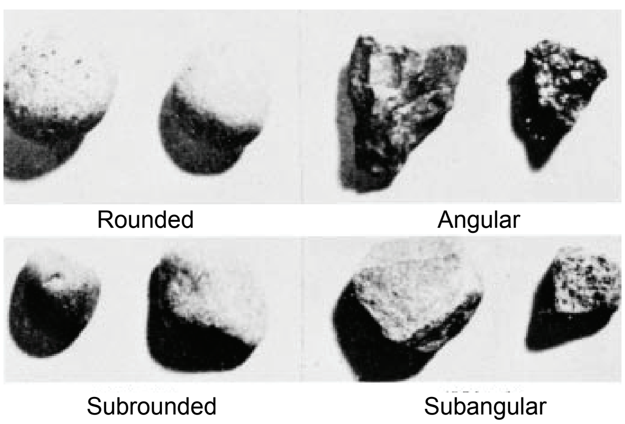
Description	Size	Familiar Example
Boulder	>12 in.	Larger than a basketball
Cobble	3 to 12 in.	Larger than a grapefruit or orange
Coarse Gravel	3/4 to 3 in.	Larger than a walnut or grape
Fine Gravel	No. 4 to 3/4 in.	Larger than a pea
Coarse Sand	No. 10 to No. 4	Larger than rock salt grain
Medium Sand	No. 40 to No. 10	Larger than openings of a window screen
Fine Sand	No. 200 to No. 40	Larger than a sugar grain

2.4.10 Particle Angularity

Use the ASTM D 2488-06 standard to describe the angularity of the sand (coarse sizes only), gravel, cobbles, and boulders, as indicated in Figure 2-13 below.

Figure 2-13
Descriptors for particle angularity

Description	Criteria
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description, but have rounded edges
Subrounded	Particles have nearly plane sides, but have well-rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges



2.4.11 Particle Shape

Use the ASTM D 2488-06 standard to describe the shape of the gravel, cobbles, and boulders *if* they meet any of the criteria in Figure 2-14.

The particle shape shall be described as follows where length, width, and thickness refer to the greatest, intermediate, and least dimensions of a particle, respectively.

Figure 2-14
Descriptors for Particle Shape

Description	Criteria
Flat	Particles with width/thickness > 3
Elongated	Particles with length/width > 3
Flat and Elongated	Particles meet criteria for both flat and elongated

2.4.12 Plasticity (for Fine-Grained Soils)

Use the ASTM D 2488-06 standard to describe the plasticity of the material based on observations made during the toughness test, as indicated in Figure 2-15 below.

Figure 2-15
Descriptors for Plasticity

Description	Criteria
Nonplastic	A 1⁄8-in. thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

2.4.13 Dry Strength (for Fine-Grained Soils)

Use the ASTM D 2488-06 standard to determine dry strength, as indicated in Figure 2-16 below. *(See Appendix A for details on field test procedures.)*

Figure 2-16
Descriptors for Dry Strength

Description	Criteria
None	The dry specimen crumbles into powder with mere pressure of handling.
Low	The dry specimen crumbles into powder with some finger pressure.
Medium	The dry specimen breaks into pieces or crumbles with considerable finger pressure
High	The dry specimen cannot be broken with finger pressure. Specimen will break into pieces between thumb and a hard surface.
Very High	The dry specimen cannot be broken between the thumb and a hard surface.

2.4.14 Dilatancy (for Fine-Grained Soils)

Use the ASTM D 2488-06 standard to determine dilatancy, as indicated in Figure 2-17 below. *(See Appendix A for details on field test procedures.)*

Figure 2-17
Descriptors for dilatancy

Description	Criteria
None	No visible change in the specimen
Slow	Water appears slowly on the surface of the specimen during shaking and does not disappear or disappears slowly upon squeezing
Rapid	Water appears quickly on the surface of the specimen during shaking and disappears quickly upon squeezing

2.4.15 Toughness (for Fine-Grained Soils)

Use the ASTM D 2488-06 standard to determine toughness, as indicated in Figure 2-18 below. *(See Appendix A for details on field test procedures.)*

Figure 2-18
Descriptors for toughness

Description	Criteria
Low	Only slight pressure is required to roll the thread near the plastic limit. The thread and the lump are weak and soft.
Medium	Medium pressure is required to roll the thread to near the plastic limit. The thread and the lump have medium stiffness.
High	Considerable pressure is required to roll the thread to near the plastic limit. The thread and the lump have very high stiffness

2.4.16 Structure

Use the ASTM D 2488-06 standard to describe the structure of intact soils, as indicated in Figure 2-19 below.

Figure 2-19
Descriptors for structure

Description	Criteria
Stratified	Alternating layers of varying material or color with layers at least ¼ in. thick; note thickness.
Laminated	Alternating layers of varying material or color with the layers less than ¼ in. thick; note thickness.
Fissured	Breaks along definite planes of fracture with little resistance to fracturing.
Slickensided	Fracture planes appear polished or glossy, sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness.
Homogeneous	Same color and appearance throughout.

2.4.17 Cementation

Use the ASTM D 2488-06 standard to describe the cementation of intact coarse-grained soils, as indicated in Figure 2-20 below.

Figure 2-20
Descriptors for cementation

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

2.4.18 Description of Cobbles and Boulders

Use the descriptive sequence for rock in Section 2.5. of this Manual to describe cobbles and boulders. The description shall include, at minimum, the following information:

- Rock identification
- Weathering
- Rock hardness
- Range of intersected lengths of core (An "intersected length" is the length of the intact core. This is not necessarily the size of the cobble or boulder.)

2.4.19 Additional Comments

Additional constituents and soil characteristics not included in the previous categories may be noted. Observations may include:

- Presence of roots or root holes
- Presence of mica, gypsum, etc.
- Presence of voids
- Surface coatings on coarse-grained particles
- Oxide staining
- Cementing agents (e.g. calcium carbonate – see Appendix A.7)
- Odor
- Depositional history (i.e. Alluvium, Colluvium, Aeolian, Lacustrine, Fill)
- Geologic formation name or soil survey unit name

All soils shall be examined to see if they contain materials indicative of man-made fills. Man-made fill items shall be listed in each of the soil descriptions. Common fill indicators include glass, brick, clay pipe, dimensioned lumber, concrete debris, in-place pavement sections, asphalt debris, metal, plastics, plaster, etc. Other items that may suggest fill include buried vegetation mats, tree limbs, stumps, etc.

The size and distribution of fill indicators shall be noted. The limits (depth range) of fill material shall be determined and identified at each exploration location.

2.4.20 Other Drilling Observations

Other observations, not included in the descriptive sequence, may include:

- Caving or sloughing of borehole or trench sides
- Difficulty in drilling or excavating, etc.
- Generic name (e.g., hard pan, fault gouge, etc.)
- Ground water inflow, elevation(s), and estimated rate(s)
- Loss of drill fluid circulation

2.5 Rock Identification Procedures for Borehole Cores

Rock identification procedures presented in this section are based on a hybrid of the International Society of Rock Mechanics (ISRM) (1981) standards and the Bureau of Reclamation (2001) standards. The detail of description provided for a particular material shall be dictated by the complexity and objectives of the project. Optional descriptors should be considered by the geoprofessional on a project by project basis.

Intensely weathered or decomposed rock that is friable and that can be reduced to gravel size or smaller by normal hand pressure shall also be classified as a soil. The material shall be identified and described as rock followed by the soil identification or classification, and description in parenthesis.

For example:

> *IGNEOUS ROCK (GRANITE), massive, light gray to light yellowish brown, intensely weathered, soft, unfractured, (Lean CLAY with SAND (CL), medium stiff, moist, mostly clay, little coarse SAND, medium plasticity).*

Note, color is not repeated in the descriptive sequence for soil.

Although not included in the descriptive sequence, Core Recovery (REC) and Rock Quality Designation (RQD) shall be recorded and presented on the boring logs. Core Recovery shall be reported for all rock coring operations as described in Appendix A.9. RQD shall be recorded and presented on the boring logs in accordance with Appendix A.10.

2.5.1 Rock Identification and Descriptive Sequence for Borehole Cores

Use the descriptors and the descriptive sequence, shown in Figure 2-21, when identifying rock specimens collected from exploratory boreholes.

Figure 2-21
Rock Identification and Descriptive Sequence

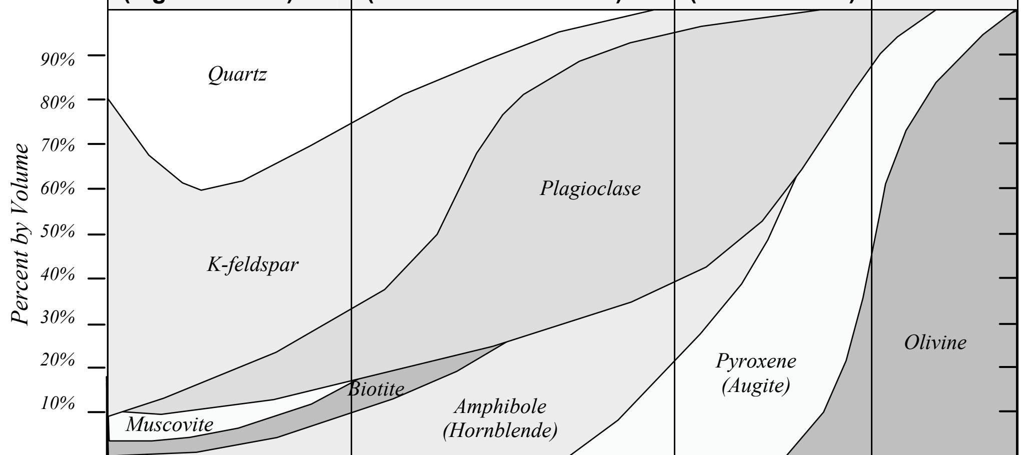
Sequence	Identification Components	Refer to Section	Required	Optional
1	Rock Name	2.5.2	●	
	Description Components			
2	Rock Grain-size	2.5.3		○
3	Bedding Spacing	2.5.4	●	
4	Color	2.5.5	●	
5	Texture	2.5.6		○
6	Weathering Descriptors for Intact Rock	2.5.7	●	
7	Rock Hardness	2.5.8	●	
8	Fracture Density	2.5.9	●	
9	Discontinuity Type	2.5.10		○
10	Discontinuity Condition (Weathering, Infilling and Healing)	2.5.11		○
11	Discontinuity Dip Magnitude	2.5.12		○
12	Rate of Slaking (Jar Slake Test)	2.5.13		○
13	Odor	2.5.14		○
14	Additional Comments	2.5.15		○

2.5.2 Rock Name

Rock name based on field identification in this section is taken from those presented by Zumberge et al. (2003). As a general practice, a staff geologist should be consulted if there are questions of the correct lithology. Rock name shall be reported using a combination of the *family name* (e.g. sedimentary, igneous, metamorphic), followed by the *rock identification*. The identification can be approximated using Figures 2-22, 2-23, or 2-24, or specifically identified by a qualified geologist.

Figure 2-22
Field identification of Igneous rock

	Felsic (Light Colored)	**Intermediate (Intermediate-Colored)**	**Mafic (Dark-Colored)**	**Ultramafic**



Texture	**Rock Identification**			
Phaneritic	Granite	Diorite	Gabbro	Peridotite
Phaneritic with Phenocrysts	Granite Porphyry	Diorite Porphyry	Gabbro Porphyry	Peridotite Porphyry
Aphanitic	Rhyolite	Andesite	Basalt	
Aphanitic with Phenocrysts	Rhyolite Porphyry	Andesite Porphyry	Basalt Porphyry	
Vesicular	Pumice, Scoria			
Glassy	Obsidian			
Non-vesicular	Agglomerate, Breccia, Tuff			

Figure 2-23
Field identification of Sedimentary rock

Origin	Textural Features and Particle Size	Composition	Diagnostic Features	Color	Rock Identification
Inorganic Detrital Materials	Clastic (Boulders, Gravels, Pebbles and granules embedded in a matrix of cemented sand grains	Angular rock or mineral fragments			Sedimentary Breccia
		Rounded rock or mineral fragments			Conglomerate
	Clastic (Coarse sand and granules)	angular fragments of feldspar mixed with quartz and other mineral grains	K-feldspar common		Arkose
	Clastic (Sand size particles)	Rounded to subrounded quartz grains		white, buff, pink, brown, tan	Quartz Sandstone
		Calcite and/or dolomite grains	effervesces freely with cold dilute HCl	light-colored	Calcarenite
	Clastic (Sand size particles mixed with clay size particles)	Quartz and other mineral grains mixed with clay		dark gray to gray-green	Wacke (Lithic Arenite)
	Clastic (Silt and clay size particles)	Mineral constituents may be identifiable with a hand lens	usually well stratified	varies	Siltstone
		Mineral constituents not identifiable	fissile, may be scratched with fingernail, usually well stratified	varies	Shale
		Mineral constituents not identifiable	massive (earthy), may be scratched with a fingernail	varies	Claystone
Inorganic Chemical Precipitates	Dense (Crystalline or Oolitic)	Calcium Carbonate	effervesces freely with cold dilute HCl, may contain fossils, generally lacks stratification	white, gray, black	Limestone
	Dense or Crystalline	Calcium Magnesium Carbonate	powder effervesces weakly with cold dilute HCl, may contain fossils, generally lacks stratification	varies, but similar to Limestone	Dolomite
	Dense (Porous)	Silica	conchoidal fracture, scratches glass	black, white, gray, red	Chert
	Dense (Amorphous)	Hydrous Calcium Sulfate	commonly can be scratched with a fingernail	varies, commonly pink, buff, white	Rock Gypsum
	Crystalline	Sodium Chloride	crystalline, salty taste	white to gray	Rock Salt
Organic Detrital Materials	Earthy (Bioclastic)	Calcium Carbonate	effervesces freely with cold dilute HCl, easily scratched with a fingernail	white	Chalk
		Silica	does not react with HCl, soft, commonly stratified	gray to white	Diatomite
	Bioclastic	Calcium Carbonate	effervesces freely with cold dilute HCl, shell fragments in a massive or crystallne matrix		Fossiliferous Limestone
		Calcium Carbonate	effervesces freely with cold dilute HCl, shell fragments cemented with little or no matrix material		Coquina
	Fibrous (Bioclastic)	Plant fibers	soft, porous, low specific gravity	brown	Peat
	Dense (Bioclastic)	Mineral free carbonaceous plant matter	harder than peat, moist	brownish to black	Lignite
		Mineral free carbonaceous plant matter	harder than lignite, dull luster, smudges fingers when handled	black	Bituminous Coal

The names of rocks derived from inorganic detrital materials may be appended to indicate the cementing agent, e.g., arkose with calcite cement.

Figure 2-24
Field identification of Metamorphic rock

Texture	Diagnostic Features	Composition	Color	Rock Identification
Foliated	slaty texture with slaty cleavage, dense, microscopic grains		variable, black and dark gray common	Slate
	phyllitic texture, fine grained to dense, "shiny" appearance	micaceous minerals are dominant		Phyllite
	schistose texture, medium to fine grained, "sparkling" appearance, porphyroblasts common	chlorite, biotite, muscovite, garnet and dark elongate silicate minerals, talc, feldspar commonly absent		Schist
	gneissic texture, coarse grained, foliation present as macroscopic grains arranged in light and dark bands	abundant quartz and feldspar in light bands and hornblende, augite, garnet or biotite in dark bands		Gneiss
	granulitic texture, medium to coarse grained, even grained, foliation present in quartzo-feldspathic rocks			Granulite
Foliated or Nonfoliated	medium to coarse-grained	mostly crystals of amphibole, sometimes feldspar, mica and talc		Amphibolite
Nonfoliated	crystalline, scratches glass, breaks across grains as easily as around them	quartz	color variable, white, pink, buff, brown, red, purple	Quartzite
	dense, dark colored		various shades of gray, gray-green, to nearly black	Hornfels
	texture of conglomerate but breaks across coarse grains as easily as around them	granules, pebbles or cobbles are commonly granitic or jasper, chert, quartz or quartzite		Metaconglomerate
	crystalline, scratches glass, breaks across grains as easily as around them, fossils in some	calcite or dolomite	white, pink, gray	Marble
	microcrystalline texture, usually with smooth wavy surfaces	serpentine, sometimes with crysotile	shades of green	Serpentinite
	granulitic texture, medium to coarse grained, even grained, foliation lacking in pyroxene-plagioclase bearing rocks			Granulite
	shiny luster, conchoidal fracture		black	Anthracite Coal

2.5.3 Rock Grain-size descriptors

The rock grain-size descriptors that follow are based on USBR (2001) standards.

Figure 2-25
Rock grain-size descriptors for Crystalline Igneous rock and Metamorphic rock

Description	Average Crystal Diameter
Very coarse grained or pegmatitic	> 3/8 in
Coarse-grained	3/16 – 3/8 in
Medium-grained	1/32 – 3/16 in
Fine-grained	0.04 – 1/32 in
Aphanitic	<0.04 in

Figure 2-26
Rock grain-size descriptors for Sedimentary and Pyroclastic Igneous rock

USCS (soils only) Particle Size	Size (inches)	Sedimentary (epiclastic) Rounded, subrounded, subangular: Particle or Fragment	Sedimentary (epiclastic): Lithified Product	Volcanic (pyroclastic): Fragment	Volcanic (pyroclastic): Lithified Product
Boulder		Boulder	Boulder Conglomerate	Block (Angular)	Volcanic Breccia
	12				
	10				
Cobble		Cobble	Cobble Conglomerate	Bomb (Rounded)	Agglomerate
	3				
	2.5				
Coarse Gravel		Pebble	Pebble Conglomerate		
	0.8				
Fine Gravel				Lapilli	Lapilli Tuff
	0.19				
	0.16				
Coarse Sand		Granule	Granule Conglomerate	Coarse Ash	Coarse Tuff
	0.08				
Medium Sand		Very Coarse Sand	Sandstone (Very Coarse, Coarse, Medium, Fine, or Very Fine)		
	0.04				
		Coarse Sand			
	0.02				
		Medium Sand			
	0.0165				
Fine Sand					
	0.0098				
		Fine Sand			
	0.0049				
	0.0029	Very Fine Sand		Fine Ash	Fine Tuff
	0.0025				
Fines Non-plastic Silt / Plastic Clay		Silt	Siltstone, Shale		
	0.0002				
		Clay	Claystone, Shale		

2.5.4 Bedding Spacing Descriptors

Bedding planes are discontinuities along which rock mass failure may occur. They also influence the hydraulic conductivity and shear strength of the rock mass.

The bedding thickness or spacing, modified from USBR (2001), shall be used as indicated in Figure 2-27 below.

Figure 2-27
Bedding Spacing Descriptors

Description	Thickness/Spacing
Massive	Greater than 10 ft.
Very thickly bedded	3 to 10 ft.
Thickly bedded	1 to 3 ft.
Moderately bedded	3-5/8 in. to 1 ft.
Thinly bedded	1-1/4 to 3-5/8 in.
Very thinly bedded	3/8 to 1-1/4 in.
Laminated	Less than 3/8 in.

2.5.5 Rock Colors

Use the color name from the *Munsell Rock Color Chart,* which is based on the National Bureau of Standards/Inter Society Color Council system, to describe the rock at the time of sampling. If the sample contains layers or patches of varying colors, record that information and describe and all colors observed.

For additional information, see ASTM D 1535-06, *Standard Practice for Specifying Color by the Munsell System*.

2.5.6 Textural Descriptors

Textural adjectives are employed to describe the size and shape of voids within the rock mass that are visible to the unaided eye. These voids are relevant to the estimation of the hydraulic conductivity, unconfined compressive strength, and the weathering susceptibility of the intact rock.

Use the USBR (2001) standard to describe the size and shape of voids, as indicated in Figure 2-28 below.

Figure 2-28
Textural Descriptors

Description	Criteria
Pitted	Pinhole to 3/8 in. openings.
Vuggy	Small opening (usually lined with crystals) ranging in diameter from 3/8 in. to 4 in.
Cavity	An opening larger than 4 in., size descriptions are required, and adjectives such as small, or large, may be used, if defined.
Honeycombed	If numerous enough that only thin walls separate individual pits or vugs, this term further describes the preceding nomenclature to indicate cell-like form.
Vesicular	Small openings in volcanic rocks of variable shape formed by entrapped gas bubbles during solidification.

2.5.7 Weathering Descriptors for Intact Rock

Weathering increases the clay content of the intact rock and the amount of separation at grain boundaries. Weathered rock masses have lower unconfined compressive strength, lower intact rock shear strength, lower shear strength along discontinuities, higher hydraulic conductivity, and are more likely to fail through the intact rock. Use USBR (2001) weathering descriptors, as indicated in Figure 2-29 below.

Figure 2-29

Weathering Descriptors for Intact Rock

	Diagnostic Features					
	Chemical weathering-discoloration and/or oxidation		Mechanical weathering-grain boundary conditions (disaggregation) primarily for granitics and some coarse-grained sediments	Texture and solutioning		
Description	**Body of rock**	**Fracture surfaces**		**Texture**	**Solutioning**	**General characteristics**
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No solutioning	Hammer rings when crystalline rocks are struck.
Slightly Weathered to Fresh						
Slightly Weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull	Minor to complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals may be noted	Hammer rings when crystalline rocks are stuck. Body of rock not weakened.
Moderately to Slightly Weathered						
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are “rusty,” feldspar crystals are “cloudy”	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck. Body of rock is slightly weakened.
Intensely to Moderately Weathered						
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation, see grain boundary conditions	All fracture surfaces are discolored or oxidized, surfaces friable	Partial separation, rock is friable; in semiarid conditions granitics are disaggregated	Texture altered by chemical disintegration (hydration, argillation)	Leaching of soluble minerals may be complete	Dull sound when struck with hammer, usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures, or veinlets. Rock is significantly weakened.
Very Intensely Weathered						
Decomposed	Discolored or oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay		Complete separation of grain boundaries (disaggregated)	Resembles a soil, partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand. Resistant minerals such as quartz may be present as “stringers” or “dikes.”
NOTE: Combination descriptors (such as "slightly weathered to fresh") are permissible where equal distribution of both weathering characteristics is present over significant intervals or where characteristics present are "in between" the diagnostic feature. However, combination descriptors shall not be used where significant identifiable zones can be delineated. Only two adjacent descriptors shall be combined.						

2.5.8 Rock Hardness

Use the modified USBR (2001) descriptors to describe the hardness of intact rock, as indicated in Figure 2-30 below.

Figure 2-30

Descriptors for Rock Hardness

Description	Criteria
Extremely Hard	Specimen cannot be scratched with a pocket knife or sharp pick; can only be chipped with repeated heavy hammer blows
Very Hard	Specimen cannot be scratched with a pocket knife or sharp pick. Breaks with repeated heavy hammer blows.
Hard	Specimen can be scratched with a pocket knife or sharp pick with difficulty (heavy pressure). Heavy Hammer blows required to break specimen
Moderately Hard	Specimen can be scratched with a pocket knife or sharp pick with light or moderate pressure. Core breaks with moderate hammer blows
Moderately Soft	Specimen can be grooved 1/6 in. deep with a pocket knife or sharp pick with moderate or heavy pressure. Breaks with light hammer blow or heavy manual pressure.
Soft	Specimen can be grooved or gouged easily with a pocket knife or sharp pick with light pressure, can be scratched with fingernail. Breaks with light to moderate manual pressure.
Very Soft	Specimen can be readily indented, grooved or gouged with fingernail, or carved with a pocket knife. Breaks with light manual pressure.

2.5.9 Fracture Density

Fractures are defined in Section 2.5.10. The fracture density is based on the spacing of all of the fractures observed in recovered core lengths from boreholes. This measurement excludes mechanical breaks and incipient joints/fractures. It also excludes features not identified as fractures, such as shears, faults, foliations, and bedding plane separations, etc.

Descriptive criteria presented below are based on borehole cores where lengths are measured along the core axis. Use the USBR (2001) fracture density standard, as indicated in Figure 2-31 below.

Figure 2-31

Descriptors for Fracture Density

Description	Observed Fracture Density
Unfractured	No fractures.
Very slightly fractured	Lengths greater than 3 ft.
Slightly to very slightly fractured	
Slightly fractured	Lengths from 1 to 3 ft. with few lengths less than 1 ft. or greater than 3 ft.
Moderately to slightly fractured	
Moderately fractured	Lengths mostly in 4 in. to 1 ft. range with most lengths about 8 in.
Intensely to moderately fractured	
Intensely fractured	Lengths average from 1 to 4 in. with scattered fragmented intervals with lengths less than 4 in.
Very intensely to intensely fractured	
Very intensely fractured	Mostly chips and fragments with a few scattered short core lengths.
NOTE: Combination descriptors (such as “very intensely to intensely fractured”) are used where equal distribution of both fracture density characteristics is present over a significant interval or exposure, or where characteristics are “in between” the descriptor definitions. Only two adjacent descriptors may be combined.	

2.5.10 Discontinuity Type

A single description, or range of descriptors, shall be used to describe the discontinuities observed over the length of the reported fracture density.

Discontinuity: A collective term used for all planes including fractures, joints, faults, shears, bedding planes and foliations. Contacts between rock bodies of different lithologies may also be considered discontinuities.

Fracture: A term used to describe any break in geologic material, excluding shears and shear zones. Additional fracture terminology is provided in Figure 2-32, below.

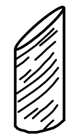
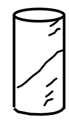
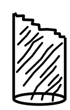
Shear: A structural break where differential movement has taken place along a surface, or zone of failure by a shear couplet, is termed a shear. Shears are sometimes characterized by striations, slickensides, gouge, breccia, mylonite, or any combination of these. Often direction, amount of displacement, and continuity may not be known because of limited exposures or observations.

Fault: A shear with significant continuity that can be correlated between observations is a fault. Faults demonstrate high spatial continuity, and therefore occur over significant portions of given sites, foundation areas, or regions. The observed fault feature may be a segment of a fault or fault zone, as defined in the literature. The designation of a shear as a fault or fault zone is a site-specific determination.

Shear/Fault Zone: A shear or fault that exhibits significant width when measured perpendicular to the plane of the shear or fault. The zone may consist of gouge, breccia, or many related faults or shears together with fractured and crushed rock between the shears or faults, or any combination of these. In the literature, many fault zones are referred to as faults.

Shear/Fault Disturbed Zone: An associated zone of fractures and/or folds adjacent to a shear or a shear zone where the country rock has been subjected to only minor cataclastic action and may be mineralized. If adjacent to a fault or fault zone, the term *fault-disturbed zone* is used. Occurrence, orientation, and aerial extent of these phenomena depend on the depth of burial (pressure and temperature) during shearing, brittleness of the geo-materials, and the stress conditions.

Figure 2-32
Descriptors for Discontinuity Type

Description		Criteria
Joint (JT)		A relatively planar fracture along which there has been little or no shearing displacement.
Foliation Joint (FJ) or Bedding Joint (BJ)		A relatively planar fracture that is parallel to foliation or bedding along which there has been little or no shearing displacement.
Bedding Plane Separation		A separation along bedding after extraction or exposure due to stress relief or slaking.
Incipient Joint (IJ) or Incipient Fracture (IF)		A joint or fracture that does not continue through the specimen or is not seen with the naked eye. However when the specimen is wetted and then allowed to dry, the joint or fracture trace is evident. When core is broken, it breaks along an existing plane.
Random Fracture (RF)		A natural break (fracture) with a generally rough, very irregular, non-planar surface which does not belong to a joint set.
Mechanical Break (MB)		A break due to drilling, blasting, or handling. Mechanical breaks parallel to bedding or foliation are called *Bedding Breaks* (BB) or *Foliation Breaks* (FB), respectively. Recognizing mechanical breaks may be difficult. The absence of oxidation, staining, or mineral fillings, and often a hackly or irregular surface are clues for recognition.
Fracture Zone (FZ)		Numerous, very closely intersecting fractures. Often fragmented core cannot be fitted together.

2.5.11 Discontinuity Condition (Weathering, Infilling and Healing)

Weathering: Descriptors for discontinuity weathering or alteration of fracture surfaces and fracture fillings (excluding soil materials) are the same as those used for weathering and alteration of intact rock (per Section 2.5.7, Figure 2-29, third column).

Discontinuity Infilling: Descriptors for hardness of fillings, gouges and/or fracture surfaces are the same as those presented for intact rock or consistency of soils.

Discontinuity Healing: Discontinuities may be filled with air, water, soil, or a crystalline mineral material that provides a significant tensile and shear strength to the discontinuity. Discontinuity healing can be observed when there is a color contrast with the bordering intact rock. Features often referred to as veins are healed discontinuities.

In addition to an observation of the amount of the discontinuity that has been healed, the healing material should be observed and recorded. The amount and material of the healing discontinuity is relevant to the estimation of discontinuity shear strength, discontinuity hydraulic conductivity, and to the ease with which the rock can be excavated (e.g., open excavation, tunnel, or borehole).

Use the USBR (2001) standard, as indicated below in Figure 2-33 below, to describe the discontinuity condition.

Figure 2-33
Descriptors for Discontinuity Healing

Descriptor	Criteria
Totally Healed	All fragments bonded, discontinuity is completely healed or recemented to a degree at least as hard as surrounding rock.
Moderately Healed	Greater than 50 percent of fractured or sheared material, discontinuity surface or filling is healed or recemented, and/or strength of healing agent is less hard than surrounding rock.
Partially Healed	Less than 50 percent of fractured or sheared material, discontinuity surface or filling is healed or recemented.
Not Healed	Discontinuity surface, fracture zone, sheared material, or filling is not healed or recemented. Rock fragments or filling (if present) held in place by their own angularity and/or cohesiveness.

2.5.12 Discontinuity Dip Magnitude

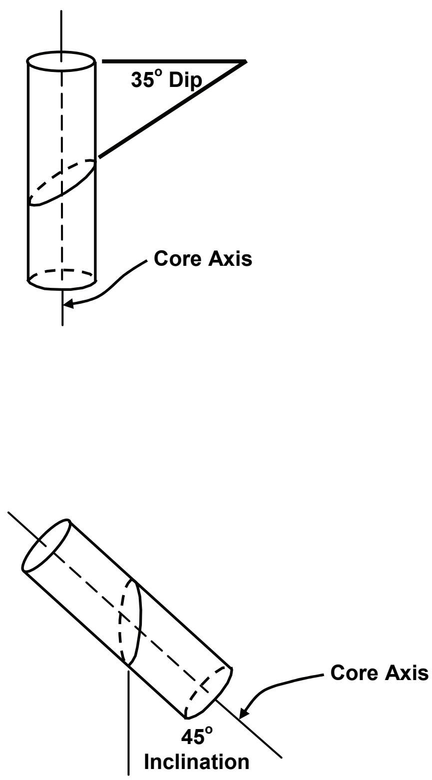
Observation of the magnitude of discontinuity dip made with non-oriented core is useful for anticipating difficulties that may arise from boring piles or shafts in rock masses that contain discontinuities that are oriented close to vertical.

Use the USBR (2001) standard, as indicated below in Figure 2-34 below, to describe the magnitude of discontinuity dip.

Figure 2-34
Discontinuity Dip Magnitude

Vertical Hole: Apparent dip is measured and reported.

Angle Hole: True dip is usually not known, angle is measured from core axis and is called inclination.



2.5.13 Rate of Slaking

Some rock types are subject to degradation when exposed to weathering processes, particularly repeated wetting and drying cycles. Rocks that exhibit slaking may disintegrate into gravel size particles, or they may disaggregate completely to the individual constituent particles: clay, silt, and sand.

Rocks that are prone to slaking include: shale, siltstone, claystone, weakly welded tuff, and highly weathered crystalline igneous and metamorphic rocks. Slaking behavior is relevant to the performance of cut slopes and the stability of bored piles/drilled shafts. See Appendix A for test procedures.

Figure 2-35
Rate of Slaking

Jar Slake Index, I_J	Observed Behavior
1	Degrades to a pile of flakes or mud
2	Breaks rapidly and forms many chips
3	Breaks slowly and forms few chips
4	Breaks rapidly and develops several fractures
5	Breaks slowly and develops few fractures
6	No change to condition of the rock fragment

2.5.14 Odor

Rocks containing significant amounts of organic material usually have a distinctive odor of decaying vegetation. This is especially apparent in fresh samples, but if the samples are dried, the odor often may be revived by heating a moistened sample. Petroleum products or other chemicals may also influence the odor of the sample. Describe the odor, if organic, and identify anything unusual, such as odor of a petroleum product or other chemical.

2.5.15 Additional Comments

Additional rock characteristics not included in the previous categories may be noted.

2.5.16 Other Drilling Observations

Other observations (not included in the descriptive sequence) that may be presented on the LOTB or BR as notes or remarks include:

- Time for core run
- Difficulty in drilling or excavating, etc.
- Generic name (e.g., hard pan, fault gouge, etc.)
- Ground water inflow, elevation(s), and estimated rate(s)
- Loss of drill fluid circulation

2.6 Sample Preparation and Identification for Laboratory Testing and Storage

Geoprofessionals who drill, sample, preserve, and transport soil samples play an important role in ensuring the quality of the laboratory test results. When performing field investigations, the geoprofessional shall be familiar with the procedures contained within the following ASTM standards:

- ASTM D 1586-99, "Test Method for Penetration Test and Split-Barrel Sampling of Soils"
- ASTM D 1587-00, "Practice for Thin-Walled Tube Sampling of Soils"
- ASTM D 3550-01, "Practice for Ring-Lined Barrel Sampling of Soils"
- ASTM D 4220-00, "Standard Practices for Preserving and Transporting Soil Samples"

The information that follows explains the procedures and information required to submit soil samples and request testing services from the Caltrans Geotechnical Laboratory, an AASHTO Materials Reference Laboratory (AMRL) accredited facility located in Sacramento.

2.6.1 Sample Preparation and Identification for Laboratory Testing and Storage

All samples shall be named according to the following convention:

Hole ID – SNN – T

Where,

Hole ID: Refer to Section 2.3

S: The Sample Type Code, as defined in Figure 2-36, which generally follow ASTM D 6453

NN: 2-digit sample number (01–99), sequenced consecutively from the top down.

T: 1-digit tube number, starting with the bottom tube numbered as 1.

For example:

A – 06 – 105 – U02 – 3

Figure 2-36
Sample Type Codes

Code	Description
U	Undisturbed Shelby tube
P	Undisturbed Piston
S	Split spoon (includes SPT and Cal Mod Samplers)
B	Block
C	Core (both rock and soil)
D	Disturbed (include auger cuttings)
R	Reconstituted
O	Other

Label Brass and Shelby Tubes as explained below and shown in Figure 2-37, below:

- Use electrical tape to completely seal the end caps onto the sample tubes.
- Clearly label samples with permanent marker
- Place the top of the label at the top end of the tube to identify the proper orientation of the sample as shown above.
- If the sample tube is only partially filled, use a filler, such as newspaper, cloth rags, or sawdust

to prevent movement of the sample in the tube during shipping and handling.

- Whenever possible use a wax seal on the end of the sample to prevent moisture exchange with or contamination by the filler material.

Label bagged samples as explained below and shown in Figure 2-38:

- Identify the sample by writing directly on the plastic bag or attaching an adhesive label.
- Seal or tie the plastic bag properly to prevent loss of moisture.

Figure 2-37
Brass and Shelby Tube Label

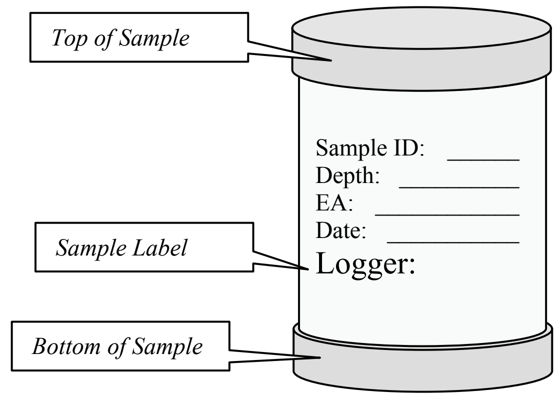
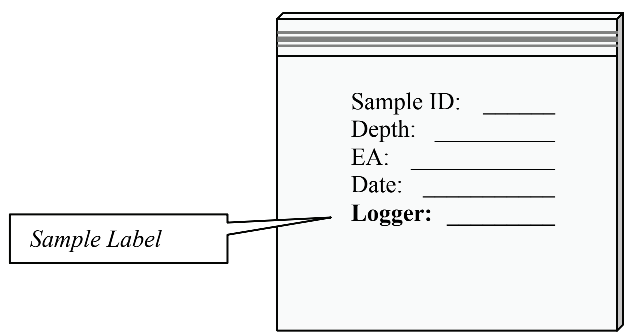


Figure 2-38
Bagged Sample Label



2.6.2 Identification of Large Soil Samples

In addition to the labeling requirements explained above, some soil samples must also be labeled with a Sample Identification Card (Caltrans Form TL-101), including:

- Samples weighing more than 5 lbs.
- Samples to be tested by the Materials Engineering and Testing Services (METS), test names are followed by "**" in Figure 2-39 below.

Place Form TL-101 inside a sealed plastic bag, then place it inside the large plastic or canvas bag that contains the sample.

Figure 2-39
Minimum Material Requirements for Various Test Methods

Test Method(s)	Test Name	Material Required	Typical Sample Size/Type	TL-101 Required
AASHTO T 265-93 (2004) ASTM D 2216-05	Moisture Content	0.5 lb	1/2 Tube	No
ASTM D 4767-04	Unit Weight	1 lb	1 Tube	No
AASHTO T 100-06	Specific Gravity	0.5 lb	1/2 Tube	No
ASTM D 422-63 (2002)	Particle-Size Analysis	1 lb	1 Tube	No
AASHTO T 89-02 AASHTO T 90-00 (2004)	Liquid Limit Plastic Limit, Plasticity Index	1 lb	1 Tube	No
ASTM D 2435-04	Consolidation Undisturbed (2.0" Diameter) (2.5" Diameter) Remolded (2.0" Diameter)	- - 80 lb	1 Tube 1 Tube 2 Full Canvas Bags	No No **Yes**
ASTM D 4546-03	Swell Potential Undisturbed (2.0" Diameter) (2.5" Diameter) Remolded (2.0" Diameter)	- - 80 lb	1 Tube 1 Tube 2 Full Canvas Bags	No No **Yes**
ASTM D 5333-03	Collapse Potential Undisturbed (2.0" Diameter) (2.5" Diameter) Remolded (2.0" Diameter)	- - 80 lb	1 Tube 1 Tube 2 Full Canvas Bags	No No **Yes**
ASTM D 3080-04	Direct Shear Undisturbed Remolded	- 80 lb	1 Tube 2 Full Canvas Bags	No **Yes**
CTM 216 (Oct 2006)	Relative Compaction (Compaction Curve Only)	80 lb	2 Full Canvas Bags	**Yes**

Test Method(s)	Test Name	Material Required	Typical Sample Size/Type	TL-101 Required
CTM 220 (Nov 2005)	Permeability Undisturbed Falling Head Remolded Falling Head Constant Head	 - 80 lb 80 lb	 1 Tube 2 Full Canvas Bags 2 Full Canvas Bags	 No **Yes** **Yes**
ASTM D 2166-06 ASTM D 2938-95 (2002)	Unconfined Compression	-	1 Tube or Core	No
ASTM D 4767-02 ASTM D 2850-03	Triaxial CU (3 points) Undisturbed (2.0" Diameter) (2.5" Diameter) Remolded (2.8" Diameter) Triaxial UU (1 point) Undisturbed (2.0" Diameter) (2.5" Diameter) Remolded (2.8" Diameter)	 - - 80 lb - - 80 lb	 3 Tubes - in series 3 Tubes - in series 2 Full Canvas Bags 1 Tube 1 Tube 2 Full Canvas Bags	No No **Yes** No No **Yes**
ASTM D 427-04	Shrinkage Limit	1 lb	1 Tube	No
ASTM D 5731-05	Point Load	-	Rock Core	No
ASTM D 4829-03	Expansion Index	40 lb	1 Full Canvas Bag	**Yes**
CTM 217 (Nov 1999) AASHTO T 176-02	Sand Equivalent**	10 lb	1/4 Full Canvas Bag	**Yes**
CTM 301 (Mar 2000) AASHTO T 190-02	R-Value**	80 lb	2 Full Canvas Bags	**Yes**
CTM 643 (Nov 1999) CTM 417 (Nov 2006) CTM 422 (Nov 2006)	Corrosion** Sulfates** Chlorides**	10 lb	1/4 Full Canvas Bag	**Yes**
EPA 9081	Organic Content** PH** Cation Exchange**	10 lb	1/4 Full Canvas Bag	**Yes**

Notes:

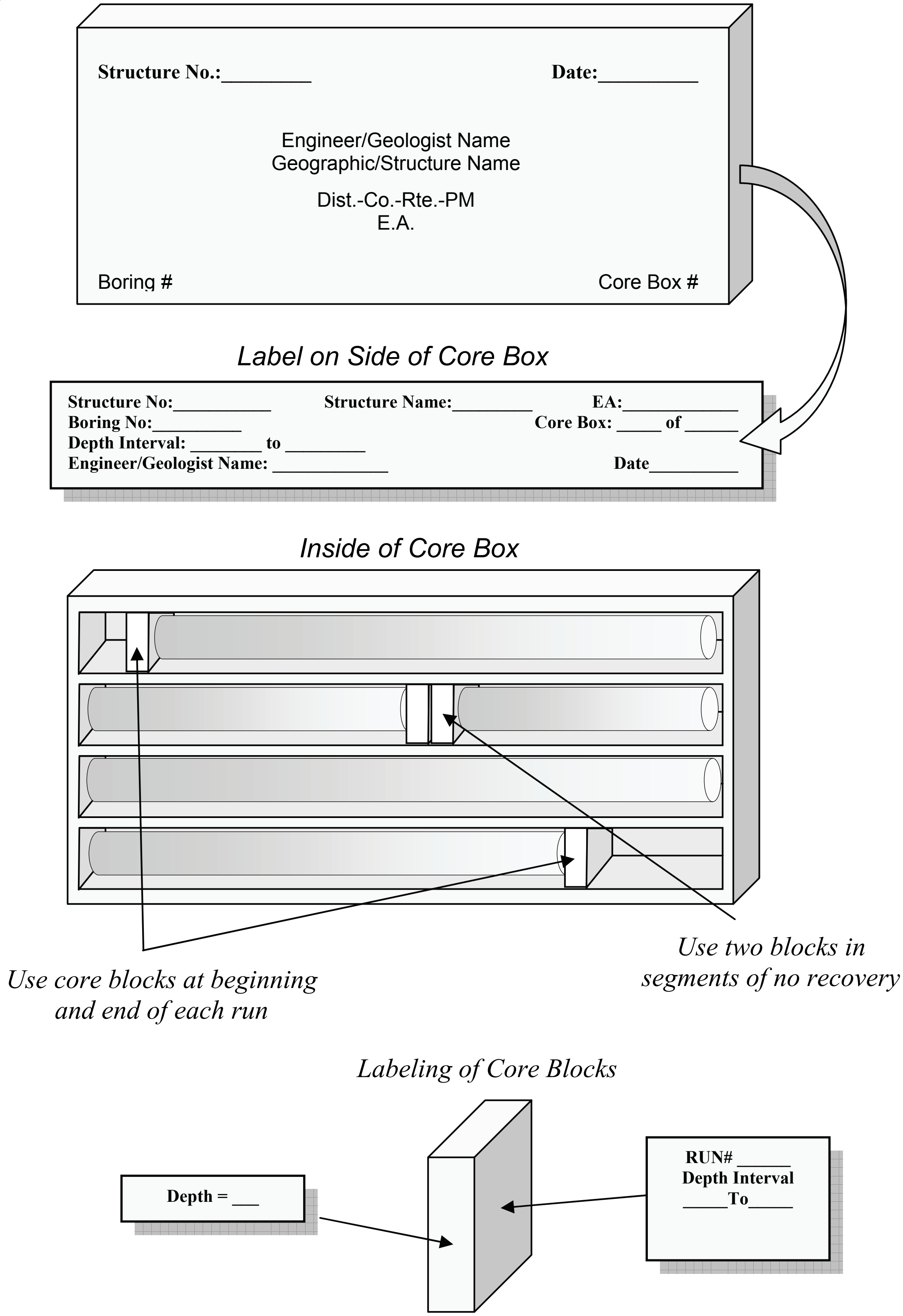
1. ** Tests performed by Materials Engineering and Testing Services (METS)
2. A 12" by 24" canvas bag completely filled contains approximately 40 lb of material.
3. A 2" by 4" tube contains approximately 1 lb. of material.
4. Minimum material weights shown for remolded samples include sufficient material for the development of a moisture density curve.

5. When calculating the number of triaxial samples that can be obtained from a Shelby tube, use a minimum sample length equal to three times the sample diameter.

2.6.3 Core Box Layout

Core boxes shall be labeled as shown in Figure 2-40.

Figure 2-40
Core Box Layout and Label



2.7 Quality Check of Field Observations and Samples

The geoprofessional shall conduct a quality check of his/her field notes and observations once back in the office. Sample descriptions and identifications shall be reviewed and revised as necessary to ensure that they are in compliance with the procedures presented in this section.

Descriptors of sample properties that are subject to change due to time or environment, such as moisture or RQD, shall not be revised. Samples that are to be stored for laboratory testing or other purposes shall be inventoried to ensure correct labeling and accounting.

Section 3: Procedures for Soil and Rock Description and/or Classification Using Laboratory Test Results

3.1 Introduction

Section 2 describes the procedures for describing and identifying soil and rock samples in the field using visual and manual methods and basic field testing tools. Most of these field procedures are sufficient to generally identify and describe the soil and rock in qualitative terms, and are appropriate for reporting in final boring records, as described later in Sections 4 and 5. In many cases these descriptors can be correlated, to some degree, to engineering parameters for use in geotechnical designs. However, the geoprofessional may want to more quantitatively and definitively characterize a particular sample using laboratory test results.

This Section addresses how to apply the results of specific laboratory tests to revise and supplement the original field observations, identifications, and descriptions. The information presented in this Section is based largely on the American Society for Testing and Materials (ASTM) D 2487-06, *Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)*.

Laboratory test results can be used to systematically refine field observations. The process of correction, calibration, and verification in developing an updated Log of Test Boring (LOTB) or Boring Record (BR) based on laboratory test results can effectively serve the purpose of self-training and self-calibration. This process is described in more detail in Section 4 of this Manual.

3.2 Revising Soil Descriptions and Assigning Soil Classification Using Laboratory Test Results

Six of the 21 attributes in the identification and descriptive sequence for soils, listed previously in section 2.4.1, may be revised with laboratory test results. They are:

- Group Name
- Group Symbol
- Consistency
- Percent or Proportion of Soils
- Particle Size Range
- Plasticity

The *Group Name* and *Group Symbol* are estimated in the field using visual and manual procedures based on ASTM D 2488-06, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure).* The field method requires the user to make judgments on a number of observations (e.g., percent of constituents by weight, whether a soil is well or poorly graded, and whether the soil is a clay or silt or some combination thereof).

Laboratory particle-size analysis, liquid limit, and plasticity index provide a quantitative basis for *classification* of the soil. Furthermore, the laboratory procedure employs a much more comprehensive listing of possible Group Names, as compared to field methods.

Consistency is estimated in the field using one or more of three methods (thumb test, Torvane, or Pocket Penetrometer), with varying levels of accuracy and repeatability. Laboratory triaxial, direct shear, and unconfined compression tests provide less subjective undrained shear strength values that can be correlated to specific consistency descriptors.

Percent gravel, sand, and fines, and range of particle sizes are estimated in the field using visual methods (e.g. jar test, visual approximation, etc.). The laboratory particle-size analysis test provides a quantitative distribution of particle sizes in proportion to the total sample weight. It is important to recognize that the sample size is significant when dealing with gravel or larger sized soils.

Plasticity is estimated in the field in order to determine Group Name and Group Symbol for fine-grained soils and to provide a plasticity descriptor. The liquid limit and plasticity index used in conjunction with ASTM D 2487-06 provide a Group Name and Group Symbol. The field-based plasticity descriptor is eliminated, as the plasticity is inherent in the Group Name and Group Symbol.

3.2.1 Soil Classification and Description Descriptive Sequence

The descriptive sequence presented in the Figure 3-1 below shall be used when classifying and describing soils. Items indicated by a check mark in the "Required" column shall be repeated to describe all the components of the subject soil to provide complete descriptive coverage. To incorporate laboratory test data in the classification and descriptive sequence, where applicable, refer to the sections in this Manual as noted in Figure 3-1. *(See "Lab" column below.)*

Figure 3-1
Classification and Description Sequence

Sequence	Classification Components	Refer to Section		Required	Optional
		Field	Lab		
1	Group Name	2.4.2	3.2.2	●	
2	Group Symbol	2.4.2	3.2.2	●	
	Description Components				
3	Consistency (for cohesive soils)	2.4.3	3.2.3	●	
4	Apparent Density (for cohesionless soils)	2.4.4		●	
5	Color (in moist condition)	2.4.5		●	
6	Moisture	2.4.6		●	
7	Percent of cobbles or boulders	2.4.7		●	
8	Percent or proportion of soils	2.4.8	3.2.4	●	
9	Particle Size Range	2.4.9	3.2.5	●	
10	Particle Angularity	2.4.10			○
11	Particle Shape	2.4.11			○
12	Hardness (for coarse sand and larger particles)	2.4.12			○
13	Plasticity (for fine-grained soils)*	2.4.13	3.2.6	●	
14	Dry Strength (for fine-grained soils)	2.4.14			○
15	Dilatency (for fine-grained soils)	2.4.15			○
16	Toughness (for fine-grained soils)	2.4.16			○
17	Calcium Carbonate (Reaction with HCl)	2.4.17			○
18	Structure	2.4.18			○
19	Cementation	2.4.19		●	
20	Description of Cobbles and Boulders	2.4.20		●	
21	Additional Comments	2.4.21			○

* *This descriptive component is not reported for the primary soil type if the liquid limit and plasticity index are available. (See Section 3.2.7)*

3.2.2 Group Name and Group Symbol

This section presents a procedure for classifying soils for engineering purposes based on laboratory determination of particle-size characteristics, liquid limit, and plasticity index. It shall be used when precise classification is required. This method is based on the ASTM version of the Unified Soil Classification System (USCS).

The ASTM procedure for classifying and describing fine-grained and coarse-grained soils is only applicable to material passing the 3-inch sieve. If the presence of cobbles or boulders or both is identified during the site exploration, the percentage of cobbles and boulders shall be reported per Section 2.4.7.

3.2.2.1 Procedure for Classification of Fine-Grained Soils

If 50% or more by dry weight of the test specimen passes the No. 200 sieve, the soil is fine-grained. Fine-grained soils are classified using the liquid limit and plasticity index in Figures 3-2 and 3-3, below.

Classify the soil as fine-grained:

- In cases where the liquid limit exceeds 110, or the plasticity index exceeds 60, the plasticity chart may be expanded by maintaining the same scale on both axes and extending the “A” line at the indicated slope.
- The soil is organic if organic matter is present in sufficient amounts to influence the liquid limit. Typically, organic soils have a dark color and an organic odor when moist and warm, and may contain visible organic matter. If the geoprofessional suspects there is sufficient organic matter to influence the soil’s classification, consult with the Caltrans Geotechnical Laboratory about additional laboratory testing.

Figure 3-2
Classification of Fine-Grained Soils

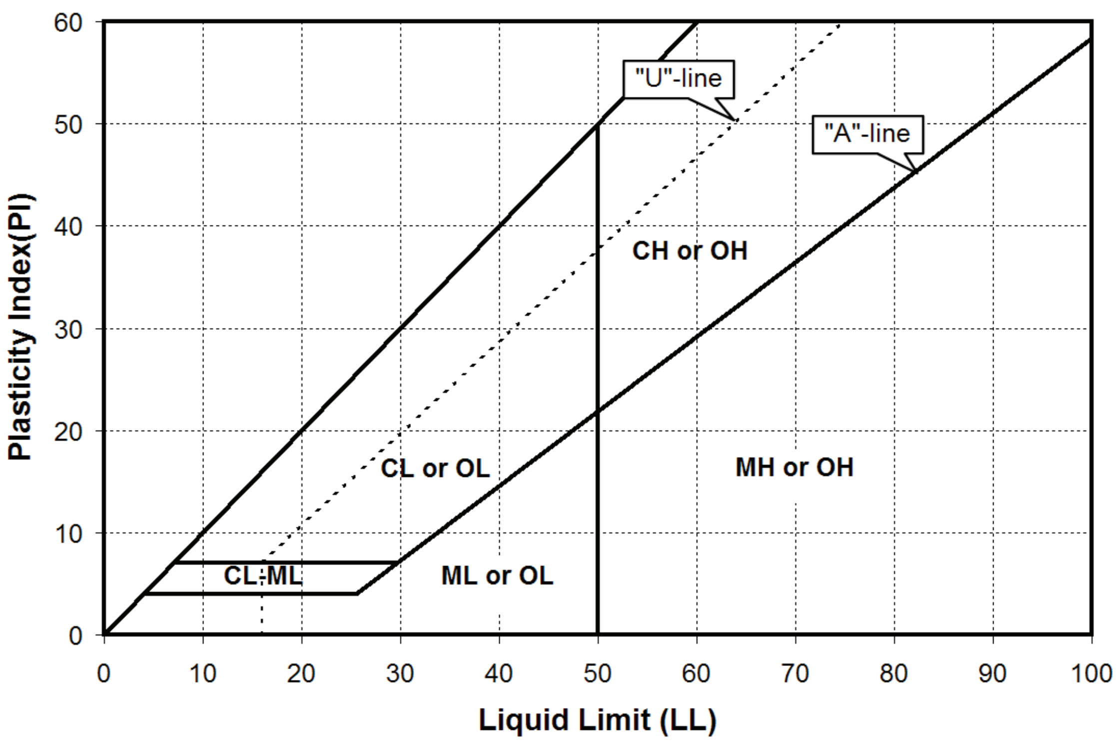


Figure 3-3

Flow chart for fine-grained soils

Liquid Limit	Organic	Plasticity Index	Group Symbol	Fines	Coarseness		Group Name
LL<50	Inorganic	PI>7 and plots on or above “A”-line	CL	<30% plus No. 200	<15% plus No. 200		Lean CLAY
					15-29% plus No. 200	% sand ≥ % gravel	Lean CLAY with SAND
						% sand < % gravel	Lean CLAY with GRAVEL
				≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	SANDY lean CLAY
						≥ 15% gravel	SANDY lean CLAY with GRAVEL
					% sand < % gravel	< 15% sand	GRAVELLY lean CLAY
						≥ 15% sand	GRAVELLY lean CLAY with SAND
		4≤PI≤7 and plots on or above “A”-line	CL-ML	<30% plus No. 200	<15% plus No. 200		SILTY CLAY
					15-29% plus No. 200	% sand ≥ % gravel	SILTY CLAY with SAND
						% sand < % gravel	SILTY CLAY with GRAVEL
				≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	SANDY SILTY CLAY
						≥ 15% gravel	SANDY SILTY CLAY with GRAVEL
					% sand < % gravel	< 15% sand	GRAVELLY SILTY CLAY
						≥ 15% sand	GRAVELLY SILTY CLAY with SAND
		PI<4 or plots below “A”-line	ML	<30% plus No. 200	<15% plus No. 200		SILT
					15-29% plus No. 200	% sand ≥ % gravel	SILT with SAND
						% sand < % gravel	SILT with GRAVEL
				≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	SANDY SILT
						≥ 15% gravel	SANDY SILT with GRAVEL
					% sand < % gravel	< 15% sand	GRAVELLY SILT
						≥ 15% sand	GRAVELLY SILT with SAND
	Organic	PI≥4 and plots on or above “A”-line	OL	<30% plus No. 200	<15% plus No. 200		ORGANIC CLAY
					15-29% plus No. 200	% sand ≥ % gravel	ORGANIC CLAY with SAND
						% sand < % gravel	ORGANIC CLAY with GRAVEL
				≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	SANDY ORGANIC CLAY
						≥ 15% gravel	SANDY ORGANIC CLAY with GRAVEL
					% sand < % gravel	< 15% sand	GRAVELLY ORGANIC CLAY
						≥ 15% sand	GRAVELLY ORGANIC CLAY with SAND
		PI<4 or plots below “A”-line		<30% plus No. 200	<15% plus No. 200		ORGANIC SILT
					15-29% plus No. 200	% sand ≥ % gravel	ORGANIC SILT with SAND
						% sand < % gravel	ORGANIC SILT with GRAVEL
				≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	SANDY ORGANIC SILT
						≥ 15% gravel	SANDY ORGANIC SILT with GRAVEL
					% sand < % gravel	< 15% sand	GRAVELLY ORGANIC SILT
						≥ 15% sand	GRAVELLY ORGANIC SILT with SAND

Figure 3-3, continued

Liquid Limit	Organic	Plasticity Index	Group Symbol	Fines	Coarseness		Group Name
LL≥50	Inorganic	Plots on or above "A"-line	CH	<30% plus No. 200	<15% plus No. 200		Fat CLAY
					15-29% plus No. 200	% sand ≥ % gravel	Fat CLAY with SAND
						% sand < % gravel	Fat CLAY with GRAVEL
				≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	SANDY fat CLAY
						≥ 15% gravel	SANDY fat CLAY with GRAVEL
					% sand < % gravel	< 15% sand	GRAVELLY fat CLAY
						≥ 15% sand	GRAVELLY fat CLAY with SAND
		Plots below "A"-line	MH	<30% plus No. 200	<15% plus No. 200		Elastic SILT
					15-29% plus No. 200	% sand ≥ % gravel	Elastic SILT with SAND
						% sand < % gravel	Elastic SILT with GRAVEL
				≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	SANDY elastic SILT
						≥ 15% gravel	SANDY elastic SILT with GRAVEL
					% sand < % gravel	< 15% sand	GRAVELLY elastic SILT
						≥ 15% sand	GRAVELLY elastic SILT with SAND
	Organic	Plots on or above "A"-line	OH	<30% plus No. 200	<15% plus No. 200		ORGANIC CLAY
					15-29% plus No. 200	% sand ≥ % gravel	ORGANIC CLAY with SAND
						% sand < % gravel	ORGANIC CLAY with GRAVEL
				≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	SANDY ORGANIC CLAY
						≥ 15% gravel	SANDY ORGANIC CLAY with GRAVEL
					% sand < % gravel	< 15% sand	GRAVELLY ORGANIC CLAY
						≥ 15% sand	GRAVELLY ORGANIC CLAY with SAND
		Plots below "A"-line		<30% plus No. 200	<15% plus No. 200		ORGANIC SILT
					15-29% plus No. 200	% sand ≥ % gravel	ORGANIC SILT with SAND
						% sand < % gravel	ORGANIC SILT with GRAVEL
				≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	SANDY ORGANIC SILT
						≥ 15% gravel	SANDY ORGANIC SILT with GRAVEL
					% sand < % gravel	< 15% sand	GRAVELLY ORGANIC SILT
						≥ 15% sand	GRAVELLY ORGANIC SILT with SAND

3.2.2.2 Procedure for Classification of Coarse-Grained Soils

If 50% or more by dry weight of the test specimen is retained on the No. 200 sieve the soil is coarse-grained. Coarse-grained soils are classified using the following procedure:

- Classify the soil as gravel if more than 50% of the coarse fraction (plus No. 200 sieve) is retained on the No. 4 sieve.
- Classify the soil as sand if 50% or more of the coarse fraction (plus No. 200 sieve) passes through the No. 4 sieve.
- If 12% or less of the test specimen passes through the No. 200 sieve, plot the cumulative particle-size distribution and compute the coefficient of uniformity, Cu, and coefficient of curvature, Cc, as given in Equations 1 and 2.

Equation 1

$$Cu = \frac{D_{60}}{D_{10}}$$

Equation 2

$$Cc = \frac{(D_{30})^2}{(D_{10} \times D_{60})}$$

Where D_{10}, D_{30}, and D_{60} are the particle-size diameters corresponding to 10, 30, and 60 percentiles passing on the cumulative particle-size distribution curve. It may be necessary to extrapolate the curve to obtain the D_{10} diameter.

Use the above results to determine the classification according to Figure 3-4 on the following page.

Figure 3-4

Flow chart for coarse-grained soils

	Fines	Grade	Type of Fines	Group Symbol	Sand/ Gravel	Group Name
Gravel	≤ 5%	Cu≥4 1≤Cc≤3		GW	< 15% sand	Well-graded GRAVEL
					≥ 15% sand	Well-graded GRAVEL with SAND
		Cu<4 1>Cc>3		GP	< 15% sand	Poorly graded GRAVEL
					≥ 15% sand	Poorly graded GRAVEL with SAND
	5-12%	Cu≥4 1≤Cc≤3	ML or MH	GW-GM	< 15% sand	Well-graded GRAVEL with SILT
					≥ 15% sand	Well-graded GRAVEL with SILT and SAND
			CL, CH or CL-ML	GW-GC	< 15% sand	Well-graded GRAVEL with CLAY (or SILTY CLAY)
					≥ 15% sand	Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
		Cu<4 1>Cc>3	ML or MH	GP-GM	< 15% sand	Poorly graded GRAVEL with SILT
					≥ 15% sand	Poorly graded GRAVEL with SILT and SAND
			CL, CH or CL-ML	GP-GC	< 15% sand	Poorly graded GRAVEL with CLAY (or SILTY CLAY)
					≥ 15% sand	Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
	> 12%		ML or MH	GM	< 15% sand	SILTY GRAVEL
					≥ 15% sand	SILTY GRAVEL with SAND
			CL or CH	GC	< 15% sand	CLAYEY GRAVEL
					≥ 15% sand	CLAYEY GRAVEL with SAND
			CL-ML	GC-GM	< 15% sand	SILTY, CLAYEY GRAVEL
					≥ 15% sand	SILTY, CLAYEY GRAVEL with SAND
Sand	≤ 5%	Cu≥6 1≤Cc≤3		SW	< 15% gravel	Well-graded SAND
					≥ 15% gravel	Well-graded SAND with GRAVEL
		Cu<6 1>Cc>3		SP	< 15% gravel	Poorly graded SAND
					≥ 15% gravel	Poorly graded SAND with GRAVEL
	5-12%	Cu≥6 1≤Cc≤3	ML or MH	SW-SM	< 15% gravel	Well-graded SAND with SILT
					≥ 15% gravel	Well-graded SAND with SILT and GRAVEL
			CL, CH or CL-ML	SW-SC	< 15% gravel	Well-graded SAND with CLAY
					≥ 15% gravel	Well-graded SAND with CLAY and GRAVEL
		Cu<6 1>Cc>3	ML or MH	SP-SM	< 15% gravel	Poorly graded SAND with SILT
					≥ 15% gravel	Poorly graded SAND with SILT and GRAVEL
			CL, CH or CL-ML	SP-SC	< 15% gravel	Poorly graded SAND with CLAY
					≥ 15% gravel	Poorly graded SAND with CLAY and GRAVEL
	> 12%		ML or MH	SM	< 15% gravel	SILTY SAND
					≥ 15% gravel	SILTY SAND with GRAVEL
			CL or CH	SC	< 15% gravel	CLAYEY SAND
					≥ 15% gravel	CLAYEY SAND with GRAVEL
			CL-ML	SC-SM	< 15% gravel	SILTY, CLAYEY SAND
					≥ 15% gravel	SILTY, CLAYEY SAND with GRAVEL

3.2.3 Consistency (Cohesive Soils)

Cohesive soil consistency descriptors shall conform to terminology and criteria established in Figure 3-5 below, generally after Das (1983) and Bureau of Reclamation standards (2001). Note that the terms to be used have been modified from those contained in both references.

The preferred procedure for the determination of consistency of cohesive soils is to obtain relatively undisturbed samples and perform laboratory triaxial, direct shear, or unconfined compression tests. The results from these tests can be correlated to specific consistency descriptors as presented in Figure 3-5 below.

A triaxial unconsolidated-undrained (UU) test is recommended for strength determination. This can be converted to an equivalent unconfined compressive strength by multiplying the Undrained Shear Strength value by 2.

Figure 3-5
Consistency

Description	Unconfined Compressive Strength (tsf)
Very Soft	< 0.25
Soft	0.25 to 0.50
Medium Stiff	0.50 to 1.0
Stiff	1 to 2
Very Stiff	2 to 4
Hard	> 4.0

3.2.4 Percent or Proportion of Soils

Percentages of gravel, sand, and fines shall be reported as percentages based on gradation and particle-size analysis (ASTM D 422-63 (2002)). Qualitative proportional descriptors (e.g. trace, some, etc.) shall not be used when gradation data is available.

Figure 3-6
Percent or proportion of soils

Descriptive Term	Size
Gravel	3 inch to No.4 Sieve
Sand	No.4 to No. 200 Sieve
Fines	Passing No. 200 Sieve

3.2.5 Particle Size

When laboratory particle size analyses are performed, the USCS soil descriptions shall be further refined using the results and the Figure 3-8 below.

Figure 3-8
Particle size

Description	Size
Boulder	>12 in
Cobble	3 to 12 in
Coarse Gravel	¾ to 3 in
Fine Gravel	No.4 to ¾ in
Coarse Sand	No.10 to No.4
Medium Sand	No.40 to No.10
Fine Sand	No. 200 to No.40
Clay and Silt	Passing No. 200

3.2.6 Plasticity (for Fine-Grained Soils)

Field estimates of plasticity shall not be included in the descriptive sequence when USCS classifications are based on liquid limit and plasticity index, since the plasticity is inherent in the group name and group symbol.

3.3 Revising Rock Identification and Description for Borehole Cores Using Laboratory Test Results

One additional component, relative strength of intact rock, can be added to the descriptive sequence for rock.

Figure 3-9

Rock Identification and Descriptive Sequence

Sequence	Identification Components	Refer to Section		Required	Optional
		Field	Lab		
1	Rock Name	2.5.2		●	
	Description Components				
2	Rock Grain-size	2.5.3			○
3	Bedding Spacing	2.5.4		●	
4	Color	2.5.5		●	
5	Texture	2.5.6			○
6	Weathering Descriptors for Intact Rock	2.5.7		●	
7	Relative Strength of Intact Rock		3.3.1	●	
8	Rock Hardness	2.5.8		●	
9	Fracture Density	2.5.9		●	
10	Discontinuity Type	2.5.10			○
11	Discontinuity Condition (Weathering, Infilling and Healing)	2.5.11			○
12	Discontinuity Dip Magnitude	2.5.12			○
13	Rate of Slaking (Jar Slake Test)	2.5.13			○
14	Odor	2.5.14			○
15	Additional Comments	2.5.15			○

3.3.1 Strength of Intact Rock

Absent discontinuities, the strength of intact rock is best determined using unconfined compression laboratory testing.

Figure 3-10

Descriptors for Relative Strength of Intact Rock

Description	Uniaxial Compressive Strength (psi)
Extremely Strong	> 30,000
Very Strong	14,500 – 30,000
Strong	7,000 – 14,500
Medium Strong	3,500 – 7,000
Weak	700 – 3,500
Very Weak	150 – 700
Extremely Weak	< 150

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Section 4:
Methods of Presentation of Subsurface Information

4.1 Introduction

The process of creating boring logs, i.e., Log of Test Borings (LOTB) and Boring Records (BR) can be summarized in four steps:

- Field sampling and descriptions *(Section 2)*
- Quality check of field descriptions *(Section 2)*
- Refinement of descriptions, and classification of soil, based on laboratory test results, if performed *(Section 3)*
- Preparation of the boring logs *(Sections 4 and 5)*

This section provides details and guidance for incorporating laboratory test data and preparing boring logs. Figure 4-1 is a schematic representation of the process from obtaining subsurface information to the creation of boring logs.

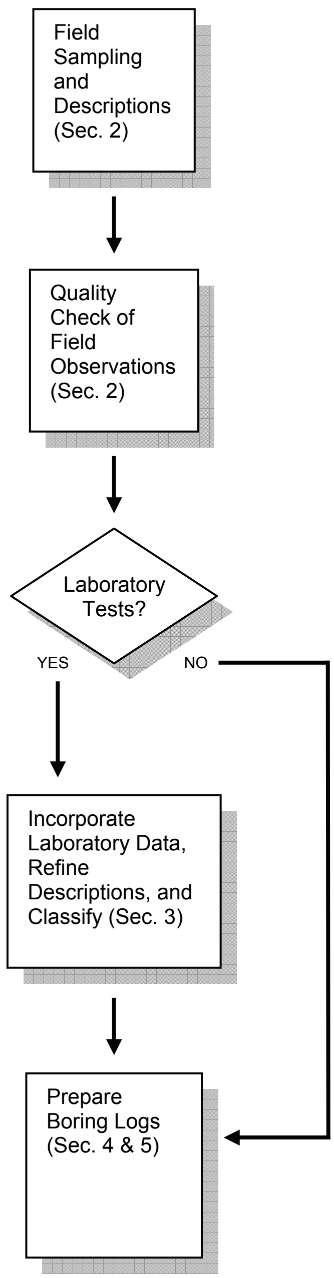
4.2 Factual vs. Interpretive Subsurface Data

FHWA guidelines state: “factual subsurface data which is pertinent to the project subsurface conditions should be presented in an easily understood fashion on the contract documents.” However, there is an inherent level of professional interpretation in describing subsurface conditions that cannot be avoided.

Some examples:

- Field description and identification procedures, according to ASTM D 2488-06, require estimation and interpretation.
- Sampling may occur at discrete intervals, yet layer boundaries may fall between sampling locations. The boundaries may be identified based on visual observations of cuttings during boring advancement. There may be uncertainty as to the depth at which a material change occurs.
- Continuity of material types between discrete sampling locations is sometimes difficult to confirm.

Figure 4-1



4.3 Incorporating Laboratory Data, Refining Descriptions, and Classifying Soil

When describing soil or rock, the geoprofessional shall use the most reliable data available. Such data could be field-generated, or a combination of field- and laboratory-generated data. If laboratory tests are performed, and in the opinion of the geoprofessional, those test results represent the actual conditions of the soil or rock, those test results shall control the identification, description, or classification.

Laboratory tests are usually not performed on every sample, especially on contiguous samples within a layer of similar material. Professional judgment should be used to apply test results from one sample to the descriptor of contiguous samples within a boring when the field observations were such that the geoprofessional considers that particular attribute of the material to be consistent across the contiguous samples.

For example, three contiguous samples were determined to be "medium stiff" using the thumb method. However, a triaxial test on one of those samples indicated that the material was in fact "stiff." In this example, the consistency descriptor for the entire layer should be "stiff."

4.3.1 Subsurface Data Presentation Method

A "layer presentation" method shall be used to present soil and rock descriptions on boring logs. *(See Figure 4-3.)* This method presents a single primary description for a layer spanning one or more contiguous sample locations. However, the layer description may vary with depth as observations and/or laboratory testing at sample locations warrant. This method is used to simplify the boring log presentation format and provide clarity, especially to prospective bidders.

4.3.2 General Rules and Considerations

The following general rules apply to the layer presentation method:

- A change in a soil's Group Symbol or a rock type shall result in a new layer within the boring log.
- Reliable laboratory test results shall be used when performed to determine the applicable descriptors within the descriptive sequence (i.e. Group Name and Symbol, consistency, gradation properties, plasticity, and rock strength).
- Individual descriptors for contiguous samples with the same descriptions and classifications should be adjusted based on the test results of one or more representative samples. Use of more than one test sample is encouraged.
- When specifying a descriptive range, the range shall not span more than one step on the range of descriptors. For example, "stiff to very stiff" would be acceptable; but "soft to hard" would not be acceptable because the range is too broad to be useful.
  Exception: Any range of colors is acceptable.
- The descriptive sequence used to describe a layer of soil or rock shall describe that layer in its entirety. Where changes are noted at depths within the layer, those changes shall replace the preceding descriptor and shall apply from that depth to the bottom of the layer.
- The layer presentation of the boring log shall enable the reader to derive individual sample descriptions based on the layer description and sample information provided.

4.3.3 Example

The process for developing boring logs has been presented in detail throughout this Manual In general, field sample descriptions are corrected and calibrated based on laboratory results, layer boundaries are determined by grouping samples within the same group symbol, sample descriptions are consolidated into a single layer description, and, finally, description changes are noted with depth within layers.

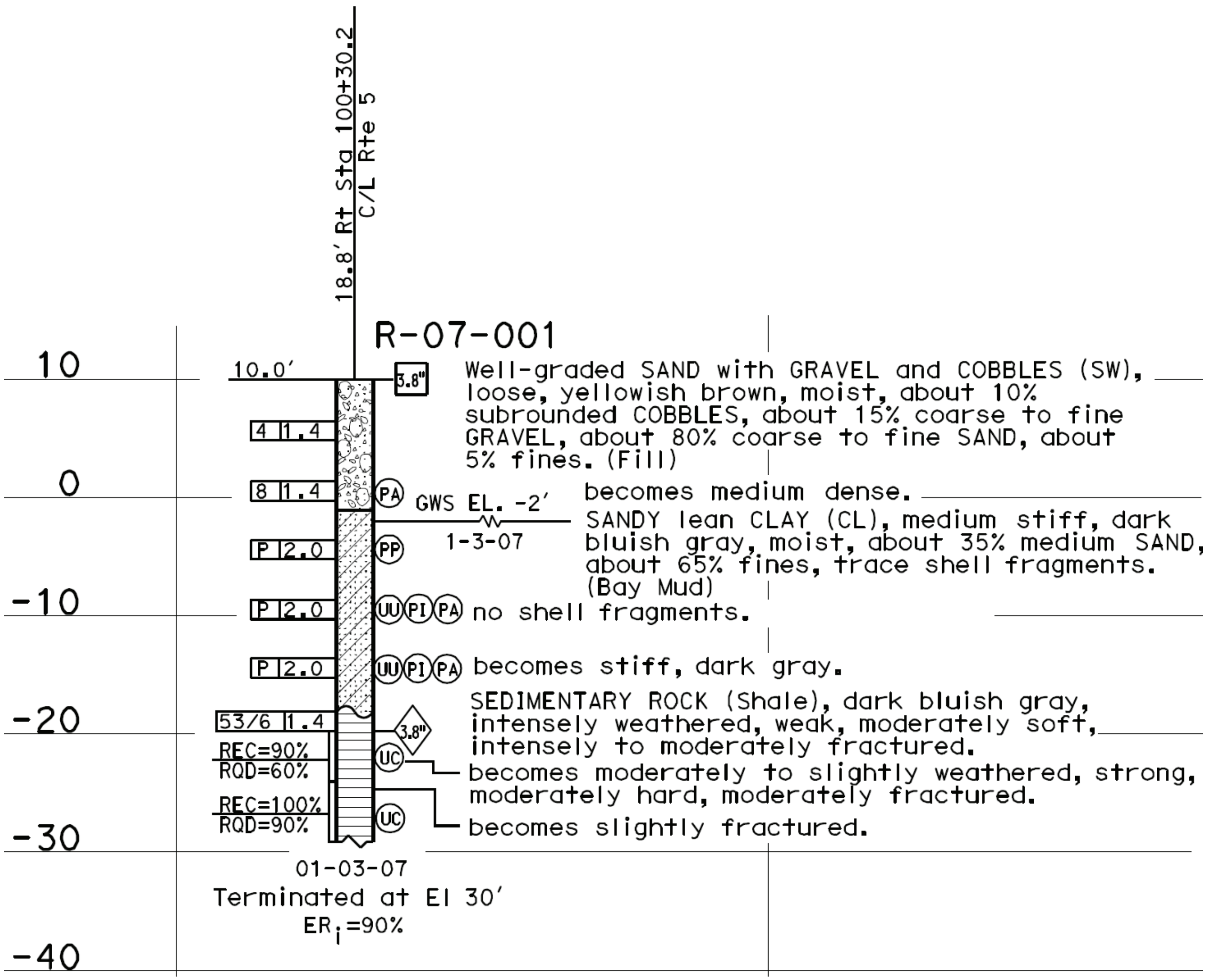
The following example demonstrates how a geoprofessional develops a layer presentation, based on field descriptions and laboratory test results.

Figure 4-2

Depth (ft.)	Sample	Field Testing	Field Description and Identification	Lab Testing	Lab Corrected Description and Identification or Classification	Final Layer Presentation
3.5-5	R-07-001-S01	SPT	Well-graded SAND with GRAVEL and COBBLES (SW), loose, yellowish brown, moist, about 10% subrounded COBBLES, about 15% coarse to fine GRAVEL, about 80% coarse to fine SAND, about 5% fines, (fill)		Well-graded SAND with GRAVEL and COBBLES (SW), loose, yellowish brown, moist, about 10% subrounded COBBLES, about 15% coarse to fine GRAVEL, about 80% coarse to fine SAND, about 5% fines, (fill)	Well-graded SAND with GRAVEL and COBBLES (SW), loose, yellowish brown, moist, about 10% subrounded COBBLES, about 15% coarse to fine GRAVEL, about 80% coarse to fine SAND, about 5% fines, (fill)
8.5-10	R-07-001-S02	SPT	Well-graded SAND with GRAVEL and COBBLES (SW), medium dense, yellowish brown, moist, about 10% subrounded COBBLES, about 95% coarse to fine SAND, about 5% fines, (fill)	PA	Well-graded SAND with GRAVEL and COBBLES (SW), medium dense, yellowish brown, moist, about 10% subrounded COBBLES, about 95% coarse to fine SAND, about 5% fines, (fill)	becomes medium dense
13.5-15	R-07-001-U03	PP	SANDY lean CLAY (CL), medium stiff, dark bluish gray, moist, about 35% medium SAND, about 65% fines, trace shell fragments, (bay mud)		SANDY lean CLAY (CL), medium stiff, dark bluish gray, moist, about 35% medium SAND, about 65% fines, trace shell fragments, (bay mud)	SANDY lean CLAY (CL), medium stiff, dark bluish gray, moist, about 35% medium SAND, about 65% fines, trace shell fragments, (bay mud)
18.5-20	R-07-001-U04	PP	SANDY lean CLAY (CL), soft, dark bluish gray, moist, about 35% medium SAND, about 65% fines, (bay mud)	UU, PA, PI	SANDY lean CLAY (CL), medium stiff, dark bluish gray, moist, about 35% medium SAND, about 65% fines, (bay mud)	as above except no shell fragments
23.5-25	R-07-001-U05	PP	SANDY lean CLAY (CL), medium stiff, dark gray, moist, about 35% medium SAND, about 65% fines, trace shell fragments, (bay mud)	UU, PA, PI	SANDY lean CLAY (CL), stiff, dark gray, moist, about 35% medium SAND, about 65% fines, trace shell fragments, (bay mud)	becomes stiff, dark gray
28-29	R-07-001-S06	SPT	SEDIMENTARY ROCK (SHALE), dark bluish gray, intensely weathered, moderately soft, intensely to moderately fractured		SEDIMENTARY ROCK (SHALE), dark bluish gray, intensely weathered, moderately soft, intensely to moderately fractured	SEDIMENTARY ROCK (SHALE), dark bluish gray, intensely weathered, moderately soft, intensely to moderately fractured
29-34	R-07-001-C07		SEDIMENTARY ROCK (SHALE), dark bluish gray, moderately to slightly weathered, moderately hard, moderately fractured	UC	SEDIMENTARY ROCK (SHALE), dark bluish gray, moderately to slightly weathered, strong, moderately hard, moderately fractured	becomes moderately to slightly weathered, strong, moderately hard, moderately fractured
34-39	R-07-001-C08		SEDIMENTARY ROCK (SHALE), dark bluish gray, moderately to slightly weathered, moderately hard, slightly fractured	UC	SEDIMENTARY ROCK (SHALE), dark bluish gray, moderately to slightly weathered, strong, moderately hard, slightly fractured	becomes slightly fractured

The LOTB for the preceding example would appear as follows:

Figure 4-3



Section 5:
Boring Log and Legend Presentation Formats

5.1 Introduction

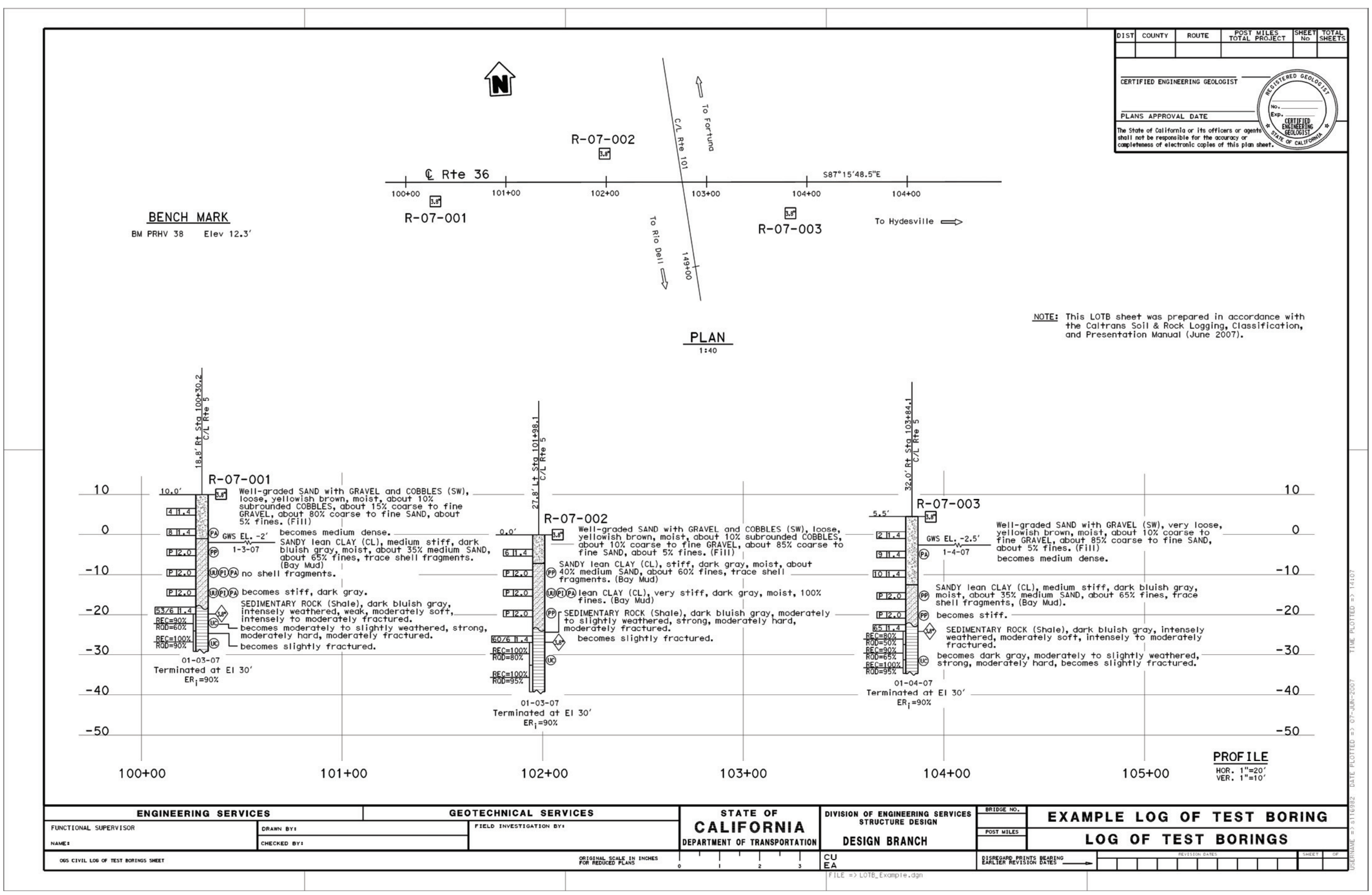
The Department uses the following formats to present subsurface information:

- Log of Test Boring (LOTB), and/or
- Boring Record (BR).

An LOTB is typically associated with a structure facility and is attached to Project Plans. A BR is typically associated with an earthwork facility and is attached to a Geotechnical Report. If a subsurface exploration was performed, there shall be at least one type of log presentation.

5.2 Log of Test Boring

Figure 5-1
Example of LOTB



5.2.1 Contents and Characteristics of the LOTB

The Log of Test Boring (LOTB) document is presented as an attachment to project plans and characterized by the following attributes:

- Presents the boring logs on an elevation scale.
- Presents a plan view showing the location of each boring relative to an alignment and/or existing or planned facility.
- Presents the type of drilling methods used to perform the investigation, the type of sampling performed, and how the sampler was advanced.
- Presents the location and description, both graphical and written, of the types of soil and rock encountered within the borehole.
- Presents the types of field and laboratory testing performed.
- Field and Laboratory test data, if presented, appear at the end of the descriptive sequence.
- Optimized for printing on full-size plan sheets (24" x 36") and typically reproduced on 11" x 17" sized paper.
- Allows presentation of more than one boring log per plan sheet.
- Is accompanied by LOTB legend sheets.

5.2.2 Notes on the LOTB

Each LOTB sheet shall contain a note section for the geoprofessional to present notes deemed to be of interest to the reader. Content of notes is left to the discretion of the geoprofessional except that the one of the following two notes shall be placed on each LOTB sheet:
If the procedures of this manual were followed without exception, then the note shall read:

"This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (Date)"

If an exception to the procedures of this manual has been approved and implemented, then the note shall be modified to read:

"This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (Date) except as noted in (Section) of (Report Title) dated (Date)"

Optional notes may include:

- Changes in drilling equipment
- Site observations
- Other drilling observations from Sections 2.4.20 or 2.5.16.

5.2.3 LOTB Sheet Formatting

LOTB sheets shall be prepared in accordance with this manual and the Caltrans *Plans Preparation Manual*. The LOTB sheet border shall present the following:

5.2.3.1 Signature Block (Upper Right Corner)

a) The State of California Registered Civil Engineer, Certified Engineering Geologist, or Registered Geologist seal with the signature, date, license number, and registration certificate expiration date of the engineer or geologist in responsible charge of the LOTB sheet;
b) Caltrans District, County, and Route;
c) Name and address of consultant firm in responsible charge of the LOTB sheet (if applicable);
d) Name and address of the lead local agency (if applicable); and
e) A disclaimer stating "The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet."

(The Office Engineer will provide the Post Miles Total Project, Sheet Number, Total Sheets, and Plans Approval Date.)

5.2.3.2 Title Block (Bottom, from left to right)

a) Notes stating "DIVISION OF ENGINEERING SERVICES" and "GEOTECHNICAL SERVICES." For consultant-prepared LOTB sheets, instead of those notes, show the name of the Design

Oversight (i.e., OSFP/OSCM Senior Liaison) Engineer and sign-off date.

b) "FUNCTIONAL SUPERVISOR": The name of the person in charge of the functional unit responsible for providing oversight of the registered engineer or geologist who developed the LOTB sheet.
c) "DRAWN BY": The name of the person who prepared (drafted) the LOTB sheet
d) "CHECKED BY": The name of the person who performed the quality control check of the LOTB sheet
e) "FIELD INVESTIGATION BY": The name(s) of the field investigator(s);
f) A note stating "STATE OF CALIFORNIA, DEPARTMENT OF TRANSPORTATION" with a scale below the sub-block and a label on the left side stating "ORIGINAL SCALE IN INCHES FOR REDUCED PLANS." For consultant-prepared LOTB sheets, the note shall state "PREPARED FOR THE STATE OF CALIFORNIA, DEPARTMENT OF TRANSPORTATION."
g) A note stating "DIVISION OF ENGINEERING SERVICES STRUCTURE DESIGN." For consultant-prepared LOTB sheets, instead of this note, show the name of the Project Engineer;
h) The Caltrans Contract Expenditure Authorization (CU and EA) numbers;
i) The State-assigned Bridge (or Structure) Number, Kilometer Post, and the State-assigned Bridge (or Structure) Name;
j) The initial drawn by and subsequent revision dates; and
k) A label stating "LOG OF TEST BORINGS _ OF _" (if applicable).

(The Office Engineer will provide the Sheet Number and Total Sheets Number.)

5.2.3.3 Plan View

a) Plan View shall be shown at the top of the first LOTB sheet. When the site is sufficiently large or complex, the first LOTB sheet should be used entirely for the Plan View.
b) When multiple LOTB sheets are drafted, they shall be numbered with reference to the stationing of the control line (i.e., showing sheet No. 1 with the lowest stationing and the last sheet with the highest stationing).
c) A distinct Plan View of the project site that is independent of the Profile View shall be shown on the LOTB.
d) Show the location, description, and elevation of the benchmark used for determining the top of boring elevations at the top left side of the Plan View under the heading "BENCHMARK". Identify the vertical datum (National Geodetic Vertical Datum, U.S. Geological Survey, U.S. Coast & Geodetic Survey, District, etc.) used to determine the benchmark elevations.
e) Show the scale directly below the Plan View label.
f) Show a North arrow.
g) Lines or control lines shown in the Plan View shall be consistent with those shown on the General Plan sheet.
h) Show stationing and names for control lines. Stationing shall increase from left to right. Show a minimum of two stations on all lines.
i) Show control line intersection stationing and bearings.
j) Show names and directions of nearest cities.
k) Show names and directions of stream flows when applicable.
l) Plot boring locations with symbols as shown in the legend to identify drilling methods (e.g., auger hole, rotary hole, cone penetration). The Hole Identification shall be presented with each symbol.
m) Boring locations are to be identified by reference line, station, and offset. Coordinates, such as Northing and Easting, may also be shown on the LOTB sheets.

5.2.3.4 Profile View

a) Show the control line, increasing from left to right, horizontally across the bottom of the Profile View.
b) Show the elevations and grid lines on both the left and right margins. Numerical values shall be in multiples of 10 (i.e. 20, 10, 0, -10, -20).

c) Show the Hole Identification, top of hole elevation, stationing, and offset at the top of each boring log.
d) Show types and diameters of each boring as shown in the legend.
e) Show the completion date of boring (m/d/y) at the bottom of each boring log.
f) Show "Terminated at EL. XX'" to indicate the bottom of boring elevation.
g) Show the SPT hammer energy ratio, "Hammer Energy Ratio (ER_i) = XX%," at the bottom of each boring.
h) Show date and elevation of groundwater measurement.
i) Show results from field penetration tests at relevant elevations along the boring log.
j) Show types of field and laboratory tests with symbols as indicated in the legend, at relevant elevations along the right side of the boring log.
k) Show the Profile scales (horizontal and vertical) under the heading "PROFILE".

5.2.3.5 Additional information to be included

a) Show standard note identifying the logging practice as follows:

   "This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual"

   When a LOTB presents information that deviates from the Caltrans Soil & Rock Logging, Classification, and Presentation Manual standards, the standard note shall be modified to read as follows:

   "*This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual except as noted in (section) of (Report Title) dated (Date)"*

b) Descriptions of types of samplers used for the field exploration.

5.2.4 As-Built LOTB Sheet Formatting

As-Built LOTB sheet(s) shall be prepared according to the following standards.

5.2.4.1 Obtaining and Reproducing the As-Built LOTB Sheet

a) Reproducible copies of As-Built LOTB sheets may be obtained from the Microfilm Services Units in the Caltrans District Offices. If the As-Built LOTB sheets provided to Local Agencies or consultants by the Caltrans District Offices are not legible, a full sized copy should be requested from Geotechnical Services.
b) As-Built LOTB sheets shall be size "D" (24" by 36"). The As-Built LOTB title block shall be sized to fit and placed over any open space (preferably toward the top) on the As-Built LOTB sheet.
c) Information on the As-Built LOTB sheet shall be clear and legible. In order to improve the legibility of the information, it may be necessary to darken the line work and the notations.

5.2.4.2 Typical Modifications to As-Built LOTB Sheets

a) If As-Built LOTB sheets are shown in metric units, the offset and stationing location of each boring must be converted to imperial units. A table shall be added showing the dual dimensions (Metric and English) of each boring. The table shall show the station and offset in relation to the new English line. The General Plan will show the current English control line.

5.2.4.3 The As-Built LOTB Title Block shall include the following information for the current project

a) A note stating "GEOTECHNICAL SERVICES -- DIVISION OF ENGINEERING SERVICES" (if applicable).
b) Caltrans District, County, Route, Post Miles - Total Project, State-assigned Bridge (or Structure) Number and Name, and Expenditure Authorization (CU and EA)

numbers. The Office Engineer will provide the Sheet Number and Total Sheets Number.

c) The State of California Registered Civil Engineer or Registered Geologist seal with the signature, date, license number, and registration certificate expiration date of the engineer or geologist in responsible charge of the LOTB sheet.

d) A note stating, "As-Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. It does not attest to the accuracy or validity of the information contained in the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party."

e) A sub-box stating "LOG OF TEST BORINGS _ OF _" (if applicable).

e) A note stating "A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA" (if applicable).

5.2.5 The LOTB Legend Sheets

The soil and rock legend sheets are standard forms that provide convenient references for the *required* soil and rock description, identification, and/or classification components presented in this Manual. References for *optional* descriptors do not appear on the legend sheets; however, they are explained in this Manual. To correctly interpret the LOTB, the reader shall be familiar with this Manual.

There are two legend sheets, one predominantly for soil and the other for rock, as shown in the Figures 5-2 and 5-3.

The legend sheets define the format for the graphical presentation of a boring log and differentiate among the various borehole and sounding types. The legend sheets also present the symbols used to identify laboratory tests.

GEOTECHNICAL SERVICES – DIVISION OF ENGINEERING SERVICES

As-Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that is this a true and accurate copy of the original document. It does not attest to the accuracy or validity of the information contained in the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

DIST.	COUNTY	ROUTE	POST MILES – TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
08	SBD	210	10.00-15.00	24	25

REGISTERED ENGINEER – CIVIL DATE

MAIN STREET OVERCROSSING

LOG OF TEST BORINGS 5 OF 6

NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA	CU: 12 EA: 432563	BRIDGE NO. 12-3456

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Figure 5-2

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

REGISTERED PROFESSIONAL ENGINEER No. Exp. CIVIL STATE OF CALIFORNIA

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE, 2007)

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GM	SILTY GRAVEL SILTY GRAVEL with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly graded SAND Poorly graded SAND with GRAVEL
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic/Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTING

C	Consolidation (ASTM D 2435-04)
CL	Collapse Potential (ASTM D 5333-03)
CP	Compaction Curve (CTM 216-06)
CR	Corrosivity Testing (CTM 643, CTM 422, CTM 417)
CU	Consolidated Undrained Triaxial (ASTM D 4767-02)
DS	Direct Shear (ASTM D 3080-04)
EI	Expansion Index (ASTM D 4829-03)
M	Moisture Content (ASTM D 2216-05)
OC	Organic Content-% (ASTM D 2974-07)
P	Permeability (CTM 220-05)
PA	Particle Size Analysis (ASTM D 422-63) (2002)
PI	Plasticity Index (AASHTO T 90-00) Liquid Limit (AASHTO T 89-02)
PL	Point Load Index (ASTM D 5731-05)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301-00)
SE	Sand Equivalent (CTM 217-99)
SG	Specific Gravity (AASHTO T 100-06)
SL	Shrinkage Limit (ASTM D 427-04)
SW	Swell Potential (ASTM D 4546-03)
TV	Pocket Torvane
UC	Unconfined Compression-Soil (ASTM D 2166-06) Unconfined Compression-Rock (ASTM D 2938-95) (2002)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850-03)
UW	Unit Weight (ASTM D 4767-04)
VS	Vane Shear (AASHTO T 223-96) (2004)

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60}-Value (Blows / 12 in.)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

PARTICLE SIZE

Description		Size
Boulder		> 12 in.
Cobble		3 to 12 in.
Gravel	Coarse	3/4 to 3 in.
	Fine	No. 4 to 3/4 in.
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

CONSISTENCY OF COHESIVE SOILS

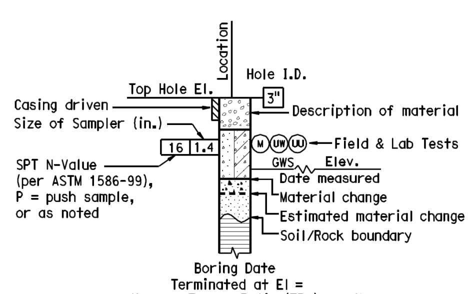
Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

PLASTICITY OF FINE-GRAINED SOILS

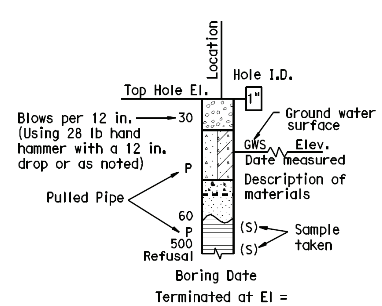
Description	Criteria
Nonplastic	A 1/8-in. thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

BOREHOLE IDENTIFICATION

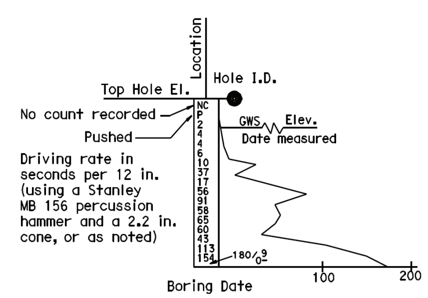
Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
●	D	Dynamic Cone Penetration Boring
▲	CPT	Cone Penetration Test (ASTM D 5778-95)
	O	Other



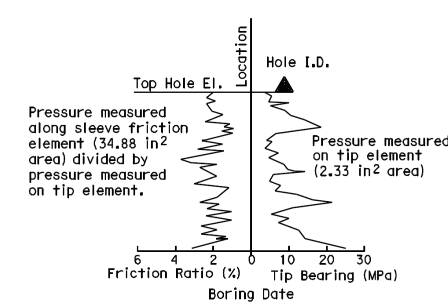
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

ENGINEERING SERVICES	GEOTECHNICAL SERVICES	STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	DIVISION OF ENGINEERING SERVICES STRUCTURE DESIGN DESIGN BRANCH	BRIDGE NO. POST MILE	SOIL LEGEND LOG OF TEST BORINGS
PREPARED BY					
CHECKED BY					

GS LOTB SOIL LEGEND — ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3 — CU EA — DISREGARD PRINTS BEARING EARLIER REVISION DATES — REVISION DATES — SHEET OF

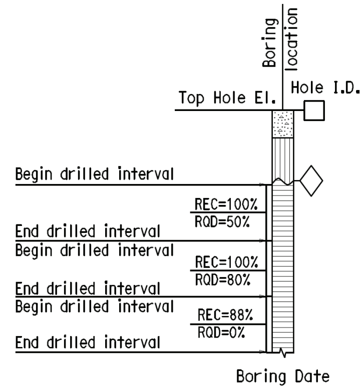
Figure 5-3

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE, 2007)

PERCENT CORE RECOVERY (REC) & ROCK QUALITY DESIGNATION (RQD)

$$REC = \frac{\sum \text{Length of the recovered core pieces (in.)}}{\text{Total length of core run (in.)}} \times 100\%$$

$$RQD = \frac{\sum \text{Length of intact core pieces} \geq 4 \text{ in.}}{\text{Total length of core run (in.)}} \times 100\%$$



RELATIVE STRENGTH OF INTACT ROCK

Term	Uniaxial Compressive Strength (PSI)
Extremely Strong	> 30,000
Very Strong	14,500 - 30,000
Strong	7,000 - 14,500
Medium Strong	3,500 - 7,000
Weak	700 - 3,500
Very Weak	150 - 700
Extremely Weak	< 150

BEDDING SPACING

Description	Thickness / Spacing
Massive	Greater than 10 ft
Very thickly bedded	3 to 10 ft
Thickly bedded	1 to 3 ft
Moderately bedded	4 in. to 1 ft
Thinly bedded	1 in. to 4 in.
Very thinly bedded	3/8 in. to 1 in.
Laminated	Less than 3/8 in.

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

REGISTERED PROFESSIONAL ENGINEER No. Exp. CIVIL STATE OF CALIFORNIA

LEGEND OF ROCK MATERIALS

- IGNEOUS ROCK
- SEDIMENTARY ROCK
- METAMORPHIC ROCK

ROCK HARDNESS

Description	Criteria
Extremely Hard	Specimen cannot be scratched with a pocket knife; no steel marks left on surface.
Very Hard	Specimen cannot be scratched with a pocket knife; steel marks left on surface.
Hard	Specimen can be scratched with a pocket knife with difficulty (heavy pressure).
Moderately Hard	Specimen can be scratched with pocket knife with light or moderate pressure.
Moderately Soft	Specimen can be grooved 1/6 in. deep with a pocket knife with moderate or heavy pressure.
Soft	Specimen can be grooved or gouged easily by a pocket knife with light pressure, can be scratched with fingernail.
Very Soft	Specimen can be readily indented, grooved or gouged with fingernail, or carved with a pocket knife.

FRACTURE DENSITY

Description	Observed Fracture Density
Unfractured	No fractures.
Very slightly fractured	Lengths greater than 3 feet.
Slightly fractured	Lengths from 1 to 3 feet with few lengths less than 1 foot or greater than 3 feet.
Moderately fractured	Lengths mostly in 4 in. to 1 foot range with most lengths about 8 in.
Intensely fractured	Lengths average from 1 to 4 in. with scattered fragmented intervals with lengths less than 4 in.
Very intensely fractured	Mostly chips and fragments with a few scattered short core lengths.

Combination descriptors (such as "Very intensely to intensely fractured") are used where equal distribution of both fracture density characteristics is present over a significant interval or exposure, or where characteristics are "in between" the descriptor definitions. Only two adjacent descriptors may be combined.

WEATHERING DESCRIPTORS FOR INTACT ROCK

Description	Diagnostic features					General Characteristics
	Chemical Weathering-Discoloration and/or oxidation		Mechanical Weathering-Grain boundary conditions (disaggregation) primarily for granitics and some coarse-grained sediments	Texture and Solutioning		
	Body of Rock	Fracture Surfaces		Texture	Solutioning	
Fresh	No discoloration, not oxidized.	No discoloration or oxidation.	No separation, intact (tight).	No change.	No solutioning.	Hammer rings when crystalline rocks are struck.
Slightly Weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull.	Minor to complete discoloration or oxidation of most surfaces.	No visible separation, intact (tight).	Preserved.	Minor leaching of some soluble minerals may be noted.	Hammer rings when crystalline rocks are struck. Body of rock not weakened.
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty," feldspar crystals are "cloudy."	All fracture surfaces are discolored or oxidized.	Partial separation of boundaries visible.	Generally preserved.	Soluble minerals may be mostly leached.	Hammer does not ring when rock is struck. Body of rock is slightly weakened.
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in-situ disaggregation, see grain boundary conditions.	All fracture surfaces are discolored or oxidized, surfaces friable.	Partial separation, rock is friable; in semiarid conditions granitics are disaggregated.	Texture altered by chemical disintegration (hydration, argillation).	Leaching of soluble minerals may be complete.	Dull sound when struck with hammer, usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures, or veinlets. Rock is significantly weakened.
Decomposed	Discolored or oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay.		Complete separation of grain boundaries (disaggregated).	Resembles a soil, partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete.		Can be granulated by hand. Resistant minerals such as quartz may be present as "stringers" or "dikes."

Combination descriptors (such as "slightly weathered to fresh") are used where equal distribution of both weathering characteristics is present over significant intervals or where characteristics present are "in between" the diagnostic feature. However, combination descriptors should not be used where significant, identifiable zones can be delineated. Only two adjacent descriptors may be combined. "Very intensely weathered" is the combination descriptor for "intensely weathered to decomposed."

ENGINEERING SERVICES | PREPARED BY | CHECKED BY

GEOTECHNICAL SERVICES

STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

DIVISION OF ENGINEERING SERVICES STRUCTURE DESIGN

DESIGN BRANCH

BRIDGE NO. | POST MILE

ROCK LEGEND

LOG OF TEST BORINGS

GS LOTB ROCK LEGEND

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU EA

DISREGARD PRINTS BEARING EARLIER REVISION DATES

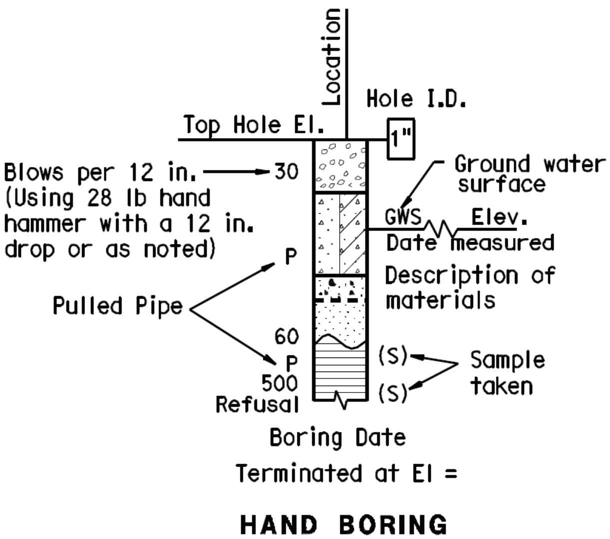
REVISION DATES | SHEET OF

Four general hole-type formats are graphically presented as follows:

5.2.5.1 Hand Boring

Hand Driven (HD) (1-inch soil tube) and Hand Auger (HA) borings shall be presented using the following format:

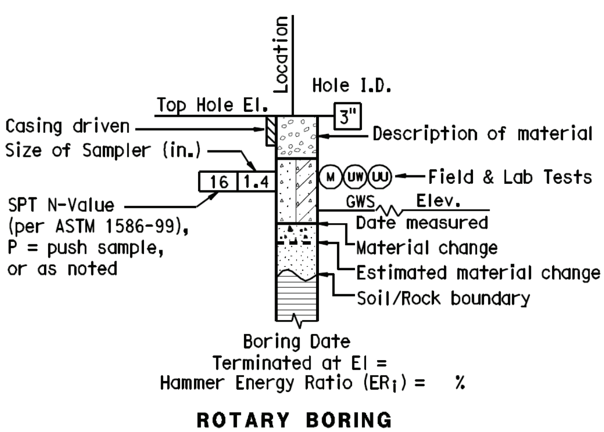
Figure 5-4



5.2.5.2 Rotary Boring

Rotary Drilled Boring or Diamond Core (R), Rotary Percussion Boring (Air) (P), Auger Boring (A), shall be presented using the following format:

Figure 5-5



Notes:

If laboratory tests are not shown as being performed, the soil descriptions presented in the LOTB are based solely on the visual practices described in this Manual.

Changes in material with depth shall be noted using the following terms:

Figure 5-6

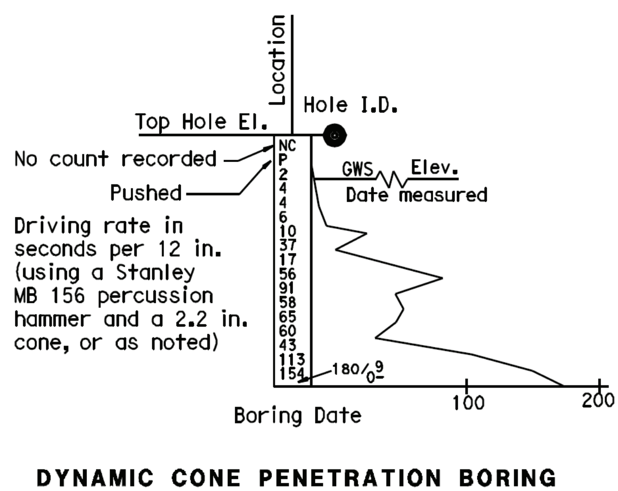
Definitions for changes in material

Term	Definition
Material Change	Change in material is observed in the sample or core, and the location of change can be accurately measured.
Estimated Material Change	Change in material cannot be accurately located because either the change is gradational or because of limitations in the drilling/sampling methods used.
Soil/Rock Boundary	Material changes from soil characteristics to rock characteristics and that change can be measured or estimated.

5.2.5.3 Dynamic Cone Penetration Sounding

The Dynamic Cone Penetration Sounding (D) shall be presented using the following format:

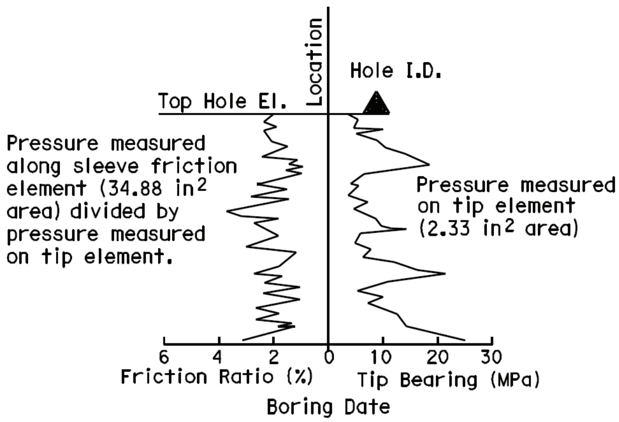
Figure 5-7



5.2.5.4 Cone Penetration Test (CPT) Sounding

A Cone Penetration Test (CPT) sounding shall be presented using the following format:

Figure 5-8

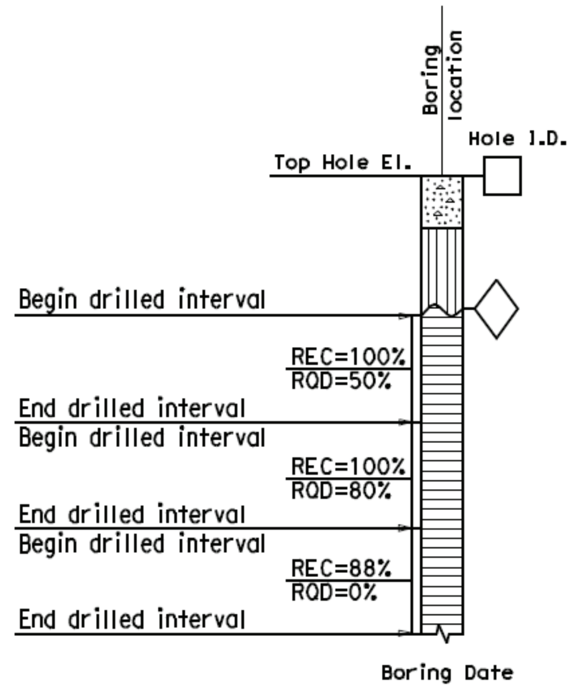


CONE PENETRATION TEST (CPT) SOUNDING

5.2.5.5 Rock Coring

Rock coring logs shall be presented using the following format:

Figure 5-9



5.2.5.6 Hole Type Symbols

Hole type is identified within the hole identification numbering convention (see Section 2.3) and symbolized on the LOTB as follows:

Figure 5-10

BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size (double square)	A	Auger Boring
Size (square)	R P	Rotary drilled boring Rotary percussion boring (air)
Size (diamond)	R	Rotary drilled diamond core
Size (rectangle)	HD HA	Hand driven (1-inch soil tube) Hand Auger
●	D	Dynamic Cone Penetration Boring
▲	CPT	Cone Penetration Test (ASTM D 5778-95)
(dashed square)	O	Other

5.2.5.7 Graphical Representation of Material Types

Soil Group Name and Group Symbol and Rock Type are symbolized on the LOTB as follows:

Figure 5-11

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GM	SILTY GRAVEL SILTY GRAVEL with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly graded SAND Poorly graded SAND with GRAVEL
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic / Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY elastic ELASTIC SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

ROCK GRAPHIC SYMBOLS
IGNEOUS ROCK
SEDIMENTARY ROCK
METAMORPHIC ROCK

5.3 Boring Records

Figure 5-12

LOGGED BY **Sue Pervisor** / BEGIN DATE **1-1-07** / COMPLETION DATE **1-2-07**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **34° 0' 24.62" / -117° 7' 62.55" WGS 84**	HOLE ID **R-07-001**
DRILLING CONTRACTOR **Gregg Drilling & Testing, Inc.**	BOREHOLE LOCATION (Station, Offset, Line) **Offset 24R C/L Rte 36**	SURFACE ELEVATION **20.7 ft NAVD 88**
DRILLING METHOD **Rotary Wash**	DRILL RIG **CS 2000 (track)**	BOREHOLE DIAMETER **8.5 in. (soil); 4 in. (rock)**
SAMPLER TYPE(S) AND SIZE(S) (ID) **SPT (1.4"), Shelby (2.87"), HQ core**	SPT HAMMER TYPE **Safety semi-automatic, 140 lb, 30-inch drop**	HAMMER EFFICIENCY, ERi **90%**
BOREHOLE BACKFILL AND COMPLETION **Neat cement grout backfill**	GROUNDWATER READINGS DURING DRILLING **12 ft** AFTER DRILLING (DATE) **21 ft on 1-4-07**	TOTAL DEPTH OF BORING **39.0 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per Foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	0	Well-graded SAND with GRAVEL and COBBLES (SW); loose; yellowish brown; moist; about 10% subrounded COBBLES; about 15% fine to coarse GRAVEL; about 80% fine to coarse SAND; about 5% fines [FILL].												
	1													
18.72	2													
	3			S01	2 2 2	4	94							
16.72	4													
	5													
14.72	6													
	7													
12.72	8													
	9	At 8.5 ft, becomes medium dense.		S02	4 6 6	12	100		5	105				PA
10.72	10													
	11	SANDY lean CLAY (CL); medium stiff; dark bluish gray; moist; about 35% medium SAND; about 65% fines; trace shell fragments [BAY MUD].												
8.72	12													
	13													
6.72	14			U03		0.25	100		22	100	PP = 0.55 TV = 0.25			
	15													
4.72	16													
	17													
2.72	18													
	19	At 18.5 ft, with no shell fragments.		U04			100		20	100	PP = 0.45 UU = 0.50			PA, PI
0.72	20													
	21													
-1.28	22													
	23													
-3.28	24	At 23.5 ft, becomes stiff; dark gray.		U05			100		20	101	PP = 0.45 UU = 0.60			PA, PI
	25													

(continued)

Department of Transportation Division of Engineering Services Geotechnical Services Office of Geotechnical Design - North	REPORT TITLE **BORING RECORD**				HOLE ID **R-07-001**
	DIST. **12**	COUNTY **Orange**	ROUTE **I-405**	POSTMILE **R34.1/R39.2**	EA **12-047643**
	PROJECT OR BRIDGE NAME **I-880 Realignment, Whitman Dr., O. C.**				
	BRIDGE NUMBER **53-0045**	PREPARED BY **Sue Pervisor**		DATE **5-25-07**	SHEET **1 of 2**

Figure 5-12 (continued)

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per Foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	25		SANDY lean CLAY (CL) *(continued)*.												
-5.28	26														
	27														
-7.28	28		SEDIMENTARY ROCK (Shale), grayish blue, intensely weathered, weak, moderately soft, intensely to moderately fractured, [BEDROCK].		S06	10 53/6"		100							SPT refusal at 6"; switch to coring.
	29		At 29 ft, becomes moderately to slightly weathered, strong, moderately hard, moderately fractured.		C07			90	60						
-9.28	30														PL
	31														
-11.28	32														
	33														
-13.28	34		At 34 ft, becomes slightly fractured.		C08			100	90						
	35														
-15.28	36														
	37														
-17.28	38														
	39		Bottom of Borehole at 39.0 ft. Boring terminated at planned depth.												
-19.28	40														
	41														
-21.28	42														
	43														
-23.28	44														
	45														
-25.28	46														
	47														
-27.28	48														
	49														
-29.28	50														
	51														
-31.28	52														
	53														
-33.28	54														
	55														

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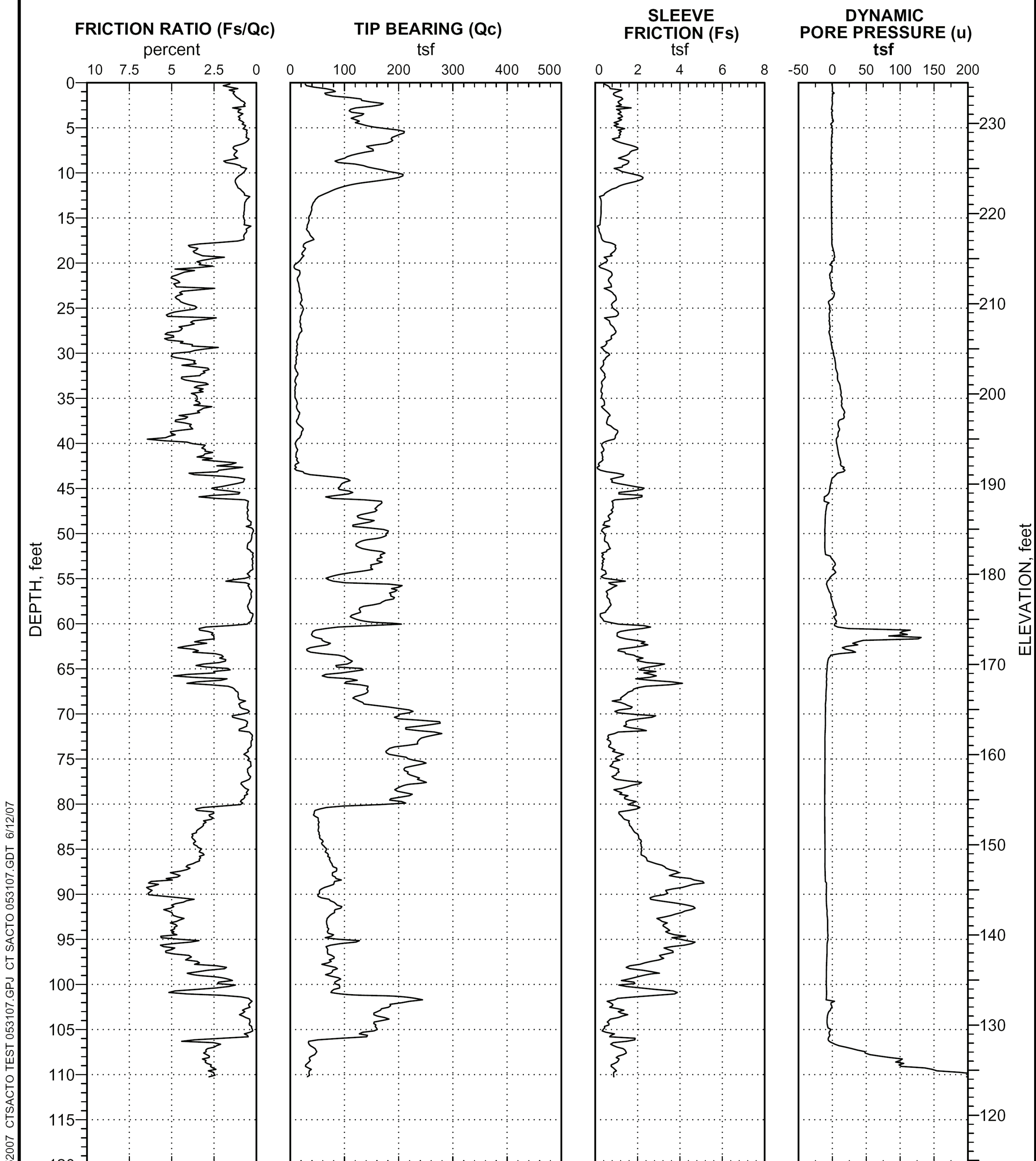
REPORT TITLE				HOLE ID
BORING RECORD				**R-07-001**
DIST. **12**	COUNTY **Orange**	ROUTE **I-405**	POSTMILE **R34.1/R39.2**	EA **12-047643**
PROJECT OR BRIDGE NAME **I-880 Realignment, Whitman Dr., O. C.**				
BRIDGE NUMBER **53-0045**	PREPARED BY **Sue Pervisor**		DATE **5-25-07**	SHEET **2 of 2**

Figure 5-13



Department of Transportation
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Office of Geotechnical Design - West

REPORT TITLE				HOLE ID
BORING RECORD				**CPT-07-001**
DIST. **04**	COUNTY **Alameda**	ROUTE **Rte 36**	POSTMILE **R34.1/R39.2**	EA **04-047643**
PROJECT OR BRIDGE NAME **Rte 36 Realignment Project**				
BRIDGE NUMBER **53-0045**	PREPARED BY **Sue Pervisor**		DATE **5-20-07**	SHEET **1 of 1**

5.3.1 Content and Characteristics of the BR

A Boring Record (BR) document is presented as an attachment to a geotechnical report and is characterized by the following attributes:

- Presents a single borehole record or CPT sounding.
- Presents the borings to an elevation scale.
- Presents the type of drilling method used to perform the investigation, the type of sampling performed, and how the sampler was advanced.
- Presents the location and description, both graphical and written, of the types of soil and rock encountered within the borehole.
- Accommodates the presentation of select field and laboratory test results.
- Optimized for printing on 8.5" x 11" sheets
- Is accompanied by BR Legend Sheets.

5.3.2 Notes on the BR

If the procedures of this manual were followed without exception, then the following note shall appear on the first page of the BR:

"This Boring Record was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (Date)"

If an exception to the procedures of this manual has been approved and implemented, then the note shall be modified to read:

"This Boring Record was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (Date) except as noted in (Section) of (Report Title) dated (Date)"

Optional notes are left to the discretion of the geoprofessional and, if are specific to an elevation or depth, should be presented at the appropriate location in the "Remarks" column. These notes may include:

- Changes in drilling equipment
- Other drilling observations from Sections 2.4.22 or 2.5.17.

Notes that are more general in content, such as a site observation, should be placed within the body of the geotechnical report.

5.3.3 The Boring Record Legend Sheets

The soil and rock legend sheets are standard forms that provide convenient references for the *required* soil and rock description, identification, and/or classification components presented in this Manual. References for *optional* descriptors do not appear on the legend sheets; however, they are explained in this Manual. To correctly interpret the BR, the reader shall be familiar with this Manual.

There are three legend sheets: one predominantly for soil and the other for rock, as shown in the following figures.

The legend sheets define the format for the graphical presentation of a boring log and differentiate among the various borehole and sounding types. The legend sheets also present the symbols used to identify laboratory tests.

Figure 5-14

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GM	SILTY GRAVEL SILTY GRAVEL with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly graded SAND Poorly graded SAND with GRAVEL
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic / Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY elastic ELASTIC SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435-04)
CL	Collapse Potential (ASTM D 5333-03)
CP	Compaction Curve (CTM 216 - 06)
CR	Corrosion, Sulfates, Chlorides (CTM 643 - 99; CTM 417 - 06; CTM 422 - 06)
CU	Consolidated Undrained Triaxial (ASTM D 4767-02)
DS	Direct Shear (ASTM D 3080-04)
EI	Expansion Index (ASTM D 4829-03)
M	Moisture Content (ASTM D 2216-05)
OC	Organic Content (ASTM D 2974-07)
P	Permeability (CTM 220 - 05)
PA	Particle Size Analysis (ASTM D 422-63 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (AASHTO T 89-02, AASHTO T 90-00)
PL	Point Load Index (ASTM D 5731-05)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301 - 00)
SE	Sand Equivalent (CTM 217 - 99)
SG	Specific Gravity (AASHTO T 100-06)
SL	Shrinkage Limit (ASTM D 427-04)
SW	Swell Potential (ASTM D 4546-03)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166-06) Unconfined Compression - Rock (ASTM D 2938-95)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850-03)
UW	Unit Weight (ASTM D 4767-04)
VS	Vane Shear (AASHTO T 223-96 [2004])

SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT)
- Standard California Sampler
- Modified California Sampler
- Shelby Tube
- Piston Sampler
- NX Rock Core
- HQ Rock Core
- Bulk Sample
- Other (see remarks)

DRILLING METHOD SYMBOLS

- Auger Drilling
- Rotary Drilling
- Dynamic Cone or Hand Driven
- Diamond Core

WATER LEVEL SYMBOLS

- First Water Level Reading (during drilling)
- Static Water Level Reading (short-term)
- Static Water Level Reading (long-term)

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REPORT TITLE				
BORING RECORD LEGEND				
DIST. **12**	COUNTY **Orange**	ROUTE **I-405**	POSTMILE **R34.1/R39.2**	EA **12-047643**
PROJECT OR BRIDGE NAME **I-880 Realignment, Whitman Dr., O. C.**				
BRIDGE NUMBER **53-0045**	PREPARED BY	DATE	SHEET **1 of 3**	

Figure 5-15

CONSISTENCY OF COHESIVE SOILS				
Descriptor	**Unconfined Compressive Strength (tsf)**	**Pocket Penetrometer (tsf)**	**Torvane (tsf)**	**Field Approximation**
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	0.25 - 0.50	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	0.50 - 1.0	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	1.0 - 2.0	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	2.0 - 4.0	1.0 - 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS	
Descriptor	**SPT N_{60} - Value (blows / foot)**
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE	
Descriptor	**Criteria**
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS	
Descriptor	**Criteria**
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

SOIL PARTICLE SIZE		
Descriptor		**Size**
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS	
Descriptor	**Criteria**
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CEMENTATION	
Descriptor	**Criteria**
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

NOTE: This legend sheet provides descriptors and associated criteria for required soil description components only. Refer to Caltrans Soil and Rock Logging, Classification, and Presentation Manual (July 2007), Section 2, for tables of additional soil description components and discussion of soil description and identification.

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PROJECT OR BRIDGE NAME **I-880 Realignment, Whitman Dr., O. C.**					
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Figure 5-16

ROCK GRAPHIC SYMBOLS
IGNEOUS ROCK
SEDIMENTARY ROCK
METAMORPHIC ROCK

BEDDING SPACING

Descriptor	Thickness or Spacing
Massive	> 10 ft
Very thickly bedded	3 to 10 ft
Thickly bedded	1 to 3 ft
Moderately bedded	3-5/8 inches to 1 ft
Thinly bedded	1-1/4 to 3-5/8 inches
Very thinly bedded	3/8 inch to 1-1/4 inches
Laminated	< 3/8 inch

WEATHERING DESCRIPTORS FOR INTACT ROCK

	Diagnostic Features					
	Chemical Weathering-Discoloration-Oxidation		Mechanical Weathering and Grain Boundary Conditions	Texture and Solutioning		
Descriptor	Body of Rock	Fracture Surfaces		Texture	Solutioning	General Characteristics
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No solutioning	Hammer rings when crystalline rocks are struck.
Slightly Weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull	Minor to complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals may be noted	Hammer rings when crystalline rocks are struck. Body of rock not weakened.
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty"; feldspar crystals are "cloudy"	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck. Body of rock is slightly weakened.
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation (refer to grain boundary conditions)	All fracture surfaces are discolored or oxidized; surfaces are friable	Partial separation, rock is friable; in semi-arid conditions, granitics are disaggregated	Altered by chemical disintegration such as via hydration or argillation	Leaching of soluble minerals may be complete	Dull sound when struck with hammer; usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures or veinlets. Rock is significantly weakened.
Decomposed	Discolored of oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay		Complete separation of grain boundaries (disaggregated)	Resembles a soil; partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand. Resistant minerals such as quartz may be present as "stringers" or "dikes".

Note: Combination descriptors (such as "slightly weathered to fresh") are used where equal distribution of both weathering characteristics is present over significant intervals or where characteristics present are "in between" the diagnostic feature. However, combination descriptors should not be used where significant identifiable zones can be delineated. Only two adjacent descriptors shall be combined. "Very intensely weathered" is the combination descriptor for "decomposed to intensely weathered".

RELATIVE STRENGTH OF INTACT ROCK

Descriptor	Uniaxial Compressive Strength (psi)
Extremely Strong	> 30,000
Very Strong	14,500 - 30,000
Strong	7,000 - 14,500
Medium Strong	3,500 - 7,000
Weak	700 - 3,500
Very Weak	150 - 700
Extremely Weak	< 150

ROCK HARDNESS

Descriptor	Criteria
Extremely Hard	Specimen cannot be scratched with pocket knife or sharp pick; can only be chipped with repeated heavy hammer blows
Very hard	Specimen cannot be scratched with pocket knife or sharp pick; breaks with repeated heavy hammer blows
Hard	Specimen can be scratched with pocket knife or sharp pick with heavy pressure; heavy hammer blows required to break specimen
Moderately Hard	Specimen can be scratched with pocket knife or sharp pick with light or moderate pressure; breaks with moderate hammer blows
Moderately Soft	Specimen can be grooved 1/6 in. with pocket knife or sharp pick with moderate or heavy pressure; breaks with light hammer blow or heavy hand pressure
Soft	Specimen can be grooved or gouged with pocket knife or sharp pick with light pressure, breaks with light to moderate hand pressure
Very Soft	Specimen can be readily indented, grooved, or gouged with fingernail, or carved with pocket knife; breaks with light hand pressure

CORE RECOVERY CALCULATION (%)

$$\frac{\Sigma \text{ Length of the recovered core pieces (in.)}}{\text{Total length of core run (in.)}} \times 100$$

RQD CALCULATION (%)

$$\frac{\Sigma \text{ Length of intact core pieces > 4 in.}}{\text{Total length of core run (in.)}} \times 100$$

FRACTURE DENSITY

Descriptor	Criteria
Unfractured	No fractures
Very Slightly Fractured	Lengths greater 3 ft
Slightly Fractured	Lengths from 1 to 3 ft, few lengths outside that range
Moderately Fractured	Lengths mostly in range of 4 in. to 1 ft, with most lengths about 8 in.
Intensely Fractured	Lengths average from 1 in. to 4 in. with scattered fragmented intervals with lengths less than 4 in.
Very Intensely Fractured	Mostly chips and fragments with few scattered short core lengths

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PROJECT OR BRIDGE NAME: I-880 Realignment, Whitman Dr., O. C.				
BRIDGE NUMBER 53-0045	PREPARED BY	DATE	SHEET 3 of 3	

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Appendix A: Field Test Procedures

A.1 Pocket Penetrometer

The Pocket Penetrometer test is conducted using the following general instructions:

- To begin test, remove protective cap, push ring against body so that low side reads 0.
- Slowly insert piston until engraved mark is level with soil.
- Observe the reading in TSF (KG/SQ CM) using low side of ring, (side closest to the piston end): record reading and repeat.
- For weak soils, use 1" adapter foot, multiply reading by 0.0625.

A.2 Torvane

The Torvane test is conducted using the following general instructions:

- To start test, push indicator counter clockwise to zero stop.
- Select reasonably flat surface at least 1 inch in diameter.
- Using midsize vane, one revolution equals 1 TSF (1KG/SQ CM).
- One revolution using small and large vane equals respectively 2.5 and 0.2 TSF (KG/SQ CM).
- Press pocket vane shear tester into soil to depth of blade; maintain constant vertical pressure while turning knob clockwise at rate to develop failure within 5 to 10 seconds.
- After failure develops, release remaining spring tension slowly. Pointer will indicate maximum shear value until manually reset.

A.3 Dry Strength

From the specimen, select enough material to mold into a ball about 1 in. (25 mm) in diameter. Mold the material until it has the consistency of putty, adding water if necessary. From the molded material, make at least three test specimens. A test specimen shall be a ball of material about 1⁄2 in. (12 mm) in diameter. Allow the test specimens to dry in air, or sun, or by artificial means, as long as the temperature does not exceed 60°C. If the test specimen contains natural dry lumps, those that are about 1⁄2 in. (12 mm) in diameter may be used in place of the molded balls. Test the strength of the dry balls or lumps by crushing between the fingers. Note the strength as none, low, medium, high, or very high in accordance with the criteria in the table in Section 2.4.14. If natural dry lumps are used, do not use the results of any of the lumps that are found to contain particles of coarse sand. The presence of high-strength water-soluble cementing materials, such as calcium carbonate, may cause exceptionally high dry strengths. The presence of calcium carbonate can usually be detected from the intensity of the reaction with dilute hydrochloric acid.

A.4 Dilatancy

From the specimen, select enough material to mold into a ball about 1⁄2 in. (12 mm) in diameter. Mold the material, adding water if necessary, until it has a soft, but not sticky, consistency. Smooth the soil ball in the palm of one hand with the blade of a knife or small spatula. Shake horizontally, striking the side of the hand vigorously against the other hand several times. Note the reaction of water appearing on the surface of the soil. Squeeze the sample by closing the hand or pinching the soil between the fingers, and note the reaction as none, slow, or rapid in accordance with the criteria in the table in Section 2.4.15. The reaction is the speed with which water appears while shaking, and disappears while squeezing.

A.5 Toughness

Following the completion of the dilatancy test, the test specimen is shaped into an elongated pat and rolled by hand on a smooth surface or between the palms into a thread about 1⁄8 in. (3 mm) in diameter. (If the sample is too wet to roll easily, it should be spread into a thin layer and allowed to lose some water by evaporation.) Fold the sample threads and reroll repeatedly until the thread crumbles at a diameter of about 1⁄8 in. The thread will crumble at a diameter of 1⁄8 in. when the soil is near the plastic limit. Note the pressure required to roll the thread near the plastic limit. Also, note the strength of the thread. After the thread crumbles, the pieces should be lumped together and kneaded until the lump crumbles. Note the toughness of the material during kneading. Describe the toughness of the thread and lump as low, medium, or high in accordance with the criteria in the table in Section 2.4.16.

A.6 Jar Slake Index Test

Slaking behavior of intact rock specimens is quantified as an index. A laboratory index test called the Slake Durability Test (ASTM D 4644-04) is the most rigorous method of measuring this behavior. A simple, but less sensitive method can be employed in the field or in the office to screen specimens of the Slake Durability test. The "Jar Slake Test" method is presented here. A water filled jar and a watch are all that are required to perform this simple test. The steps are as follows:

- A fragment of rock is immersed in enough water to cover it by 15 mm. It is best if the rock is oven dried. It has been reported that damp material is relatively insensitive to degradation in this test when compared with dry material.
- After immersion, the fragment is observed continuously for the first 10 minutes and carefully during the first 30 minutes. When a reaction occurs, it is often during the first 30 minutes. A final observation is made after 24 hours.
- The condition of the piece is categorized (complete breakdown, partial breakdown, no change), as shown in the table in Section 2.5.14 (Air Force Manual 1983).

A.7 Calcium Carbonate

Because calcium carbonate is a common cementing agent, it is important to report its presence, which is done on the basis of the reaction with dilute hydrochloric acid (HCl). Use the ASTM D 2488-06 standard to describe the reaction with HCl, as indicated in Figure 2-19 below.

Figure 2-19
Descriptors for calcium carbonate reaction

Description	Criteria
None	No visible reaction.
Weak	Some reaction, with bubbles forming slowly.
Strong	Violent reaction, with bubbles forming immediately.

A.8 Standard Penetration Test

Standard Penetration Tests (SPT) shall be conducted according to the following test methods:

- ASTM D 1586-99, Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils
- ASTM D 6066-96, Standard Practice for Determining the Normalized Penetration Resistance of Sands for Evaluation of Liquefaction Potential

The following guidance is provided:

- Blow counts shall be reported on the LOTB and BR as observed in the field, N, with no corrections.

  Where,

  N The sum of the hammer blows required to drive the sampler over the test interval from 0.5 to 1.5 ft below the cleanout depth.

- Hammer efficiency shall be noted on the LOTB and BR to allow the reader to determine N_{60} or $(N_1)_{60}$.

  Where,

  ER_i Hammer energy ratio

N_{60} Penetration resistance adjusted to a 60 % drill rod energy ratio per ASTM D 6066-96: $N_{60} = N_{measured} \times (ER_i / 60)$

$(N_1)_{60}$ Penetration resistance adjusted for energy and normalized to a 1 ton/ft^2 stress level.

- Blow counts for each of the 6 inch increments shall be recorded in the field, but not necessarily reported on the LOTB and BR. The 2nd and 3rd driving intervals shall be summed and reported.

  For example:

  1st 6 in. interval: 10 blows
  2nd 6 in. interval: 15 blows
  3rd 6 in. interval: 18 blows
  N reported as "33"

- For partial increments, the depth of penetration shall be reported to the nearest 1 inch, in addition to the number of blows.

  For example:

  1st 6 in. interval: 20 blows
  2nd 6 in. interval: 40 blows
  3rd 6 in. interval: 60 blows for 2 inches, then refusal
  N reported as "100/8"

- If the seating interval (1st 6 in. interval) is not achieved, note refusal.

  For example:

  1st 6 in. interval: 50 blows for 2 inches, then refusal
  N reported as "REF"

- If a substantial change in material is encountered over the course of driving the sampler, the 2nd and 3rd driving intervals can be reported separately.

  For example:

  1st 6 in. interval: 10 blows
  2nd 6 in. interval: 20 blows
  3rd 6 in. interval: 60 blows for 3 inches, then refusal
  N reported as "20/6, 60/3"

A.9 Core Recovery (REC)

The core recovery value (REC), with few exceptions, provides an indication of the success of the coring operation in recovering the cored rock. Portions of the cored rock mass may not be recovered because the fluid used in the drilling operations transports portions of the rock mass during the coring operation or the rotation of the core barrel traps and grinds away portions of the rock mass. Diminished core recovery can also be attributed to voids within the rock mass. Core recovery is expressed as a percentage.

$$\text{REC} = \frac{\Sigma\,(\text{Length of the recovered core pieces, inches})(100\%)}{\text{Total length of the core run, inches}}$$

A.10 Rock Quality Designation (RQD)

Rock Quality Designation is an index that relates to the degree of fracturing in a rock mass as observed in a core specimen. A high value of RQD is indicative of a less fractured rock mass or a rock mass having widely spaced fractures. RQD is valid for core diameters from 1.4 to 3.35 inches. This RQD criteria is generally based on ASTM D 6032-02.

$$\text{RQD} = \frac{\Sigma\,(\text{Length of intact core pieces} \geq 4 \text{ inches})(100\%)}{\text{Total length of the core run, inches}}$$

Used alone, RQD is not sufficient to provide an adequate description of rock mass quality. The RQD does not account for joint orientation, tightness, continuity, and gouge material. The RQD shall be used in combination with other geological and geotechnical input.

The RQD denotes the percentage of intact rock retrieved from a borehole of any orientation. All pieces of intact rock core equal to or greater than 4 inches long are summed and divided by the total length of the core run. An intact core is any segment of core between two open, natural discontinuities.

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Appendix B: Field Logging Aids

B.1 Field Sample Logging Forms

Forms are provided to assist the sample logging process in the field. One form is used specifically for soil samples, while the other used for rock samples.

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Caltrans

Soil Sample Field Description & Identification

Instructions:

- Field Description and Identification based upon *Soil and Rock Logging, Classification, and Presentation Manual*, dated July 1, 2007.
- Shaded fields are *required*, non-shaded fields are *optional*.
- Intensely weathered or decomposed rock that is friable and that can be reduced to gravel size or smaller by normal hand pressure shall be identified and described as rock followed by the soil identification or classification, and description in parenthesis.

Project	**Logged by**	**Date**	**Hole ID**

District	**County**	**Route**	**Postmile**	**EA**	**Sample Depth**	**Sample ID**

Drilling Method	**Sampler Type and Size**

SPT Blow Counts		**Hammer energy ratio (ER_i)**	**Pocket Penetrometer Measurement**
1st 6 in. interval			
2nd 6 in. interval	$N_{measured}$ =	$N_{60} = N_{measured} \times (ER_i/60)$	**Torvane Measurement**
3rd 6 in. interval			

Group Name	**Group Symbol**	**Color**

Consistency
☐Very Soft
☐Soft
☐Medium Stiff
☐Stiff
☐Very Stiff
☐Hard

Apparent Density
☐Very loose
☐Loose
☐Medium dense
☐Dense
☐Very dense

Moisture
☐Dry
☐Moist
☐Wet

Structure
☐Stratified
☐Laminated
☐Fissured
☐Slickensided
☐Blocky
☐Lensed
☐Homogeneous

Cementation
☐Weak
☐Moderate
☐Strong

Additional Description

Percent (est.) 100% Total by Volume of entire sample

Boulders — %

Rock Identification

Intersected Lengths of Core

Weathering
☐Fresh
☐Slightly Weathered to Fresh
☐Slightly Weathered
☐Moderately to Slightly Weathered
☐Moderately Weathered
☐Intensely to Moderately Weathered
☐Intensely Weathered
☐Very Intensely Weathered
☐Decomposed

Rock Hardness
☐Extremely Hard
☐Very Hard
☐Hard
☐Moderately Hard
☐Moderately Soft
☐Soft
☐Very Soft

Angularity
☐Angular
☐Subangular
☐Subrounded
☐Rounded

Shape
☐Flat
☐Elongated
☐Flat and Elongated

Cobbles — %

Rock Identification

Intersected Lengths of Core

Weathering
☐Fresh
☐Slightly Weathered to Fresh
☐Slightly Weathered
☐Moderately to Slightly Weathered
☐Moderately Weathered
☐Intensely to Moderately Weathered
☐Intensely Weathered
☐Very Intensely Weathered
☐Decomposed

Rock Hardness
☐Extremely Hard
☐Very Hard
☐Hard
☐Moderately Hard
☐Moderately Soft
☐Soft
☐Very Soft

Angularity
☐Angular
☐Subangular
☐Subrounded
☐Rounded

Shape
☐Flat
☐Elongated
☐Flat and Elongated

Percent (est.) 100% Total by Weight of soils portion

Gravel — %

(or) **Proportion**
☐Trace
☐Few
☐Little
☐Some
☐Mostly

Size
☐Coarse
☐Fine

Angularity
☐Angular
☐Subangular
☐Subrounded
☐Rounded

Shape
☐Flat
☐Elongated
☐Flat and Elongated

Sand — %

(or) **Proportion**
☐Trace
☐Few
☐Little
☐Some
☐Mostly

Size
☐Coarse
☐Medium
☐Fine

Angularity
(for coarse only)
☐Angular
☐Subangular
☐Subrounded
☐Rounded

Fines — %

(or) **Proportion**
☐Trace
☐Few
☐Little
☐Some
☐Mostly

Plasticity
☐Nonplastic
☐Low
☐Medium
☐High

Dry Strength
☐None
☐Low
☐Medium
☐High
☐Very High

Dilatency
☐None
☐Slow
☐Rapid

Toughness
☐Low
☐Medium
☐High

Rock Sample Field Description & Identification

Instructions:

- Field Description and Identification based upon *Soil and Rock Logging, Classification, and Presentation Manual*, dated July 1, 2007.
- Shaded fields are *required*, non-shaded fields are *optional*.
- Intensely weathered or decomposed rock that is friable and that can be reduced to gravel size or smaller by normal hand pressure shall be identified and described as rock followed by the soil identification or classification, and description in parenthesis.

Project				**Logged by**	**Date**	**Hole ID**
District	**County**	**Route**	**Postmile**	**EA**	**Sample Depth**	**Sample ID**

Drilling or Coring Method	**Sampler Type and Size**

Length of Core Run	**Length of the recovered core pieces**	**Length of intact core pieces > 4 inches**
Recovery (REC)	**Rock Quality Designation (RQD)**	

Rock Identification	
Color	**Odor**
Additional Description	

Family Name
- ☐Sedimentary
- ☐Igneous
- ☐Metamorphic

Bedding Spacing
- ☐Massive
- ☐Very thickly bedded
- ☐Thickly bedded
- ☐Moderately bedded
- ☐Thinly bedded
- ☐Very thinly bedded
- ☐Laminated

Texture
- ☐Pitted
- ☐Vuggy
- ☐Cavity
- ☐Honeycombed
- ☐Vesicular

Rock Grain-Size

(Crystalline Igneous and Metamorphic Rock)
- ☐Very coarse grained or pegmatitic
- ☐Coarse-grained
- ☐Medium-grained
- ☐Fine-grained
- ☐Aphanitic

(Sedimentary Rock)
- ☐Boulder, Boulder Conglomerate
- ☐Cobble, Cobble Conglomerate
- ☐Pebble, Pebble Conglomerate
- ☐Granule, Granule Conglomerate
- ☐Very Coarse Sand
- ☐Coarse Sand
- ☐Medium Sand
- ☐Fine Sand
- ☐Very Fine Sand
- ☐Silt, Siltstone, Shale
- ☐Clay, Claystone, Shale

(Pyroclastic Igneous Rock)
- ☐Block (Angular), Volcanic Breccia
- ☐Bomb (Rounded), Agglomerate
- ☐Lapilli, Lapilli Tuff
- ☐Coarse Ash, Coarse Tuff
- ☐Fine Ash, Fine Tuff

Weathering
- ☐Fresh
- ☐Slightly Weathered to Fresh
- ☐Slightly Weathered
- ☐Moderately to Slightly Weathered
- ☐Moderately Weathered
- ☐Intensely to Moderately Weathered

(Requires completion of *Soil Sample Field Description and Identification* form)
- ☐Intensely Weathered
- ☐Very Intensely Weathered
- ☐Decomposed

Rock Hardness
- ☐Extremely Hard
- ☐Very Hard
- ☐Hard
- ☐Moderately Hard
- ☐Moderately Soft
- ☐Soft
- ☐Very Soft

Fracture Density
- ☐Unfractured
- ☐Very slightly fractured
- ☐Slightly to very slightly fractured
- ☐Slightly fractured
- ☐Moderately to slightly fractured
- ☐Moderately fractured
- ☐Intensely to moderately fractured
- ☐Intensely fractured
- ☐Very intensely to intensely fractured
- ☐Very intensely fractured

Discontinuity Type
- ☐Joint (JT)
- ☐Foliation Joint (FJ) or Bedding Joint (BJ)
- ☐Bedding Plane Separation
- ☐Incipient Joint (IJ) or Incipient Fracture (IF)
- ☐Random Fracture (RF)
- ☐Mechanical Break (MB)
- ☐Fracture Zone (FZ)

Discontinuity Dip Magnitude

Jar Slake Index, I_j
- ☐1
- ☐2
- ☐3
- ☐4
- ☐5
- ☐6

Discontinuity Weathering
- ☐Fresh
- ☐Slightly Weathered to Fresh
- ☐Slightly Weathered
- ☐Moderately to Slightly Weathered
- ☐Moderately Weathered
- ☐Intensely to Moderately Weathered
- ☐Intensely Weathered
- ☐Very Intensely Weathered
- ☐Decomposed

Discontinuity Healing
- ☐Totally Healed
- ☐Moderately Healed
- ☐Partially Healed
- ☐Not Healed

Healing Material:

Discontinuity Infilling

Rock:
- ☐Extremely Hard
- ☐Very Hard
- ☐Hard
- ☐Moderately Hard
- ☐Moderately Soft
- ☐Soft
- ☐Very Soft

Soils:
- ☐Very Soft
- ☐Soft
- ☐Medium Stiff
- ☐Stiff
- ☐Very Stiff
- ☐Hard

Appendix C:
Procedural Documents

State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

M e m o r a n d u m

Flex your power!
Be energy efficient!

To: ALL STAFF
GEOTECHNICAL SERVICES
DIVISION OF ENGINEERING SERVICES

Date: June 15, 2007

From: JAMES E. DAVIS
Deputy Division Chief
Geotechnical Services

Subject: Exception Process for the Soil & Rock Logging, Classification, and Presentation Manual

All Geotechnical Services staff, including consultants performing work on behalf of the Department, shall follow the procedures in the Soil & Rock Logging, Classification, and Presentation Manual (ref: memo to all staff dated June 15, 2007). Although the manual attempts to provide for most geotechnical conditions, there may be reason to deviate from its procedures or terminology. This memorandum provides the procedure that staff shall follow in order to gain approval for such deviations.

The policy terminology used in the manual is defined as follows:

Term	Standard Type	Definition
Shall, Required	Mandatory	*Mandatory Standard*. The associated provisions **must** be used. There is no acceptable alternative.
Should	Advisory	*Advisory Standard*. The associated provisions are preferred practices.
May, Optional	Permissive	*Permissive Standard.* Use or application of the associated provisions is left to the discretion of the Geoprofessional.

In cases where exceptions to mandatory or advisory standards are proposed by the geotechnical professional, a *Request for Exception* form shall be completed and signatures retained prior to finalizing or issuing any related document, such as a Log of Test Boring or Geotechnical Report. The completed *Request for Exception* form shall be placed in the project file and any related geotechnical report shall discuss the exception. When a LOTB or Boring Record presents information that deviates from mandatory or advisory standards, the standard note:

"This LOTB sheet (Boring Record) was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (date)"

Shall be modified to read as follows:

"This LOTB sheet (Boring Record) was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (date) except as noted in (section) of (Report Title) dated (Date)"

Attachments

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Soil & Rock Logging, Classification, and Presentation Manual
Request for Exception

Name:
Office:
District-EA: -
Project Name:
Manual Section No.:

Description of Exception Being Requested:

Discussion of Why the Exception to Policy is Necessary:

Should the Manual be revised to allow for the exception? Please discuss.

____________________	____________________
Branch Chief	Office Chief (Advisory & Mandatory Standards)
____________________	____________________
Deputy Division Chief (Mandatory Standards)	Date Approved

cc: Project File, GS Corporate

Caltrans Soil & Rock Logging, Classification, and Presentation Manual

Committee Charter and Standard Procedures (June 2007)

Committee Members:	Tom Whitman, Chairperson David Jang Bob Price Hector Valencia Mark Hagy Craig Hannenian Loren Turner (DRI)
Committee Sponsor:	Tim Pokrywka

Purpose of Committee

The Soil & Rock Logging, Classification, and Presentation Manual Committee, formed by the Geotechnical Services Management Team (GSMT), shall maintain the manual as follows.

- The manual shall be kept current with respect to the state of practice of all standards and procedures presented in the manual.

- The committee shall consider all requests for modifications to the manual and respond to those requests in a timely manner.

Annual Review

The committee shall perform an annual review of the manual due to the GSMT in June of each year. The review shall consist of the following:

- Review of the manual's references and considerations of changes to reflect updated references, if any.
- Review of approved *"Request for Exception"* forms and consideration for revision to the manual to accommodate approved exceptions.
- Solicitation of comments by our partners such as the Association of Drilled Shaft Contractors (ADSC), Consultants and Construction and consideration of their issues.

Standard Procedure for Modifying the Manual

1. Staff shall post requests for modifications on the *Caltrans Geotechnical Services Discussion Board, Soil & Rock Logging, Classification and Presentation Manual* Category located at:

   http://cap1.dot.ca.gov/forum/GeotechnicalServices/index.php

   The request for modification shall include at least the following information:
   - Name of requestor
   - Office
   - Manual Section Number
   - Description of Proposed Change
   - Discussion of Reason or Need for the Proposed Change
   - If applicable, reference to an approved *Request for Exception* relating to the Proposed Change

2. The committee chairperson will monitor the discussion board, review the proposal and assign any additional research to an appropriate person. The person will typically be the Committee Member representing the requesting office, but may be others depending on the proposal topic.
3. If the proposal is minor, the chairperson may implement the revision without input from the committee. Otherwise, depending on the proposal's complexity, the chairperson will either schedule a committee meeting, solicit comments on the discussion board, or ask for each member's recommendation on whether to implement the proposed change or not.
4. Acceptability of proposed changes will be by majority vote of the committee members present at the meeting, or by those who respond to the chairperson by the stated deadline.
5. Unless deemed urgent by the committee chairperson, revisions will be made annually and coincide with the annual report to the GSMT. An exception to this shall be during the first 12 months after initial release where the committee will be expected to address comments on an ongoing basis.

Committee Member Roles and Responsibilities

Committee members pledge to actively participate in maintenance of the manual according to this committee charter. It is expected that each committee member respond to correspondence in a timely manner so not to delay progress of related business.

Member Signatures

Thomas Whitman

David Jang

Robert Price

Hector Valencia

Mark Hagy

Craig Hannenian

Loren Turner

APPENDIX

B Contract Administration

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State of California
Agency

Business, Transportation and Housing

Memorandum

To: ALL CONSTRUCTION ENGINEERS
ALL RESIDENT ENGINEERS
ALL BRIDGE CONSTRUCTION ENGINEERS
ALL STRUCTURE REPRESENTATIVES

Date: May 23, 2000

File: Construction Program
Directive CPD-00-5

From: **DEPARTMENT OF TRANSPORTATION**
CONSTRUCTION PROGRAM
MS 44

Subject: Clarifications to Differing Site Conditions Specifications

Revisions are being proposed to specifications regarding Differing Site Conditions (DSC) to clarify the administrative procedures required for analyzing a notice of a DSC by a Contractor.

Section 5-1.116, "Differing Site Conditions," of the Standard Specifications requires the Engineer to determine whether a Differing Site Condition exists. The current specifications define a differing site when subsurface or latent conditions either differ "materially from those indicated in the contract…", also referred to as a Type 1, or differ "materially from those ordinarily encountered and generally recognized as inherent in the work provided for in the contract…", also referred to as a Type 2.

When Contractors provide notice of a DSC, Contractors frequently assert that the existing conditions differ from what is indicated on the project plans, specifically the Log of Test Borings (LOTB). It is not uncommon for the districts to respond by referencing Section 2-1.03, "Examination of Plans, Specifications, Contract, and Site Work," of the Standard Specifications, and to inform the Contractor that the LOTB is not part of the project plans, therefore finding no merit in the notice. While it is correct that the LOTB is not part of the project plans, it does not prohibit the use of the LOTB to derive information for the determination of a DSC. The LOTB should not be the sole basis for making such a determination. If other pertinent information is available, such as materials reports and above ground site investigations, that information should also be used in the determination.

A revision is proposed to Section 5-1.116 that will clarify and allow both parties to consider all available information to assist in determining if a Type 1 DSC exists. The specification will be revised to say, "…differing materially from those indicated in the contract, ***the log of test borings or other record of geotechnical data obtained by the Department's investigation of subsurface conditions, the 'Materials Information,' or an examination of the conditions above ground at the site…***". Certain paragraphs of Section 2-1.03 affected by this revision have also been amended.

The following instructions are issued to Resident Engineers and Structure Representatives:

Construction Engineers
Resident Engineers
Bridge Construction Engineers
Structure Representatives
May 23, 2000
Page 2

1. The Resident Engineer or Structure Representative will notify the appropriate materials staff (district or ESC, Division of Structures Foundations) both prior to the start of the work and when the Contractor has filed a DSC notice.

2. In reviewing a Contractor's DSC dispute, Resident Engineers and Structure Representatives shall take into consideration all information available to the Contractor, including the log of test borings or other record of geotechnical data, other materials information, and information from above ground site conditions, and shall not reject the Contractor's Notice of Differing Site Condition on the basis that the LOTB is not a part of the contract.

3. Future contracts will contain the revised specifications to Paragraphs 2, 5, and 6 of Section 2-1.03 and Paragraph 1 of Section 5-1.116.

4. Resident Engineers and Structure Representatives shall act to notify the Contractor of Differing Site Conditions in favor of the State, in accordance with the revised specifications, when the conditions encountered are warranted.

Questions regarding this memorandum may be directed to Peter Vacura of the Construction Program at (916) 654-2593, or David Keim of the Division of Structure Construction at (916) 227-8814.

____
ROBERT PIEPLOW
Acting Program Manager
Construction

____
RALPH SOMMARIVA, Chief
Division of Structure Construction
Engineering Service Center

c: District Division Chiefs, Construction
Area Construction Managers, Structures

State of California | Business, Transportation and Housing Agency

Memorandum

To: DEPUTY DISTRICT DIRECTORS, Construction
AREA BRIDGE CONSTRUCTION MANAGERS
SENIOR CONSTRUCTION ENGINEERS
RESIDENT ENGINEERS
STRUCTURE REPRESENTATIVES

Date: February 21, 2002

File: Division of Construction
CPD 01-12

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF CONSTRUCTION - MS 44

Subject: Review of Differing Site Condition Disputes by a Management Review Committee

The purpose of this memorandum is to describe a revised procedure for the resolution of differing site condition (DSC) disputes. This procedure shall become effective immediately.

DSC disputes can be particularly complex, difficult to analyze, and require the consideration of various sources of information. In addition, DSC disputes often occur during the subsurface work performed early in a project, and, therefore, can be protracted disputes that are costly to the Department of Transportation (Department) when not resolved early. For these and other reasons, DSC disputes are a relatively common dispute during both the claim administration and arbitration phases of dispute resolution.

During the progress of the work, if conditions are encountered at the site that are believed to meet the standards of a DSC, then the party discovering those conditions shall promptly notify the other party in writing of the specific differing conditions before they are disturbed. Upon written notification from the contractor, the resident engineer will investigate the conditions and determine if the conditions materially differ from the work under the contract and if an adjustment to the contract will be made. If the contractor does not agree with the resident engineer's determination and would like to pursue the issue further, Standard Special Provision 5-1.012, "Differing Site Conditions," requires the contractor to file a notice of potential claim (NOPC) within 15 days from the resident engineer's determination; otherwise the decision of the resident engineer shall be deemed to have been accepted by the contractor as correct.

In order to resolve DSC disputes earlier and to clarify the Department's position on the dispute, the attached process has been developed. This process will take place after the contractor files an NOPC regarding a DSC and will involve a management review committee early in the potential claim process. The management review committee will consist of the deputy district director, construction (chairperson), the structure construction area manager, and the construction coordinator. The process will consist of three primary steps:

1. Upon receipt of an NOPC pertaining to a DSC dispute, the resident engineer prepares a draft response to the NOPC and submits the response to the deputy district director, construction within five days.

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DEPUTY DISTRICT DIRECTORS, Construction, et al
February 21, 2002
Page 2

2. The management review committee will review the resident engineer's draft response and provide any comments to the resident engineer within 10 days. If necessary to clarify information related to the DSC dispute, the management review committee or the contractor can initiate further communication with the other party during this period.

3. The resident engineer will incorporate any response from the management review committee into the NOPC response and submit the response to the contractor within five days.

For on-going contracts with the dispute review board (DRB) specification, the timeframe described above must be shortened from a total of 20 days to 15 days. The DRB specification will be revised to allow the resident engineer 20 days to respond in writing to the contractor's NOPC.

In addition to the personnel referenced above, other positions could be involved in the process of elevating DSC disputes by fulfilling their normal responsibilities in administering the contract, including construction engineers, construction managers, claims engineers and Engineering Services personnel. The construction engineer, as the supervisor of the resident engineer who must approve or concur with any contract change order, should be involved in the analysis of the DSC from the earliest stages. The construction manager should ensure adequate resources and training have been provided to successfully analyze and administer DSC disputes, and could also provide direct support and assistance to the construction engineer. The claims engineer could provide expertise in dispute resolution processes and in analyzing the DSC dispute for merit and cost. Finally, Engineering Services personnel could provide technical expertise in areas such as structure design and construction, materials engineering and testing, and structural foundations. For the resident engineer to consistently provide a response to the NOPC within 20 days, the appropriate personnel should be involved early in the process, including when the resident engineer is preparing a response to the contractor's initial DSC notice.

If you have any questions regarding this process, please call Scott Jarvis, Chief, Office of Contract Management at (916) 651-6284.

Original Signed by:

ROBERT PIEPLOW
Chief
Division of Construction

Attachment

c: SJarvis

bc: RPieplow; JMcMillan; RSheena

SJ:sf

"Caltrans improves mobility across California"

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: DEPUTY DISTRICT DIRECTORS, Construction
CONSTRUCTION MANAGERS
SENIOR CONSTRUCTION ENGINEERS
RESIDENT ENGINEERS
DOLORES VALLS, Deputy Division Chief,
Structure Construction

Date: May 10, 2004

File: Division of Construction
CPD 04-6

From: ROBERT PIEPLOW
Chief
DIVISION OF CONSTRUCTION

Subject: Application of the Tunnel Safety Orders

The California Department of Transportation (Department) is required to obtain tunnel classifications from the California Department of Industrial Relations, Division of Occupational Safety and Health (Cal/OSHA), Mining and Tunneling unit before bid advertisement for all operations covered by Sections 8400 through 8469, "Tunnel Safety Orders," of the *California Code of Regulations*. The intent of the tunnel classification is to inform the contractor of the potential hazards and safety precautions required.

The Tunnel Safety Orders are available via the internet at the Department of Industrial Relations web site:

http://www.dir.ca.gov/Title8/sub20.html

The Department has recently received clarification from the Cal/OSHA Mining and Tunneling unit citing which types of operations are applicable to the Tunnel Safety Orders. The Tunnel Safety Orders are applicable only when human entry will occur during a construction operation at any of the following locations:

- Tunnels: Culverts greater than 760 mm (30 inches) in diameter;
- Shafts: Excavations where the depth, (a) is at least twice the greatest cross-sectional dimension, and (b) exceeds 6.1 m (20 feet);
- Raises: Vertical or inclined underground excavation driven from bottom to top;
- Underground chambers and premises appurtenant thereto;
- Boring and pipe-jacking operations 760 mm (30 inches) in diameter or greater.

Some of the common types of activities where human entry is likely and often require a tunnel classification include:

- Pipe jacking or boring operations

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DEPUTY DISTRICT DIRECTORS, et al
May 10, 2004
Page 2

- Culvert rehabilitation
- Large diameter pile construction
- Pump house vaults
- Cut-and-cover operations connecting to an existing underground installation
- Well construction
- Deep structure footings/shafts
- Cofferdam excavations
- Waste slab construction

Review both on-going and already advertised projects for operations that may require a tunnel classification. Issue contract change orders on all projects where a tunnel classification is required and a tunnel classification was not included in the special provisions. Issue the change order as a "change in character of work" in accordance with Section 4-1.03C, "Changes in Character of Work," of the *Standard Specifications*. If a tunnel classification has not been obtained and the contractor is currently performing an operation where the Tunnel Safety Orders are applicable, halt operations until the Cal/OSHA Mining and Tunneling unit is contacted and a tunnel classification is obtained. Processing time for a tunnel classification can take up to two weeks so contact should be made as quickly as possible. Section 8422, "Tunnel Classifications," of the *California Code of Regulations* lists the information that must be submitted to the local Cal-OSHA Mining and Tunneling unit. A listing of the Cal/OSHA Mining and Tunneling unit offices is attached.

The contractor may be entitled to compensation for compliance with the requirements of the Tunnel Safety Orders only where the requirements exceed the requirements of the *Construction Safety Orders*. The *Construction Safety Orders* are available via the internet at the Department of Industrial Relations web site:

http://www.dir.ca.gov/Title8/sub4.html

If you have any questions regarding this construction procedure directive, please contact Greg Berry, Safety Coordinator, Division of Construction, at (916) 654-4580.

Attachments: Mining and Tunneling unit offices

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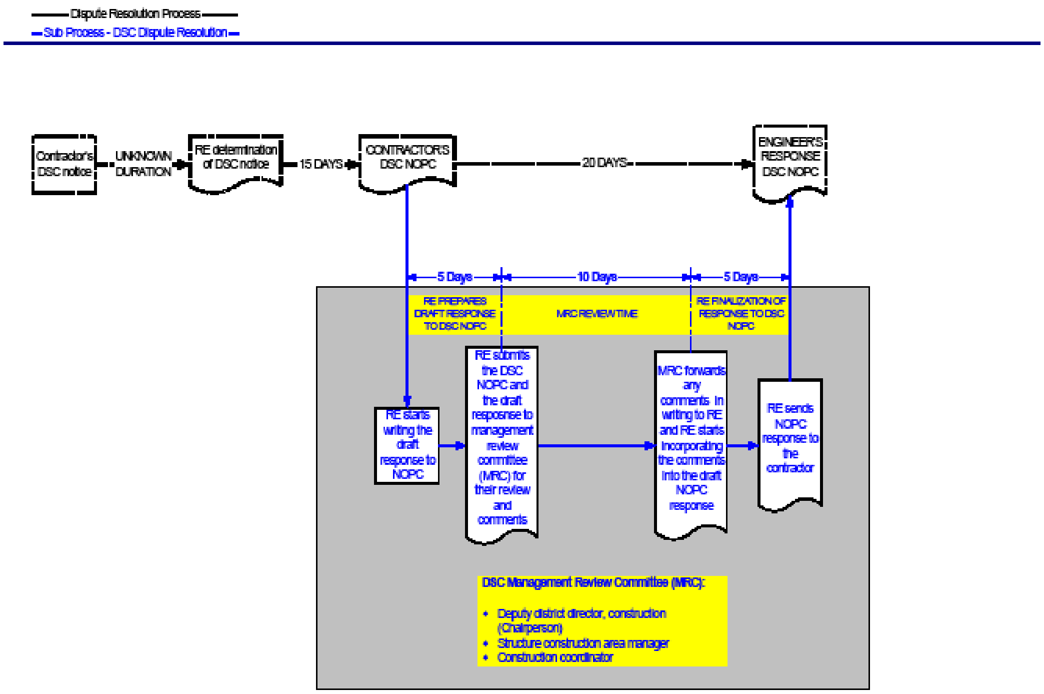
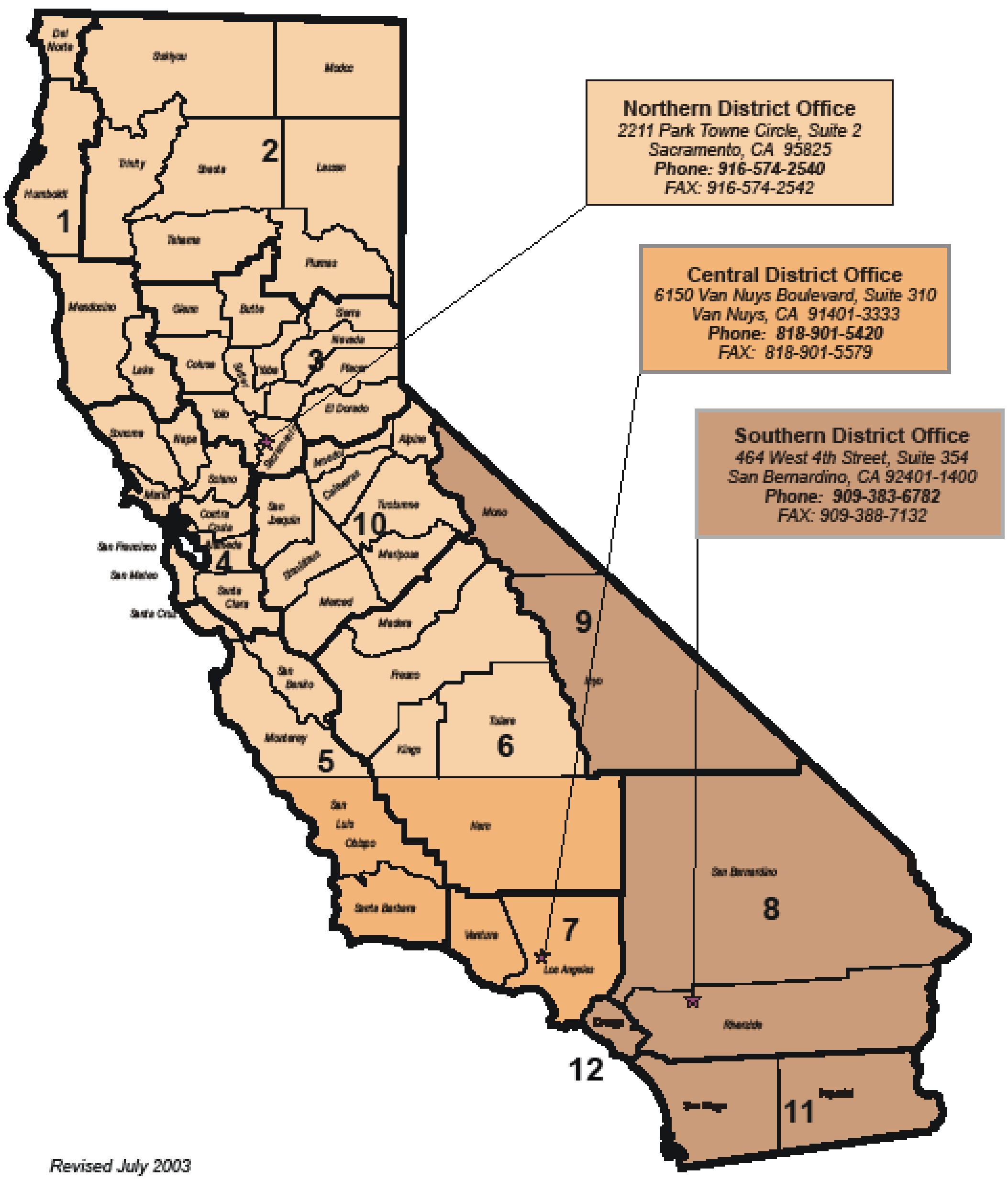
DSC disputes review by Management Review Committee (MRC)



California Mining and Tunneling Districts



APPENDIX

C Footing Foundations

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Please note that these conversion tables are approximate. They can be used by characterizing the soil as being either predominately granular or cohsesive. If possible, the conversion of the Penetration Index (N value) should be checked by using is-situ or laboratory tests.

GRANULAR SOILS

COMPACTNESS	VERY LOOSE		LOOSE		MEDIUM		DENSE		VERY DENSE
Relative Density, D_d		15%		35%		65%		85%	
Standard Penetration Resistance, N = Blows/ft*		4		10		30		50	
Angle of Internal Friction, ϕ		28		30		36		41	
Unit Weight (PCF) Moist	100		95-125		110-130		110-140		130+
Submerged	60		55-65		60-70		65-85		75+

VERY LOOSE: A reinforcing rod can be pushed into soil several feet.
DENSE: Difficult to drive a 2x4 stake with a sledge hammer.

* N = Blows/Ft as measured by the standard penetration test (See Appendix B).

$$\text{Relative Density, } D_d = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

e = existing void ratio of mass being considered.
e_{max} = void ratio of same mass in its loosest state.
e_{min} = void ratio of same mass in its most compact state.

COHESIVE SOILS

CONSISTENCY	VERY SOFT		SOFT		MEDIUM		STIFF		VERY STIFF		HARD
q_u = unconfined comp. strength (PSF)		500		1000		2000		4000		8000	
Standard Penetration Resistance, N = Blows/Ft *		2		4		8		16		32	
Unit Weight (PCF) Saturated	100-120				110-130				120-140		130+

VERY SOFT: Exudes from between fingers when squeezed in hand.
SOFT: Molded by light finger pressure.
MEDIUM: Molded by strong finger pressure.
STIFF: Indent by thumb.
VERY STIFF: Indent by thumb nail.
HARD: Difficult to indent by thumb nail.

* N = Blows/Ft as measured by the standard penetration test (See Appendix B).

To be used only as a rough guide.

STATE OF CALIFORNIA -- BUSINESS, TRANSPORTATION, AND HOUSING AGENCY PETE WILSON, Gov

DEPARTMENT OF TRANSPORTATION
Nevada City Construction Office
P. O. Box 691
Nevada City, CA 95959

September 10, 1991

03-NEV-49-21.9
03-295604 F-P049(95)
S. Yuba River Br.

David A. Mowat Company
Highway 49
Nevada City, CA

Gentlemen:

This letter is to clear up any possible misunderstanding about field revision of the elevation of spread footings. You are reminded that Section 51-1.03 of the Standard Specifications states that "the elevations of the bottoms of footings shown on the plans shall be considered as approximate only..."

The Engineer will establish final footing elevations at the earliest time possible consistent with the progress of the work, and that you will be informed in writing of the Engineer's decision.

You are reminded that should you elect to do any work or order any materials before receiving the Engineer's decision regarding spread footing elevations, you do so at your own risk and assume the responsibility for the cost of alterations to such work or materials in the event that revisions are required.

If you have any questions about this or any other matter, please call me at (916) 265-9413.

Sincerely,

John Rodrigues
Resident Engineer
David R. Keim
by David R. Keim
Structures Representative

cc: OSC
03 Const
DKDefoe
File c:\wp50\pr3\letters\09-10-91.1

STATE OF CALIFORNIA-BUSINESS, TRANSPORTATION AND HOUSING AGENCY

California Test 112
March 1998

DEPARTMENT OF TRANSPORTATION
ENGINEERING SERVICE CENTER
Transportation Laboratory
P.O. Box 19128
Sacramento, California 95819

METHOD FOR INSTALLATION AND USE OF EMBANKMENT SETTLEMENT DEVICES

This 19-page document is available on the DES-Materials Engineering and Services website:
http://www.dot.ca.gov/hq/esc/ctms/pdf/CT_112.pdf

OVERVIEW

The installation, maintenance, and data collection procedures for the various embankment settlement devices used to monitor subsurface settlement are described in this method. Analysis of the settlement data is included as a separate part of this method.

Settlement devices are used to monitor the rate and magnitude of settlement occurring at a point within or beneath an embankment during and subsequent to construction. The data obtained from these devices are used to determine the allowable loading rate during embankment construction and the appropriate time for removal of surcharge and/or commencement of permanent structure construction.

This method is divided into the following parts:

1. Fluid Level Settlement Devices
2. Pipe Riser Settlement Device
3. Settlement Data Analysis
4. Safety and Health

The fluid level, vented standpipe unit may be used at most locations. A sealed standpipe unit must be installed at locations where groundwater may interfere with the operation of the unit or where excess pore water is expected from the use of dredged material or wet soil in embankment construction. Where it is possible, the tube length between standpipe and indicator unit should generally be limited to a maximum of 100 m. Installations over longer distances can be made but are not advisable under normal circumstances, since it may result in inconsistent test data. Factors such as larger size tubes, change of platform location, or changes in elevation of the water line may have to be considered (see NOTE).

NOTE: There may be job conditions with respect to terrain, long tube length between standpipe and indicator unit, or anticipated large settlements that require special installations.

The pipe riser settlement device is used for monitoring fill settlement over soft foundation soils where the fluid level settlement devices are not feasible because of flat terrain, width of embankment construction, or other features which would make installation of a fluid level type of settlement platform undesirable. The pipe riser settlement device is a direct-reading unit which is exposed for the full duration of fill construction and surcharge removal. Because of the vulnerability of this unit to damage by the contractor's operations, the

- 1 -

pipe riser settlement device should be used only on those projects where the fluid level type of settlement device would be impractical.

PART 1. FLUID LEVEL SETTLEMENT DEVICES

SECTION 1 - VENTED STANDPIPE UNIT

A. APPARATUS

1. Vented standpipe unit (Figure 1)

2. Indicator unit (Figure 2)

3. Polyethylene tubing, 10-mm inside diameter

4. Hand tools - shovel, bar, posthold auger, hammer, adjustable wrenches, etc.

5. Water container (approximately 4-L capacity)

B. INSTALLATION

1. Select a location for the standpipe unit on the ground after approximately 0.3 m of fill has been placed above original ground and generally within the area where the maximum height of embankment will be placed. See Figure 4.

2. Select a point outside of the toe of the proposed embankment for the indicator unit. See Figure 4. Select this location so that sufficient vertical distance will be available for lowering the indicator unit as the standpipe unit settles. A hand level may be used to estimate the desired elevations for the indicator unit.

3. Because of terrain, excessive anticipated settlement, or other causes, it may be necessary to place the standpipe unit in the embankment at varying elevations above the original ground. In these cases, record the vertical distance between the base of the standpipe unit and original ground to allow proper consideration for embankment compression in the settlement analysis.

4. After embankment has been placed 1.0 to 1.5 m above the desired elevation for the standpipe unit, prepare a pit and trench in the embankment for the standpipe unit and tubing. See Figure 4. The bottom of the pit should normally be about 0.3 m above original ground. The trench should be cut to the same depth at the pit and should have a slight downward slope to the indicator unit location. Make sure that the trench is clear of any future construction, such as pile driving, ripping, ditching, etc.

5. Upon completion of the excavation, remove all rocks and large clods from the trench. Prepare a smooth, level area in the pit using fine embankment material.

6. Assemble the standpipe unit as shown in Figure 1. *Do not attach the pipe cap*. Firmly seat the standpipe unit on a prepared level area.

7. Install the indicator unit post at the previously selected point for the indicator unit. This post can be either a metal sign post or 4-by-4.

8. Using a hand level, attach the indicator unit to the post so that the 0.7-m graduation on the indicator unit scale is approximately level with the top of the spill tube on the standpipe unit.

9. Push the 10-mm water line through the metal tube conduit in the center of the vented standpipe unit until the end is approximately 5 mm above the top. Push the 10-mm air vent line through the other conduit until approximately 20 mm extends out the top. See Figure 1.

10. Unroll the water and air vent lines loosely in the trench from the standpipe to the indicator unit. It might be desirable to encase both lines in 19-mm flexible metal conduit for additional protection under rocky material.

11. Cut and attach the water and air lines to the indicator unit as shown in Figures 2 and 3. Then fill the system by pouring

water in the sight tube of the indicator unit (Figure 2) until water comes out of the top of the spill tube of the standpipe unit with no air bubbles showing in the line. Generally, 4 L of water is more than adequate for 100 m of tubing. When filling, attempt to keep the water level in the sight tube near the 0.7-m graduation. Do not allow the water level to drop below the bottom of the sight tube since this would allow air to enter the system.

NOTE: It is helpful if someone can watch the overflow at the standpipe unit while the system is being filled to look for evidence of entrapped air and to signal when the system is full.

If there is evidence of air bubbles entrapped in the water line, continue charging the system with water until the air is purged through the standpipe unit. After charging the system with water and purging the water line of all air, attach the indicator unit on the post to provide an initial reading of approximately 0.7 m.

Adjacent to the bottom of the indicator unit, place a reference nail in the post at the elevation of the 0.0-m graduation. This provides a reference point for surveys and relocation of the indicator unit. Complete the assembly of the standpipe unit by attaching the pipe cap as shown in Figure 1.

12. Cut the air line at approximately the 0.7-m graduation of the indicator unit. Then loop the air line inside of the indicator unit over the lock hasp. See Figure 2. The end of the air line should be pointing downward to prevent the entrance of water or debris. This air line must be free of water at all times since it serves to equalize atmospheric pressure at the standpipe unit and the indicator unit.

    During cold weather when the air line is too stiff to be looped, cut the air line at the 0.3-m graduation mark. Then insert the end of a 0.3-m length of 6-mm plastic tubing in the end of the air line and loop the smaller tubing inside the indicator unit.

13. Carefully backfill the trench and pit with material that is free from large rocks or sharp objects and compact by hand for a depth of at least 0.3 m. Special care must be taken around the base of the standpipe unit to prevent separating the base plate from the plywood platform and to prevent breaking or distorting the plastic tubing.

14. After hand backfilling and compacting for a depth of 0.3 m has been completed, mechanical methods may be used to finish the backfilling operation until the trench is level with the existing fill height. In those cases when the standpipe unit extends above the existing fill height, attach a marker post to the unit and mound fill material around it until it is completely covered. In no case should compaction equipment be allowed directly over an installation until a minimum of 0.3 m of compacted material has been placed over the standpipe unit.

SECTION 2 - SEALED STANDPIPE UNIT

A. APPARATUS

1. Sealed standpipe unit (Figure 5)

2. Plastic drain tubing, 13-mm inside diameter

3. Vented standpipe unit (as described in Part 1, Section 1-A)

B. INSTALLATION

Installation is similar to that for the vented standpipe unit with the following exceptions:

1. Install the device as shown in Figure 6.

2. Follow the procedure in Part 1, Sections B-1 through B-5.

3. Assemble the standpipe unit as shown in Figure 5. Do not attach the outer galvanized pipe. Firmly seat the standpipe unit on the prepared area in the pit.

4. Follow the procedure in Part 1, Sections B-7 and B-8.

5. Attach the 10-mm water tube to the baseplate as shown in Figure 5.

6. Unroll the water, air vent, and drain tubes loosely in the trench from the standpipe to the indicator unit.

7. Follow the procedure in Part 1, Sections B-10 and B-11.

8. Complete the assembly of the sealed standpipe unit by attaching the outer galvanized pipe, other fittings, air vent, and drain tubes.

   NOTE: It may be desirable in some cases to fill around the sealed units with sand so that the tubes will be supported at their attachment points.

9. Cut the drain tube near the base of the indicator unit post. Position the drain tube so that water flows out freely and intrusion of soil or debris is prevented.

10. Follow the procedure in Part 1, Sections B-13 and B-14.

SECTION 3 - POST-INSTALLATION PROCEDURES

A. COLLECTION OF DATA

1. As soon as possible after the settlement device has been installed, determine the elevation of the reference to ±2 mm by survey. This elevation should be checked periodically to correct settlement readings for settlement of indicator unit.

2. Settlement Readings

   a. Note the height of water in the sight tube.

   b. Pour sufficient water in the sight tube to raise the water level approximately 50 mm.

   c. Take a reading at 1 h. The water level after adding the water should drop to the first reading or slightly above it.

   d. If little or no change is observed in the water level, or if the water level is below the 0.01-m graduation, refer to Part 2, Section 3.

3. Record the data as indicated in Figure 7. The form is normally used to record chronological data from a single settlement unit installed to monitor settlement. Instructions in filling out the form follow:

   a. Settlement Data Report (Figure 7)

      Column 1 - Enter the date of the reading.

      Column 2 - Record the water level reading from the graduated scale on the indicator unit (after adding water, as indicated above).

      Column 3 - Record the latest elevation of either the 0.0-m graduation on the indicator unit, or the nail reference as determined by the survey.

      Columns 4 and 5 - These columns are not used.

      Column 6 - Enter the changes in water level elevation and reference elevation since the last reading. This is the sum of the differences between the current and immediately previous entries in Columns 2 and 3.

      Column 7 - Enter the total settlement as *minus* the elevation change since installation. This is obtained by changing the sign of the current data in Column 6 and adding this value to the previous entry in Column 7.

Column 8 - Enter the height of the fill at the surface as determined by survey (optional).

Column 9 - Enter the height of fill above original ground to the nearest 0.1 m.

Column 10 - Enter the number of calendar days elapsed since the settlement device was installed.

Column 11 - Enter any information that would be helpful in the analysis of data as shown. If it is necessary to lower the indicator unit on the post, enter the date and the vertical distance lowered; be sure to include the corrected values in Columns 2, 3, and 6.

B. MAINTENANCE

1. Most important to the continued functioning of fluid level settlement devices is the use of as little water as necessary when recharging the system before reading. For this reason, use only enough water to raise the level in the sight tube approximately 50 mm. Continuous additions of greater quantities of water will probably cause flooding of the standpipe unit.

2. If the water level in the sight tube does not drop after adding water, check the unit over a period of several days. Do not, however, add an excessive amount of water; just observe the system to see if the unit is slow to respond.

   a. If the unit is not operating properly, remove the indicator box from the post and raise it up about 0.3 m. Disconnect the water line from the sight tube and attach the line upright on the post. Inspect the bottom of the sight tube and connector for debris. Remove any obstructions and reassemble the unit without losing water from the water line. After assembly, lower the indicator unit until water is observed in the sight tube, then recharge the system with clean water as necessary.

   b. If the device is still not operating satisfactorily and the sealed standpipe unit is being used, plug the top of the sight tube and attempt to force compressed air through the air line and out the drain line. Do not use greater air pressure than necessary to obtain a small flow through the lines. Do not allow the water in the sight tube to overflow. Keep the top of the sight tube sealed during this operation.

   c. If all other attempts to correct the malfunction fail, disconnect and drain the water line. Then apply compressed air at low pressure to the air line in an attempt to remove debris from the water line. If the sealed standpipe unit is used, plug the drain line during this operation. Occasionally, force air through the water line to clear the lines if no return is observed when pressurizing the air line. If successful in clearing obstructions from the water line in this manner, considerable care is required while recharging the system with water to not use too much water and to not introduce large voids in the system. For this reason, recharging the unit should be performed only by personnel experienced in this type of activity.

3. If the water level in the sight tube is below the 0.01-m graduation or if there is no water in the sight tube, look for leaks around the connection between the sight tube and the water line. If no leak is seen, measure the vertical difference between the 0.0-mm graduation on the indicator unit and the reference point. Remove the unit from the post and lower it approximately 0.5 m or until water is observed in the sight tube. If possible, and without adding water, adjust the height of the indicator unit on the post so that the water level in the sight tube is approximately at the 0.7-m graduation.

Add a small quantity of water and check the water level before attaching the indicator unit to the post. After adjusting the height of the indicator unit, again measure the vertical distance between the 0.0-m graduation on the indicator unit and the reference point, and record the correction on the settlement data form (Figure 7, Column 3).

4. Be sure to replace the cover on the indicator unit after each reading to prevent excessive loss by evaporation and contamination by debris.

5. Occasionally, it may be necessary to protect the air and water lines from rodents or pests. If such a problem exists, protect these lines in flexible conduit extending from the bottom of the indicator unit to below the ground surface. Although this should be done during installation, the conduit can be added later if extreme care is taken not to lose water continuity as described above.

PART 2. PIPE RISER SETTLEMENT DEVICE

A. APPARATUS

1. Pipe riser settlement device (Figure 8)

2. Hand tools - shovel, bar, hammer, pipe wrenches, etc.

B. INSTALLATION

1. It will usually be necessary to determine the location for installing the settlement device by survey. If settlement readings are to be continued after completion of the fill and removal of surcharge, it is imperative that the unit be located directly beneath the median of divided travel lanes or the shoulder of other roadways.

2. After approximately 1 m of embankment material has been placed, excavate a pit to a depth of approximately 0.5 m above original ground at the previously determined location for the settlement device. Prepare a firm, level area free of large rocks or clods for the settlement device at the bottom of the pit.

3. Assemble the settlement device as shown in Figure 8. Attach a 19-mm pipe floor flange to the center of the wood platform with bolts or lag screws. Then screw a 1.8 m length of 19-mm pipe into the floor flange. Place a pipe coupling on the top of the 19-mm pipe and tighten all joints in the assembly using a pipe wrench.

4. Measure and record the distance from the top of the pipe coupling to the top of the wood platform. Then slip the 38 mm by 1.5-m protective sleeve, which may be either rigid polyvinyl chloride (PVC) or iron pipe, over the control pipe until it is about 0.5 m above the floor flange. Place a duct seal or other seal to hold the protective sleeve in place. See Figure 8. Do not attach the protective sleeve to the wood platform or the control pipe. This protective sleeve is used to absorb the friction between the fill material and the settlement unit and, therefore, must be free to move independently from the wood platform and control pipe.

5. Firmly seat the settlement device on the prepared area in the bottom of the pit. Then fill and compact by hand using fine embankment material free of large rocks and clods around the settlement device to a depth of 0.3 m.

6. Using a spirit level, check to make sure the control pipe is reasonably plumb, then carefully fill the pit with embankment material and compact in place.

7. Attach a post to the top of the protective sleeve to alert construction equipment operators of the obstruction.

   NOTE: It has been found that a 1.8-m long 2-by-4 painted with alternate 0.3 m wide stripes of red and white is satisfactory for this use. It is recommended that flagging be attached to the top of this post. The post should be attached so that it can be easily removed and reattached as additional pipe is added during embankment construction.

C. COLLECTION OF DATA

1. As soon as possible after the settlement device has been installed, determine the elevation of the top of the 19-mm pipe coupling attached to the control pipe. Normally, the elevations required will be obtained by a survey party.

2. During embankment construction, the elevation of the top of the control pipe should be determined by survey approximately twice weekly. After embankment construction is completed, the elevation should be determined frequently enough to indicate significant changes in the rate of settlement. Normally, the time between surveys will be weekly immediately subsequent to completion of the embankment, and the interval between surveys will increase with time.

3. During fill placement, it will be necessary to extend the lengths of the control pipe and protective sleeve. When extending the control pipe, use the following procedure:

    a. Determine the elevation to the top of the existing control pipe coupling.

    b. Remove the protective post, attach a coupling to the length of control pipe to be added, and tighten the joint with pipe wrenches.

    c. Insert the added length in the coupling on top of the existing control pipe and tighten the joint by using one pipe wrench on the existing coupling and one pipe wrench on the added length of control pipe. While tightening the joint, do not allow the coupling between the control pipe and the added length to turn. Turn only the added length of control pipe.

    d. Measure the added length of control pipe, including the coupling. If possible, check this distance by determining the elevation of the control pipe.

    e. Record the length of additional control pipe added under Column 5 on the form shown in Figure 9. Be sure to add this length to the previous value shown in Column 4.

    f. Add and secure a 1.5-m length of protective sleeve to the existing sleeve and secure to the top of the post.

4. Enter all data on the form shown in Figure 9 as follows:

    Column 1 - Record the reading date.

    Column 2 - This column is not used.

    Column 3 - This column is not used.

    Column 4 - Record the elevation to top of control pipe as determined by survey.

    Column 5 - Record the length of the control pipe.

    Column 6 - Record the height of the riser above the original ground (Column 4 minus Column 5).

    Column 7 - Record the total settlement to the nearest 0.002 m. This figure is obtained by subtracting the figure in Column 6 for the day being read from the figure at the top of Column 6 (elevation at the time of installation).

    Column 8 - Record the elevation of the surface of the fill as determined by survey (that is optional).

    Column 9 - Record the height of the fill above original ground to the nearest 0.1 m.

Column 10 - Record the number of calendar days elapsed since the settlement device was installed.

Column 11 - Record any information that would be helpful in the analysis of data. Be sure to indicate in this column the date and length added to the control pipe.

PART 3. SETTLEMENT DATA ANALYSIS SCOPE

The procedure for plotting and analyzing settlement data obtained from all types of settlement devices is described in this method. Comprehensive settlement analyses are complex and require extensive knowledge of soil mechanics and soil structure of the area under study. Considerable information, however, can be obtained by the simplified method described in this part.

1. Plot the data on a semi-logarithmic chart as shown in Figure 10. Note that the scale for days is on the logarithmic abscissa of the chart and both settlement and fill height are scaled arithmetically on the ordinate.

2. Note that during construction, the rate of settlement increases in approximate proportion to the fill load applied. This is generally true in all cases where the rate of loading embankment is nearly constant. If embankment construction is suspended for an appreciable length of time, the negative slope indicating rate of settlement should become more positive or flatter until embankment construction resumes. In no case, however, should the rate of settlement curve assume a positive slope.

   a. A sudden increase in the rate of settlement during construction is an indication of impending failure and would dictate that fill loading be stopped immediately.

   b. If the rate of settlement remains excessive after suspending fill operations, additional corrective measures must be taken to reduce the rate of settlement.

      NOTE: This may include removing a portion of the embankment or constructing berms or struts. Such measures usually require a comprehensive analysis and, for that reason, the problem must be brought to the attention of the Project Engineer without delay.

3. After embankment construction has been completed, the rate of settlement will decrease with time, especially for soft foundation soils. However, a marked decrease in the rate of settlement may be noticed until an appreciable amount of time has elapsed since completion of the embankment.

   a. Any significant increase in the rate of settlement after completion of the embankment is sufficient cause for immediate corrective action as described above.

   b. When the plotted data indicate that the slope of the rate of settlement curve is essentially horizontal, the embankment surcharge may be removed and/or permanent structure construction may be started. For example, from data shown in Figure 10, a practical minimal rate of settlement was obtained at about 360 days; at this time the embankment surcharge was removed as shown.

4. Data should be collected throughout the life of the contract. Longer data-collection periods are necessary if significant rates of settlement are measured.

   a. The time interval between readings may be increased as the indicated rate of movement decreases.

   b. Collection of data may be required for several years on selected projects. Long-term settlement data are frequently useful in the design of embankments where similar conditions are encountered.

PART 4. SAFETY AND HEALTH

Prior to handling, testing or disposing of any waste materials, testers are required to read: Part A, Section 5.0, Part B, Sections 5.0, 6.0 and 10.0 and Part C, Section 1.0 of Caltrans Laboratory Safety Manual. Users of this method do so at their own risk.

REFERENCES:

None

End of Text (California Test 112 contains 19 pages)

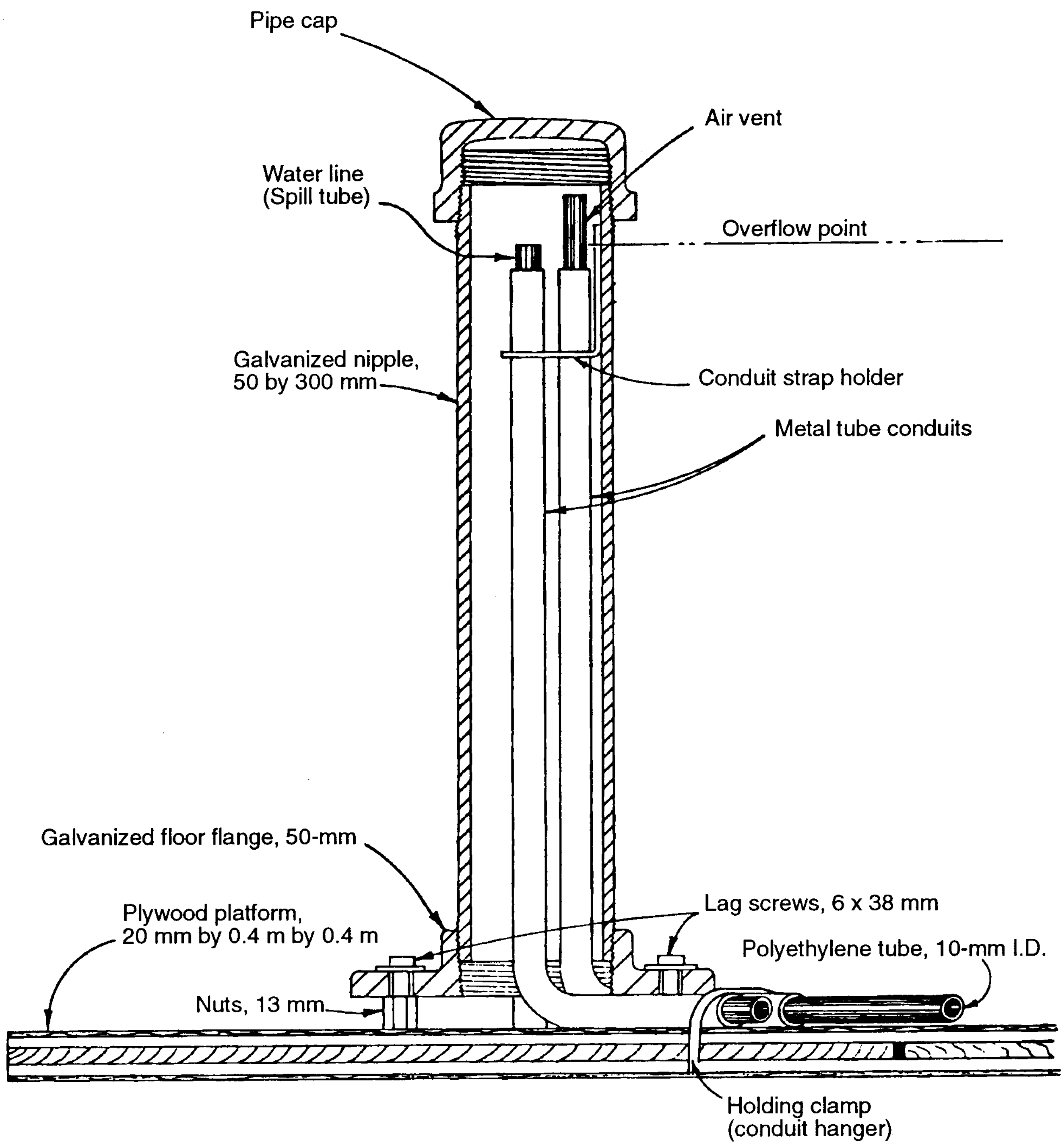


FIGURE 1 - VENTED STANDPIPE UNIT

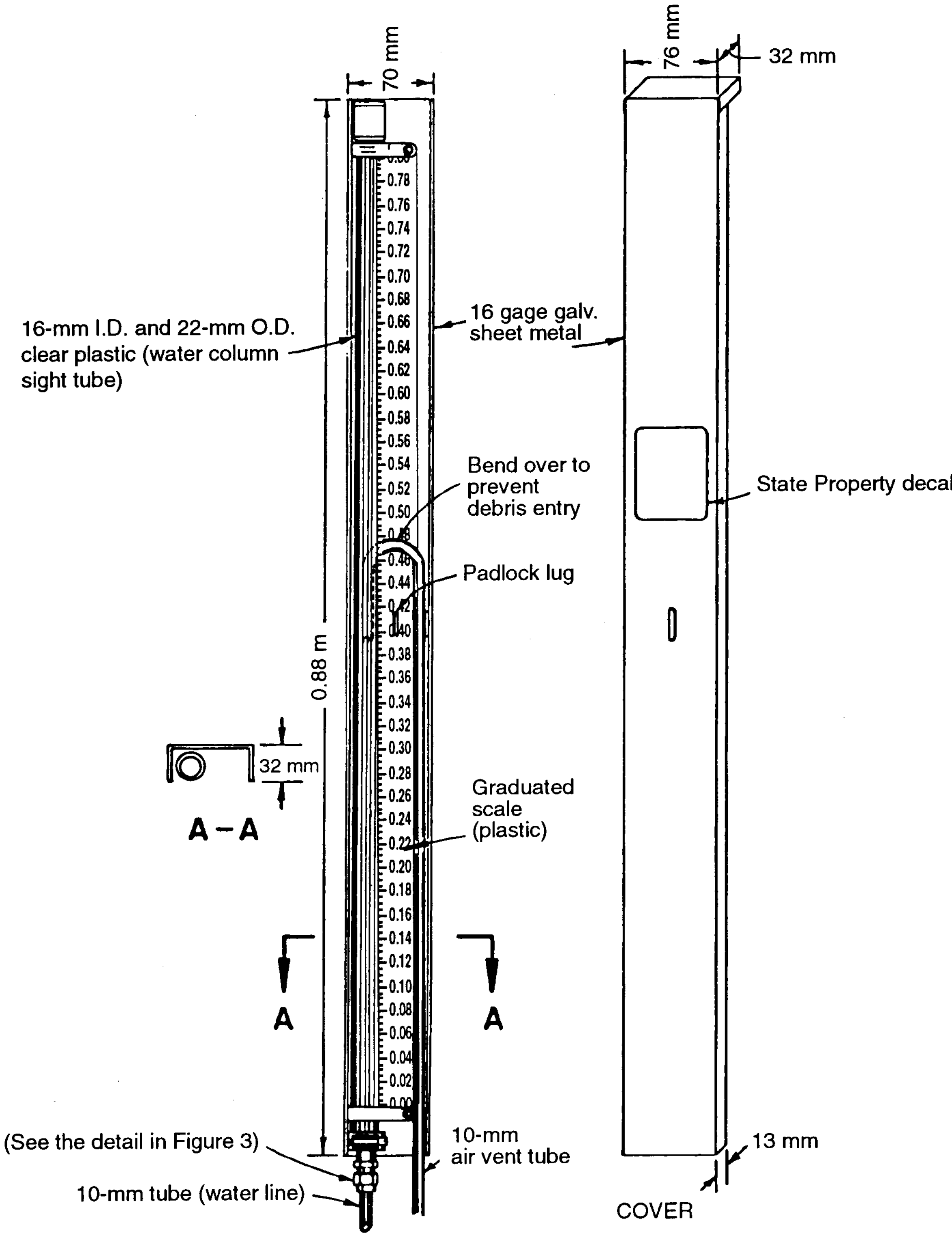


FIGURE 2 - INDICATOR UNIT

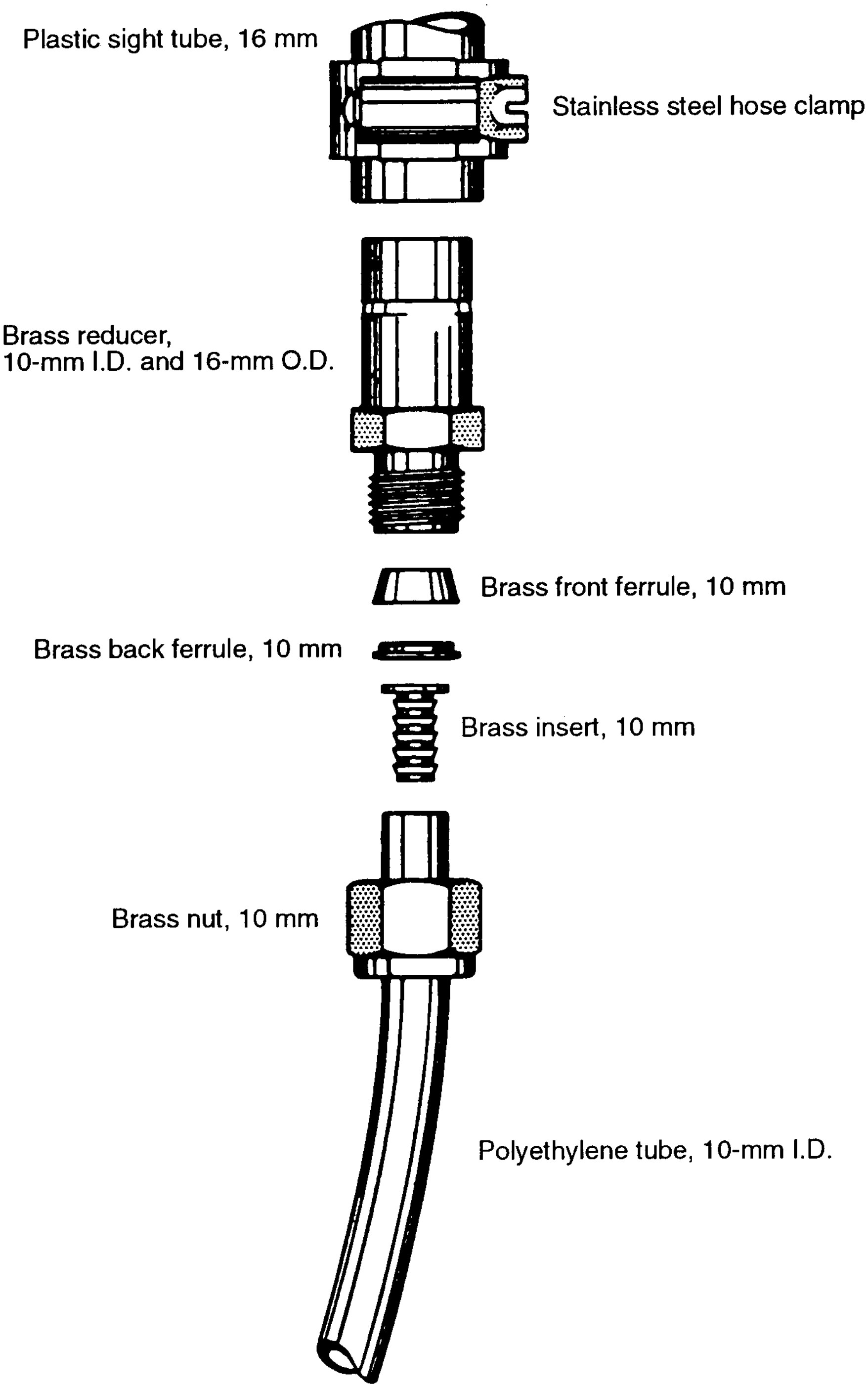
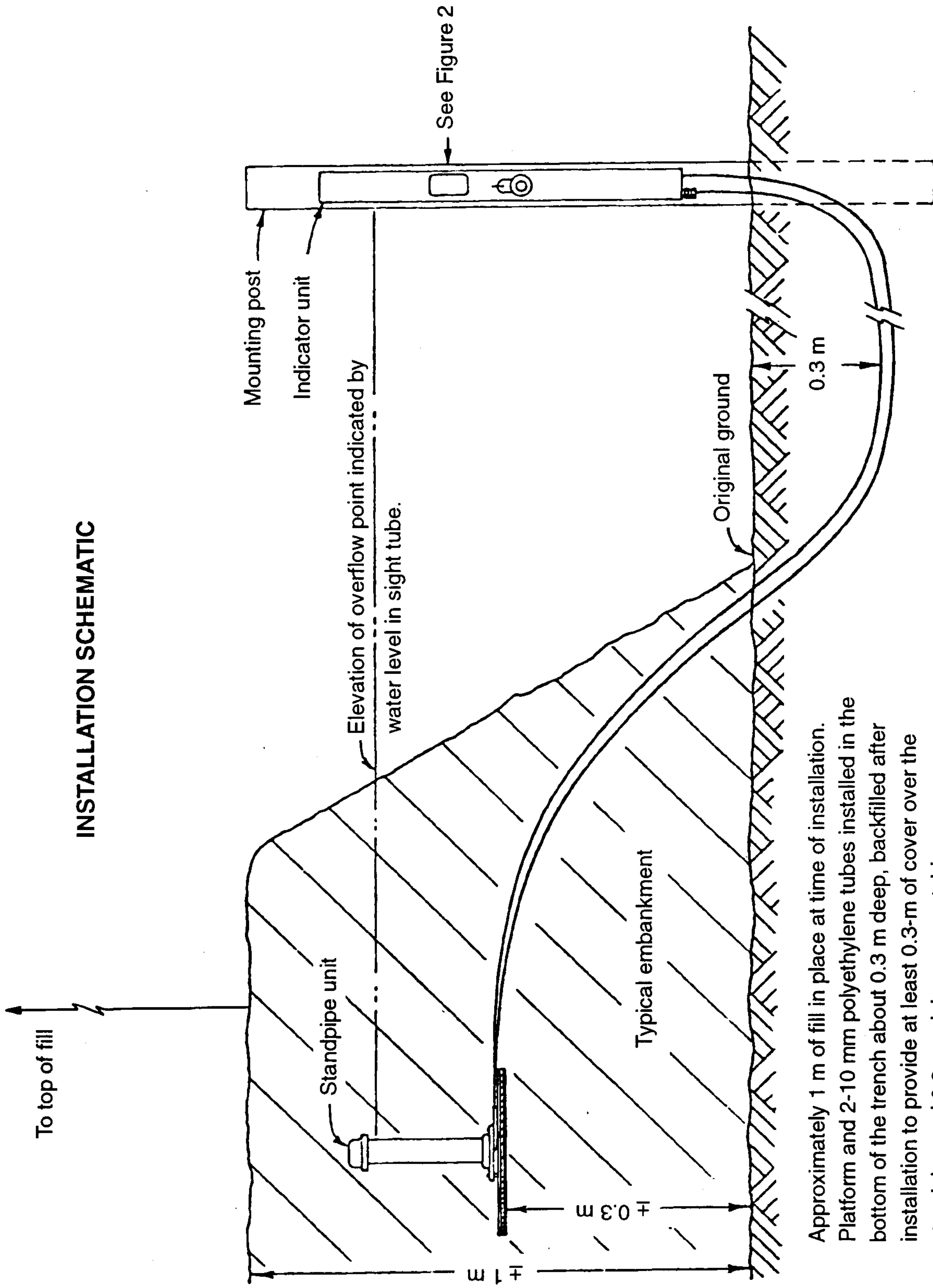


FIGURE 3 - DETAIL OF INDICATOR UNIT

INSTALLATION SCHEMATIC



Approximately 1 m of fill in place at time of installation.
Platform and 2-10 mm polyethylene tubes installed in the bottom of the trench about 0.3 m deep, backfilled after installation to provide at least 0.3-m of cover over the standpipe and 0.3-m minimum over tubing run.

FIGURE 4 - SETTLEMENT INDICATOR DEVICE (VENTED TYPE)

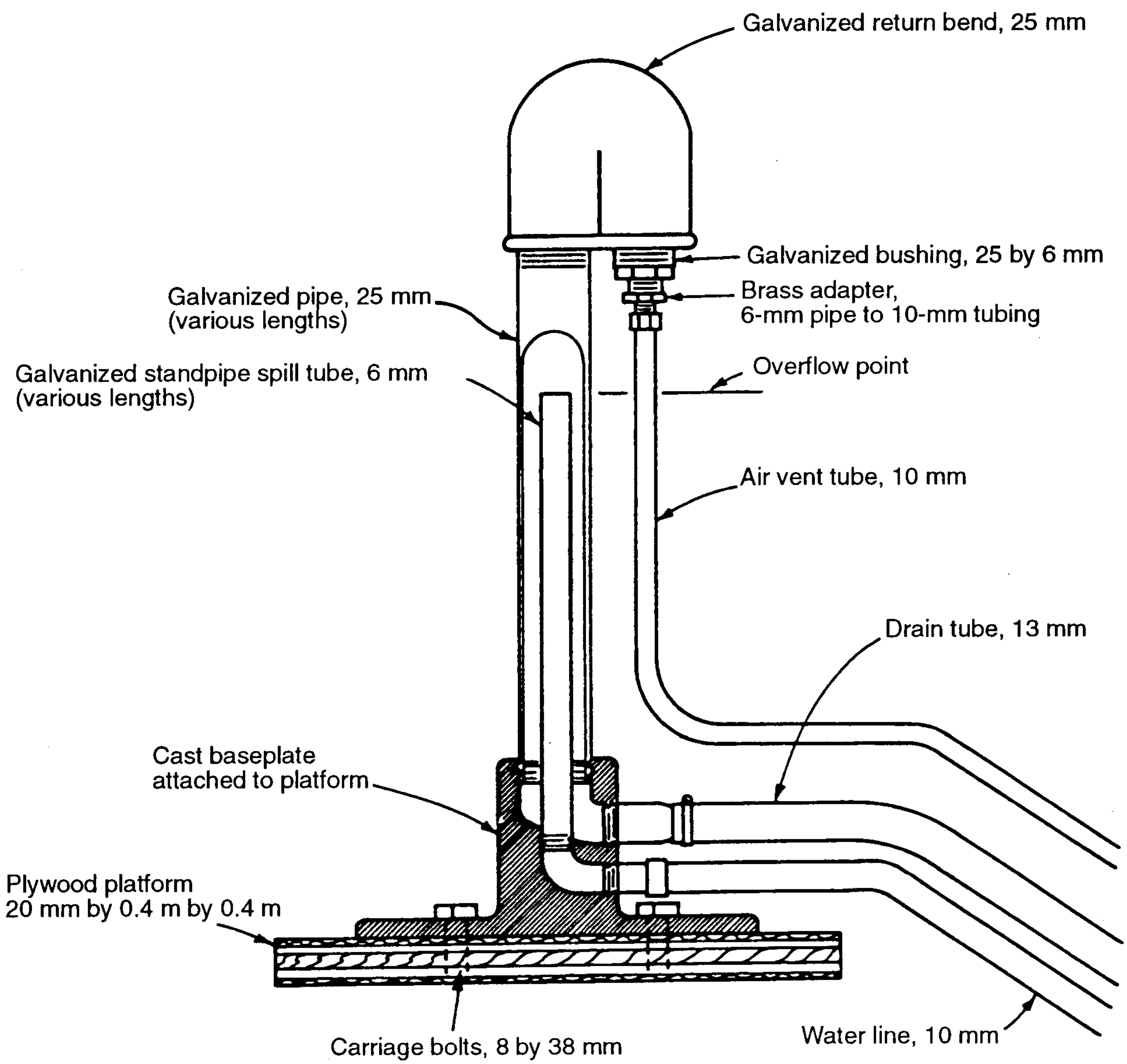


FIGURE 5 - SEALED STANDPIPE

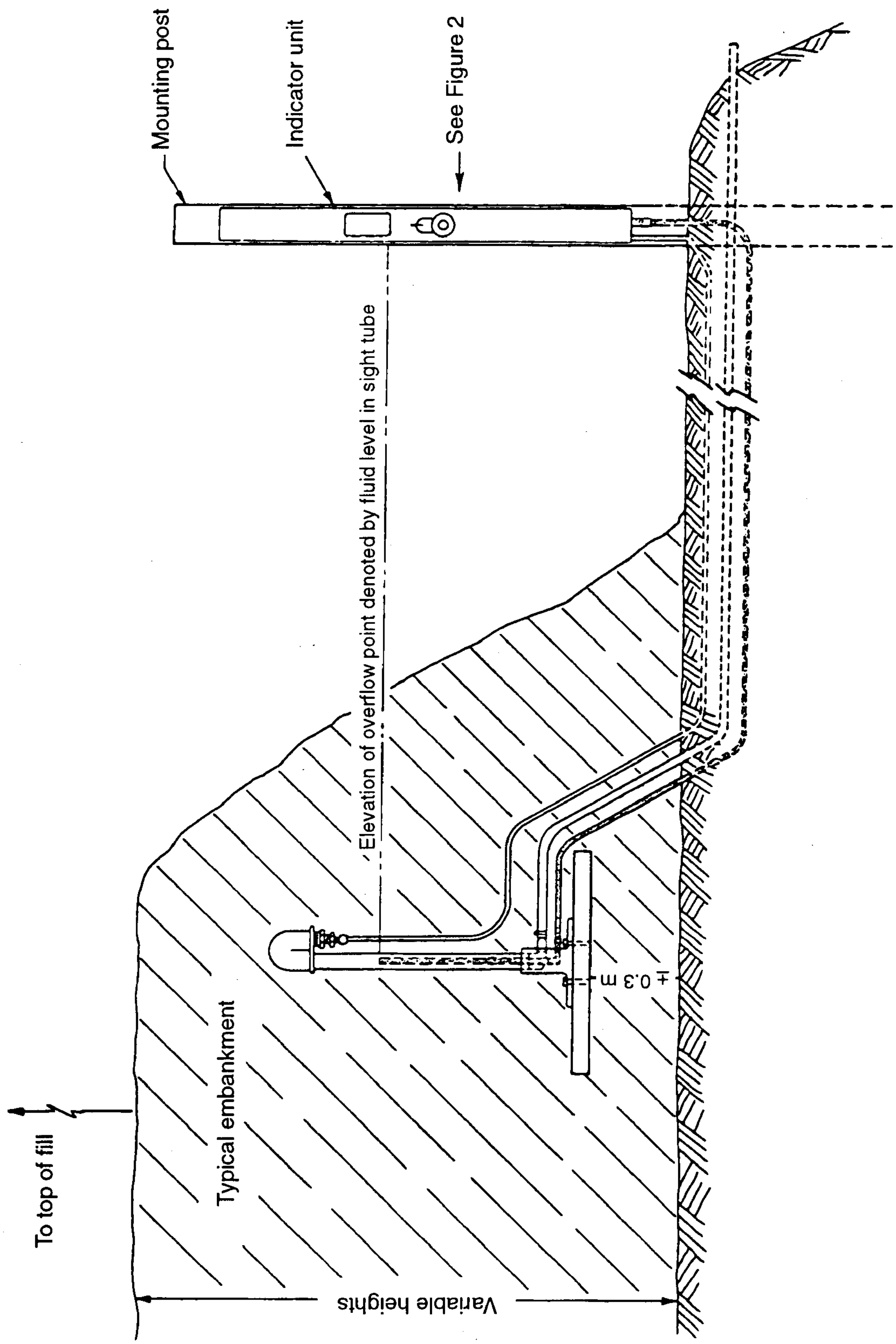


FIGURE 6 - SETTLEMENT INDICATOR DEVICE (SEALED FLUID LEVEL TYPE)

STATE OF CALIFORNIA — SETTLEMENT DATA — DEPARTMENT OF TRANSPORTATION

Job Stamp	Install Loc.	Comments:
	Br. No. / Unit No.	
	Sta. & Offset	
	Design Ht. (m) 6.4 / Surcharge (m) 1.8	
	Waiting Period (days) 270	
	Controlled Loading 1 m per week	

Date	Water Tube Reading (m)	Reference Nail Elevation (m)	RISER PIPE		Elevation Change	Settlement	FILL HEIGHT		Elapsed Time (days)	Remarks
			Top of Pipe Elevation (m)	Length of Pipe (m)			Surface Elevation	Fill Above Original Ground		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
3/3/94	0.758	238.006			0.000	0.000	238.0	0.0	0	Platform set on O.G.
3/4/94	0.744				-0.014	0.014	238.9	0.9	1	
3/7/94	0.738				-0.006	0.020	239.8	1.8	4	
3/11/94	0.716				-0.022	0.042	240.7	2.7	8	
3/18/94	0.658				-0.058	0.100	241.7	3.7	15	
3/25/94	0.600				-0.058	0.158	242.6	4.6	22	
4/1/94	0.548	238.006			-0.052	0.210	243.5	5.5	29	
4/8/94	0.502				-0.046	0.256	244.4	6.4	36	
4/15/94	0.470				-0.032	0.288	245.3	7.3	43	
4/22/94	0.448				-0.022	0.310	246.2	8.2	50	Surcharge completed
4/26/94	0.430	238.000			-0.024	0.334	246.2	8.2	54	
5/12/94	0.408	238.000			-0.022	0.356	246.2	8.2	70	
5/25/94	0.396				-0.012	0.368	246.2	8.2	83	
6/8/94	0.384				-0.012	0.380	246.2	8.2	97	
7/6/94	0.388				0.004	0.376	246.2	8.2	125	
8/1/94	0.378	237.990			-0.020	0.396	246.2	8.2	151	
9/2/94	0.372				-0.006	0.402	246.2	8.2	183	
10/6/94	0.366	237.982			-0.014	0.416	246.2	8.2	217	
11/7/94	0.360				-0.006	0.422	246.2	8.2	249	
12/1/94	0.356	237.982			-0.004	0.426	246.2	8.2	273	Rain since last reading: 94mm
1/5/95	0.354	237.982			-0.002	0.428	246.2	8.2	308	Settlement terminated: 1/17/95
9/28/95	0.348	237.982			-0.006	0.434	244.4	6.4	574	Surcharge removed: 2/1/95

FIGURE 7 - SETTLEMENT DATA REPORT

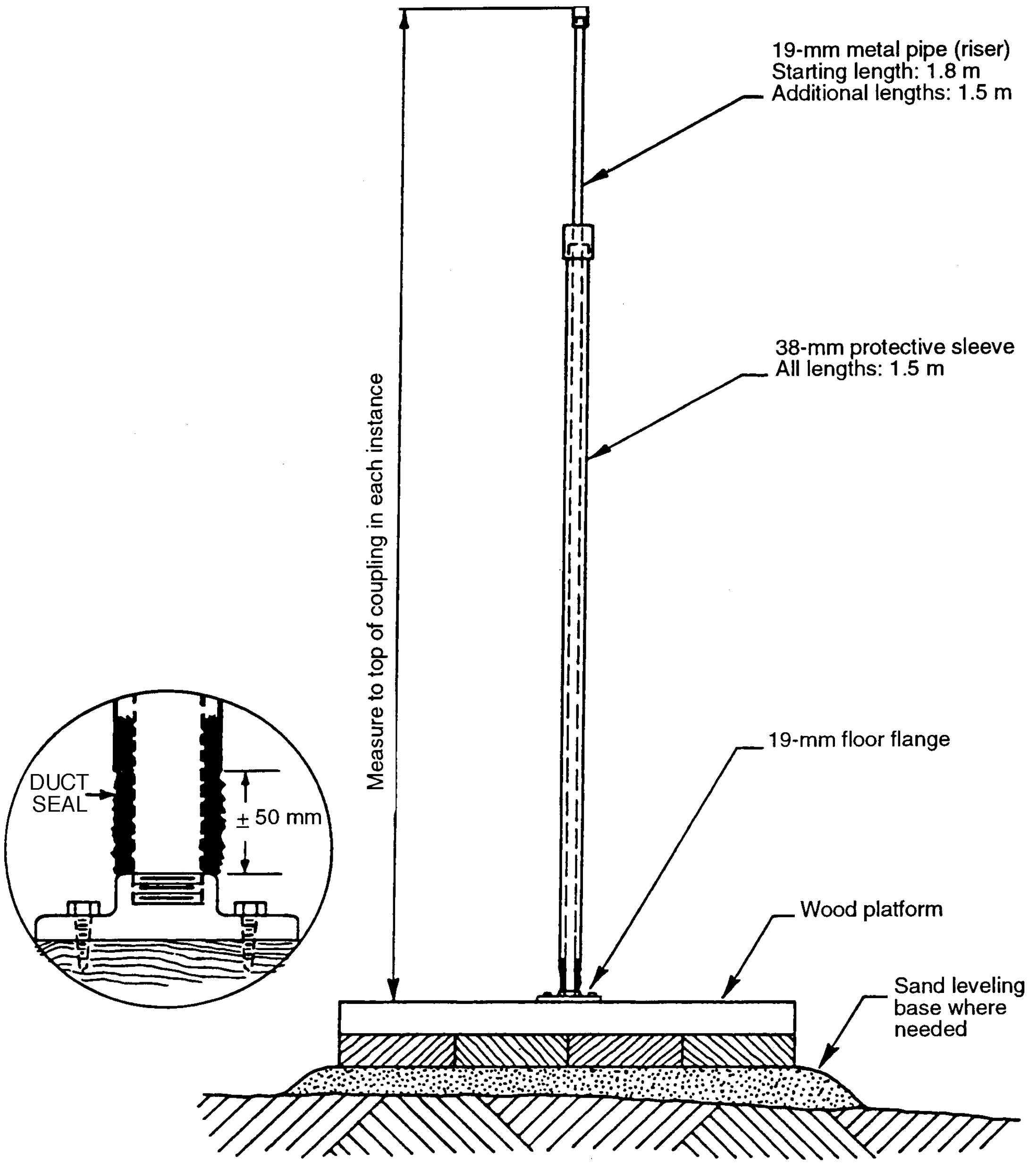
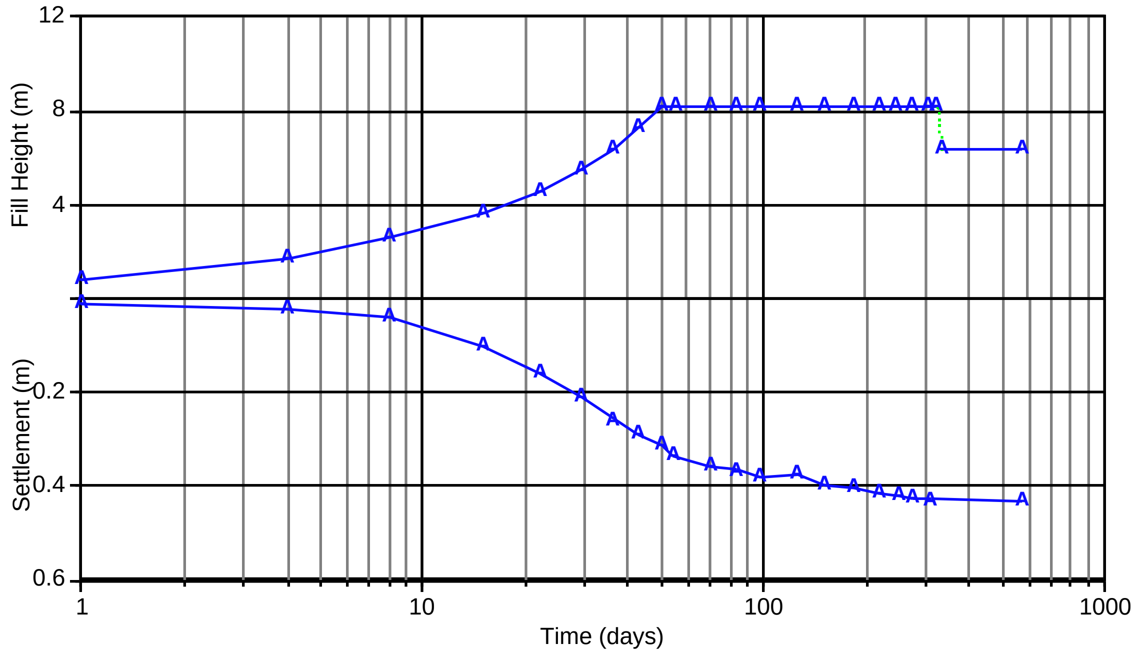


FIGURE 8 - PIPE RISER SETTLEMENT DEVICE

STATE OF CALIFORNIA	SETTLEMENT DATA		DEPARTMENT OF TRANSPORTATION
Job Stamp	Install Loc.		Comments:
	Br. No.	Unit No.	
	Sta. & Offset		
	Design Ht. (m) 15.1	Surcharge (m) 0	
	Waiting Period (days)		
	Controlled Loading		

Date	Water Tube Reading (m)	Reference Nail Elevation (m)	RISER PIPE		Elevation Change	Settlement	FILL HEIGHT		Elapsed Time (days)	Remarks
			Top of Pipe Elevation (m)	Length of Pipe (m)			Surface Elevation	Fill Above Original Ground		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
2/25/95			4.700	3.264	1.436	0.000		1.5	0	
2/27/95			6.304	4.880	1.424	0.012		2.4	2	
3/3/95			7.912	6.492	1.420	0.016		3.0	6	
3/6/95			7.916	6.492	1.424	0.012		4.6	9	
3/10/95			7.906	6.492	1.414	0.022		6.1	13	
3/20/95			9.498	8.102	1.396	0.040		7.8	23	
3/27/95			12.650	11.338	1.312	0.124		10.2	30	
4/6/95			17.300	16.176	1.124	0.312		14.9	40	
4/15/95			17.246	16.176	1.070	0.366		15.1	49	Fill completed
4/29/95			17.210	16.176	1.034	0.402		"	63	
5/7/95			17.184	16.176	1.008	0.428		"	71	
5/13/95			17.178	16.176	1.002	0.434		"	77	
5/23/95			17.160	16.176	0.984	0.452		"	87	
5/29/95			17.164	16.176	0.988	0.448		"	93	
6/6/95			17.152	16.176	0.976	0.460		"	101	
6/12/95			17.178	16.176	1.002	0.434		"	107	
6/20/95			17.138	16.176	0.962	0.474		"	115	
6/26/95			17.124	16.176	0.948	0.488		"	121	
7/17/95			17.124	16.176	0.948	0.488		"	142	
7/24/95			17.118	16.176	0.942	0.494		"	149	
8/4/95			17.108	16.176	0.932	0.504		"	160	

FIGURE 9 - SETTLEMENT DATA REPORT

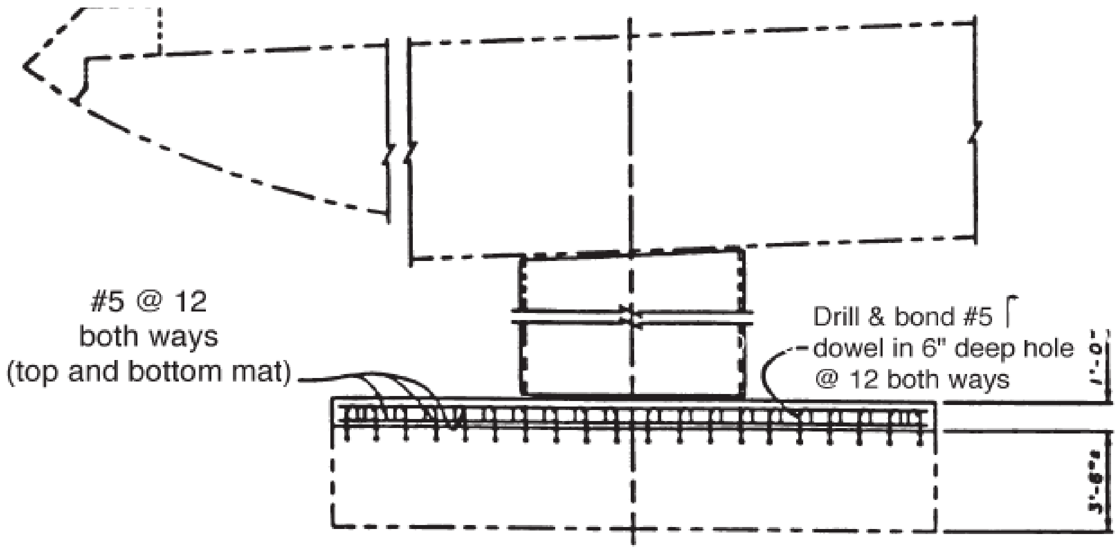


Settlement Data

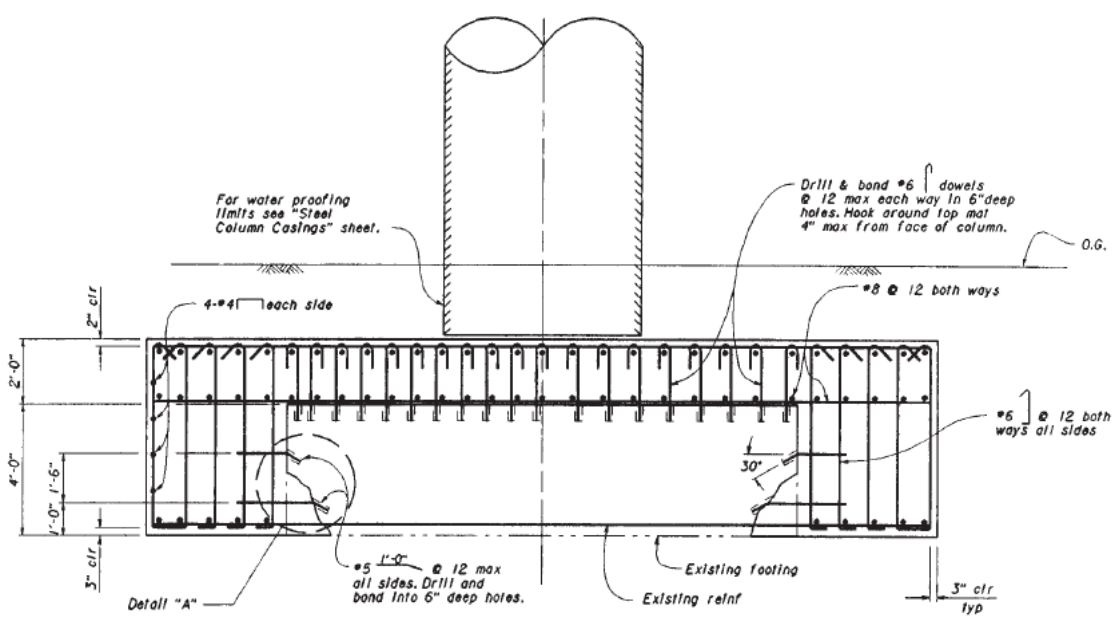
San Ramon Road OC | Abutment 1 | Bridge No. 49-193
05-039733 | 05-SLO-101-47.2/52.5
Station: 2.3 m Rt. 8+50 F-30 Line | Installed: 3/3/94
1 m Vertical Loading per Week | Waiting Period Not to Exceed 270 days

FIGURE 10 - SETTLEMENT DATA ANALYSIS

Throughout the 1990's the Department underwent a massive seismic retrofit program. Retrofits of footings designed and built prior to 1973 were required to address deficiencies. These retrofits required the installation of a top mat of reinforcing steel to address tensile loads at the top of the footing due to seismic forces. In some cases footing dimensions were increased and/or perimeter piles added. These additional piles provide additional resistance to bending moment in the structure and provide additional restraint against rotation. Typical spread footings seismic retrofits are shown in the Figures below.



SECTION D-D



ELEVATION

APPENDIX

D Pier Column & Pile Shaft

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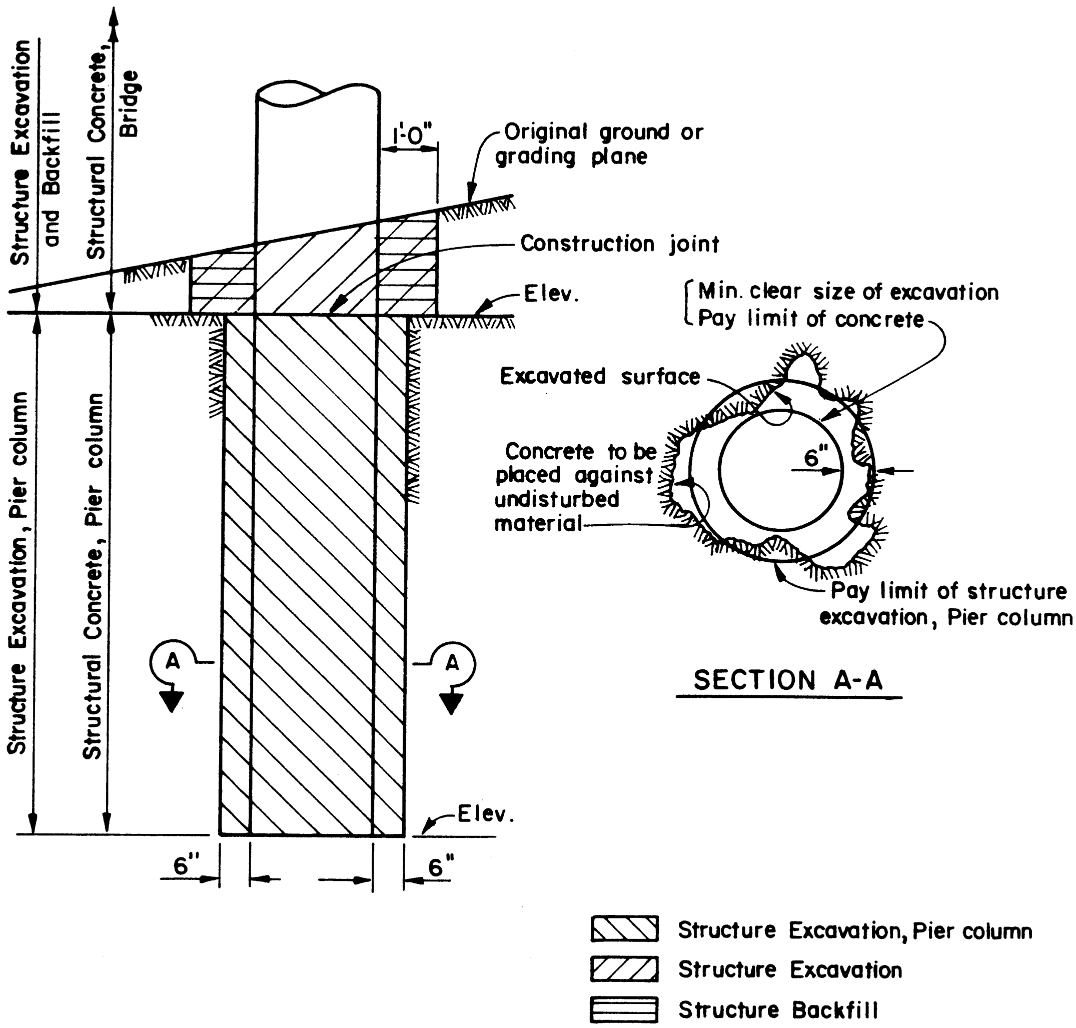
PIER COLUMN



NOTE TO DESIGNERS:

1. Use Pier Column in hard material where conventional CIDH methods are not feasable.
2. Elevations shown are upper and lower limits of hard material excavation.
3. Construction joint to be placed 1' min. below finished grade.

Pier Column - Caltrans Bridge Design Details, page 7-20

Blasting – Example

What follows is an example of how the Department uses blasting in the construction of bridge foundations. In the past, blasting has been performed to facilitate the construction of spread footings and pier columns. The photograph below is an example of what can be considered a hybrid of the two. Pier W2 of the San Francisco-Oakland Bay Bridge East Span Construction project uses blasting to construct a foundation in rock. The foundation is in excess of 60' deep and 80' square.

Pier W2, part of the E2-T1 project (EA 04-0120L4) is part of the new San Francisco Oakland Bay Bridge. Blasting was used for structure excavation of these piers. Construction of W2 structures was complete in Sept 2004 at a cost of $24.1 million.

Special Provisions – EA 04-0120C4

The following specification was taken from the Special Provisions of the above referenced Contract. It is understood that projects of this type are unique and extreme in their application of engineering principals but still utilize construction operations utilized on projects throughout the State. Note that special provisions are unique to each contract; and as such may vary from what is presented below. In some geologic regions blasting may be the only option however Industry continues to implement new technologies. In some cases, new tools such as the rotator and/or oscillator (refer to Chapter 6) may be more appropriate and more environmentally considerate.

Special Provisions – EA 04-0120C4

BLASTING

Attention is directed to, "Project Information," and "Photo Survey of Existing Facilities," of these special provisions, regarding the Blasting Demonstration Report, and photo survey of the existing facilities.

Attention is directed to "Order of Work," of these special provisions regarding transportation and use of explosives.

If the Contractor elects to use blasting for structure excavation (bridge) at Piers W2 , project blasting shall conform to Sections 7-1.10, "Use of Explosives," and 19-2.03, "Blasting," of the Standard Specifications and these special provisions.

The Contractor shall control project blasting effects (fly rock, ground motion, and air noise levels) within the safe limits so as not to cause damage to neighboring improvements.

Blasting Plan Submittal

The Contractor shall submit a blasting plan to the Engineer detailing how he proposes to control fly rock, air noise level, and ground motion peak particle velocity. No blasting operations, including drilling, shall start until the Engineer has reviewed and approved the blasting plan.

The Contractor shall submit the blasting plan in accordance with the provisions in "Working Drawings," of the special provisions not less than 30 working days before commencing blasting activity or at any time the Contractor proposes to change the drilling and blasting methods. The Contractor shall provide 10 working days for the Engineer to complete the review of the blasting plan. In the event that additional blasting plans are required, the Contractor shall provide 5 working days for the review of each additional plan.

The blasting plan shall provide for limiting ground motion to a maximum peak particle of 100 mm/sec at the existing E1 Pier of the San Francisco Oakland Bay Bridge (Bridge No. 33-0025), and 50 mm/sec at the Torpedo Building

(Building 262). Controlling fly rock, air noise levels, and ground motor peak particle velocities as specified herein shall not relieve the Contractor of his responsibility for assuring the complete safety of his operation.

The blasting plan shall indicate the type and method of instrumentation proposed by the Contractor to determine air noise levels, and ground motion peak particle velocity at the nearest improvements. The blasting plan shall also provide for a pre-blast reconnaissance survey of all adjacent improvements.

Approval of the Contractor's blasting plan or blasting procedures shall not relieve the Contractor of any of his responsibility under the contract for assuring the complete safety of his operations with respect to neighboring improvements, or for the successful completion of the work in conformance with the requirements of the plans and specifications.

If the Engineer fails to complete the review within the time allowed, and if, in the opinion of the Engineer, the Contractor's controlling operations are delayed or interfered with by reason of the delay, an extension of time commensurate with the delay will be granted as provided in Section 8-1.07, "Liquidated Damages," of the Standard Specifications.

Qualifications

The blasting supervisors (blaster in charge) shall have a minimum of 10 years experience, directly related to the specific types of blasting they are supervising.

All blasters and supervisors shall be properly qualified and licensed in accordance with applicable federal, State, and local government regulations.

The Contractor shall retain the services of an experienced seismologist or engineering consultant with at least 10 years experience in monitoring blasting operations and interpreting ground vibration, air overpressure, and water pressure amplitudes for similar construction projects.

The Contractor shall retain the services of an experienced specialist who will conduct the pre-blast inspections of private properties as specified herein. The specialist shall have performed similar pre-construction survey services on at least three projects of similar scope and complexity.

Pre-Blast Condition Survey

The Contractor shall perform a pre-blast survey of specified buildings and structures, and utilities within 100 meters or which may potentially be at risk from blasting damage. The survey method used shall be acceptable to the Contractor's insurance company. The Contractor shall perform the pre-blast survey within 30 working days in advance of the planned commencement or resumption of blasting operations and pre-blast records shall be made available to the Engineer for review. The Contractor prior to the beginning of the blast shall notify occupants of the local buildings. The pre-blast survey shall, as a minimum, contain the following:

A. The name of the person making the inspection.

B. *The names of the property owner and occupants, the addresses of the property, the date and time of the inspection.*
C. *A complete description of the structure(s) or other improvement(s) including culverts and bridges.*
D. *A detailed interior inspection with each interior room (including attic and basement spaces) designated and described. All existing conditions of the walls, ceiling and floor such as cracks, holes and separations shall be noted.*
E. *A detailed exterior inspection fully describing the existing conditions of all foundations, walls, roofs, doors, windows, and porches.*
F. *A detailed listing, inspection and documentation of existing conditions of garages, outbuildings, sidewalks and driveways.*
G. *A detailed listing of highway signposts, light fixtures and overhead power lines.*
H. *A survey of any wells or other private water supplies including total depth and existing water surface levels.*

The Contractor shall perform a re-survey of all locations whenever blasting operations are either terminated or suspended for a period in excess of 30 working days. The documentation may consist of either a written report, or videotape with voice narration. The videotape, if used, must include date and time displayed on the image. The Contractor shall provide copies of the pre-blast inspection report or videotape documentation to the Engineer at the time that the blasting plan is submitted.

The Contractor shall control project blasting so that vibration, flyrock, ground and vibration motion, and air noise levels do not cause damage to nearby structures including highway sign posts, light fixtures and parked vehicles, undue annoyance to nearby residents, or danger to employees on the project. The Contractor shall use controlled blasting techniques and designs and shall coordinate the traffic control during blasting operation. The Contractor shall be responsible for all damage resulting from blasting.

Vibration Control and Monitoring

When blasting within proximity of buildings, structures, or utilities that may be subject to damage from blast-induced ground vibrations, the Contractor shall control ground vibrations by the use of properly designed delay sequences and allowable charge weights per delay. Allowable charge weights per delay shall be based on vibration levels that will not cause damage. The Contractor shall perform trial blasts to select allowable charge weights per delay by measuring vibration levels. The Contractor shall select proper control method to limit over break. The trial blasts shall be carried out in conformance with the blasting test section requirements, modified as required to limit ground vibrations to a level which will not cause damage. The blasting test section requirements require that two seismographs be used, one placed on the end of the shot and one placed at 90 degrees behind the shot to establish vibration levels and their relation to the measurement location. The Contractor shall have full responsibility to control over break.

Whenever vibration damage to adjacent structures is possible, the Contractor shall monitor each blast with an approved seismograph located, as approved, between the blast area and the structures subject to the blast site. The seismograph used shall be capable of recording particle velocities for three mutually perpendicular components of vibration in the range generally found with controlled blasting.

The Contractor shall employ a qualified vibration specialist to establish safe vibration limits. The vibration specialist shall also interpret the seismograph records to ensure that the seismograph data are utilized effectively in the control of the blasting operations with respect to the existing structures. The vibration specialist used shall be subject to the Engineer's approval.

The Contractor shall provide vibration monitoring at the following locations:

A. *Existing E1 pier of San Francisco-Oakland Bay Bridge*
B. *Torpedo Building (Building 262)*
C. *Navy Building 1*
D. *Coast Guard Building 27*

The measuring devices should be positioned at the closest face of structure or body of water to the blast site.

Data recorded for each shot shall be furnished to the Engineer prior to the next blast and shall include the following information:

A. *Identification of instrument used.*
B. *Name of qualified observer and interpreter.*
C. *Distance and direction of recording station from blast area.*
D. *Type of ground at recording station and material on which instrument is sitting.*
E. *Maximum particle velocity in each component.*
F. *A dated and signed copy of seismograph readings record.*

At the Contractor's option, shot designs may be based upon scaled distance following the chart below. The scaled distance is the ratio of distance in feet from the blast site to the site to be protected to the square root of the maximum explosive weight used for each delay of 9 milliseconds or more.

Blast Design Table

Distance to site to be protected	*Scaled distance factor*
0 to 91 meters	*22.57 m/kg$^{1/2}$*
91 to 1,524 meters	*24.94 m/kg$^{1/2}$*
1,524 meters	*29.4 m/kg$^{1/2}$*

Environment Protection

Sound Pressure Level (SPL) due to blasting shall not be greater than 180 dB (decibels) in the water at a distance of 10 meters from any point on the shoreline at Yerba Buena Island. The Contractor shall design blasting plan to meet SPL performance limitations and shall perform trial blasts to select allowable charge weights per delay based on measured values of SPL. The Engineer will conduct acoustical monitoring and marine mammal monitoring during all blasting activities. The safe distance for marine mammals due to blasting effects is herein referred to as the Marine Mammal Safety Zone (MMSZ). The MMSZ will be established at a 50-meter radii from the shoreline adjacent to the blasting area, and may be increased or decreased in size based on results of acoustical monitoring. The purpose of the marine mammal monitoring is to prohibit blasting activity if marine mammals are present within the MMSZ. In addition, the Engineer will monitor for Pacific herring spawning event within a 200-meter distance from the shoreline adjacent to the blasting area. If spawning is observed, blasting activity will be prohibited. Work shall not resume until the Engineer notifies the Contractor, which is expected to be approximately 14 calendar days from the time of spawning.

The Contractor shall provide two working days advance notice to the Engineer before each day he is planning to blast. The marine mammal monitoring shall commence at least 15 minutes before blasting begins. The Engineer will have the sole discretion to direct Contractor with approval to proceed with blasting operation prior to each and every blast.

The Department will conduct surveys and monitoring of bird activity before and during blasting activities as part of an agreement with the resource agencies.

Air Blast and Noise Control

The Contractor shall install an air blast monitoring system between the main blasting area and the nearest structure subject to blast damage or annoyance. The equipment used to make the air blast measurements shall be the type specifically manufactured for that purpose. Noise levels shall be held below 125 dbA at the nearest structure or designated location. The Contractor shall use appropriate blast hole patterns, detonation systems, and stemming to prevent venting of blasts and to minimize air blast and noise levels produced by the blasting operations. The decibel level shall be lowered if it proves to be too high based on damage or complaints. The Contractor shall furnish a permanent, signed and dated record of the noise level measurement to the Engineer immediately after each shot.

Flyrock Control

Before the firing of any blast in areas where flying rock may result in personnel injury or unacceptable damage to property, parked vehicles or the work, the Contractor shall cover the rock to be blasted with approved blasting mats, soil, or other equally serviceable material, to prevent flyrock.

If flyrock leaves the construction site and lands on private property all blasting operations will cease until a qualified consultant, hired by the Contractor, reviews the site and determines the cause and solution to the flyrock problem. Before blasting proceeds, a written report shall be submitted by the Contractor to the Engineer for approval.

Video Recordings of Blasts

Videotape recordings will be taken of each blast. The tapes or sections of tapes will be indexed in a manner to properly identify each blast. At the option of the Engineer, copies of videotapes of blasts will be furnished on a weekly basis.

The Contractor shall keep accurate records of each blast. Blasting records shall be made available to the Engineer at all times and shall contain the following data as a minimum:

A. *Blast Identification by numerical and chronological sequence.*
B. *Location (referenced to stationing), date and time of blast.*
C. *Type of material blasted.*
D. *Number of holes.*
E. *Diameter, depth and spacing of holes.*
F. *Height or length of stemming.*
G. *Types of explosives used.*
H. *Type of caps used and delay periods used.*
I. *Total amount of explosives used.*
J. *Maximum amount of explosives per delay period of 9 milliseconds or greater.*
K. *Powder factor (pounds of explosive per cubic yard of material blasted).*
L. *Method of firing type.*
M. *Weather conditions (including wind direction).*
N. *Direction and distance to nearest structure or structures of concern.*
O. *Type and method of instrumentation.*
P. *Location and placement of instruments.*
Q. *Instrumentation records and calculations for determination of ground motion particle velocity or for charge size based on scaled distance.*
R. *Measures taken to limit air noise and fly rock.*
S. *Any unusual circumstances or occurrences during blast.*
T. *Measures to limit over break*
U. *Name of contractor.*
V. *Name and signature of responsible blaster.*

Blasting Guards

The Contractor shall provide sufficient blasting guards and station them around the blasting area during blasting to assure that people and structures are not endangered. Traffic during blasting shall be controlled by the Contractor.

Blasting operations may be suspended by the Engineer for any of the following:

A. Safety precautions, monitoring equipment and traffic control measures are inadequate.
B. Ground motion particle velocity or air noise exceeds the limits specified.
C. Blasting control plan have not been approved.
D. Required records are not being kept.
E. Excessive outbreak as determined by the Engineer

Suspension of blasting operations shall in no way relieve the Contractor of his responsibilities under the terms of this contract. Blasting operations shall not resume until modifications have been made to correct the conditions that resulted in the suspension.

Blasting complaints shall be accurately recorded by the Contractor as to complainant, address, date, time, nature of the complaint, name of person receiving the complaint, the complaint investigation conducted, and the disposition of the complaint. The Contractor shall make the complaint available to the Engineer as soon as practical, but no later than at the beginning of the following day's work shift.

PAYMENT

Full compensation for blasting including all the requirements as specified herein, shall be considered as included in the contract price paid per cubic meter for structure excavation (bridge) and no separate payment will be made therefor.

Photo 1. Pier W2. Before detonation. Blasting mats on top.

Photo 2. W2. Detonation

Photo 3. Detonation

Photo 4. Flyrock contained by blasting mats.

PROJ/RTE/PM: 04-SF-80-13.2 at Yerba Buena Island	
PROJECT NO. 04-0120C4	Video-photos from Leonard Fiji,
Senior Bridge Engineer	

Pile Shaft (Type II) - Case Study

Contract No. 04-470804
04-SJ, Ala-205, 580-10.0/0.4 (near the City of Tracy)
580/205 Separation (Bridge No. 33-0693R)
Construction started in 2006.

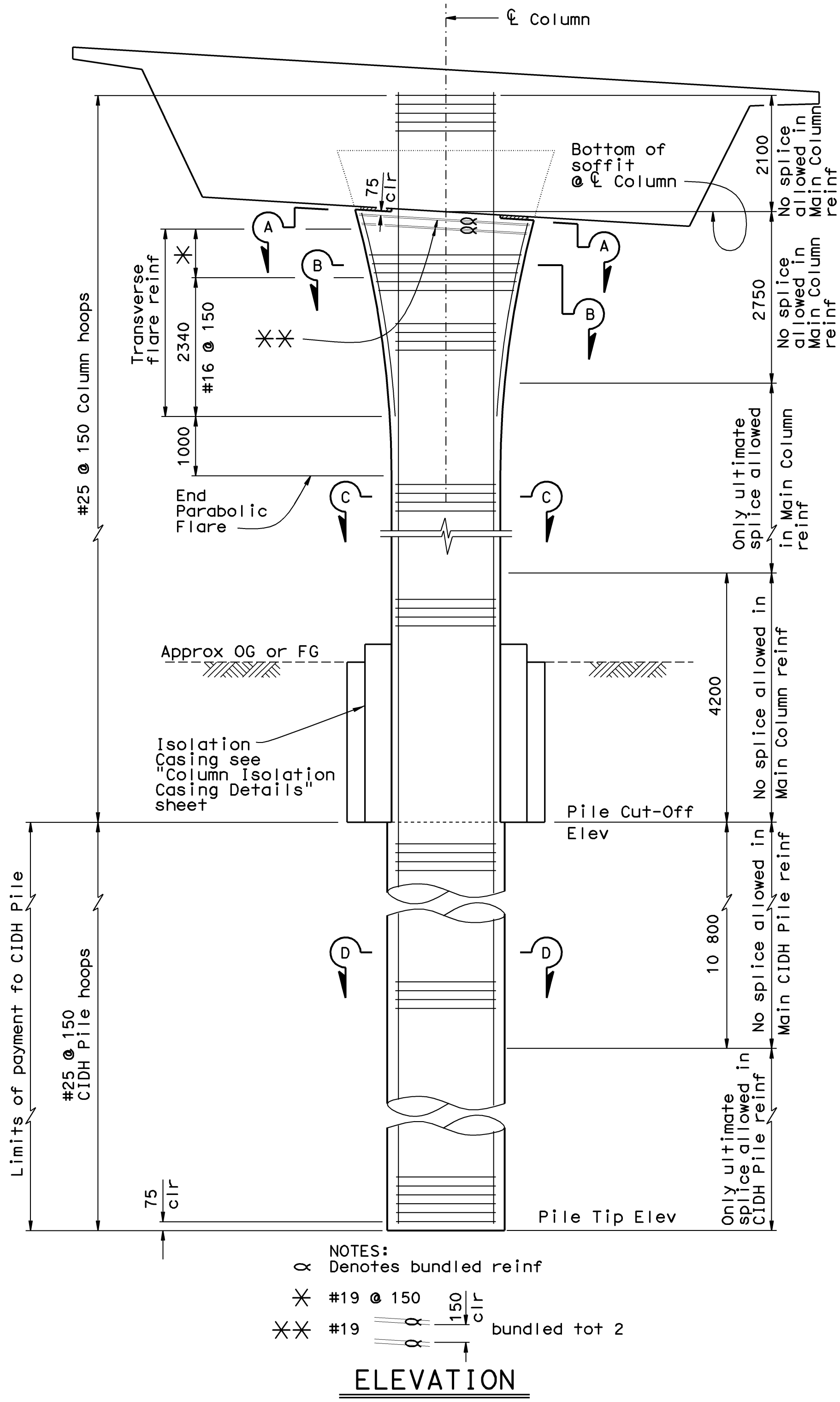
Pile Shaft Project: Although this project has large diameter CIDH piles, this project is not considered a pier column. It **does not** have contract pay items for structure excavation (pier column) and structure concrete (pier column). Conventional methods were used to drill the CIDH pile.

However, the **pile shaft** design was chosen because limited space constraints next to the existing freeway and change of elevation differences between Abutment 1 to Abutment 8. If a pile cap foundation was chosen, then the pile caps would have had to have been excavated 10 to 20 feet beneath the existing freeway to account for the different column stiffness. This excavation would have been problematic due to space constraints in the middle of the existing freeway. Due to the limited space constraints, single column pile shaft foundations were considered easier to construct than a conventional pile cap with standard plan piles. Due to the different column lengths, isolation casing were required at certain locations to account for the different stiffness of short and long columns. Also, the claystone formation underlying the project site was conducive to drilled shaft construction since caving issues would be reduced after using temporary casings to stabilize softer/looser near surface soils (Comments by Tim Alderman, Caltrans Geologist).

Description of Bridge Work: construct a 7-span cast-in-place prestressed concrete box girder bridge approximately 373 meters in length and 12.6 meters in width.
Pile shaft diameter: 1980-mm for Bents 2, 3, & 4; 2280-mm for Bents 5, 6, & 7.
Pile shaft length: 26.75 meters (Bent 4) to 39.8 meters (Bent 2)
Column heights: 8.9 meters (Bent 4); 19.2 meters (Bent 7)
Isolation casings: Bents 2, 3 & 4.

Construction Issues:
Groundwater was anticipated and encountered. Two-cranes were needed to lift the pile shaft column rebar into place. Windy conditions affected crane operations.

(Photos contributed by Gon Choi, Consultant Engineer.)



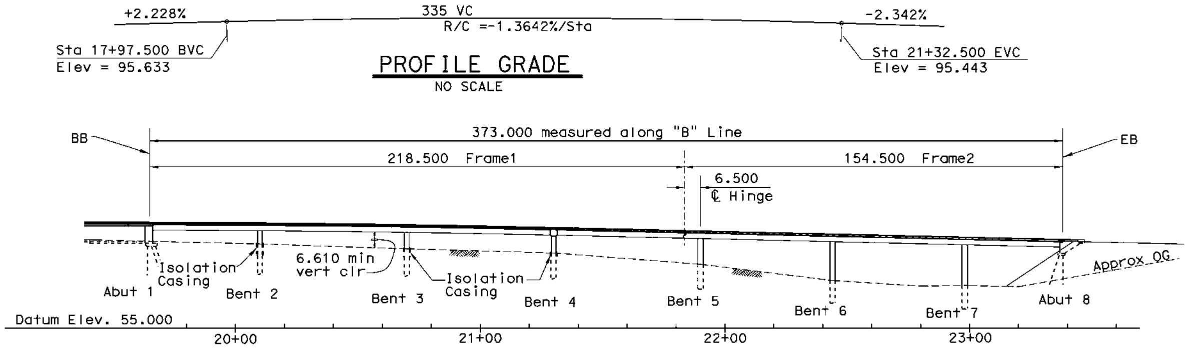
Bents 2 & 3 Column Details – 580/205 Separation, Contract No. 04-470804

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SJ,Ala	205,580	L0.0/0.4, 24.5/24.7,0.0/R4.0	199	272

REGISTERED CIVIL ENGINEER 8-10-05 DATE

11-28-05
PLANS APPROVAL DATE

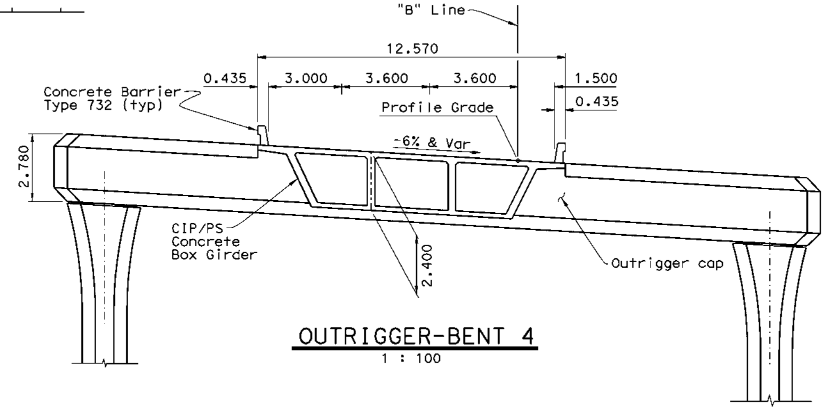
REGISTERED PROFESSIONAL ENGINEER RIHUI YANG No. C61576 Exp. 06/30/07 CIVIL STATE OF CALIFORNIA



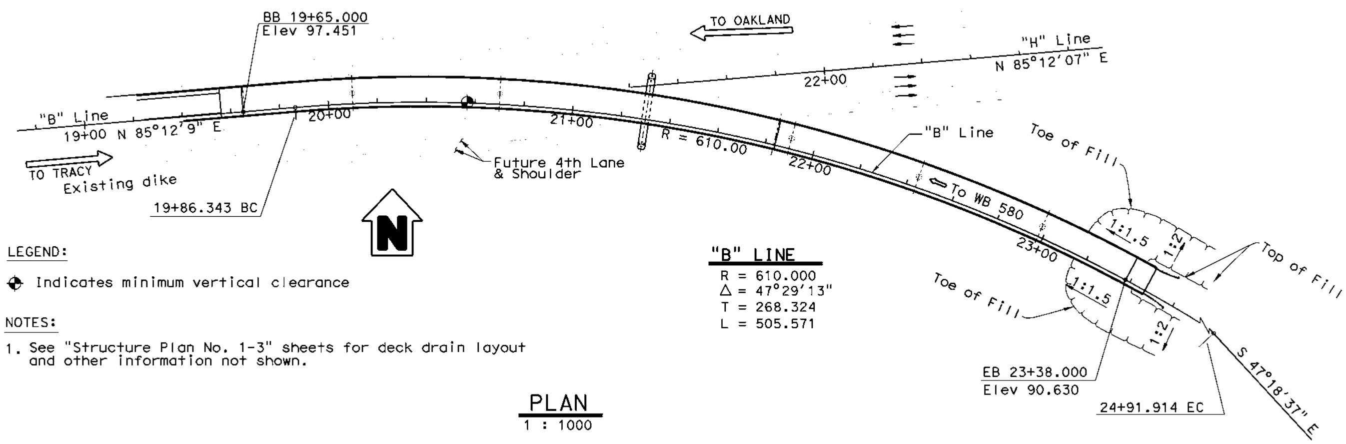
DEVELOPED ELEVATION
1 : 1000

QUANTITIES

STRUCTURE EXCAVATION (BRIDGE)	421	m^3
STRUCTURE BACKFILL (BRIDGE)	349	m^3
FURNISH STEEL PILING (HP 250 X 85)	863	m
DRIVE STEEL PILE (HP 250 X 85)	39	EA
1.98 m CAST-IN DRILLED-HOLE CONCRETE PILING	127	m
2.28 m CAST-IN-DRILLED-HOLE CONCRETE PILING	107	m
PRESTRESSING CAST-IN-PLACE CONCRETE	LUMP SUM	
STRUCTURAL CONCRETE, BRIDGE FOOTING	106	m^3
STRUCTURAL CONCRETE, BRIDGE	3 745	m^3
STRUCTURAL CONCRETE, APPROACH SLAB (TYPE N)	74	m^3
PTFE/ELASTOMERIC BEARING	4	EA
JOINT SEAL ASSEMBLY (MR 101 MM - 160 MM)	36	m
BAR REINFORCING STEEL (BRIDGE)	662 173	kg
ISOLATION CASING	21 242	kg
MISCELLANEOUS METAL (RESTRAINER - CABLE TYPE)	2 139	kg
MISCELLANEOUS METAL (BRIDGE)	5 363	kg
CONCRETE BARRIER (TYPE 732)	763	m



OUTRIGGER-BENT 4
1 : 100



LEGEND:

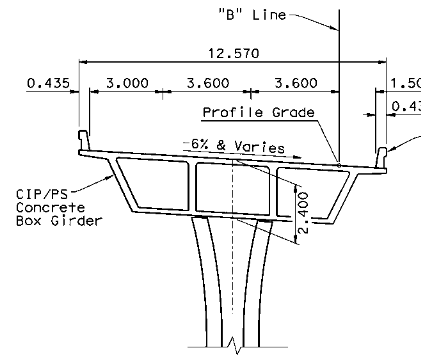
Indicates minimum vertical clearance

NOTES:

1. See "Structure Plan No. 1-3" sheets for deck drain layout and other information not shown.

"B" LINE
R = 610.000
Δ = 47°29'13"
T = 268.324
L = 505.571

PLAN
1 : 1000



TYPICAL SECTION
1 : 100

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

Kevin Keady
DESIGN ENGINEER

	BY	CHECKED		BY	CHECKED
DESIGN	R. Yang	K. Keady	LOAD FACTOR DESIGN	LIVE LOADING: HS20-44 AND ALTERNATIVE AND PERMIT DESIGN LOAD	
DETAILS	R. Yang/RN	B. Robinson	LAYOUT	R. Yang	K. Keady
QUANTITIES	T. Bui	B. Rowley	SPECIFICATIONS	Karla Meier	PLANS AND SPECS COMPARED Karla Meier

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF STRUCTURES
STRUCTURE DESIGN 17

BRIDGE NO. 33-0693R
KILOMETER POST 0.8

580/205 SEPARATION
GENERAL PLAN

STRUCTURES DESIGN GENERAL PLAN SHEET (METRIC) (REV.12-1-01)

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 10115
EA 470801

DISREGARD PRINTS BEARING EARLIER REVISION DATES

CMP casing used to prevent cave-in during drilling of the CIDH concrete piles.

Contractor placing SlurryPro CDP with CMP casing used to prevent cave-in @ Bent 5 of the 580/205

Drilling at Bent 3 of the 580/205 Separation Bridge using a 1.98m diameter auger.

Drilling at Bent 7 with Steel casing to prevent cave-in.

Link Belt LS-518 Crane with Steven M Hain Co 450K, Series 1 drill.

2.13-m diameter Clean out bucket cleaning out Bent 7 of the 580/205 Separation Bridge.

Cave-in at Bent 3 of the 580/205 Separation Bridge.

Drilling at Bent 5.

Iron Workers splicing the #43 rebar with BarGrip couplers.

Iron Workers making rebar cage for Bent 4R CIDH/COLUMN.

Rebar coupler splice equipment in use.

Iron Workers making the rebar cage at Bent 5.

Critical lift safety meeting held just before the cage lift.

Two cranes; Bent 7 – Cage 20 pick.

At the middle of the cage lift.

Vertical rebar cage lifted in the air.

Slurry storage and setting tanks.

4-inch tremie tubes.

CIDH pour with tremie tube in the middle of the pile and slurry is being pump back to the tank.

Column pour

Guy wire placed near the top & middle of the rebar cage.

Guy wire on a short column.

Guy wire placed on the column forms.

Guy wire in place during column pour.

Bent 5: 180-ft long rebar cage with 3" gamma gamma PVC inspection tubes installed.

Bent 7 cage; Bent 6 drilling.

Bent 5 Column - After the forms were stripped.

Bent 5 - side view of the column.

APPENDIX

E Driven Piles

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GATES FORMULA COMMENTARY

Projects with driven pile foundations specify the "Gates Formula" to determine nominal resistance. No longer will contracts utilize the ENR formula. This change is incorporated in the "Amendments To July 1999 Standard Specifications" (Section 49-1.08) found in the front of the Special Provisions. The change is also discussed in Bridge Construction Memo (BCM) 130-4.0 dated June 14, 2007.

Why change from ENR to Gates Formula?

- Factor of safety from ENR (Engineering News Record) varies from ½ to 20. With low factor of safety, capacity of the pile is actually driven to be under the factored design load. Lack of capacity has result in excessive settlement. Extremely high factor of safety often cause damage to the pile and result in contractor claims and also is a waste of time and energy.

- California was actually one of the last States using the ENR formula.

- ENR does not properly account for down drag or the overburden effects and resistance associated with zones that may scour or liquefy.

Advantage of Gate's Formula

- This formula predicts the static capacity of the pile significantly more accurately than the ENR Formula because it provides a significantly lower coefficient of variation.

Additionally, since the formula utilizes ultimate capacity and not an unfactored safe load, the formula can account for the effects of downdrag, scour, and liquefaction.

The Gates formula (US Customary) is:

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

R_u = Calculated nominal resistance/ultimate compressive capacity in kips
E_r = Energy rating of hammer at observed field drop height in foot pounds
N = Number of blows in the last foot (maximum of 100)

Additional Notes:

Caltrans Memo To Designer 3-1 was updated in July 2008. During constructability reviews, it is very important that the Structure Construction reviewer checks the pile data table on the plan sheets for notes on downdrag and liquifaction.

A very good reference showing the differences in formulas (Gates, ENR, Haley, Janbu, etc) is the "Comparison of Methods for Estimating Pile Capacity, Report No. WA-RD 163.1", Final Report dated August 1988, by the Washington State Department of Transportation. In lieu of that, examples of comparisons are shown below.

PILE DRIVING FORMULAS

GATES FORMULA

$$P = \left((1.83 * (E_r)^{1/2} * \log_{10}(0.83 * N)) - 124\right) z$$

Where, P = safe load in kips
E_r = energy of driving in foot pounds
N = number of hammer blows in the last foot
z = conversion factor for units and safety with this formula

ENGINEERING NEWS (ENR)

$$P = \frac{2E}{(s + 0.1)}$$

Where, P = safe load in pounds
E = rated energy in foot-pounds
s = penetration per blow in inches

This formula was derived from the original Engineering News formula for drop hammers on timber piles, which was:

$$P = \frac{WH}{(s + c)}$$

Where, W = weight of ram in pounds
H = length of stroke in inches
c = elastic losses in the cap, pile, and soil in inches

It was modified to correct units and apply other factors to compensate for modern equipment.

JANBU FORMULA

$$P = \left(\frac{WH}{k_u s}\right) z$$

Where, P = safe load in pounds
W = weight of ram in pounds
H = length of stroke in inches
s = penetration per blow in inches
k_u = factor derived from the following,

$$k_u = C_d \left[1 + \sqrt{1 + (\lambda / C_d)}\right]$$

$$C_d = 0.75 + 0.15 \left(W_p / W\right)$$

$$\lambda = \frac{WHL}{AEs^2}$$

when, W_p = weight of pile in pounds
L = length of pile in inches
A = area of pile in square inches
E = modulus of elasticity of pile in pounds per square inch

z = conversion factor for units and safety with this formula

HILEY FORMULA

$$P = \left(\frac{e_f WH}{s + \frac{1}{2}(c_1 + c_2 + c_3)}\right)\left(\frac{W + n^2 W_p}{W + W_p}\right) z$$

Where, P = safe load in pounds
e_f = efficiency of hammer (%)
W = weight of ram in pounds
H = length of stroke in inches
s = penetration per blow in inches
c_1 , c_2 , c_3 = temporary compression of pile cap and head, pile, and soil, respectively in inches
n = coefficient of restitution
W_p = weight of pile in pounds
z = conversion factor for units and safety with this formula

PACIFIC COAST FORMULA

$$P = \frac{E_n\left(\dfrac{W + kW_p}{W + W_p}\right)z}{s + \dfrac{PL}{AE}}$$

Where, P = safe load in pounds
E_n = energy of driving in inch pounds
W = weight of ram in pounds
W_p = weight of pile in pounds
s = penetration per blow in inches
L = length of pile in inches
A = area of pile in square inches
E = modulus of elasticity of pile in pounds per square inch
k = 0.25 for steel piles
0.10 for other piles
z = conversion factor for units and safety with this formula

COMPARISON OF FORMULAS

Given Problem Conditions

Hammer Data: Delmag D36-32

Maximum Energy =	83,880 ft·lbs
Hammer/Ram Weight =	7,938 lbs
Maximum Stroke =	10.42 ft
Penetration or Set=	0.844 inches

Length of Pile = 80 feet

- Assume hard driving -

Case 1:	12” PC/PS concrete pile
Case 2:	12 BP 53 Steel Piles

GATES FORMULA

For Case 1 & 2:

$$P = \left((1.83 * (E_r)^{1/2} * \log_{10}(0.83 * N)) - 124\right)z$$

$$= \left((1.83 * (83{,}880)^{1/2} * \log_{10}(0.83 * (12/0.844)) - 124\right)\frac{1}{2\left(2\,{}^{\text{kip}}\!/_{\text{ton}}\right)}$$

$$= \left((1.83 * 289.62 * 1.072) - 124\right)\frac{1}{2\left(2\,{}^{\text{kip}}\!/_{\text{ton}}\right)}$$

$$= (568.122 - 124)\frac{1}{2\left(2\,{}^{\text{kip}}\!/_{\text{ton}}\right)}$$

$$= \frac{444.122\text{ kips}}{2\left(2\,{}^{\text{kip}}\!/_{\text{ton}}\right)} \approx \underline{\underline{111.0\text{ tons}}}$$

ENGINEERING NEWS (ENR) FORMULA

For Case 1 & 2:

$$P = \frac{2E}{(s + 0.1)}$$

$$= \frac{2(83{,}880.0\text{ ft} \cdot \text{lbs})}{0.844\text{ in} + 0.1}$$

$$= 177{,}712\text{ lbs} \approx \underline{\underline{70\text{ tons}}}$$

JANBU FORMULA

Case 1:

$$P = \left(\frac{WH}{k_u s}\right) z = \left(\frac{WH}{k_u s}\right) \frac{1}{3\left(2000 \,{}^{\mathrm{lbs}}\!/\!_{\mathrm{ton}}\right)}$$

$$= \left(\frac{7{,}938\ \mathrm{lbs}(10.42\ \mathrm{ft} \times 12\ {}^{\mathrm{in}}\!/\!_{\mathrm{ft}})}{2.697(0.844\ \mathrm{in})}\right) \frac{1}{3\left(2000 \,{}^{\mathrm{lbs}}\!/\!_{\mathrm{ton}}\right)}$$

$$= \left(\frac{435{,}931\ \mathrm{lbs}}{3(2000\ {}^{\mathrm{lbs}}\!/\!_{\mathrm{ton}})}\right) \approx \underline{\underline{72.66\ \mathrm{tons}}}$$

$$c_d = 0.75 + 0.15(W_p / W)$$
$$= 0.75 + 0.15(11{,}600\ \mathrm{lbs}/7{,}938\ \mathrm{lbs})$$
$$= 0.969$$

$$\lambda = \frac{WHL}{AEs^2}$$
$$= \frac{7{,}938\ \mathrm{lbs}(10.42\ \mathrm{ft} \times 12\ {}^{\mathrm{in}}\!/\!_{\mathrm{ft}})(80\ \mathrm{ft} \times 12\ {}^{\mathrm{in}}\!/\!_{\mathrm{ft}})}{(144\ \mathrm{in}^2)(4.4 \times 10^6\ {}^{\mathrm{lbs}}\!/\!_{\mathrm{in}^2})(0.844\ \mathrm{in})^2}$$
$$= 2.111$$

$$k_u = c_d\left[1 + \sqrt{1 + (\lambda / c_d)}\right]$$
$$= 0.969\left[1 + \sqrt{1 + (2.111/0.969)}\right]$$
$$= 2.697$$

Case 2:

$$P = \left(\frac{WH}{k_u s}\right) z = \left(\frac{WH}{k_u s}\right) \frac{1}{3\left(2000 \,{}^{\mathrm{lbs}}\!/\!_{\mathrm{ton}}\right)}$$

$$= \left(\frac{7{,}938\ \mathrm{lbs}(10.42\ \mathrm{ft} \times 12\ {}^{\mathrm{in}}\!/\!_{\mathrm{ft}})}{2.581(0.844\ \mathrm{in})}\right) \frac{1}{3\left(2000 \,{}^{\mathrm{lbs}}\!/\!_{\mathrm{ton}}\right)}$$

$$= \left(\frac{455{,}578\ \mathrm{lbs}}{3(2000\ {}^{\mathrm{lbs}}\!/\!_{\mathrm{ton}})}\right) \approx \underline{\underline{75.93\ \mathrm{tons}}}$$

$$c_d = 0.75 + 0.15(W_p / W)$$
$$= 0.75 + 0.15(4{,}240\ \mathrm{lbs}/7{,}938\ \mathrm{lbs})$$
$$= 0.830$$

$$\lambda = \frac{WHL}{AEs^2}$$
$$= \frac{7{,}938\ \mathrm{lbs}(10.42\ \mathrm{ft} \times 12\ {}^{\mathrm{in}}\!/\!_{\mathrm{ft}})(80\ \mathrm{ft} \times 12\ {}^{\mathrm{in}}\!/\!_{\mathrm{ft}})}{(15.58\ \mathrm{in}^2)(30 \times 10^6\ {}^{\mathrm{lbs}}\!/\!_{\mathrm{in}^2})(0.844\ \mathrm{in})^2}$$
$$= 2.861$$

$$k_u = c_d\left[1 + \sqrt{1 + (\lambda / c_d)}\right]$$
$$= 0.830\left[1 + \sqrt{1 + (2.861/0.830)}\right]$$
$$= 2.581$$

HILEY FORMULA

Case 1:

$$P = \left(\frac{e_f WH}{s + \frac{1}{2}(c_1 + c_2 + c_3)}\right)\left(\frac{W + n^2 W_p}{W + W_p}\right) z$$

$$= \left(\frac{e_f WH}{s + \frac{1}{2}(c_1 + c_2 + c_3)}\right)\left(\frac{W + n^2 W_p}{W + W_p}\right) \frac{1}{2.75(2000\,{}^{\text{lbs}}\!/\!_{\text{ton}})}$$

$$= \left(\frac{1.00(7{,}938\text{ lbs})(10.42\text{ ft} \times 12\,{}^{\text{in}}\!/\!_{\text{ft}})}{0.844\text{ in} + \frac{1}{2}(0.37\text{ in} + 0.32\text{ in} + 0.10\text{ in})}\right)\left(\frac{7{,}938\text{ lbs} + (0.25^2)(11{,}600\text{ lbs})}{7{,}938\text{ lbs} + 11{,}600\text{ lbs}}\right) \frac{1}{2.75(2000\,{}^{\text{lbs}}\!/\!_{\text{ton}})}$$

$$= \frac{355{,}090\text{ lbs}}{2.75(2000\,{}^{\text{lbs}}\!/\!_{\text{ton}})} \approx \underline{\underline{64.6\text{ tons}}}$$

Case 2:

$$P = \left(\frac{e_f WH}{s + \frac{1}{2}(c_1 + c_2 + c_3)}\right)\left(\frac{W + n^2 W_p}{W + W_p}\right) z$$

$$= \left(\frac{e_f WH}{s + \frac{1}{2}(c_1 + c_2 + c_3)}\right)\left(\frac{W + n^2 W_p}{W + W_p}\right) \frac{1}{2.75(2000\,{}^{\text{lbs}}\!/\!_{\text{ton}})}$$

$$= \left(\frac{1.00(7{,}938\text{ lbs})(10.42\text{ ft} \times 12\,{}^{\text{in}}\!/\!_{\text{ft}})}{0.844\text{ in} + \frac{1}{2}(0.0\text{ in} + 0.48\text{ in} + 0.10\text{ in})}\right)\left(\frac{7{,}938\text{ lbs} + (0.55^2)(4{,}240\text{ lbs})}{7{,}938\text{ lbs} + 4{,}240\text{ lbs}}\right) \frac{1}{2.75(2000\,{}^{\text{lbs}}\!/\!_{\text{ton}})}$$

$$= \frac{662{,}508\text{ lbs}}{2.75(2000\,{}^{\text{lbs}}\!/\!_{\text{ton}})} \approx \underline{\underline{120.5\text{ tons}}}$$

PACIFIC COAST FORMULA

Case 1:

$$P = \frac{E_n\left(\frac{W + kW_p}{W + W_p}\right)z}{s + \frac{PL}{AE}}$$

$$= \frac{E_n\left(\frac{W + kW_p}{W + W_p}\right)}{s + \frac{PL}{AE}} \times \frac{1}{4(2000\ {}^{\text{lbs}}\!/_{\text{ton}})}$$

$$= \frac{83{,}880\ \text{ft}\cdot\text{lbs}(12\ {}^{\text{in}}\!/_{\text{lbs}})\left(\frac{7{,}938\ \text{lbs} + 0.1(11{,}600\ \text{lbs})}{7{,}938\ \text{lbs} + (11{,}600\ \text{lbs})}\right)}{0.844\ \text{in} + \frac{P(80\ \text{ft}\times 12\ {}^{\text{in}}\!/_{\text{ft}})}{(144\ \text{in}^2)(4.4\times 10^6)}} \times \frac{1}{4(2000\ {}^{\text{lbs}}\!/_{\text{ton}})}$$

$$= \frac{468{,}711\ \text{in}\cdot\text{lbs}}{0.844\ \text{in} + P(1.52\times 10^{-6}\ {}^{\text{in}}\!/_{\text{lbs}})} \times \frac{1}{4(2000\ {}^{\text{lbs}}\!/_{\text{ton}})}$$

$$= \frac{343{,}511\ \text{lbs}}{4(2000\ {}^{\text{lbs}}\!/_{\text{ton}})} \approx \underline{\underline{42.94\ \text{tons}}}$$

Case 2:

$$P = \frac{E_n\left(\frac{W + kW_p}{W + W_p}\right)z}{s + \frac{PL}{AE}}$$

$$= \frac{E_n\left(\frac{W + kW_p}{W + W_p}\right)}{s + \frac{PL}{AE}} \times \frac{1}{4(2000\ {}^{\text{lbs}}\!/_{\text{ton}})}$$

$$= \frac{83{,}880\ \text{ft}\cdot\text{lbs}(12\ {}^{\text{in}}\!/_{\text{lbs}})\left(\frac{7{,}938\ \text{lbs} + 0.25(4240\ \text{lbs})}{7{,}938\ \text{lbs} + (4240\ \text{lbs})}\right)}{0.844\ \text{in} + \frac{P(80\ \text{ft}\times 12\ {}^{\text{in}}\!/_{\text{ft}})}{(15.58\ \text{in}^2)(30\times 10^6)}} \times \frac{1}{4(2000\ {}^{\text{lbs}}\!/_{\text{ton}})}$$

$$= \frac{743{,}720\ \text{in}\cdot\text{lbs}}{0.844\ \text{in} + P(2.1\times 10^{-6}\ {}^{\text{in}}\!/_{\text{lbs}})} \times \frac{1}{4(2000\ {}^{\text{lbs}}\!/_{\text{ton}})}$$

$$= \frac{430{,}395\ \text{lbs}}{4(2000\ {}^{\text{lbs}}\!/_{\text{ton}})} \approx \underline{\underline{53.8\ \text{tons}}}$$

TABLE WITH RESULTS OF FORMULA COMPARISON

Pile Formula		CASE 1 12" PC/PS Concrete Pile		CASE 2 HP12x53 Steel Pile	
	Pile Length	80.0 ft	40.0 ft	80.0 ft	40.0 ft
GATES		111.0 tons	111.0 tons	111.0 tons	111.0 tons
ENR		88.9 tons	88.9 tons	88.9 tons	88.9 tons
JANBU		72.7 tons	91.5 tons	75.9 tons	92.7 tons
HILEY		64.6 tons	88.0 tons	120.5 tons	135.7 tons
PACIFIC COAST		42.9 tons	63.5 tons	53.8 tons	73.3 tons

EXAMPLE 1: CALCULATION OF MINIMUM HAMMER ENERGY

Given:

Hammer Data: Delmag D36-32
Hammer/Ram Weight = 7938 lbs
Manufacturer's Maximum Energy Rating = 83,880 ft·lbs

Nominal Resistance = 390 kips

CHECK: Hammer Energy per Standard Specification 49-1.05.

From the GATES Equation,

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

Rearranging for N:

$$N = \frac{10^{\left(R_u + 124 \big/ 1.83\sqrt{E_r} \right)}}{0.83}$$

$$N = \frac{10^{\left(390+124 \big/ 1.83\sqrt{83,880} \right)}}{0.83}$$

$$= \frac{10^{\left(514 \big/ 530 \right)}}{0.83}$$

$$= \frac{10^{0.9698}}{0.83}$$

$$= 11.23 \approx 11 \ \text{blows}/\text{ft}$$

$$s = \text{penetration per blow in inches}$$

$$= N^{-1}\left(12 \ \text{in}/\text{ft}\right)$$

$$= \left(11.23 \ \text{blows}/\text{ft}\right)^{-1}\left(12 \ \text{in}/\text{ft}\right)$$

$$= \underline{\underline{1.07 \ \text{in}/\text{blow} > 0.125 \ \text{in}/\text{blow}}}$$

$$\boxed{\therefore \text{Meets the energy requirements of S.S. 49-1.05.}}$$

EXAMPLE 2: CALCULATIONS FOR ESTABLISHING A BLOW COUNT CHART

Given:

Hammer Data: Delmag 36-32
Hammer/Ram Weight = 7938 lbs
Maximum Stroke = 10.42 ft

Nominal Resistance = 390 kips

ASSUMPTION(S): $E_r = \text{Ram Weight} \times \text{Observed Field Drop Height}$
$\text{Observed Field Drop Height} = 6\text{ ft}$

From the GATES Equation,

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

Rearranging to solve for N:

$$N = \frac{10^{\left(\frac{R_u + 124}{1.83\sqrt{E_r}} \right)}}{0.83} \qquad E_r = 6\text{ ft}(7938\text{ lbs}) = 47{,}628\text{ ft}\cdot\text{lbs}$$

$$N = \frac{10^{\left(\frac{390+124}{1.83\sqrt{47{,}628}} \right)}}{0.83} = \frac{10^{\left(\frac{514}{399} \right)}}{0.83} = \frac{10^{1.287}}{0.83} = \underline{\underline{23.33 \approx 23 \text{ blows/ft}}}$$

Calculations for the chart data are completed by using the Excel spreadsheet, *PileEquation-Gates.xls (updated 7/26/2007)*, downloaded from the OSC Intranet website. See next page for calculation results of the spreadsheet.

PILE DRIVING (US CUSTOMARY)

Bridge/Structure No.:	09-8765	Project No.	04-123456
Bridge/Structure Name:	Sample Rd UC (widen)		
Abut/Bent/RW/SW No.:	Abut 1		

Blue font = input/changeable data

Hammer Data

Hammer Type:		Delmag 36-32
Hammer Weight (lbs)	=	7938 lbs
Nominal Resistance† (kips)	=	390 kips

GATES Formula

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

—OR—

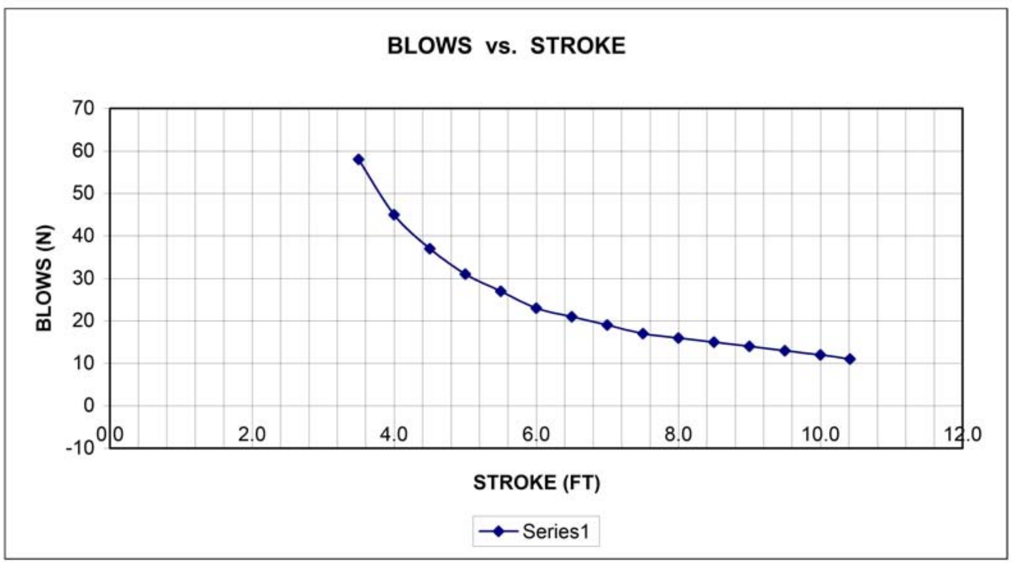
$$N = \frac{10^{\left(R_u+124 / 1.83\sqrt{E_r}\right)}}{0.83}$$

where: R_u = nominal resistance of pile (kips)
E_r = energy developed by hammer at observed stroke (ft·lbs)
N = number of hammer blows in the last foot (max. ≤ 100)

E_r = WH

where: W = hammer weight (lbs)
H = stroke or drop height of hammer (ft)

(H) Stroke (feet)	(E_r) Energy (ft-lbs)	(N) Blows (per foot)	
	-	-	
	-	-	
10.4	82,690	11	
10.0	79,380	12	
9.5	75,411	13	
9.0	71,442	14	
8.5	67,473	15	
8.0	63,504	16	
7.5	59,535	17	
7.0	55,566	19	
6.5	51,597	21	
6.0	47,628	23	
5.5	43,659	27	
5.0	39,690	31	
4.5	35,721	37	
4.0	31,752	45	
3.5	27,783	58	
	-	-	
	-	-	
	-	-	
	-	-	
	-	-	
	-	-	
	-	-	



†S.S. 49-1.08: *When the pile nominal resistance is omitted from the plans or the special provisions, timber piles shall be driven to a nominal resistance of 180 kips, and steel and concrete piles shall be driven to a nominal resitance of 280 kips. (May 2006)*

Date printed: 3/4/2008

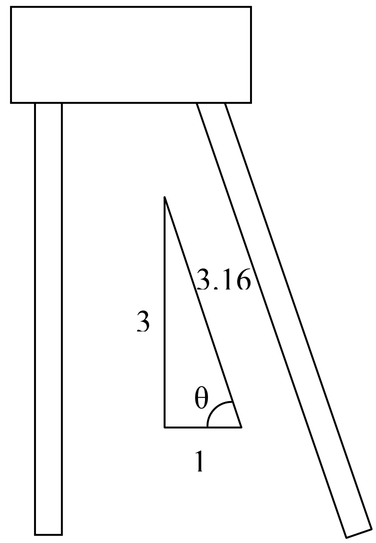
EXAMPLE 3: BATTERED PILE BLOW COUNT CHART

Given:

Hammer Data: Delmag 36-32
Hammer/Ram Weight = 7938 lbs
Maximum Stroke = 10.42 ft

Nominal Resistance = 390 kips

Battered pile: 1:3



ASSUMPTION(S): $E_r = \text{Ram Weight} \times \text{Observed Field Drop Height}$

$\text{Observed Field Drop Height} = 9\text{ ft}$

As in the previous example, rearranging the Gates Formula gives,

$$N = \frac{10^{\left(\frac{R_u + 124}{1.83\sqrt{E_r}}\right)}}{0.83} = \frac{10^{\left(\frac{390+124}{1.83\sqrt{67{,}775.8}}\right)}}{0.83} = \frac{10^{\left(\frac{514}{476}\right)}}{0.83} = \frac{10^{1.0798}}{0.83} = 14.48 \approx \underline{\underline{14\ \text{blows/ft}}}$$

$$\theta = \sin^{-1}\left(\frac{3}{3.16}\right) = 71.565°$$

$$E_r = 7938\text{ lbs}\left(9\text{ ft} \times \sin 71.565°\right) = 67{,}775.8\text{ ft}\cdot\text{lbs}$$

Calculations for the chart data are completed by using a MODIFIED version of the Excel spreadsheet, *PileEquation-Gates.xls (updated 7/26/2007)*. See next page.

PILE DRIVING (US CUSTOMARY)

Bridge/Structure No.: 09-8765 Project No. 04-123456
Bridge/Structure Name: Sample Rd UC (widen)
Abut/Bent/RW/SW No.: Abut 1

Blue font = input/changeable data

Hammer Data

Hammer Type:		Delmag 36-32
Hammer Weight (lbs)	=	7938 lbs
Nominal Resistance[†] (kips)	=	390 kips
Pile Battered?(Y/N)	Y	H: 1 V: 3

GATES Formula

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

—OR—

$$N = \frac{10^{\left(\frac{R_u+124}{1.83\sqrt{E_r}}\right)}}{0.83}$$

where: R_u = nominal resistance of pile (kips)
E_r = energy developed by hammer at observed stroke (ft-lbs)
N = number of hammer blows in the last foot (max. ≤ 100)

E_r = WH

where: W = hammer weight (lbs)
H = stroke or drop height of hammer (ft)

(H) Stroke (feet)	(E_r) Energy (ft-lbs)	(N) Blows (per foot)	
	-	-	
	-	-	
10.4	78,469	12	
10.0	75,306	13	
9.5	71,541	14	
9.0	67,776	14	
8.5	64,011	16	
8.0	60,245	17	
7.5	56,480	18	
7.0	52,715	20	
6.5	48,949	22	
6.0	45,184	25	
5.5	41,419	29	
5.0	37,653	34	
4.5	33,888	40	
4.0	30,123	50	
3.5	26,357	65	
	-	-	
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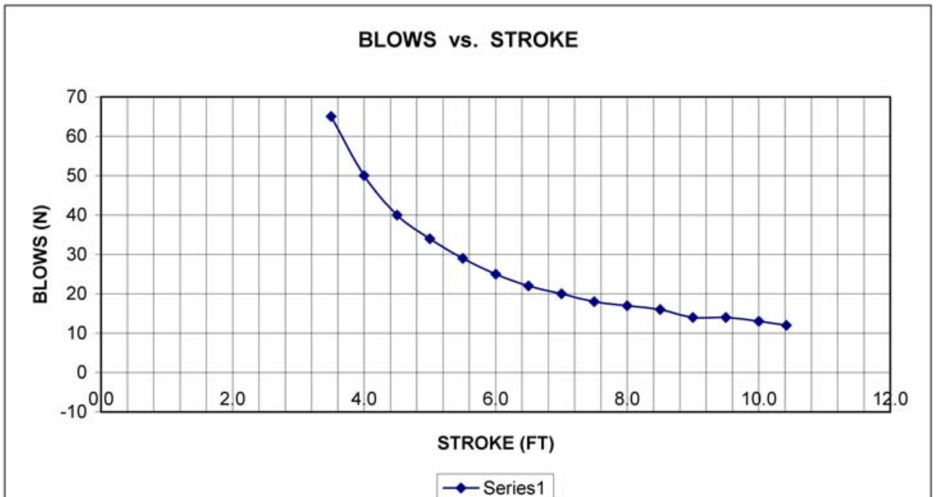
	-	-	
	-	-	



[†]S.S. 49-1.08: When the pile nominal resistance is omitted from the plans or the special provisions, timber piles shall be driven to a nominal resistance of 180 kips, and steel and concrete piles shall be driven to a nominal resitance of 280 kips. (May 2006)

Date printed: 3/5/2008 RMG

EXAMPLE 4: CALCULATIONS FOR PILES WITH DOWNDRAG

<u>The following metric example has downdrag:</u>
(Example submitted by Joy Cheung, P.E., and Anh Luu, P.E.)

Island Parkway Overcrossing – Rte 101/Ralston Interchange
EA 04-256804, Oversight Project

Pile Data Table

Location	Pile Type 360 = 15.5"	Design Loading	Nominal Resistance		Design Tip Elevation	Specified Tip Elevation
			Compression	Tension		
Abut 1	900C Alt "X"	625 kN	1250 kN	0 kN	−24.2(1), −8.8(2)	−24.2
Bent 2	900C Alt "X"	625 kN	1250 kN	400 kN	−23.3(1), −11.2(2), −19.9(3)	−23.3
Bent 3	460 mm Square = 18"	400 kN	800 kN	0 kN	−18.35(1)	−18.35
Bent 4	460 mm Square	425 kN	850 kN	0 kN	−18.7(1)	−18.7
Bent 5	460 mm Square	425 kN	850 kN	0 kN	−18.7(1), −16.3(2)	−18.7
Bent 6	460 mm Square	425 kN	850 kN	0 kN	−18.7(1)	−18.7
Bent 7	460 mm Square	425 kN	850 kN	0 kN	−18.25(1)	−18.25
Bent 8	460 mm Square	425 kN	850 kN	0 kN	−18.25(1)	−18.25
Bent 9	460 mm Square	425 kN	850 kN	0 kN	−18.4(1)	−18.4
Bent 10	460 mm Square	400 kN	800 kN	0 kN	−18.4(1), −12.0(2)	−18.4
Abut 11	900C Alt "X"	400 kN	800 kN	0 kN	−19.9(1), −6.2(2)	−19.9

Design tip elevation is controlled by the following demands: (1)Compression, (2)Lateral, (3)Tension
The estimated downdrag load is 295 kN at abutment 1; 242 kN per pile at bent 2; 377 kN per pile at Bent 3; 243 kN per pile at bent 4, 5 and 6; 351 kN per pile at bent 7 and 8; 329 kN per pile at bent 9 and 10; and 367 kN per pile at abutment 11. For use of Gates formular, Ultimate pile capacity = Nominal resistance + 2xdowndrag load.

The Pile Data Table from the contract plans show:
Bent 2 Piles – Class 900C Alt "X" (Pile Data Table)
Nominal Resistance (Compression) = 1250 KN
Estimate Down Drag Load = 242 KN
Ultimate Pile Capacity = R_u = Nominal resistance + 2 x downdrag

Therefore:

R_u = Nominal resistance + 2 x downdrag
R_u = 1250 KN + (2 * 242KN) = 1734 KN

Contractor's proposed hammer:
<u>DELMAG D36-32</u>

Pile Hammer Data - (per specs, Contractor provides data)
Also see BCM 130-3.0 dated June 14, 2007.
Internet: www.pileco.com, www.hmc-us.com, …etc;

Pile hammer data:
Max Energy Output = 83880 ft.lbs = 83880 * 1.3558 = 113724.5 Joules
Piston Weight/Ram Weight = mass = 7938 lbs = 3600.6 kg
Maximum obtainable stroke/Piston Drop = height = 10'5" = 3.18 m
Find:

E_r = Energy rating of hammer at observed field drop height in Joules

> **It is generally accepted that the energy output of an open-end diesel hammer is equal to the ram weight times the length of stroke.

Gravitational potential energy = mass × free-fall acceleration × height = $m \cdot g \cdot H = E_r$

E_r = 3600.6 kg · 9.81 · 3.18 = 112,323 Joules < 113,724 (Max Energy)

> ** For battered pile, $E_r = m \cdot g(H \cdot \sin\theta)$

N = Number of blows per 300 millimeters (maximum of 100)

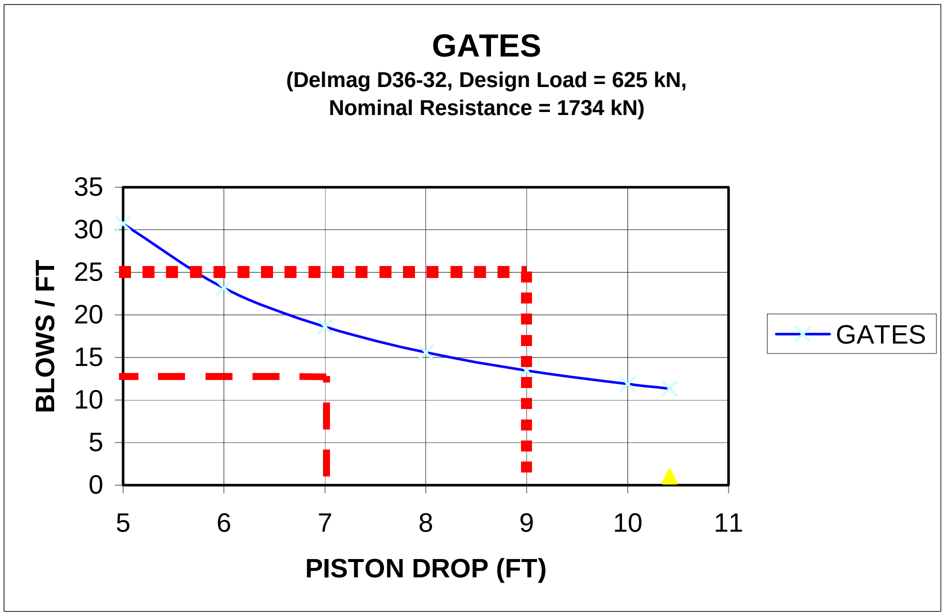
$$N = \frac{10^{\left(R_u + 550 \big/ 7\sqrt{E_r} \right)}}{0.83}$$

Set up table:

Hammer Type:	Delmag D 36-32
Design Load:	625kN
Nominal Resistance:	1734kN
Max Energy	113724Joules
Piston Wt	3600.6Kg

PISTON DROP (ft)	PISTON DROP (m)	ENERGY (joules)	Blows Per Last 300 mm. GATES
10.417	3.18	112151	11
10	3.05	107661	12
9	2.74	96895	13
8	2.44	86129	16
7	2.13	75363	19
6	1.83	64597	23
5	1.52	53831	31
4	1.22	43064	45
3	0.91	32298	79

Set up graph:



Meets Bearing Value
NOT GOOD

A very good spreadsheet (PileEquation-Gates.xls) used to calculate blows per foot using the Gates equation can be found on the OSC Intranet Homepage under, "Downloads/Forms". This spreadsheet was updated on 7/26/07.

Continue calculations:

Standard Specifications - Section 49-1.05 --Impact Hammer Minimum Energy "not less 3mm/blow at the specified bearing value…"

Use the Gates formula again…

$$R_u = (7 * (E_r)^{1/2} * \log_{10}(0.83 * N)) - 550$$

Find N.

Using E_r = 3600.6 kg * 9.81 * 3.18 = 112,323 Joules

R_u = 1250 KN + (2 * 242KN) = 1734 KN

N = 11 blows/ 300 mm

s = Penetration per blow in millimeters

= 300 mm/11 blows

≈ 27.0 mm > 3 mm **OK**.

Note: An upper limit is not specified for the Contractor to furnish an approved hammer having sufficient energy to drive piles at a penetration rate of not less than 1/8-inch per blow at the required bearing value.

End of example.

EXAMPLE 5: ESTIMATE HAMMER STROKE OF A SINGLE ACTING HAMMER

Given:

Hammer Data: Delmag 36-32
Hammer/Ram Weight = 7938 lbs
Maximum Stroke = 10.42 ft

From Field Observations: Ram Blows per Minute (bpm) = 43

From the SAXIMETER Formula,

$$H = 4.01\left(\frac{60}{\text{bpm}}\right)^2 - 0.3$$

$$H = \text{hammer stroke in feet}$$

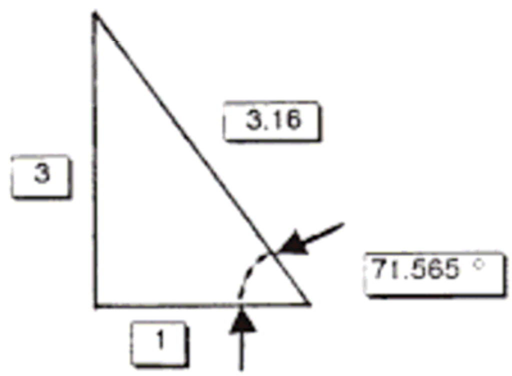
$$\text{bpm} = \text{field observation of hammer blows per minute}$$

$$\begin{aligned} H &= 4.01\left(\frac{60}{\text{bpm}}\right)^2 - 0.3 \\ &= 4.01\left(\frac{60}{43\ \text{bpm}}\right)^2 - 0.3 \\ &= 7.81 - 0.3 \\ &= 7.51 \approx \underline{\underline{7.5\ \text{ft}}} \end{aligned}$$

Example Battered Pile Blow Count Chart

BATTERED PILE

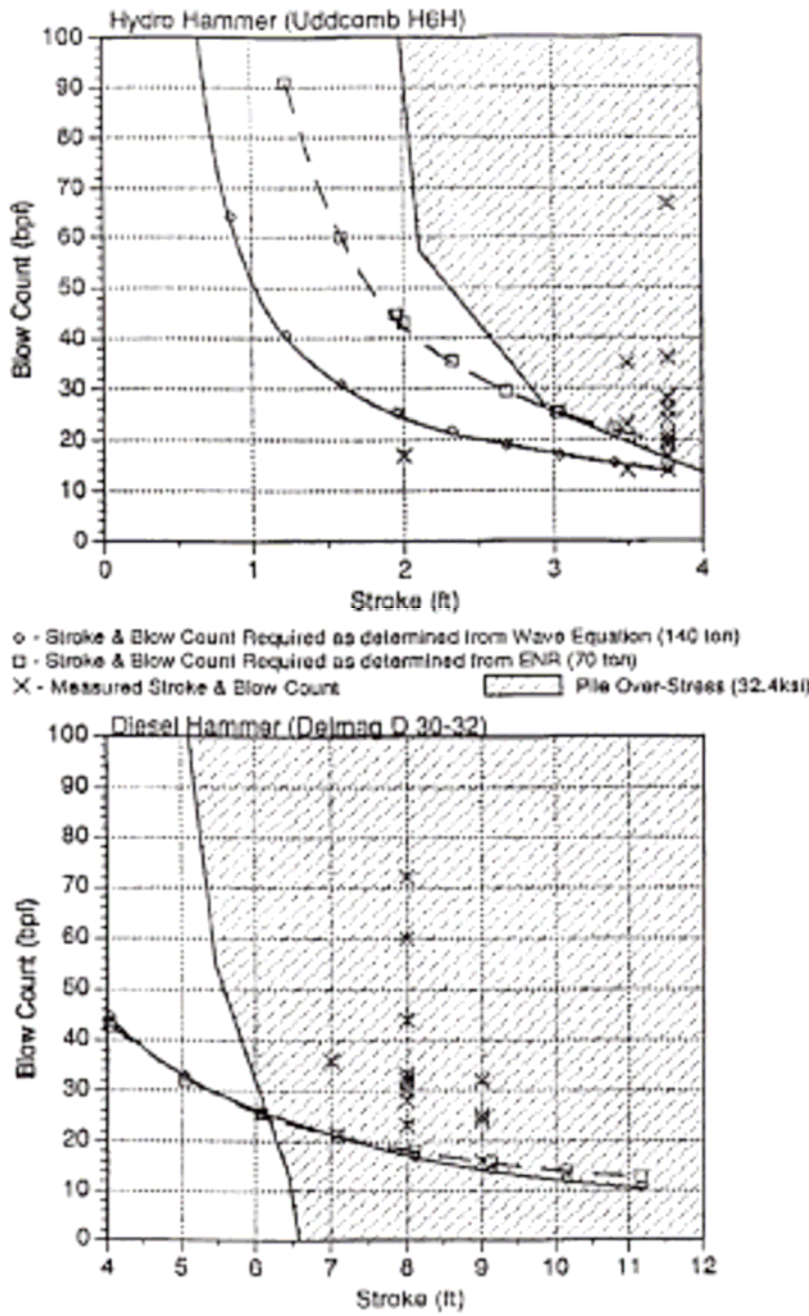
PILE CAPACITY 140000 POUNDS
HAMMER D 30-23
PISTON WEIGHT 6,600 POUNDS



$$E = W * H * \sin 71.565^{\circ}$$

STROKE FEET	BLOW S PER FOOT
10	15.0
9.5	15.9
9	16.9
8.5	18.0
8	19.3
7.5	20.8
7	22.6
6.5	24.6
6	27.1
5.5	30.2
5	34.0

Example Wave Equation Field Acceptance Charts



Field Acceptance Charts for Hydro and Diesel Hammers using both Wave Equation Analysis and ENR formula

(HP12x84, L=32'±, Predrill = 22', Very Dense Boulders & Cobbles in Sand Matrix at tip, N>70)

APPENDIX

F Static Pile Load Testing and Dynamic Monitoring

Table of Contents

Fax to: *Office of Geotechnical Support*
Foundation Testing Branch
Attn: Brian Liebich, P.E.

Fax #: (916) 227-1083
Phone #: (916) 227-1000

Pile Load Test (PLT) Request Form

Project Name:	Request By:
Dist/Co/Rte/PM:	Office Phone #: ( )
EA No. & Activity Code:	Fax #: ( )
Bridge No:	Pager #: ( )
Date of Request:	Cell Phone #: ( )

Abutment or Bent Location	Pile Numbers	Pile Diameter	Max. Test Load	Date Ready for Testing	Status
					☐ Estimated Date ☐ Actual Date
					☐ Estimated Date ☐ Actual Date
					☐ Estimated Date ☐ Actual Date

Type of Pile Load Test to be Performed

(check all that apply)

☐ Compression ☐ Tension ☐ Lateral

Nature of Pile Load Test

☐ Required by Specifications
☐ Pile Load Test Indicator Program
☐ Emergency Construction Capacity Verification

Have the test pile and anchor pile load test connections already been installed? Yes ☐ / No ☐

Please submit forms to the above FAX number by 11:00 am Friday for testing to be scheduled during the following week.
Please update all Estimated dates as dates can be confirmed or as piles become ready for testing.

For individuals with sensory disabilities, this document can be made available in Braille, large print, audiocassette or computer disk upon request. To obtain one of these alternate formats, please call (916) 227-8185 or TTY 711 or write to the EEO Officer, Division of Engineering Services, P.O. Box 168041, Mail Stop 9 Room 509, Sacramento, CA 95816-8041.

Fax to: *Office of Geotechnical Support*
Foundation Testing & Instrumentation Branch
Attn: Brian Liebich. P.E.

Fax #: (916) 227-1083
Phone #: (916) 227-1000

Pile Dynamic Analysis (PDA) Test Request Form

Project Name:	Request By:
Dist/Co/Rte/PM:	Office Phone #: ( )
EA No. & Activity Code:	Fax #: ( )
Bridge No:	Pager #: ( )
Date of Request:	Cell Phone #: ( )

Abutment or Bent Location	Pile Numbers	Pile Diameter	Pile Length	Date Ready for Testing	Status
					☐Estimated Date ☐Actual Date
					☐Estimated Date ☐Actual Date
					☐Estimated Date ☐Actual Date
					☐Estimated Date ☐Actual Date

This work to be performed for the Purpose of:

☐**Dynamic Monitoring Only** ☐**Field Acceptance Criteria**

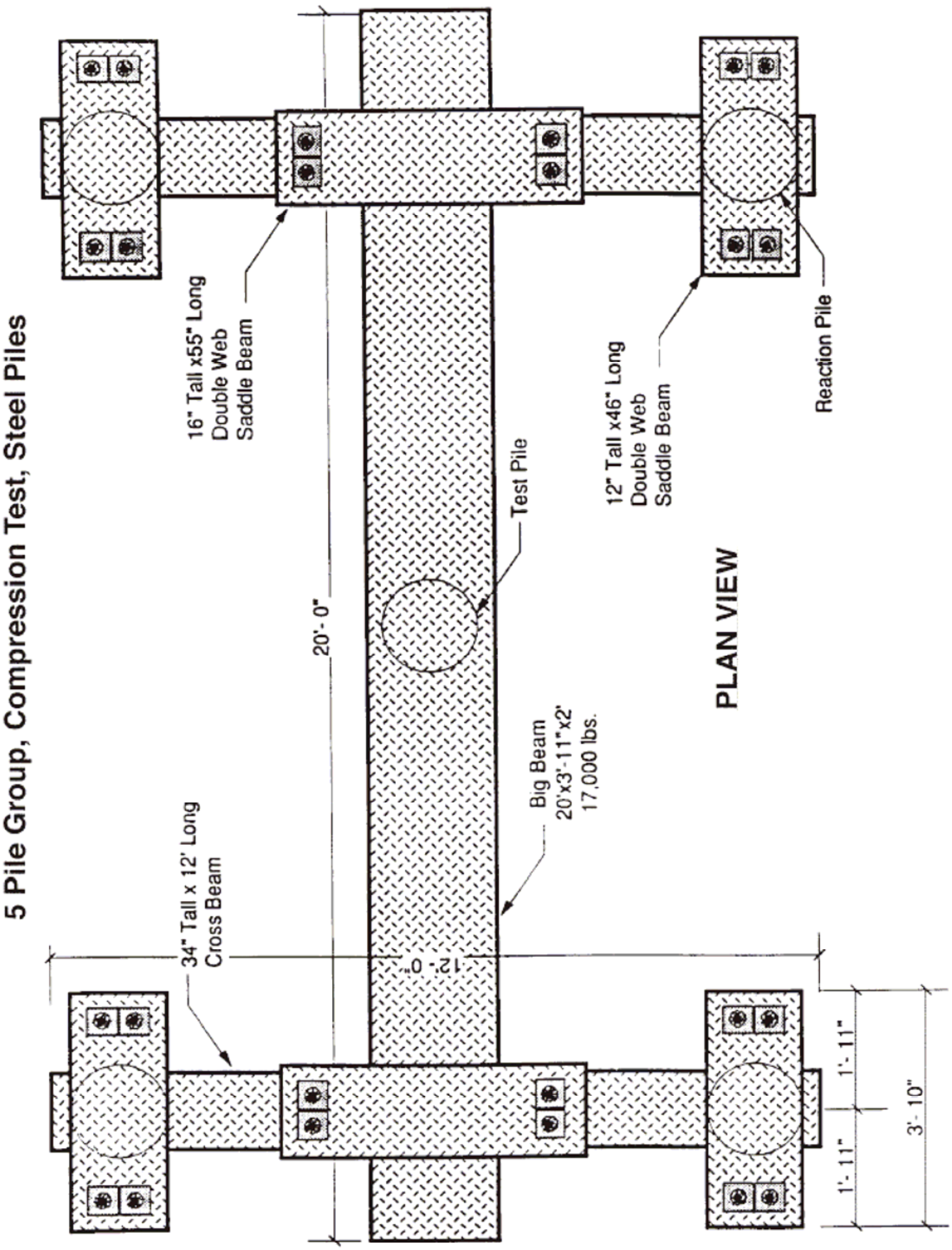
☐**Hammer Energy Verification** ☐**SPT Hammer Efficiency**

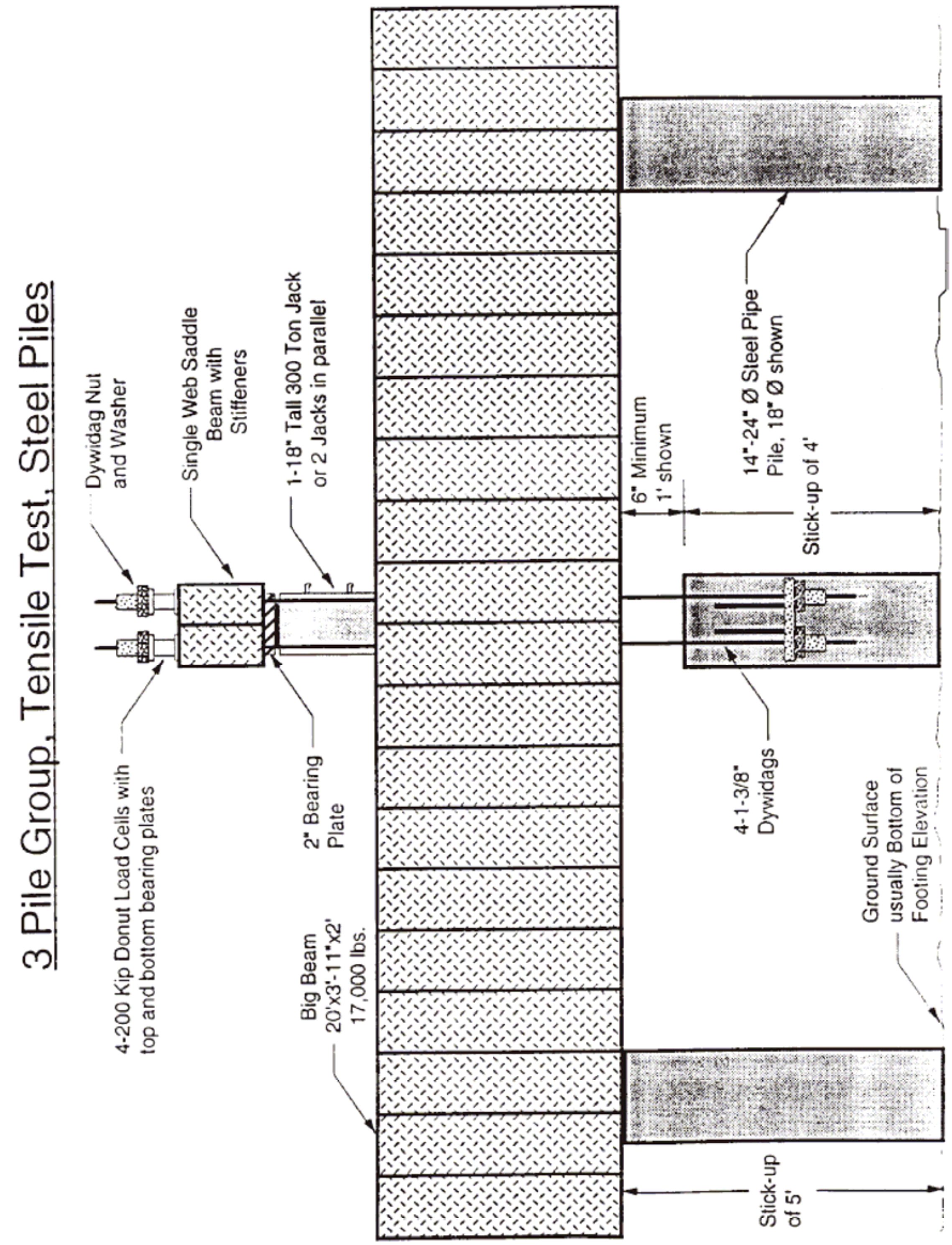
Have the PDA bolt holes already been installed? Yes☐ / No☐

Comments:

Please submit forms to the above FAX number by 11:00 am Friday for testing to be scheduled during the following week.
Please update all Estimated dates as dates can be confirmed or as piles become ready for testing.

For individuals with sensory disabilities, this document can be made available in Braille, large print, audiocassette or computer disk upon request. To obtain one of these alternate formats, please call (916) 227-8185 or TTY 711 or write to the EEO Officer, Division of Engineering Services, P.O. Box 168041, Mail Stop 9 Room 509, Sacramento, CA 95816-8041.


5 Pile Group, Compression Test, Steel Piles
16" Tall x55" Long
Double Web
Saddle Beam
Test Pile
12" Tall x46" Long
Double Web
Saddle Beam
Reaction Pile
PLAN VIEW
Big Beam
20'x3'-11"x2'
17,000 lbs.
34" Tall x 12' Long
Cross Beam
20'- 0"
12'- 0"
1'- 11"
1'- 11"
3'- 10"


3 Pile Group, Tensile Test, Steel Piles
Dywidag Nut and Washer
Single Web Saddle Beam with Stiffeners
1-18" Tall 300 Ton Jack or 2 Jacks in parallel
4-200 Kip Donut Load Cells with top and bottom bearing plates
2" Bearing Plate
Big Beam 20'x3'-11"x2' 17,000 lbs.
6" Minimum 1' shown
14"-24" Ø Steel Pipe Pile, 18" Ø shown
Stick-up of 4'
4-1-3/8" Dywidags
Ground Surface usually Bottom of Footing Elevation
Stick-up of 5'

State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: RENE HACHEY
STRUCTURE REPRESENTATIVE
CENTRAL VIADUCT (REPLACE)

Date: January 12, 2004

File: 04-SF-101-R8.0/R8.5
04–291004
Central Viaduct
(Replace)
Br. No. 34–0077

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES – MS-5

Subject: Pile Load Test Results: Bent 6 Control Zone

Attached is a report summarizing the results of the Pile Load Test for Bent 6 Control Zone, which includes all the bents of the above-referenced project.

If you have any questions or comments regarding this report, please contact Youssef Awad, P.E. at (916) 227–5454 or Calnet 8–498–5454.

For: BRIAN LIEBICH, P.E.
Senior Transportation Engineer
Foundation Testing Branch

Attachments

c: D. Valls – SC
J. Bowman – OGDW
Y. P. Kim – SD
C. Ealy – FHWA
T. Shantz – DRI

YA/ya

"Caltrans improves mobility across California"

FOUNDATION TESTING BRANCH

January 12, 2004

04-SF-101-R8.0/R8.5
04 - 291004
Central Viaduct (Replace)
Bridge Number 34 - 0077

Pile Load Test Results: Bent 6 Control Zone

January 12, 2004

Project Information

04-SF-101-R8.0/R8.5
04-291004
Central Viaduct (Replace)
Bridge Number 34-0077

Subject

Pile Load Test Results: Test Pile at Bent 6 Control Zone

Introduction

This report presents the Pile Load Test results for the Test Pile at Bent 6 Control Zone, which is located between Bent 6 and Bent 7, of the above-referenced project. A static axial compressive load test was conducted on the 2.0-meter diameter Cast-In-Drilled-Hole (CIDH) concrete Test Pile utilizing four 2.0-meter diameter CIDH Anchor (reaction) Piles. The Pile Load Test was required per Section 10-1.49 "Piling" of the Special Provisions and was to be completed before the start of pile construction at all bent locations. The purpose of the test is to verify pile compressive load design capacity and efficacy of pile installation procedure. The Test Pile and Anchor Piles are not production piles and will not be incorporated within the bridge foundation. The Construction Plans indicate that the top of the Test Pile and Anchor Piles will be removed to 1.0 meter below the ground surface after the completion of the Pile Load Test. Project site location and Pile Load Test location maps are provided for reference in Appendix A.

Foundation Description

The Central Viaduct (Replace) project foundation was originally designed to include 1.8-meter and 2.1-meter diameter CIDH piles to be constructed using conventional methods. However, the

Contractor proposed to install the CIDH piles using oscillated casings with more convenient diameters of 2.0-meter and 2.2-meter instead of the original diameters of 1.8-meter and 2.1-meter, respectively. The larger diameter piles are to be installed to the originally specified tip elevations. According to the Construction Plans, the 1.8-meter diameter CIDH piles are 14.7 meters to 27.8 meters in length and require nominal compression resistances ranging from 13,850 kN to 26,250 kN. The Specified Tip Elevations for the 1.8-meter diameter CIDH piles range from Elevation −1.0 meters to Elevation −18.0 meters. The 2.1-meter diameter CIDH piles are 28.0 meters to 28.6 meters in length and require nominal compression resistances ranging from 29,100 kN to 32,650 kN. The Specified Tip Elevations for the 2.1-meter diameter CIDH piles were at Elevation −20.0 meters. The Test Pile was 2.0-meter diameter and approximately 31.4 meters in length, and the required nominal compression resistance was 26,250 kN. The Test Pile has a constructed top of pile elevation of +12.87 meters and an approximate constructed tip elevation of 18.50 meters while the ground surface was at an approximate elevation of +11.72 meters at the time of testing.

Subsurface Conditions

The subsurface conditions at the Test Pile location were inferred from Boring 01-6, advanced in March 2001. Boring 01-6 was located near the location of the Test Pile. Below an approximate ground surface elevation of +12.48 meters, Boring 01-6 encountered in general granular soils that consisted of alternating layers of medium dense to very dense Sand, clayey Sand and silty Sand; stiff to very stiff silty Clay, Clay and Gravely Clay; and very dense Gravel. At an approximate elevation of −32.5 meters, Boring 01-6 encountered hard to soft, intensely weathered, highly fractured, and sheared Serpentinite and Shale Bedrock (Franciscan Formation) that extended to the termination depth of Boring 01-6 at an approximate elevation of −34.5 meters. A copy of Boring 01-6 is included in Appendix A. Detailed descriptions of subsurface conditions of the project site are provided in the Log of Test Borings included in the Construction Plans.

Pile Installation Summary

The pile group utilized for the static axial compressive load test included the following: one 2.0-meter diameter CIDH concrete Test Pile and four 2.0-meter diameter CIDH concrete Anchor

Piles. The Test Pile was constructed to an approximate tip elevation of −18.5 meters. The top of Test Pile elevation was approximately +12.87 meters, with approximately 1.15 meters above ground surface elevation. A corrugated metal pipe approximately 2.0 meters in diameter was used to encase the concrete of the Test Pile and Anchor Piles above ground surface. The total length of the Test Pile was approximately 31.35 meters, of which approximately 30.2 meters of the pile were below the ground surface.

Personnel from the Foundation Testing Branch of the Office of Geotechnical Support conducted Gamma Gamma Logging (GGL) of the Test Pile and Anchor Piles. Please refer to the GGL report from this Office dated December 18, 2003. GGL was performed to determine the acceptability of the Test Pile and Anchor Piles for load testing. Although minor anomalies were detected by GGL in two of the Anchor Piles, a simple repair was completed prior to conducting the pile load test.

Static Axial Compressive Load Testing

Personnel from the Foundation Testing Branch (FTB) of the Office of Geotechnical Support conducted a static axial compressive load test on the Test Pile of the Central Viaduct (Replace) project. The load test was conducted on December 30, 2003. Test procedures used by FTB personnel were in general conformance with ASTM D 1143-81 and procedures developed by this Office. The load testing consisted of a five-pile group: one 2.0-meter Test Pile and four 2.0-meter diameter Anchor (reaction) Piles. The vertical movement was monitored at the Test Pile utilizing displacement transducers placed at four locations along the perimeter of the Test Pile at an approximately 0.25 meters below the top of the Test Pile. As such, the effective tested pile length was approximately 31.1 meters.

On the Test Pile, compression testing was conducted by applying load in increments and measuring the corresponding pile displacement. A total of six cycles of loading were applied and the test was terminated when the specified capacity of the loading apparatus (35,600 kN) was reached. At the maximum tested load of about 35,600 kN, the total displacement of the Test Pile was approximately 83 mm.

Table I summarizes the results of the static axial compressive load test. A plot depicting load-displacement behavior for the Test Pile can be found in Appendix B.

Table I. Static Axial Compressive Load Test Results:
Test Pile at Central Viaduct (Replace)

Pile Name	Test Pile
Pile Location	Between Bent 6 and Bent 7
Pile Size and Type	2.0-meter diameter CIDH
Test Date	December 30, 2003
Required Nominal Resistance (per Construction Plans)	26,250 kN (~ 5900 kips)
Measured Displacement at Nominal Resistance (26,250 kN)	~ 41 mm
Measured Displacement at Design Load (13,125 kN)	~ 10 mm
Applied Load at 12.7 mm (0.5 inches) Displacement	~ 15,000 kN (~ 3370 kips)
Davisson's Failure Criterion	~ 27 mm at 21,300 kN
Maximum Tested Load	35,600 kN (~ 8000 kips)
Measured Maximum Displacement	~ 83 mm
Measured Permanent Set	~ 73 mm

Discussion

At the required nominal resistance of 26,250 kN, the total displacement of the Test Pile was approximately **41** mm. At the design load of 13,125 kN, the measured displacement was approximately 10 mm. At Caltrans 12.7 mm (0.5 inches) displacement criterion, the applied load was approximately 15,000 kN. At the maximum tested load of about 35,600 kN, the total displacement of the Test Pile was approximately 83 mm with a measured permanent set of approximately 73 mm. The Davisson's failure criterion for the Test Pile is approximately 27 mm of displacement at 21,300 kN load.

Mr. John Bowman, Project Engineering Geologist of the Office of Geotechnical Design West, was consulted to evaluate the potential of revising the specified tip elevations for piles within the control zone associated with the Test Pile, which include all bent locations of the Central Viaduct (Replace) project. Mr. Bowman determined that the pile tip elevations are to remain at the specified tip elevations indicated in the Construction Plans for all the bent locations. He also indicated that pile displacement at the specified nominal resistance was within tolerable limits.

Conclusions and Recommendations

Based upon the results of the Pile Load Test and consultation with Mr. Bowman, Project Engineering Geologist of the Office of Geotechnical Design West, this Office recommends that all CIDH piles at all bent locations of the Central Viaduct (Replace) project be conditionally released for construction. This release is subject to replacing the specified 1.8-meter and 2.1-meter pile diameters by the larger Contractor-proposed 2.0-meter and 2.2-meter diameter CIDH piles, respectively, without modification to pile specified tip elevations.

The Pile Load Test results and recommendations herein are only valid for the construction of the Contractor-proposed 2.0-meter and 2.2-meter diameter CIDH piles, as appropriate, where piles are installed to the specified tip elevations using pile installation procedures that are similar to those employed at the Test Pile.

If you have any questions or comments regarding this report, please call Youssef M. Awad at (916) 227–5454 or Calnet 8–498-5454.

REGISTERED PROFESSIONAL ENGINEER
YOUSSEF M. AWAD
No. C 62322
Exp. 09/2005
CIVIL
STATE OF CALIFORNIA

01/12/2004

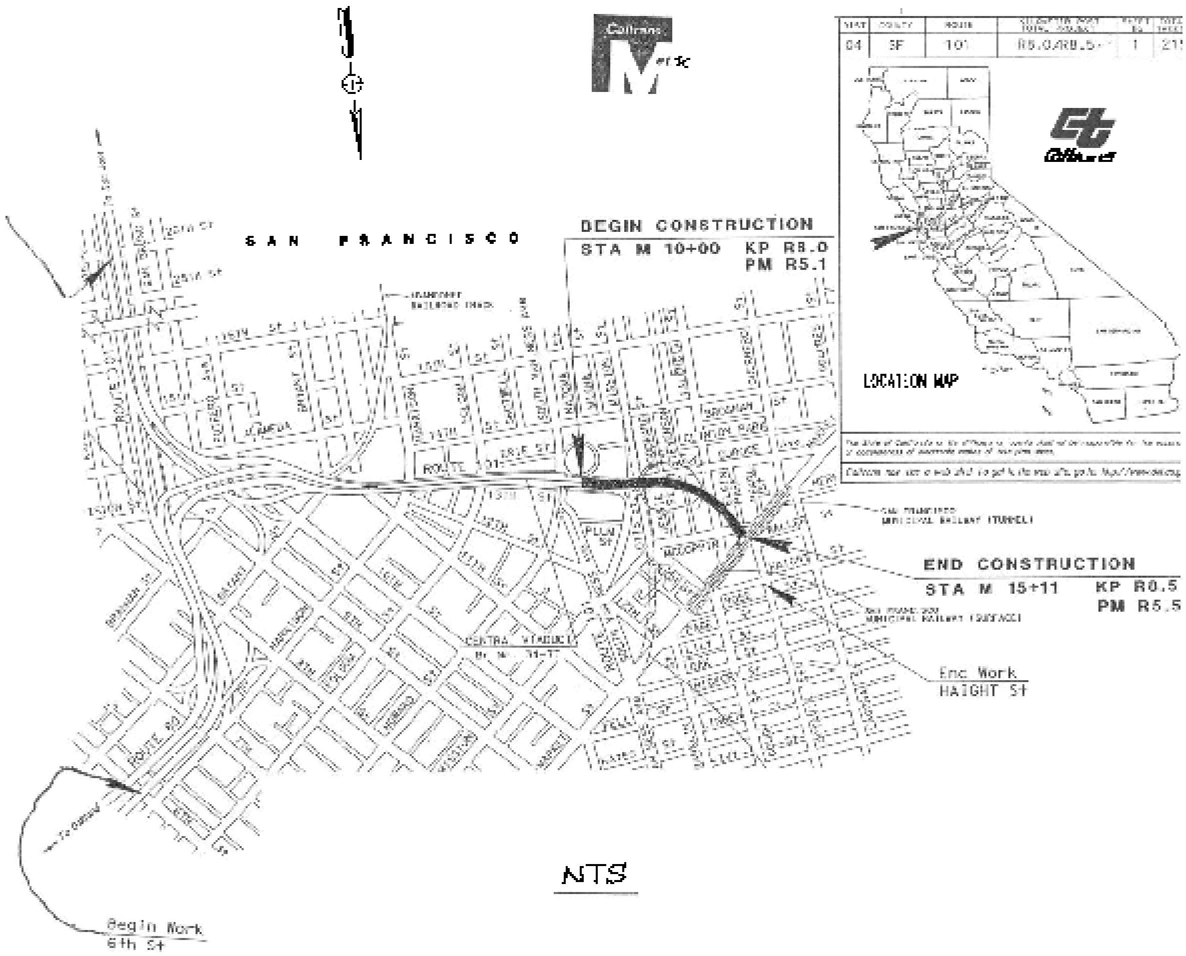
YOUSSEF M. AWAD, P.E.
Transportation Engineer, Civil
Foundation Testing Branch
Office of Geotechnical Support

Caltrans

APPENDIX A

Location Maps and Geologic Profile

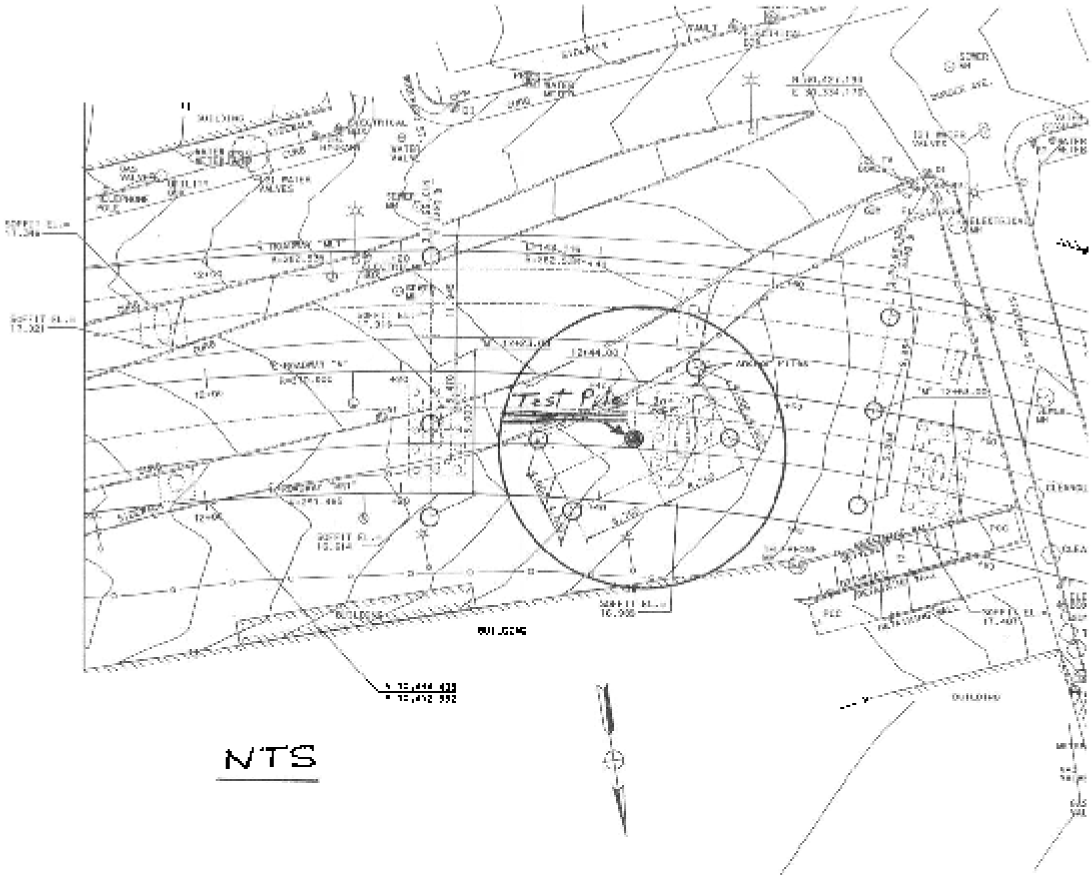
Foundation Testing Branch



GENERAL SITE LOCATION MAP
Central Viaduct (Replace)

Bridge Number 34–0077
04–SF–101–R8.0/R8.5
EA Number 04–291004

Foundation Testing Branch

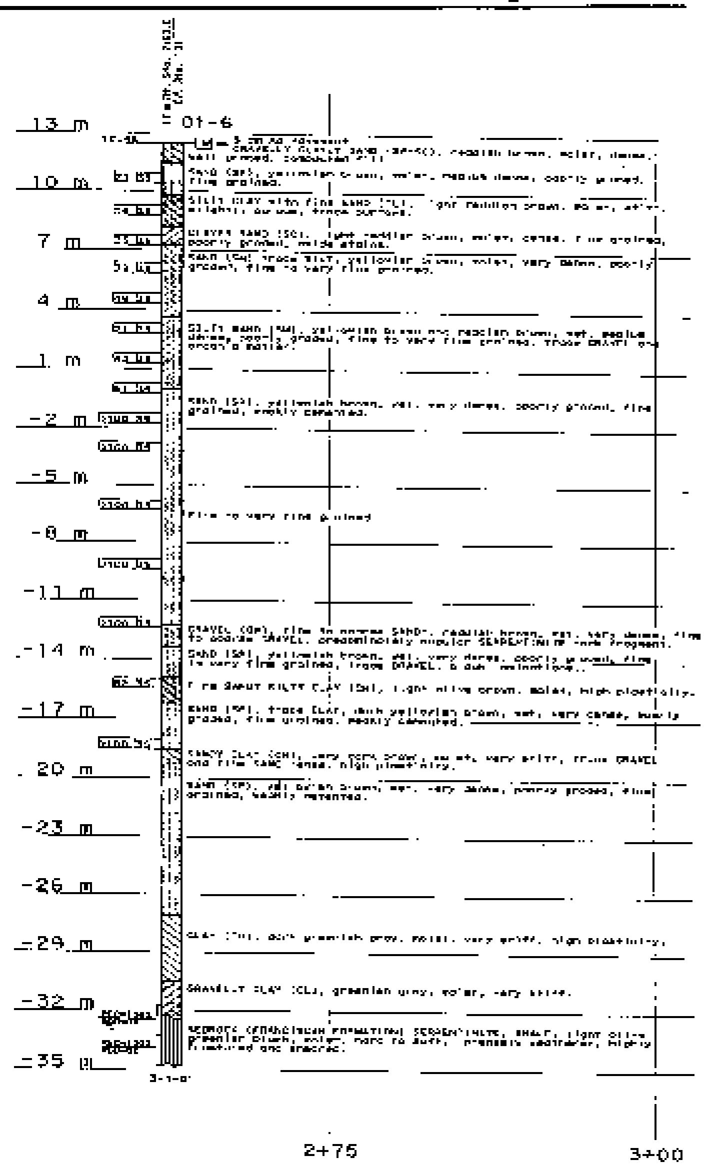


PILE LOAD TEST LOCATION MAP
Central Viaduct (Replace)
Test Pile at Bent 6 Control Zone (Between Bent 6 and Bent 7)

Bridge Number 34–0077
04–SF–101–R8.0/R8.5
EA Number 04-291004

Pile Type: 2.0-meter diameter CIDH
Date of Compression Test: 12/30/2003

Foundation Testing Branch



NTS

GEOLOGIC PROFILE
Central Viaduct (Replace)
Boring 01–6

Bridge Number 34–0077
04–SF–101–R8.0/R8.5
EA Number 04–291004

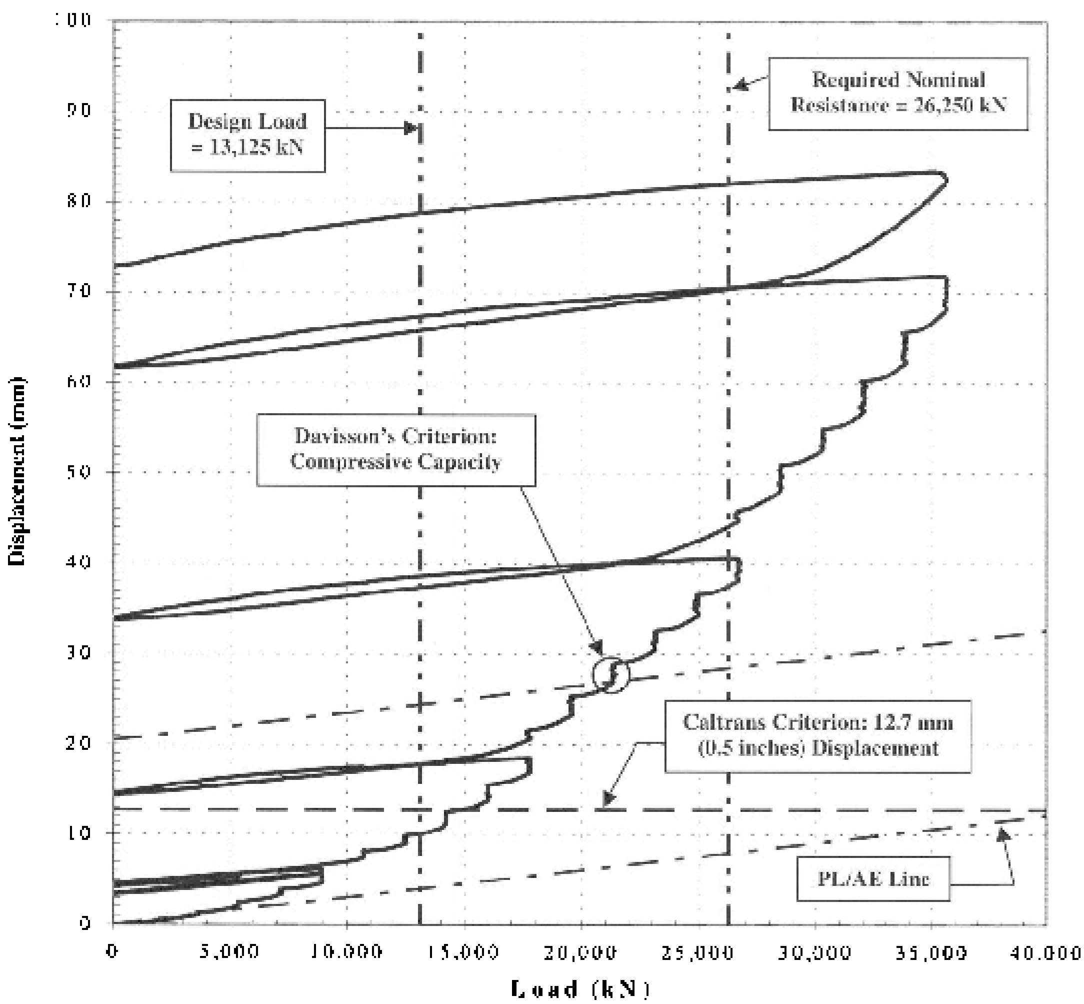
Test Pile at Bent 6 Control Zone
Pile Type: 2.0-meter diameter CIDH
Date of Compression Test: 12/30/2003

Foundation Testing Branch

APPENDIX B

Pile Load Test
Load Displacement Behavior

Foundation Testing Branch



LOAD DISPLACEMENT BEHAVIOR
Central Viaduct (Replace)
Pile Load Test at Bent 6 Control Zone

Bridge Number 34-0077
04-SF-101-R8.0/R8.5
Effective Tested Pile Length = 31.1 m

EA Number 04-291004
Pile Type: 2.0-meter diameter CIDH
Constructed Tip Elevation = -18.5 m
Date of Compression Test: 12/30/2003

Caltrans

State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: EHAB A. WAHED, P.E.
Structure Representative

Date: July 13, 2007

File: 10–SJ–5–R22.4/R25.1
10–3A1204
San Joaquin River Bridge (Widen)
Bridge Number 29-0252R

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES – MS-5

Subject: Pile Field Acceptance Criteria and Pile Dynamic Analysis: Test Pile at Pier 2 (Pile No. 3)

The attached report contains the Pile Field Acceptance Criteria and the results of Pile Dynamic Analysis for the Test Pile at Pier 2 (Pile No. 3) of the above-referenced project.

If you have any questions or comments regarding this report, please contact Jason Wahleithner, P.E. at (916) 227–5509.

BRIAN LIEBICH, P.E.
Senior Transportation Engineer
Foundation Testing Branch

Attachments

c: D. Valls – SC (email)
F. Hoffman – SD (email)
R. Buell – OGDN (email)

JDW/jdw

"Caltrans improves mobility across California"

FOUNDATION TESTING BRANCH

July 13, 2007

10-SJ-5-R22.4/R25.1
10-3A1204
San Joaquin River Bridge (Widen)
Bridge Number 29-0252R

Pile Field Acceptance Criteria and Pile Dynamic Analysis:
Test Pile at Pier 2 (Pile No. 3)

Foundation Testing Branch

July 13, 2007

Project Information

10-SJ-5-R22.4/R25.1
10-3A1204
San Joaquin River Bridge (Widen)
Bridge Number 29-0252R

Subject

Pile Field Acceptance Criteria and Pile Dynamic Analysis: Test Pile at Pier 2 (Pile No. 3)

Introduction

This report presents dynamic monitoring results for the first pile driven within the control location identified as Piers 2 through 5 of the San Joaquin River Bridge over Route 5, Bridge No. 29-0252R. The subject pile, designated as the Test Pile, was driven at the location of a production pile at Pier 2. The exact location of the Test Pile within the Pier 2 footing is shown on Foundation Plan No. 1 included in Appendix A. Bearing acceptance criteria curves for the Test Pile were developed from the results of the dynamic monitoring and are included for use as field acceptance criteria in accordance with Section 10-1.39 "PILING" of the Special Provisions.

Foundation Description

The San Joaquin River Bridge (Widen) project includes installation by impact hammer of PP 610x13-mm open-ended steel pipe piles at all pier locations. The Pier 2 and Pier 3 piles have a specified tip elevation of –22.50 meters that is controlled by compression demands. The Pier 4 and Pier 5 piles have specified tip elevations of –22.50 meters and –18.00 meters, respectively, that are controlled by liquefaction. Pile data specified in the Contract Plans are shown below in Table I.

Table I. Pile Data: PP 610x13-mm Open-Ended Steel Pipe Pile

Bridge Number	Support Location	Nominal Compression Resistance (kN)	Nominal Tension Resistance (kN)	Bottom of Footing Elevation (m)	Specified Tip Elev. (m)	Approximate Plan Pile Length*** (m)	Analyzed Pile Length* (m)
29-0252R	Pier 2**	1,800	900	–3.94	–22.50	18.69	16.8***
	Pier 3	1,800	900	–3.97	–22.50	18.66	
	Pier 4	1,800	900	–4.58	–22.50	18.05	
	Pier 5	1,800	900	2.07	18.00	20.20	

* Information according to the Contractor's Pile Driving Analysis submittal.
** Location of Test Pile.
*** Approximate Plan Pile Length includes 125 mm embedment into Pier Footing.
**** Due to Liquefaction potential, the nominal resistance value used for pile acceptance criteria shall be 1823 kN at Pier 4 and 2313 kN at Pier 5.

Subsurface Conditions

According to the As-Built Log of Test Borings (LOTB) dated November 17, 1969 and attached to the Contract Plans for Bridge Number 29-0252R, the San Joaquin River Bridge (Widen) site is underlain by alluvium consisting of loose to dense sand and medium dense silt underlain by stiff clay. The LOTB indicates that the specified tip elevation of –22.500 meters for the Test Pile is within the stiff clay layer.

Pile Installation

The Test Pile consisted of a 610-mm diameter open-ended steel pipe pile with a 13-mm wall thickness. The steel pipe material was spiral welded ASTM A252 Grade 3 material with a minimum yield stress of 310 Mpa (45 ksi). The overall length of the Test Pile was 27.44 m and consisted of two separate pipe pieces joined with a field welded splice at a distance of 7.32 m below the top of the pile. The overall pile length at Pier 2, determined from the Contract Plans, is 18.69 meters. The Test Pile was fabricated with an additional length of 8.76 meters in order to prevent submerging the PDA gages below the water within the Pier 2 cofferdam.

The driving system utilized for the Test Pile consisted of a Delmag D30-32 single-acting diesel hammer with an approximate rated energy of 94.70 kN-m, a maximum stroke of 3.22 meters, and a ram weight of 29.37 kN. The driving system was previously reviewed and approved by this Office in a report dated March 22, 2007. The driving system for the Test Pile included a steel pipe follower measuring 7.00 meters in length, 610 mm in diameter, and 13.7 mm in wall thickness. Production piles for Piers 2 through 5 will be driven with the steel pipe follower.

The Test Pile was driven at the location of Production Pile Number 3, as shown on Foundation Plan No. 1 included in Appendix A. The Pier 2 footing was excavated to the planned bottom of seal course elevation of –54 meters prior to driving. The Test Pile was driven to elevation –23.64 meters on June 27, 2007 without the use of a pre-drilled hole, a pilot hole, or center-relief drilling. The soil elevation within the open-ended pipe pile was measured following driving and found to be a distance of 3.05 meters below the original bottom of seal course elevation. Following initial drive, it was determined that the pile was over-driven approximately 1.44 meters and the actual pile tip elevation was 1.44 meters lower than planned.

The first restrike was conducted on June 28, 2007 (1-day restrike), and the pile was driven a distance of 150 mm to elevation –23.79 meters. The second and final restrike was conducted on July 6, 2007 (9-day restrike), and the pile was driven a distance of 150 mm to elevation –23.94 meters. Center-relief drilling was not required at any time during the restrikes.

Pile Dynamic Analysis

Pile Dynamic Analysis (PDA) monitoring was performed on the Test Pile for approximately the last 9.2 meters of driving during the initial installation. PDA-monitored restrikes at one-day and nine days following initial installation were subsequently conducted on the Test Pile with the same hammer to provide an estimate of the pile capacity due to setup conditions.

Two strain sensors and two accelerometers were used to monitor the pile stresses and strains. All gauges were mounted on the outer surface of the pile at about two pile diameters below the top of the pile. Personnel of the Foundation Testing Branch conducted monitoring using a Pile Driving Analyzer® by Pile Dynamics Inc. Measured strains and accelerations induced in the pile as a result of driving were used to determine various engineering parameters of interest. These significant attributes derived for each hammer blow include the maximum pile compressive stresses and hammer performance data, such as maximum energy transferred from the hammer to the pile. Plots depicting these parameters as a function of penetration are presented in Appendix B.

Table II shown below includes a summary of PDA monitoring results. Blow counts (blows per 0.3 meter) recorded by Structure Construction during initial driving and restrike events are shown on the Log Pile Sheet included in Appendix D.

Table II. PDA Monitoring Results: Test Pile at Pier 2

	For Field Acceptance*	For Determination of Setup**		
Parameter	**Initial Drive To –22.50 m**	**Initial Drive To –23.9 m**	**1-Day Restrike**	**9-Day Restrike**
Elevation of Pile Tip at Start of Monitoring	–14.47 m	–14.47 m	–23.64 m	–23.79 m
Transferred Energy (EMX) at End of Initial Drive or Beginning of Restrike	46.5 kN-m	46.5 kN-m	69.2 kN-m	40.2 kN-m
Average Stroke at End of Initial Drive or Beginning of Restrike	2.4 m	2.4 m	3.1 m	2.9 m
Peak Maximum Average Compressive Stress (CSX)	239.8 MPa	239.8 MPa	253.2 MPa	292.9 MPa
Peak Maximum Individual Compressive Stress (CSI)	260.4 MPa	260.4 MPa	271.1 MPa	292.9 MPa
Blow Counts at End of Initial Drive or Beginning of Restrike (Blow/0.3 m)	28 Bl/0.3-m	37 Bl/0.3-m	70 Bl/0.3-m	140 Bl/0.3-m
CAPWAP Derived Estimated Capacity	1,530 kN	2,448 kN	3,008 kN	3,511 kN

* Field Acceptance Determined From PDA Data Recorded At Specified Tip Elevation.
** Setup Determined From PDA Data Recorded Within Last 0.3 Meter of Actual Tip Elevation.

Discussion

The Test Pile at Pier 2 was driven to an elevation approximately 1.44 meters below the specified tip elevation. To account for additional geotechnical resistance on the Test Pile that will not exist for production piles, the pile field acceptance criteria was developed using PDA data from depths corresponding to the correct specified tip elevation. PDA data collected during the initial drive for the last several blows at the lower, actual tip elevation was used to establish the time-setup relationship, since this corresponds with the same depth as that at which restrike data was collected.

The Test Pile at Pier 2 appears to have been driven without visible damage while being monitored by a Pile Driving Analyzer®. The compressive pile driving stresses measured by the PDA did not exceed the allowable stresses within the pile during initial driving and both restrikes. The peak maximum compressive stress recorded at any gauge during initial driving and both restrikes was 292.9 MPa. This is slightly less than the allowable driving stress of 295 MPa, which corresponds to 95% of the minimum yield stress of 310 Mpa for the ASTM A252 Grade 3 steel.

Dynamic analysis of the pile indicates that uneven stresses were induced in the pile during the second restrike, identified by observations of differences between the measured stresses on either side of the pile. This indicates that bending stresses were present within the pile during driving. Pile and hammer alignment must be properly maintained to impart maximum energy to the pile and limit the potential for pile damage. The average compressive stress appears to be within acceptable levels throughout driving.

Pile Field Acceptance Criteria

Pile field acceptance criteria were developed by this Office based on the results of wave equation analysis correlated to pile dynamic measurements. Wave equation analyses were performed utilizing CAPWAP™ Version 2000-1 and GRL-WEAP™ Version 2005 software. Based on GRL-WEAP™, bearing curves or field acceptance charts, depicting estimated pile nominal compressive resistance (pile capacity) as a function of blow count and hammer stroke are presented in Appendix C. The time-setup relationship is also included in Appendix C along with an example for interpreting the relationship for use within the control group. If the required loading is not obtained during the initial driving, the time-setup relationship can be used to provide an estimate of the pile capacity due to setup conditions. Alternatively, a restrike can be performed later to verify total pile capacity with the influence of setup included. For restrikes, this observed blow count and stroke information should be developed for a 102-mm driving event and then extrapolated to blows per 0.3 meter.

Pile field acceptance criteria presented in this report relate driving resistance to stroke and blow count. Field acceptance criteria are valid only for the Delmag D30-32, single-acting Diesel hammer when operating properly. The influence of uneven or eccentric blows is not addressed by these charts but should be considered. The values identified for stroke have been correlated to effective energy transferred to the pile for the driving events monitored. If the performance of the hammer is altered or if the construction practice is altered in a way that affects imparted energy, the field acceptance criteria will not be valid.

Recommendations

This Office recommends the release for construction of all piles in the control zone associated with the Test Pile at Pier 2. The derived pile driving resistance relationship plot presented may be used to estimate driving resistance from observed blow count and ram drop height for initial driving and restrike events. The field acceptance criteria presented are site-specific and hammer-specific and should not be applied to piles outside the control zone. If the hammer is modified, or if a new hammer is utilized, additional Pile Dynamic Analysis should be performed to verify dynamic characteristics.

If you have any questions regarding this report, please call me at (916) 227-5509.

Jason D Wahleithner
7/13/2008

REGISTERED PROFESSIONAL ENGINEER
JASON DWIGHT WAHLEITHNER
No. C65572
Exp. 9/30/2008
CIVIL
STATE OF CALIFORNIA

JASON D. WAHLEITHNER, P.E.
Transportation Engineer, Civil
Foundation Testing Branch
Office of Geotechnical Support

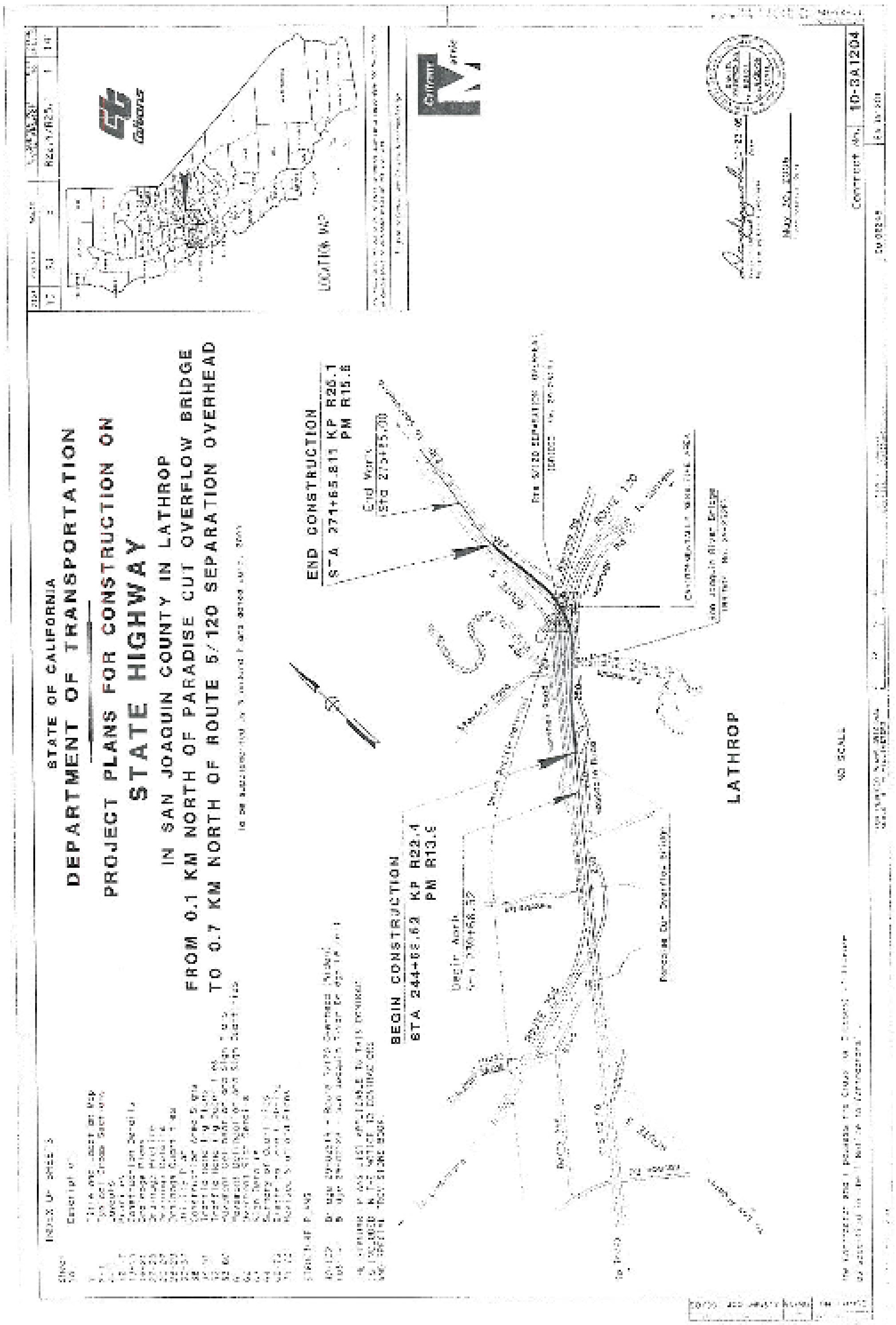
APPENDIX A

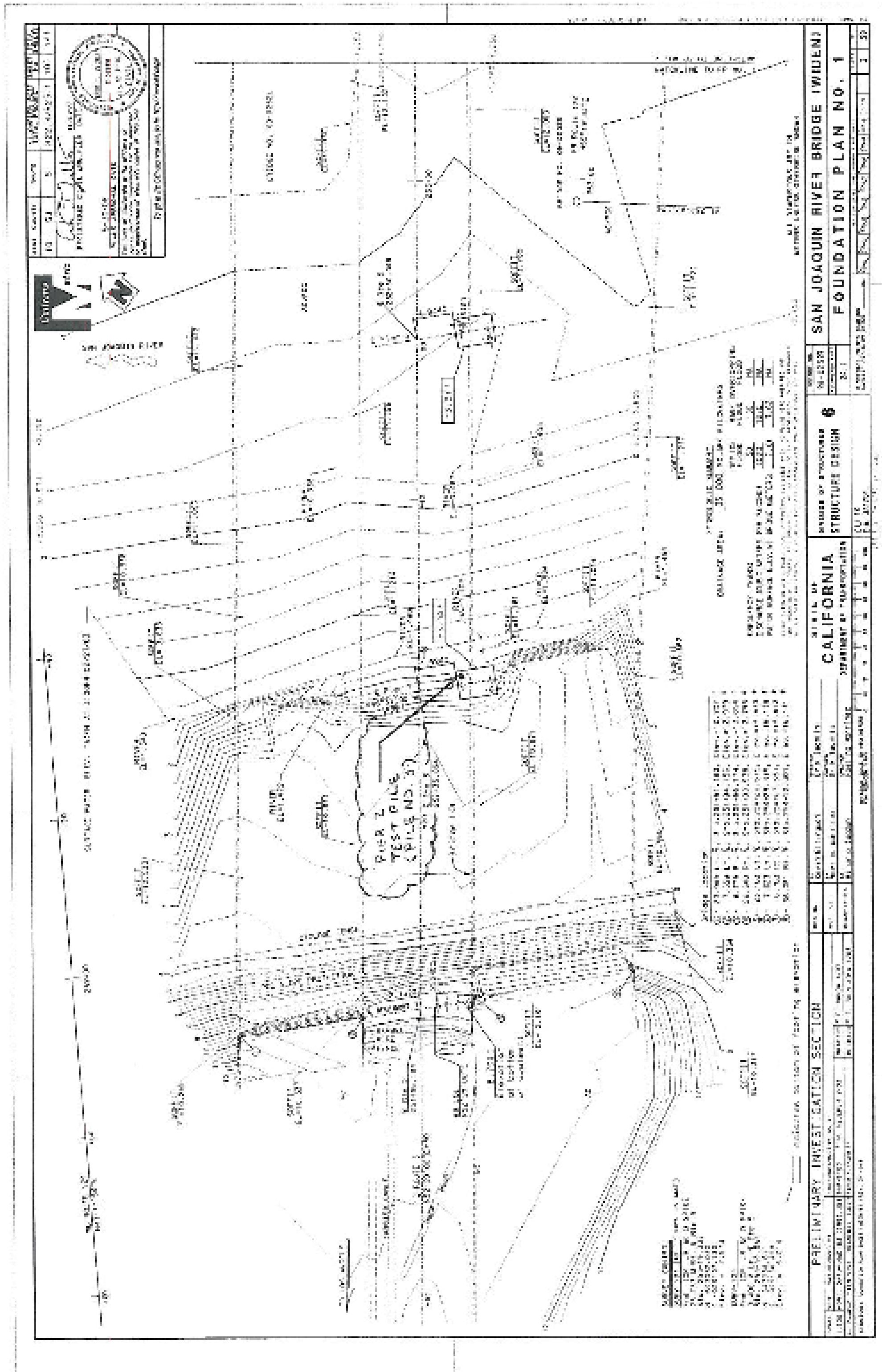
Site Location Plan
Foundation Plan

San Joaquin River Bridge (Widen)
Bridge Number 29-0252R

PDA Test Pile at Pier 2

Caltrans


STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
PROJECT PLANS FOR CONSTRUCTION ON
STATE HIGHWAY
IN SAN JOAQUIN COUNTY IN LATHROP
FROM 0.1 KM NORTH OF PARADISE CUT OVERFLOW BRIDGE
TO 0.7 KM NORTH OF ROUTE 5/120 SEPARATION OVERHEAD
END CONSTRUCTION
STA 271+65.811 KP R26.1
PM R15.6
BEGIN CONSTRUCTION
STA 244+68.63 KP R22.4
PM R13.9
LATHROP
LOCATION MAP
NO SCALE
10-3A1204


SAN JOAQUIN RIVER BRIDGE (WIDEN)
FOUNDATION PLAN NO. 1
CALIFORNIA
STRUCTURE DESIGN
PRELIMINARY INVESTIGATION SECTION
PIER 2 TEST PILE (PILE NO. 3)

Foundation Testing Branch

APPENDIX B

Pile Dynamic Analysis

San Joaquin River Bridge (Widen)
Bridge Number 29-0252R

PDA Test Pile at Pier 2

Foundation Testing Branch

SAN JOAQUIN RIVER BRIDGE 29-0252R – PIER 2
PP 610 x 13 – DELMAG D30-32 HAMMER

PDA RESULTS: END OF INITIAL DRIVE – JUNE 27, 2007

METRIC UNITS

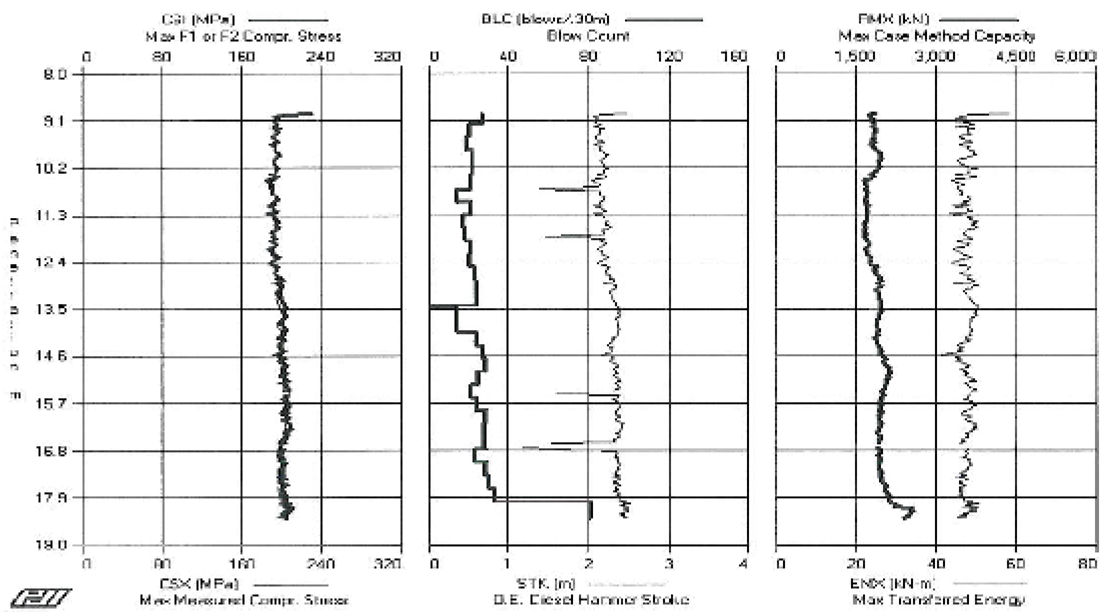
Description of Plotted Parameters:

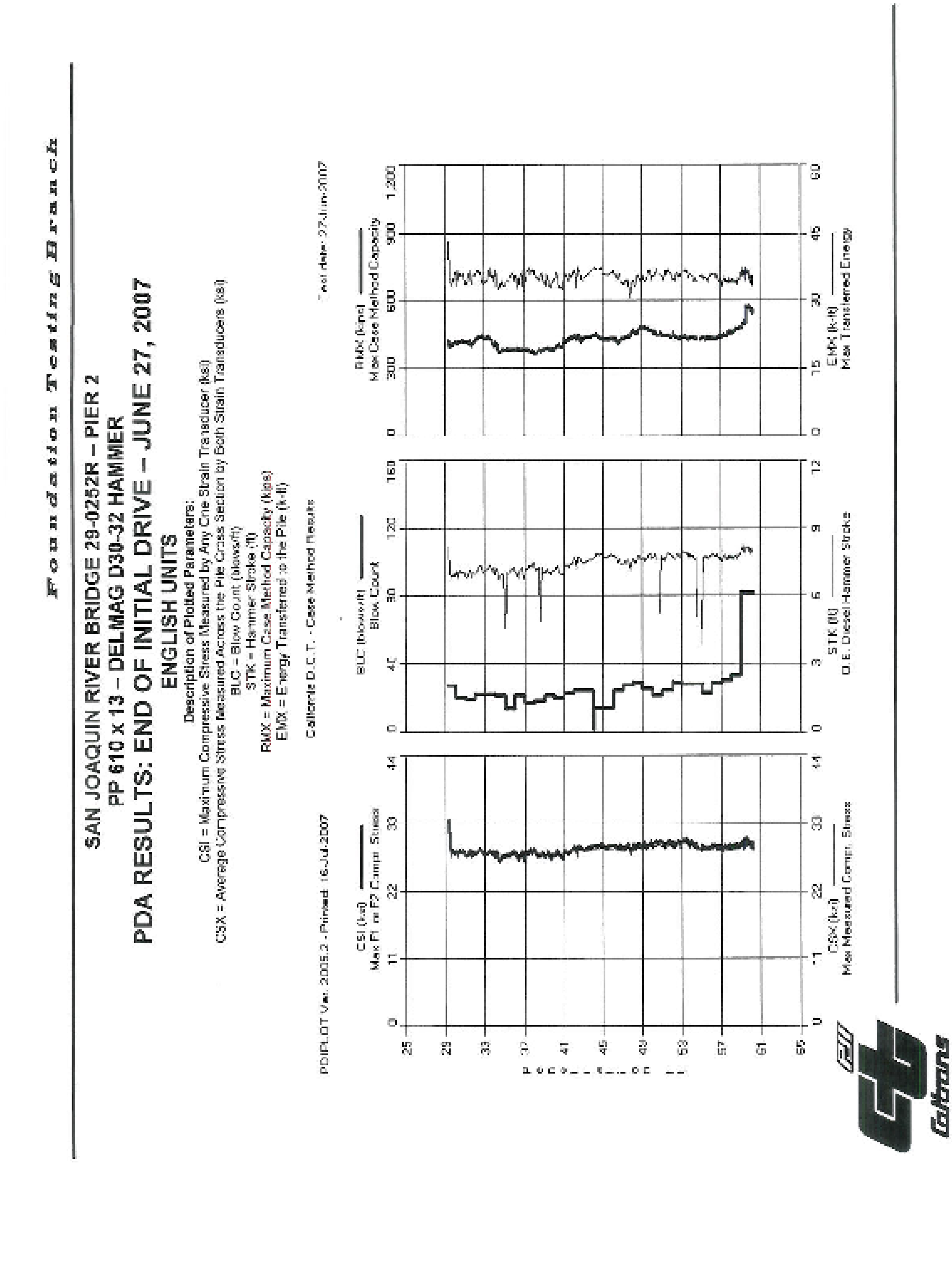
CSI = Maximum Compressive Stress Measured by Any One Strain Transducer (MPa)
CSX = Average Compressive Stress Measured Across the Pile Cross Section by Both Strain Transducers (MPa)
BLC = Blow Count (blows/0.3 m)
STK = Hammer Stroke (m)
RMX = Maximum Case Method Capacity (kN)
EMX = Energy Transferred to the Pile (kN-m)

California D.O.T. - Case Method Results

PDIPLOT Ver. 2006.2 - Printed: 16-Jul-2007

Test date: 27-Jun-2007




Foundation Testing Branch
SAN JOAQUIN RIVER BRIDGE 29-0252R – PIER 2
PP 610 x 13 – DELMAG D30-32 HAMMER
PDA RESULTS: END OF INITIAL DRIVE – JUNE 27, 2007
ENGLISH UNITS
Description of Plotted Parameters:
CSI = Maximum Compressive Stress Measured by Any One Strain Transducer (ksi)
CSX = Average Compressive Stress Measured Across the Pile Cross Section by Both Strain Transducers (ksi)
BLC = Blow Count (blows/ft)
STK = Hammer Stroke (ft)
RMX = Maximum Case Method Capacity (kips)
EMX = Energy Transferred to the Pile (k-ft)
PDIPLOT Ver. 2005.2 - Printed: 16-Jul-2007
California D.C.T. - Case Method Results
Test date: 27-Jun-2007
CSI (ksi) Max F1 or F2 Compr. Stress
CSX (ksi) Max Measured Compr. Stress
BLC (blows/ft) Blow Count
STK (ft) O.E. Diesel Hammer Stroke
RMX (kips) Max Case Method Capacity
EMX (k-ft) Max Transferred Energy

Foundation Testing Branch

SAN JOAQUIN RIVER BRIDGE 29-0252R – PIER 2

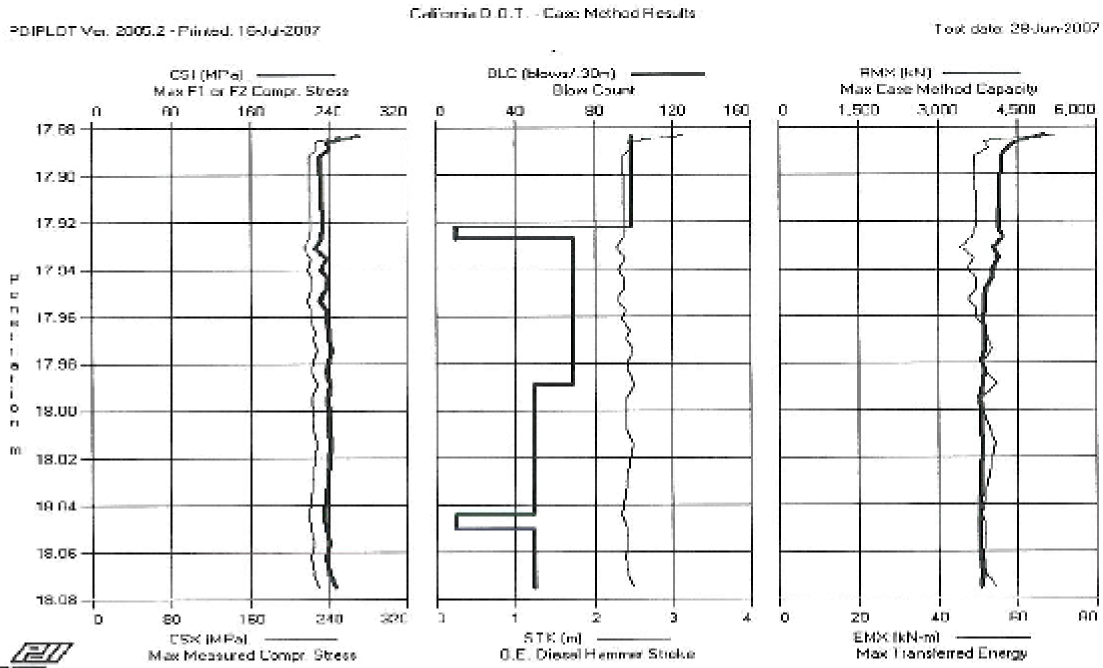
PP 610 x 13 – DELMAG D30-32 HAMMER

PDA RESULTS: 1-DAY RESTRIKE – JUNE 28, 2007

METRIC UNITS

Description of Plotted Parameters:

CSI = Maximum Compressive Stress Measured by Any One Strain Transducer (MPa)
CSX = Average Compressive Stress Measured Across the Pile Cross Section by Both Strain Transducers (MPa)
BLC = Blow Count (blows/0.3 m)
STK = Hammer Stroke (m)
RMX = Maximum Case Method Capacity (kN)
EMX = Energy Transferred to the Pile (kN-m)



Foundation Testing Branch

SAN JOAQUIN RIVER BRIDGE 29-0252R – PIER 2
PP 610 x 13 – DELMAG D30-32 HAMMER
PDA RESULTS: 1-DAY RESTRIKE – JUNE 28, 2007
ENGLISH UNITS

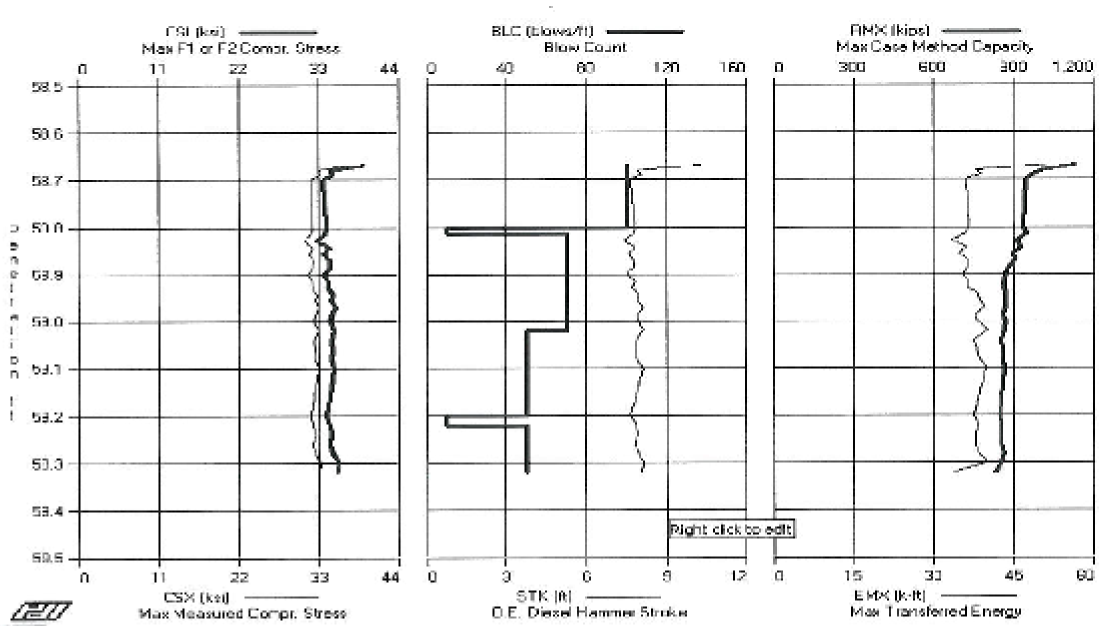
Description of Plotted Parameters:

CSI = Maximum Compressive Stress Measured by Any One Strain Transducer (ksi)
CSX = Average Compressive Stress Measured Across the Pile Cross Section by Both Strain Transducers (ksi)
BLC = Blow Count (blows/ft)
STK = Hammer Stroke (ft)
RMX = Maximum Case Method Capacity (kips)
EMX = Energy Transferred to the Pile (k-ft)

California D.C.T. - Case Method Results

PDIPLOT Ver. 2005.2 - Printed: 1-Jul-2007

Test date: 28-Jun-2007



Foundation Testing Branch

SAN JOAQUIN RIVER BRIDGE 29-0252R – PIER 2

PP 610 x 13 – DELMAG D30-32 HAMMER

PDA RESULTS: 9-DAY RESTRIKE – JULY 6, 2007

METRIC UNITS

Description of Plotted Parameters:

CSI = Maximum Compressive Stress Measured by Any One Strain Transducer (MPa)

CSX = Average Compressive Stress Measured Across the Pile Cross Section by Both Strain Transducers (MPa)

BLC = Blow Count (blows/0.3 m)

STK = Hammer Stroke (m)

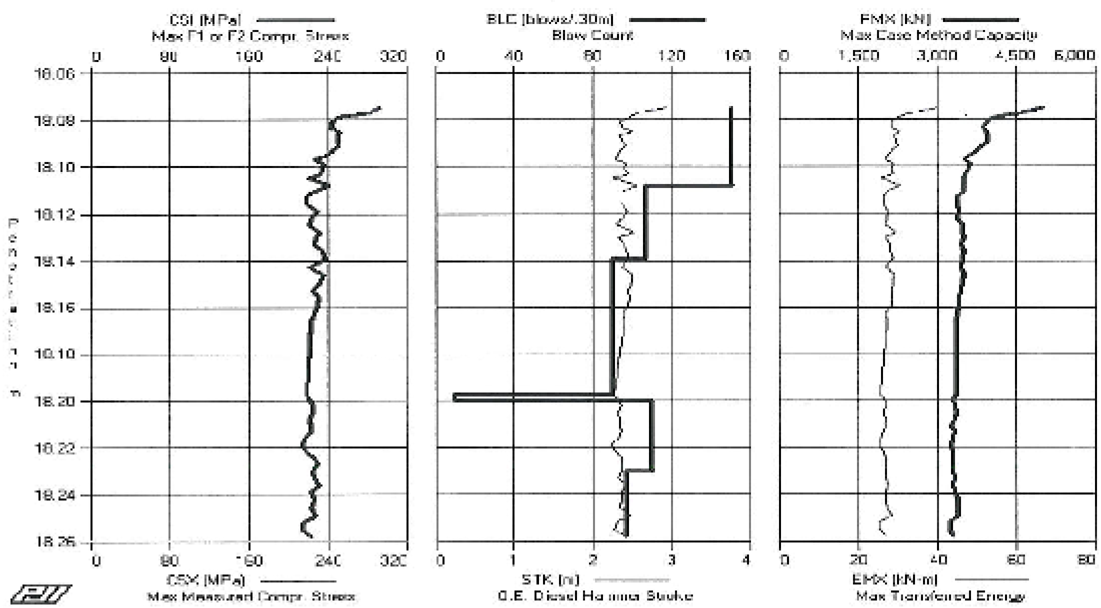
RMX = Maximum Case Method Capacity (kN)

EMX = Energy Transferred to the Pile (kN-m)

California D.O.T. Case Method Results

PDIPLOT Ver. 2005.2 - Printed: 16-Jul-2007

Test date: 6-Jul-2007



Caltrans

Foundation Testing Branch

SAN JOAQUIN RIVER BRIDGE 29-0252R – PIER 2

PP 610 x 13 – DELMAG D30-32 HAMMER

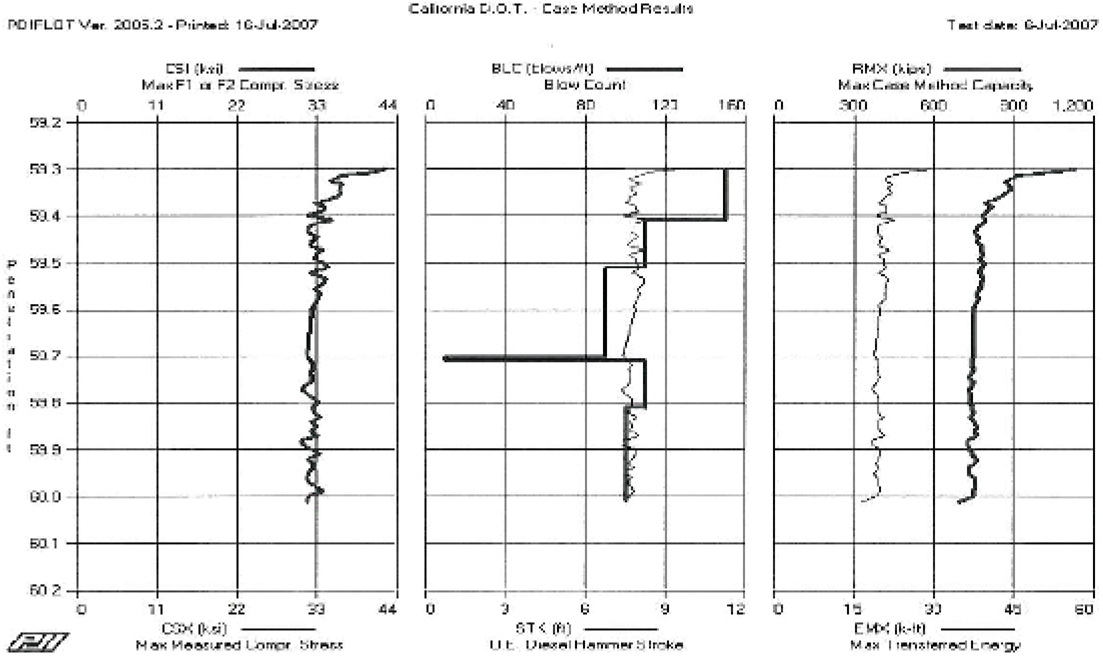
PDA RESULTS: 9-DAY RESTRIKE – JULY 6, 2007

ENGLISH UNITS

Description of Plotted Parameters:

CSI = Maximum Compressive Stress Measured by Any One Strain Transducer (ksi)
CSX = Average Compressive Stress Measured Across the Pile Cross Section by Both Strain Transducers (ksi)
BLC – Blow Count (blows/ft)
STK – Hammer Stroke (ft)
RMX = Maximum Case Method Capacity (kips)
EMX = Energy Transferred to the Pile (k-ft)

California D.O.T. - Case Method Results



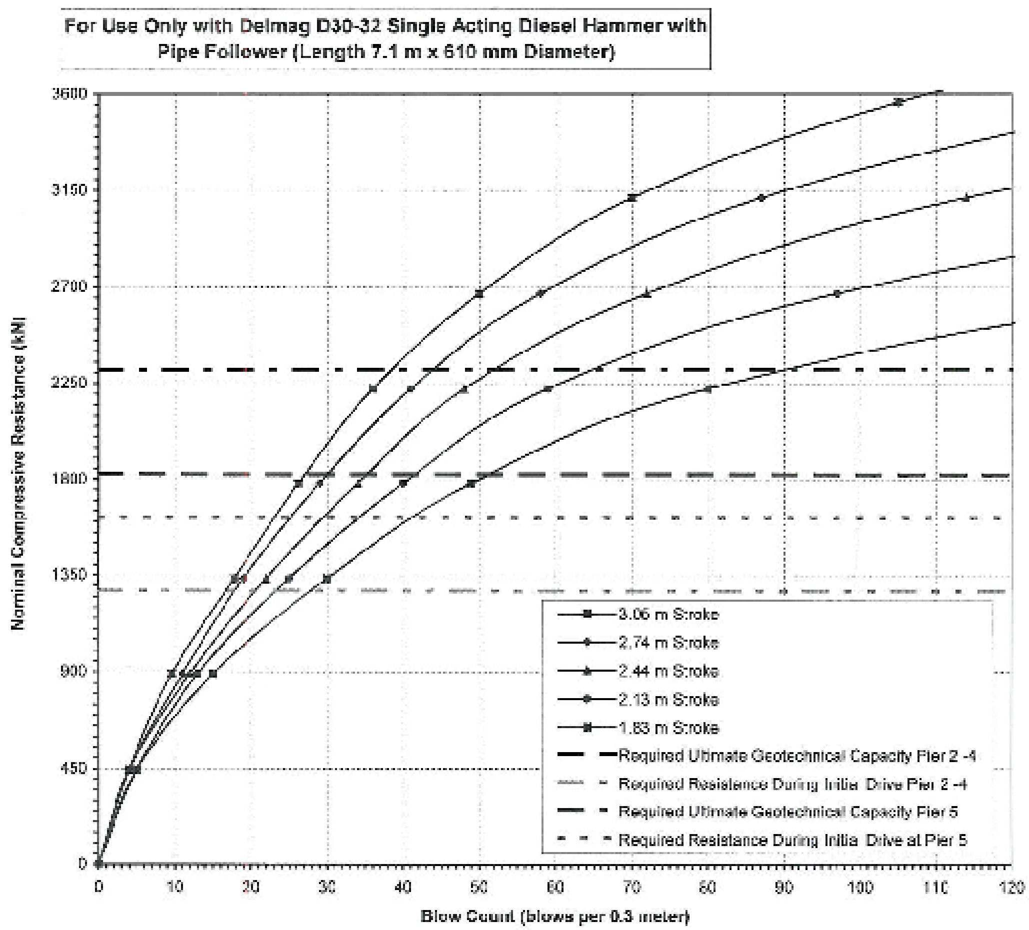
APPENDIX C

Bearing Curve (Field Acceptance Chart) – Metric Units
Bearing Curve (Field Acceptance Chart) – English Units
Time-Setup Relationship

San Joaquin River Bridge (Widen)
Bridge Number 29-0252R

PDA Test Pile at Pier 2

Foundation Testing Branch



Field Acceptance Chart: Capacity Relationship

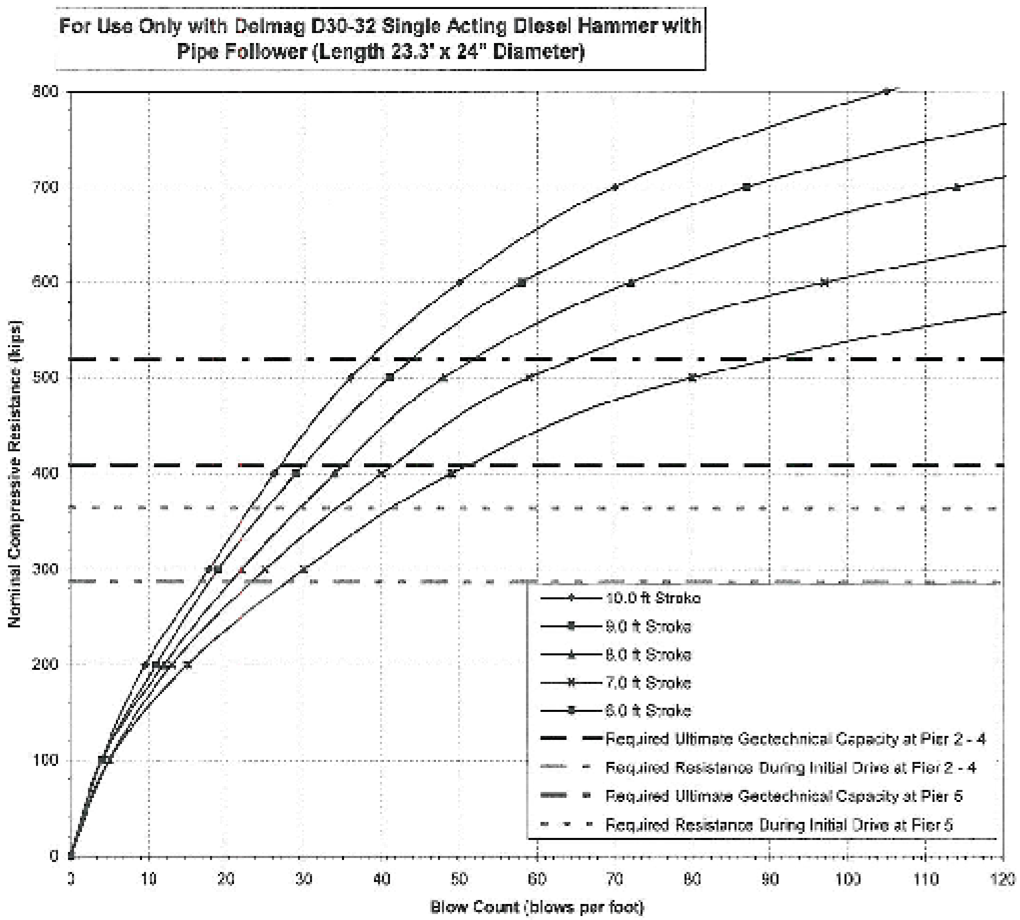
Derived From Wave Equation Analysis Program (WEAP)

Pipe Piles for Control Location Including Pier 2 Through Pier 5

EA Number 10-3A1204
Bridge Number 29-0252R
10-SJ-5-R22.4/R25.1

San Joaquin River Br. (Widen)
Delmag D30-32

Foundation Testing Branch



Field Acceptance Chart: Capacity Relationship

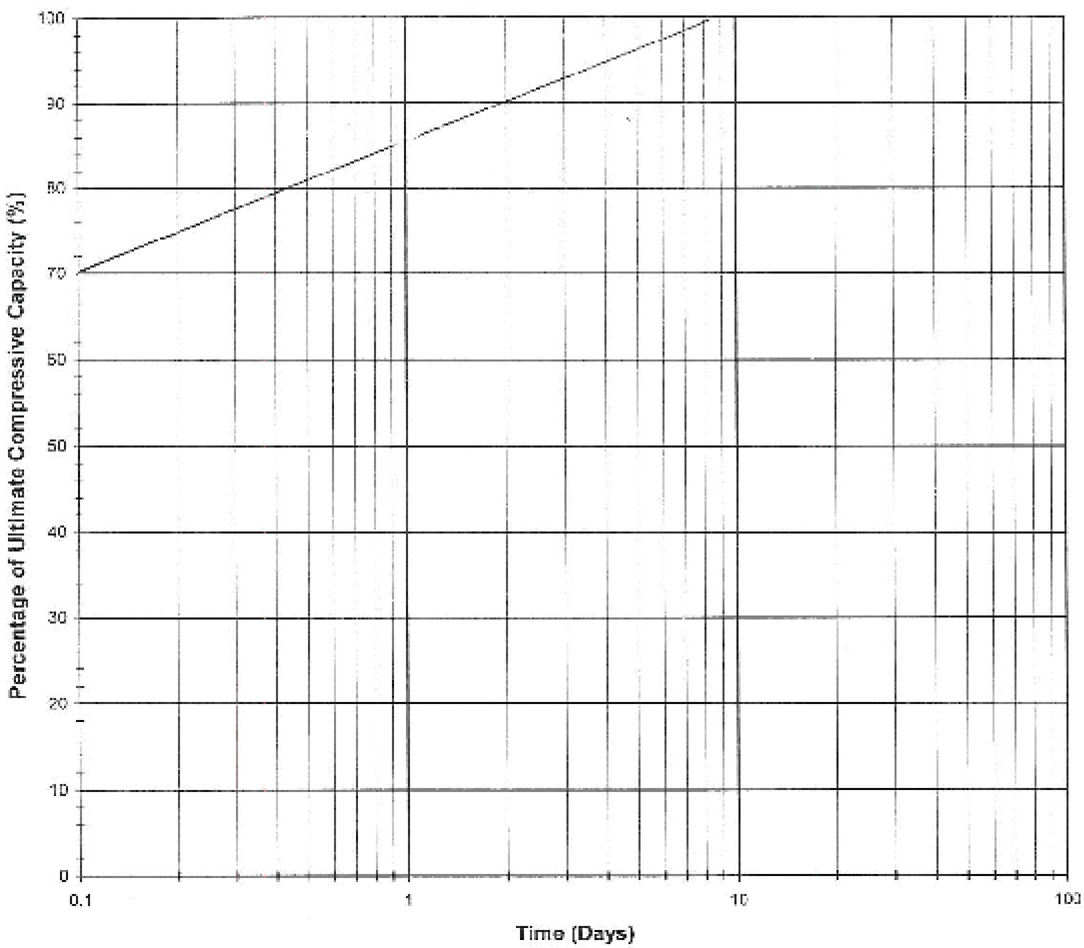
Derived From Wave Equation Analysis Program (WEAP)

Pipe Piles for Control Location Including Pier 2 Through Pier 5

EA Number 10-3A1204
Bridge Number 29-0252R
10-SJ-5-R22.4/R25.1

San Joaquin River Br (Widen)
Delmag D30-32

Foundation Testing Branch



Time-Setup Relationship

Test Pile at Pier 2: 1- and 9-Day Restrike

San Joaquin River Br. (Widen)

EA Number 10-3A1204
Bridge Number 29-0252R
10-SJ-5-R22.4/R25.1

San Joaquin River Br. (Widen)
Delmag D30-32
Driven Tip Elev. –22.500 m

Caltrans

Foundation Testing Branch

Example how to use the "Time-Setup Relationship" Curve

1) The production pile was driven to within 1 foot from the specified tip elevation or to the exact specified tip elevation.

2) 20 blows per foot were recorded with an 8.0 ft stroke at this depth. According to the Field Acceptance Chart (Bearing Graph), a Nominal Compressive Resistance of 291 kips (1,295 kN) is obtained (this is at 70% of the ultimate compressive capacity according the time-setup relationship).

3) After a setup period of 8 days, the projected Nominal Resistance in Compression can be extrapolated as:
291 kips x (1/0.70) = 416 kips

   Or in SI units:
1,295 kN x (1/0.70) = 1,850 kN

APPENDIX D

Pile Log Sheet
Log of Test Borings

San Joaquin River Bridge (Widen)
Bridge Number 29-0252R

PDA Test Pile at Pier 2

Caltrans

JOB STAMP
10-3A1204
10-SJ-5-R22.4/R25.1

DEPARTMENT OF TRANSPORTATION
LOG PILE SHEET
DH-CS C79 (REV. 11 73)

Bridge No 29-0252R Abut or ~~Bent No~~ Pier 2 Ftg ____ Pile No #3 Sheet No 1

Hammer Make APE Model D36-32 E = 62,843 Ft-Lbs

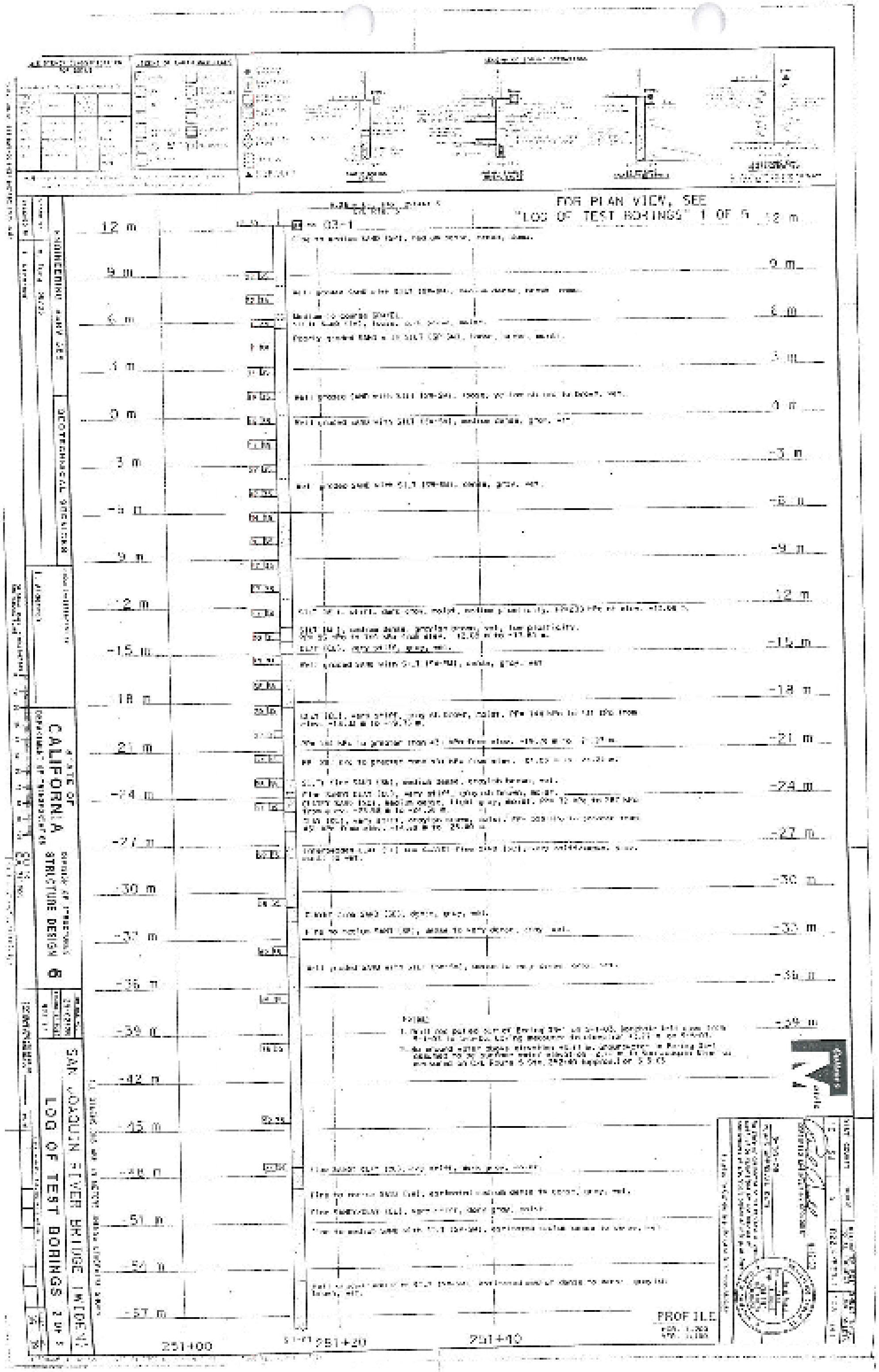
Reference Point Description Template Elev ____

4-9.5ft below 3-PDA
8:39
8:51

PENET	TIP ELEV	BLOWS/FT
1		14
5		23
10		19
15		26
20		25
21		26
22		27
23		27
24		30
25		28
26		28
27		29
28		31
29		31
30		32
31		37
32		
33		
34		
35		

PENET	TIP ELEV	BLOWS/FT

PENET	TIP ELEV	BLOWS/FT


FOR PLAN VIEW, SEE
"LOG OF TEST BORINGS" 1 OF 5
12 m
9 m
6 m
3 m
0 m
-3 m
-6 m
-9 m
-12 m
-15 m
-18 m
-21 m
-24 m
-27 m
-30 m
-33 m
-36 m
-39 m
-42 m
-45 m
-48 m
-51 m
-54 m
-57 m
251+00
251+20
251+40
PROFILE
STATE OF CALIFORNIA
STRUCTURE DESIGN
GEOTECHNICAL SERVICES
SAN JOAQUIN RIVER BRIDGE (WIDEN)
LOG OF TEST BORINGS 2 OF 5

APPENDIX

G Slurry Displacement Piles

Table of Contents

State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: GABRIEL ACERO
Structure Representative
I-15 Managed Lanes

Date: February 22, 2008

File: 11–SD–15– M38.7/M42.7
11–080924
I-15 Managed Lanes
(Green Valley Creek Bridge)
Br. No. 57-1133L

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES – MS 5

Subject: Gamma–Gamma Logging Acceptance Test Results: Right Pile at Pier 2 – Stage III

Attached is a report summarizing the Gamma–Gamma Logging acceptance test results for the Right Pile at Pier 2 – Stage III of the above referenced project.

If you have any questions or comments regarding this report, please contact Michael K. Harris, P.E. at (916) 227-1058.

BRIAN LIEBICH, P.E.
Senior Transportation Engineer
Foundation Testing Branch
Office of Geotechnical Support

Attachments

c: D. Valls – SC (E-Mail)
J. Walters – SC (E-Mail)
B. Alsamman – SC (E-Mail)
R. Granados – SC (E-Mail)
A. Ma – SC (E-Mail)
L. Valla – SD (E-Mail)
M. DeSalvatore – OGDS-2 (E-Mail)
M. Holm – SD (E-Mail)
E. Neupert – OGDS-2 (E-Mail)

MKH/mkh

"Caltrans improves mobility across California"

FOUNDATION TESTING BRANCH

February 22, 2008

11–SD–15–M38.7/M42.7
11–080924
I-15 Managed Lanes
(Green Valley Creek Bridge)
Br. No 57-1133L

Gamma-Gamma Logging Acceptance Test Results:

Right Pile at Pier 2 - Stage III

February 22, 2008

Project Information

11-SD-15-M38.7/M42.7
11-080924
I-15 Managed Lanes (Green Valley Creek Bridge)
Br. No. 57-1133L

Subject

Gamma-Gamma Logging Acceptance Test Results: Right Pile at Pier 2 – Stage III

Introduction

This report presents the results of Gamma-Gamma Logging (GGL) acceptance testing for the Pile at Pier 2 – Stage III of the above referenced project. This Cast-In-Drilled-Hole (CIDH) concrete pile is constructed with a 2.6-meter diameter permanent steel casing and a 2.4-meter diameter drilled rock socket. The pile contains eight (8) GGL inspection tubes appropriately spaced on the interior of the reinforcing bar cage. Gamma Gamma Logging was conducted in the accessible portions of all the PVC inspection tubes in the pile. GGL inspection tubes are numbered in a clockwise direction with Tube 1 most closely aligned to magnetic North unless otherwise marked in the field.

According to information provided by Structure Construction, constructed top of concrete, pile tip and casing tip elevations for the Right Pile at Pier 2 – Stage III are as shown in Table I. Pile cutoff and specified tip elevations per the Plan Sheets are also included in Table I.

Table I. Summary of Pile Information :
Right Pile at Pier 2 – Stage III

Pile Information (Meters)	Right Pile Pier 2
Specified Cutoff Elev.	104.80
Specified Casing Tip Elev.	95.60
Specified Pile Tip Elev.	80.80
Specified Pile Length	24.00
Reported Top of Concrete Elev.*	99.852
Reported Casing Tip Elev.*	95.563
Reported Pile Tip Elev.*	80.643
Reported Pile Length*	19.209
Approx. GGL-Tested Length	18.2

* Based on information from Structure Construction.

Gamma-Gamma Logging was conducted in the Right Pile at Pier 2 – Stage III by personnel of the Foundation Testing Branch of the Office of Geotechnical Support on February 20, 2008. GGL was conducted utilizing a Mt. Sopris Model HLP-2375 Gamma-Gamma Probe with a 100-millicurie Cesium 137 source.

Background

Gamma-Gamma Logging is generally viewed as one of the most accurate non-destructive test methods used to detect anomalies in CIDH piles. Substantial drops in average bulk density readings from Gamma–Gamma tests are indicative of the presence of anomalies in the material surrounding the inspection tube. For the Mt. Sopris Model HLP-2375 Gamma–Gamma Probe used by this Office, the range of detection is approximately 75 mm (3 inches) into the concrete around the inspection tube.

Discussion

A summary graph depicting variation from mean bulk density versus depth for Gamma-Gamma Logging can be found in Appendix A for the Right Pile at Pier 2 – Stage III. The mean and standard deviation criteria set was derived from Gamma-Gamma Logging readings from the tested inspection tubes in the pile, excluding portions significantly impacted by reinforcement, anomalies and water, as applicable. The understood approximate locations of PVC tube couplers have been considered in the analysis. The results of the Gamma-Gamma Logging are summarized in Table II.

Table II. Summary of Gamma–Gamma Logging Test Results: Right Pile at Pier 2 – Stage III

Pile (Section)	Approx. Depth* (Meters)	Approx. Elevation** (Meters)	GGL Tube(s)	Data Description
Right, Pier 2 (A-A)	0.0 to 1.3	99.9 to 98.6	2	Low bulk density readings detected in one (1) inspection tube. May affect up to 13% of pile cross-section.
Right, Pier 2 (B-B)	1.3 to 1.6	98.6 to 98.3	4	Anomalies detected in one (1) inspection tube. May affect up to 13% of pile cross-section.
Right, Pier 2 (–)	17.9 to Tip	82.0 to Tip	2, 3, 4, 6, 7, 8	High bulk density readings detected in six (6) inspection tubes. Consistent with water in tubes. Interpreted as not anomalous.

* Depths are referenced to top of concrete at the time of testing.
** Based on information provided by Structure Construction personnel.

Gamma–Gamma Logging within the Right Pile at Pier 2 – Stage III detected low bulk density readings in one (1) inspection tube from top of concrete to about 1.3 meters depth, affecting up to 13% of pile cross-section at this location as indicated in Table II (Section A-A). This region is located below the Specified (Plan) Cutoff Elevation for the Pile and is to be mitigated prior to construction of the top portion of the CIDH pile. The low bulk densities indicate that slurry mixing or contamination may have occurred at the as-built top of concrete (construction joint) portion of the pile in the vicinity of this tube. This condition may be mitigated by field inspection and appropriate removal of substandard material, in a procedure considered as Simple Repair.

Gamma–Gamma Logging within the Right Pile at Pier 2 – Stage III also detected anomalies in one (1) inspection tube (Tube 4) from about 1.3 meters to about 1.6 meters depth below top of concrete, as indicated in Table II (Section B-B). This region involving one (1) GGL inspection tube may affect up to 13% of pile cross-section at this location, which is within the permanent steel casing.

Gamma-Gamma Logging within the Right Pile at Pier 2 – Stage III also detected significant increases in apparent density near the bottom of six (6) inspection tubes within the pile. This is consistent with standing water inside the inspection tubes. Due to the limited extent of water in the tubes, this particular occurrence did not present a significant problem for analysis. Presence of water in GGL inspection tubes may impede the collection and analysis of accurate data for acceptance of piles. Therefore, in compliance with section 10–1.55 PILING of the Special Provisions, tubes in future CIDH piles must be completely purged of water prior to Gamma-Gamma Logging.

No other significant anomalies were detected in the tested portion of the pile.

According to information provided by Structure Construction, the Right Pile at Pier 2 - Stage III was constructed to an approximate length of 19.2 meters. Based on the reported constructed length and field measurements by personnel from this Office at the time of GGL testing, this pile exhibited approximately 0.9 meter to 1.1 meters of untested concrete above the reported pile tip. This may be attributable to debris in the tubes, irregular placement of the tubes during construction, over-drilling of the shaft without extending the reinforcement, measurement error, or some combination of these factors. With clean, properly positioned tubes and with reinforcing and pile tip constructed to plan, the untested portion represented by each inspection tube should be approximately 0.4 meter.

Recommendations

This Office recommends rejection of the Right Pile at Pier 2 – Stage III of the Green Valley Creek Bridge, based on Gamma-Gamma Logging test results. Please refer to Caltrans BRIDGE CONSTRUCTION MEMOS 130-10.0, 130-11.0 and 130-12.0 (June 14, 2007) for guidance in addressing rejected CIDH piles. Please see Pile Design Data Form for the Right Pile at Pier 2 - Stage III in Appendix C. Note that Section A-A is a candidate for "Simple Repair".

If you have any questions regarding this report, please contact Michael Harris at (916) 227-1058.

2/22/08

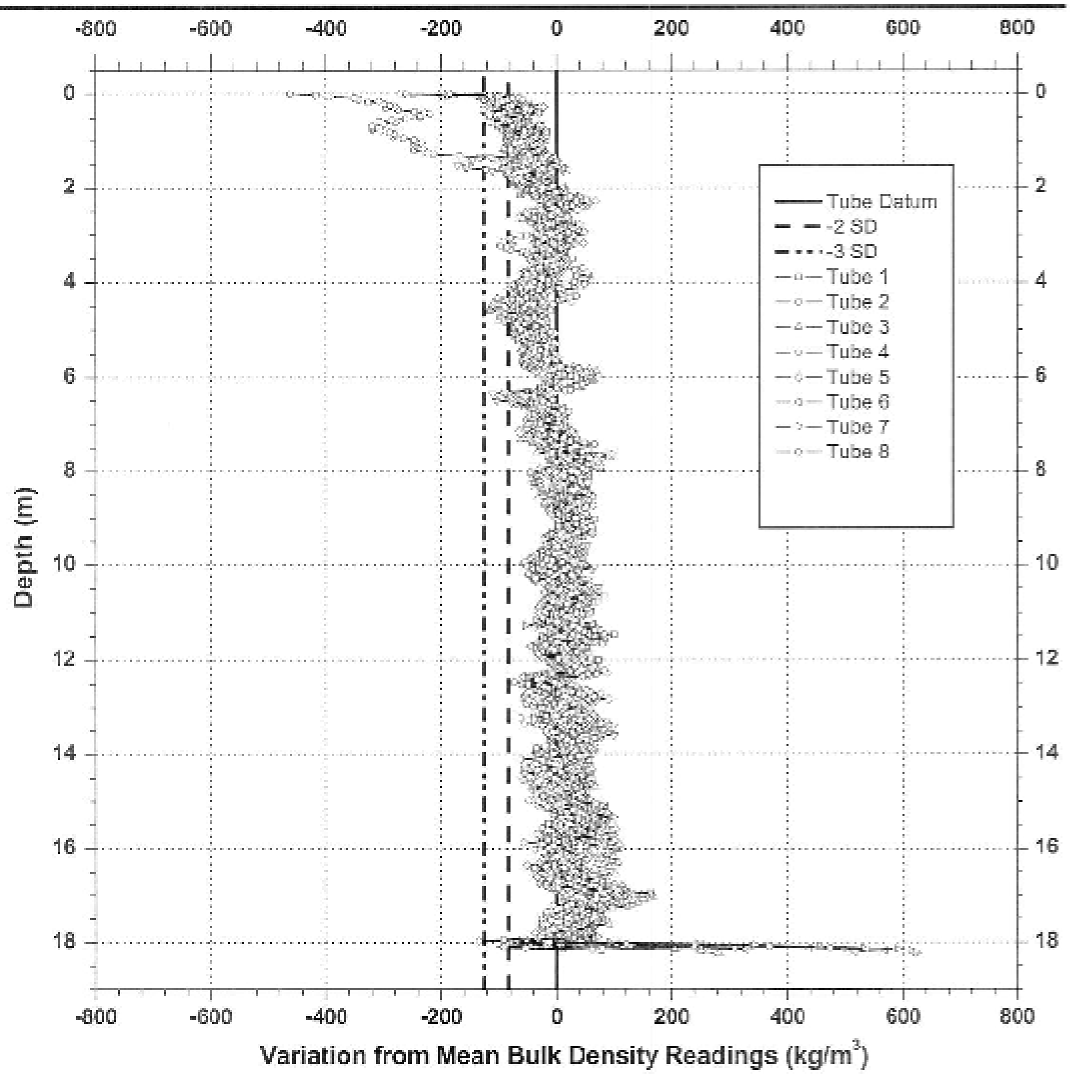
MICHAEL K. HARRIS, P.E.
Transportation Engineer, Civil
Foundation Testing Branch
Office of Geotechnical Support

REGISTERED PROFESSIONAL ENGINEER
MICHAEL KELVIN HARRIS
No. C 27924
exp. 3/31/08
CIVIL
STATE OF CALIFORNIA

APPENDIX A

Gamma Gamma Logging Test Results

Foundation Testing Branch



GAMMA-GAMMA LOGGING ACCEPTANCE TEST RESULTS
I-15 Managed Lanes (Green Valley Creek Bridge)
Right CIDH Pile at Pier 2 - Stage III

EA 11-080924
Bridge Number 57-1133L
11-SD-15-M38.7/M42.7
Date Tested: 02/20/2008

2.6/2.4 m-diameter CIDH Pile
Reported Top of Concrete Elev.= +99.852 m
Reported Pile Tip Elev.= +80.643 m
Winch/Probe/Source: 1194-3551-041

CALIFORNIA FTB

The mean and standard deviation were calculated using density readings from all tubes excluding portions significantly impacted by reinforcement, anomalies and water, as appropriate .

APPENDIX B

Pile Information

PRELIMINARY FOR BIDDING PURPOSES ONLY

PILE DATA TABLE (LEFT BRIDGE)									
LOCATION	PILE TYPE	DESIGN LOADING	NOMINAL RESISTANCE (C.SH)		CUT-OFF ELEVATIONS	SPECIFIED TIP ELEVATIONS (CASING)	DESIGN TIP ELEVATIONS (CIDH)	SPECIFIED TIP ELEVATIONS (CIDH)
			COMPRESSION	TENSION				
Abut 1	1200 mm CIDH	1950 kN	3300 kN	0	N/A	N/A	[illegible] (1) 103.50 m (2)	101.20 m
Abut 1 Lt Ret Wall, H=5.5 m	HP [illegible]	400 kN	800 kN	0	N/A	N/A	111.50 m (1)	111.30 m
Pier 2	2400 mm CIDH w/Permanent Steel Casing	N/A	21 400 kN	0	104.20 m	55.50 m	46.30 m (1) 60.60 m (2)	60.60 m
Pier 3	2400 mm CIDH w/Permanent Steel Casing	N/A	21 400 kN	0	104.20 m	56.70 m	53.47 m (1) 72.82 m (2)	[illegible]
Abut 4 Lt Ret Wall, H=[illegible]	HP 250x62	400 kN	320 kN	0	N/A	N/A	98.40 m (1)	98.40 m
Abut 4	1200 mm CIDH	1950 kN	2800 kN	0	N/A	N/A	[illegible] (1) [illegible] (2)	[illegible]

Note: Design tip elevations are controlled by the following demands: (1) Compression, (2) Lateral

PILE DATA TABLE (RIGHT BRIDGE)									
LOCATION	PILE TYPE	DESIGN LOADING	NOMINAL RESISTANCE (C.SH)		CUT-OFF ELEVATIONS	SPECIFIED TIP ELEVATIONS (CASING)	DESIGN TIP ELEVATIONS (CIDH)	SPECIFIED TIP ELEVATIONS (CIDH)
			COMPRESSION	TENSION				
Abut 1	1200 mm CIDH	1950 kN	3400 kN	0	N/A	N/A	[illegible] (1) [illegible] (2)	24.30 m
Pier 2	2400 mm CIDH w/Permanent Steel Casing	N/A	21 400 kN	0	105.30 m	45.30 m	36.30 m (1) 77.41 m (2)	77.41 m
Pier 3	2400 mm CIDH w/Permanent Steel Casing	N/A	21 400 kN	0	104.35 m	56.00 m	78.30 m (1) [illegible] (2)	[illegible]
Abut 4	1200 mm CIDH	1950 kN	3300 kN	0	N/A	N/A	[illegible] (1) 100.50 m (2)	51.70 m

Note: Design tip elevations are controlled by the following demands: (1) Compression, (2) Lateral

SPREAD FOOTING DATA TABLE (LEFT BRIDGE)				
LOCATION	RECOMMENDED SOIL BEARING PRESSURE (kPa)		MAXIMUM CONTACT PRESSURE (q max)	STRENGTH REDUCTION FACTOR (φ)
	WSD	LFD		
	GROSS ALLOWABLE SOIL BEARING PRESSURE (q all)	ULTIMATE SOIL BEARING PRESSURE (q ult)		
Abut 1 Lt Ret Wall, H=[illegible]	155 kPa	N/A	135 kPa	N/A
Abut 1 Lt Ret Wall, H=[illegible]	170 kPa	N/A	170 kPa	N/A
Abut 1 Rt Ret Wall, H=[illegible]	170 kPa	N/A	170 kPa	N/A
Abut 1 Rt Ret Wall, H=[illegible]	310 kPa	N/A	310 kPa	N/A
Abut 1 Rt Ret Wall, H=[illegible]	375 kPa	N/A	345 kPa	N/A
Abut 4 Lt Ret Wall, H=[illegible]	170 kPa	N/A	170 kPa	N/A
Abut 4 Lt Ret Wall, H=[illegible]	155 kPa	N/A	155 kPa	N/A
Abut 4 Rt Ret Wall, H=[illegible]	360 kPa	N/A	345 kPa	N/A
Abut 4 Rt Ret Wall, H=[illegible]	375 kPa	N/A	315 kPa	N/A
Abut 4 Rt Ret Wall, H=[illegible]	170 kPa	N/A	170 kPa	N/A

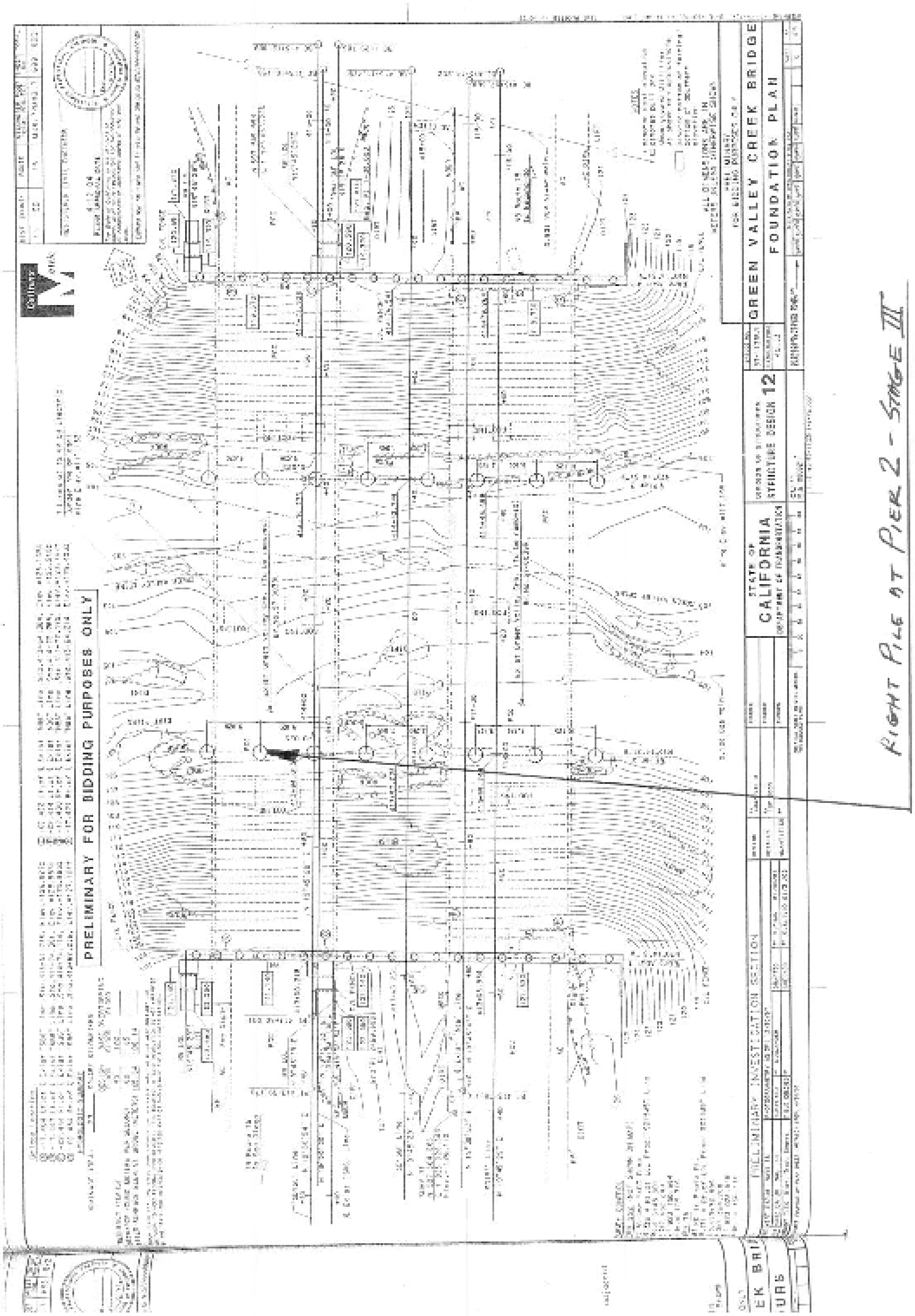
ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

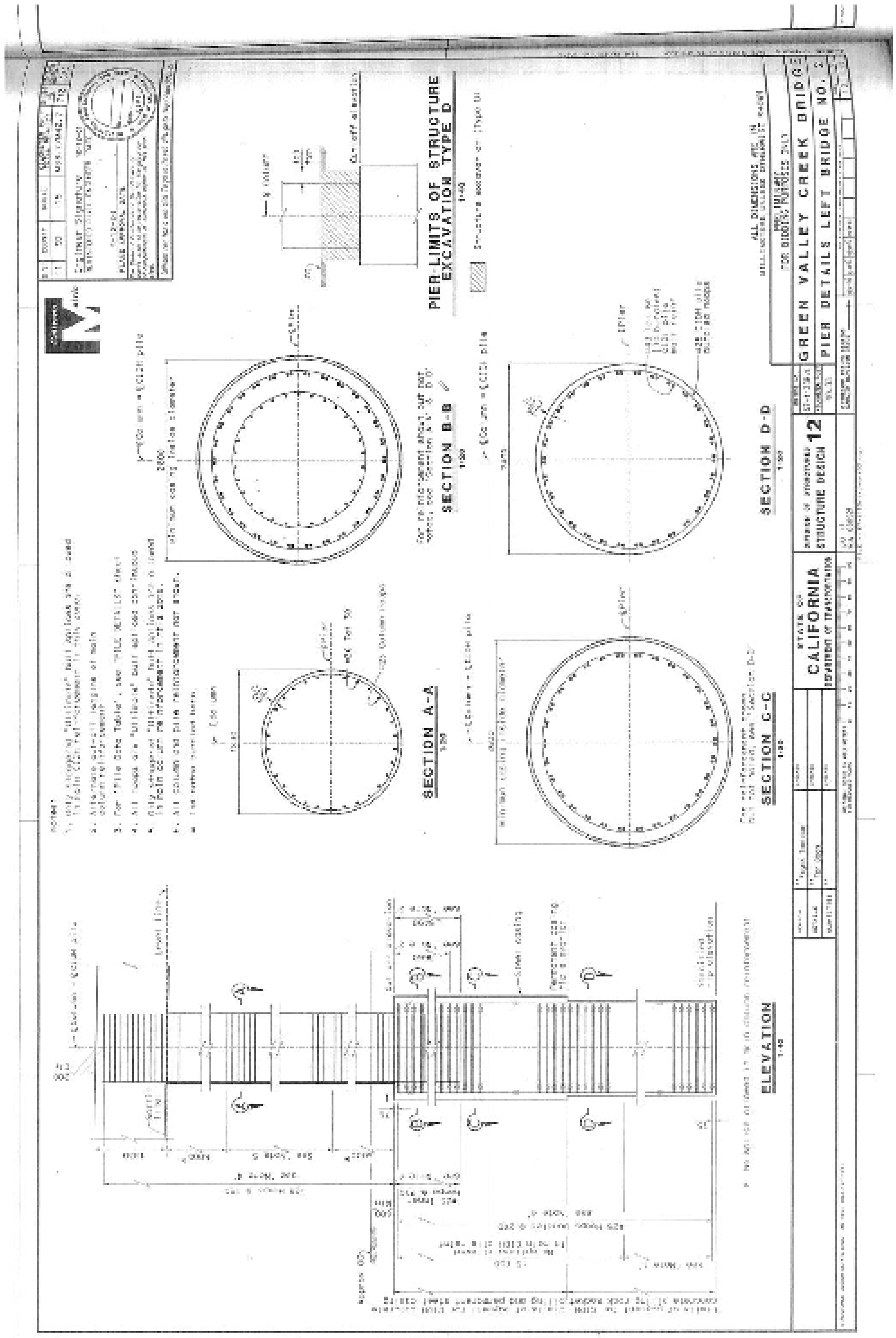
PRELIMINARY FOR BIDDING PURPOSES ONLY

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF STRUCTURES
STRUCTURE DESIGN 12

GREEN VALLEY CREEK BRIDGE
PILE DATA


PRELIMINARY FOR BIDDING PURPOSES ONLY
STATE OF CALIFORNIA
STRUCTURE DESIGN 12
GREEN VALLEY CREEK BRIDGE
FOUNDATION PLAN
PRELIMINARY INVESTIGATION SECTION
RIGHT PILE AT PIER 2 - STAGE III


ELEVATION
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
PIER-LIMITS OF STRUCTURE EXCAVATION TYPE D
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
STRUCTURE DESIGN
GREEN VALLEY CREEK BRIDGE

Foundation Testing Branch

EA 11-080924, Unit 3
I-15 Managed Lanes
San Diego

Green Valley Creek Br, Stage III, Piers 3 CIDH

Piles	2L		2M		2R
Specified Cut-off Elev. (Meters):	104.8				104.8
Specified Pile Tip Elev. (Meters):	80.8				80.8
Specified Length of Pile (Meters):	24.0				24.0
Reported Cut-off Elev. (Meters):	101.505				99.852
Reported Pile Tip Elev. (Meters):	80.8				80.643
Reported Length of Pile (Meters):	20.705				19.209
Permanent Steel Casing tip CMP	99.3				95.563
Date Poured					2/14/08

**Note:
- Ground Elevation = 104.8 +/- m

Caltrans

Page 1

APPENDIX C

CIDH Pile Design Data Form

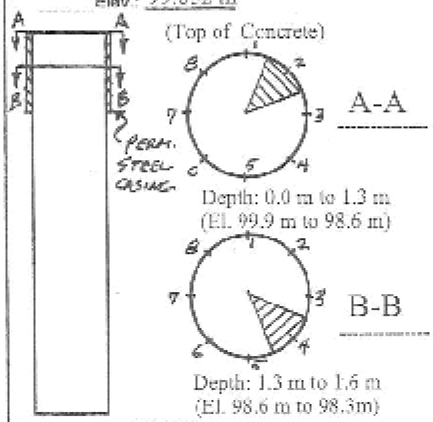
Engineering Service Center

(A) Foundation Testing Name: M. Harris Phone: 916-227-1058

Anomaly Overview

Testing Performed ☒ GGL ☐ CSL

Elev.: 99.852 m (Top of Concrete)



Depth: 0.0 m to 1.3 m (El. 99.9 m to 98.6 m)

Depth: 1.3 m to 1.6 m (El. 98.6 m to 98.3m)

Tip Elev.: 80.643 m

Anomaly Description

Section A-A: Low bulk density readings detected in one (1) GGL inspection tube. May affect up to 13% of pile cross-section.

Section B-B: Anomalies detected in one (1) GGL inspection tube. May affect up to 13% of pile cross-section.

(B) Structural Foundations or Geotech Oversight Name: ______ Phone: ______

As Designed Resistance of Pile

Compression: ______ Tension: ______

GWT Elevation: ______

Nominal Axial Load at Anomaly A-A

Compression: ______ Tension: ______

Soil Type: ______

Nominal Axial Load at Anomaly B-B

Compression: ______ Tension: ______

Soil Type: ______

(C) Structures Design or CCMB Name: ______ Phone: ______

As Designed Strength for Pile Cross Section:

Shear: ______ Moment: ______

Factored Load of Pile at Affected Area as described in (A) at Section A-A:

Shear: ______ Moment: ______

Pile is structurally ☐ Adequate ☐ Inadequate with the anomaly in place at this depth.

Factored Load of Pile at Affected Area as described in (B) at Section B-B:

Shear: ______ Moment: ______

Pile is structurally ☐ Adequate ☐ Inadequate with the anomaly in place at this depth.

Comments: ______

(D) Corrosion Name: ______ Phone: ______ If Required

For anomalies between the top of the pile and 1 meter below the lowest possible ground water surface, California Test Methods (CTM's) CTM 643 (Parts 2, 3, 4 and 6 only), CTM 417, and CTM 422 are required to assess any proposed repair. For anomalies outside these limits, and where no stray current source can be identified, no consideration of corrosion potential is required.

Corrosion Potential At Anomaly A-A: ______

Corrosion Potential At Anomaly B-B: ______

Bridge Name: I-15 Managed Lanes	Bridge No. 57-1133L Gm Valley Cr	Support #: Pier 2
Dist/Co./Route: 11-SD-15	E.A. 11-080924	Pile #: Right - Stage II
Structure Rep.: Gabriel Acero	Phone #: 858-748-4250	Fax #: 858-748-7964

Caltrans

Pile Design Data Form

Date: February 22, 2008

APPENDIX

H Tiebacks, Tiedowns & Soil Nails

Table of Contents

Tieback Construction Checklist

GENERAL OVERVIEW

Tiebacks are utilized in both temporary and permanent structures. Tiebacks are normally used to achieve higher walls or deeper excavations than can be achieved by cantilevered construction alone. In temporary construction, if soil conditions and physical development adjacent to the work area allows, tiebacks may be proposed by the Contractor to avoid the use of struts and bracing that may obstruct the work area. In temporary applications tiebacks are proposed and designed by the Contractor and included within their shoring submittal to the Structures Representative. In permanent applications tiebacks are normally utilized with soldier piling for timber lagged cantilevered tieback wall construction. As such, this checklist should be integrated with the "SOLDIER PILE CONSTRUCTION CHECKLIST".

The following checklist is intended to append Bridge Construction Memo 145-10.0, and serves as a stand-alone reference related solely to the installation of tiebacks. The Structure Representative is encouraged to employ the following checklist for tieback installations. If a problem or situation is encountered that is not addressed by this checklist, you are encouraged to contact either your Senior Bridge Engineer or the Earth Retaining Systems Specialist.

I. SOURCES OF TECHNICAL INFORMATION

A. Bridge Construction Records and Procedures Manual:
   1. 145-10.0 - Tieback Wall Construction Checklist:
      - Recent memo issued July 1, 2001.
   2. BCM 160-6.0 – Prestressed Concrete Working Drawings and Microfilms.

B. Foundation Manual:
   1. Chapter 11 – Tiebacks, Tiedowns, and Soil Nails
   2. Appendix H – Tiebacks, Tiedowns, and Soil Nails
      - Construction Checklist

C. Trenching and Shoring Manual:
      - Chapter 9 – Tiebacks: General theory on tiebacks.

D. Prestress Manual:
      - Focus on subjects including safety, prestress working drawings, strands/rods, bearing plates, wedges, jacks, stressing, grouting, and Appendices A through F.

II. SOURCES OF PROJECT SPECIFIC INFORMATION

A. Structures Pending File:

1. Foundation Report:
   Review recommended unbonded lengths for each tieback level beyond the Retaining Wall Layout Line (RWLOL).
   Note any comments concerning anticipated constructability problems:
   - Caving, presence of boulders, groundwater problems.
   - Structure Rep should verify Foundation Report comments regarding specifications or design features are incorporated into the Contract Documents.
2. Designer's notes.

B. Information Handout:

- Local, Regional, State, and Federal regulatory and permit specific requirements:
- Focus review on regulatory requirements/restrictions related to Structures contract items.

III. CONTRACTURAL REQUIREMENTS

A. Contract Special Provisions:

1. Section 5. General:
   - Section 5-1._ Local Agency/State Parks/DFG/RWQCB/USACOE Requirements
     Review requirements for all local, regional, State, and Federal regulatory agencies having jurisdiction over the work and note potential impacts to structures related work.
2. Section 8. Miscellaneous:
   - Section 8-3. Welding Quality Control:
     Note applicable shop and field welding for tieback related items:
     Bearing plates.
     Wedge plates – double pile systems.
3. Section 10. Construction Details:
   - Water Pollution Control Program – WPCP.
   - Tieback Anchors.

B. Contract Plans:

1. General Plan – Typical Section:
   - Note tieback levels and angle on inclination of tiebacks.
2. Retaining Wall Elevation:
   - Note varying tieback spacings for various tieback levels.

3. Retaining Wall Details:
   - General Notes:
     Note Design Force, T, for various tiebacks/levels.
4. Tieback Anchor Details:
   - Note differences between Alternative A & B Tiebacks.
   - Note Tieback Minimum Unbonded Length for various tiebacks/levels.
5. Log of Test Borings – LOTB:
   - Review LOTBs' with respect to wall layout line and tieback locations/elevations – the data provided may be extrapolated over the full depth of the tiebacks extending into the hillside.

C. Standard Specifications:
   1. Section 50 – Prestressing Concrete.

IV. JOB BOOKS SET-UP

A. Category 12 – Contractor's Submittals:
NOTE: Contact Documents Unit ASAP!
e-mail Manjit Sandhu at the Documents Unit as soon as possible with your latest office address/location and telephone number to avoid any unnecessary delays in receiving submittals for review and comment back to the design engineer.
   1. Tieback Working Drawing Submittals.

B. Category 20 – Contractor's Water Pollution Control Program.

C. Category 37 – Initial Tests and Acceptance Tests:
   1. Tieback Performance and Proof Testing.
   2. Grout Tests.

D. Category 41 – Reports of Inspection of Material:
   1. Tiebacks:
      - Sheathing.
      - Corrosion inhibiting grease.
      - Anchor heads.
      - Wedges.
      - Bearing plate/trumpet assemblies.
      - Grout caps – if required.

E. Category 42 – Welding Quality Control Program:
   1. Contractor's Welding Quality Control Program.
   2. CWI Inspection Reports related to tieback fabrications.

F. Category 43 – Concrete and Reinforcing Steel:
   1. Grout mix designs.

V. PRE JOB DISCUSSION WITH DESIGN & GEOLOGY/GEOTECHNICAL

A. Establish contacts with Structures Design and Geotechnical Services:
   1. Structures Design (SD) Contacts:
      Specifications – Look for RCE Stamp at beginning of Specials.
      Plans – Engineer of record on Plans.
   2. Geotechnical Services Contact:
      Name and phone number should be at bottom of Foundation Report.

B. Discuss/resolve any concerns developed either during review of project specific information, or as a result of preliminary site investigations.

VI. PRE CONSTRUCTION MEETING WITH CONTRACTOR

A. Remind Contractor of his responsibility to submit tieback working drawings, Welding Quality Control Program, Water Pollution Control Program measures related to tieback installations, and grout mix designs in a timely manner to allow sufficient time for review/comment/approval by the Structure Representative.

VII. SUBMITTAL REVIEWS

A. Tieback Working Drawings:
   1. SD responsibility to approve with Structures review/comment.

   NOTE: Memo to Designers 5-14, "Review of Working Drawings for Tieback Anchors":
   "The responsibility for checking working drawings is shared by the Designer and the Structure Representative".

   2. Verify:
      - Order of work defined in sufficient detail – see CONSTRUCTION.
      - Stressing equipment calibrated by METS within 1 year of intended use on the project.
      - Contingencies should difficult drilling/tieback installation conditions develop.
      - Testing loads specified agree with Plans/Special Provisions requirements.
      - Bearing plate dimensions provided do not result in overstress of concrete in compression (11 Mpa), steel in bending (0.55 f_y), or cast steel or iron (0.36f_y).
      - Unbonded lengths meet or exceeds the minimum length specified upon the Structures Construction Plans.
      - Check calculations:
        Lock-off shim thickness for varying design loads.
        80% elongation at test load.

- Upon receipt of TL-29s' for strands – compare As' & Es' with values on submitted working drawings.
- Corrosion inhibiting grease is specified over the unbonded, sheathed portion of the strands.
- Centralizers and spacing are noted.

B. Contractor's Water Pollution Control Program:
1. District responsibility to approve with Structures review/comment.
2. Verify regulatory/agency permit requirements w/in Section 5 of SPs' are addressed:
    - Containment of groundwater pumped from tieback drilled holes.
    - Containment of grout during grouting operations.

C. Welding Quality Control Program:
1. BCM 180-9.0.
2. Submitted WQCP forwarded to appropriate regional METS office for review/approval.
3. Structures Representative's responsibility to approve with METS review/comment/input.
4. Assure all applicable requirements and submittals per Special Provisions Section 8 are met:
    - Separate QCP for each Item of work.

VIII. CONSTRUCTION

A. Construction Sequence:
1. Soldier piles installed, face of wall excavated, timber lagging installed
    - Refer to the "SOLDIER PILE CONSTRUCTION CHECKLIST".
2. Drilling, installation, and primary grouting of tiebacks.
3. Post grouting (i.e., pressure grouting) at Contractor's discretion.
4. Concrete waler construction:
    - Bearing plate/sleeve assembly installation during concrete waler formwork construction.
    - Pour concrete waler.
    - Testing/stressing of tieback and lock-off to final service load.
    - Perform secondary grouting.
    - Install grout cap and complete third stage grouting.
4. Dual soldier pile construction:
    - Welding of bearing bar to piles.
    - Installation of wedge plate and bearing plate/sleeve assembly.
    - Testing/stressing of tieback and lock-off to final service load.
    - Perform secondary grouting.
    - Perform third stage grouting.
    - Form and pour concrete encasement over anchor assembly.

3. Prepare cut sheets for each pile:
   - Cut to required pile tip.
   - Cut to top of Class 3 Concrete (Backfill).
   - Cut to top of pile cut-off.

B. Delivery/Storage of Materials:
1. Tiebacks:
   - Perform thorough inspection of tiebacks for damage:
     Refer to repair procedures submitted by contractor for damaged sheathing.
   - Verify proper storage of tiebacks.
2. Bearing plate/sleeve (trumpet) assemblies:
   - Inspect for damage to assemblies, particularly damaged galvanizing.
   - Note different bearing plate dimensions for various tieback locations.

C. Equipment Mobilization:
1. Note and photograph all equipment mobilized to jobsite:
   - Drill rig, augers, and casings.
   - Grout plant, pumps, and compressors.
   - Testing/stressing equipment.
   - Settling tank, pumps, hoses – condition of hoses.
2. Verify all equipment mobilized conforms to Contractors tieback working drawing submittal.

D. Materials Inspection, Sampling, & Testing:
1. Each shipment of tiebacks should include:
   - TL-0624 Inspection Release Tags (orange).
   - Corrosion inhibiting grease – Certificate of Compliance, laboratory chemical analysis.
   - Corrugated HDPE exterior sheathing – Certificate of Compliance.
   - Smooth HDPE tendon sheathing – Certificate of Compliance.
2. Each shipment of steel fabrications should include:
   - TL-0624 Inspection Release Tags (orange) .
   - Certified Welding Inspection reports:
     For steel fabricated items required by Section 8 Welding Quality Control Program requirements.
   - Certificates of Compliance for fabrication and galvanizing.
3. Prior to testing/stressing, each shipment of anchor heads and strand wedges should include:
   - Anchor heads - Certificate of Compliance, material certifications, lot/batch numbers.
   - Strand wedges – Certificate of Compliance, material certifications, lot/batch numbers.

E. Drilling:
SAFTY CONCERNS:

- Review OSC Code of Safe Practices – Drilling Tiebacks and Soil Nails
- Stay away from rotating machinery.
- Maintain eye contact with drilling equipment operator.

1. Full Time Inspection Required:
    - Drilled hole activity is highest potential for Differing Site Condition – DSC to occur.
    - Begin logging holes immediately – don't wait for Contractor to file a claim.
    - Compare drill tailings (spoils) with information contained upon LOTBs'.
    - Note productivity rates for drilling advancement, soil conditions, presence of groundwater at given drilling depth/elevation.
    - Use high powered spotlight or mirror to observe soil structure over full depth of drilled hole.
    - Note any potential problem soils areas:
      Focus performance tests at tiebacks where questionable soil conditions are encountered.
    - Verify actual hole depth and actual drilled diameter:
      To estimate grout volume to assure tiebacks are being fully grouted.
      To determine maximum primary grout level outside of corrugated sheathing.

Potential Problems:

Rocks or boulders are encountered within drilled holes:

- Contractor is contractually on notice within "Tieback Anchors" of the Contract Special Provisions to anticipate and be prepared for difficult drilling conditions.
- Submitted/approved tieback working drawings should address.
- Amend as necessary in writing prior to proceeding.
- For encountering rocks and boulders, down hole pneumatic hammer drill rigs and drill bits should be employed.

Drilled holes caving:

Usually not detected until the tieback is attempted to be installed.

- Submitted/approved tieback working drawings should address.
- Amend as necessary in writing prior to proceeding.

Caving in dry holes:

- Casing system advancing with drilling.

Caving in wet holes:

- Casing system, or
- Tremie seal: 3-sack slurry & re-drill 24 hours later.

Tieback Installation:

1. Prior to installation:
    - Re-inspect each assembly for damage.
    - Verify bonded length:
      Cut sheathing in vicinity of bonded/unbonded zone, verify, and patch.
    - Ensure centralizers are installed:
      Provide one at end of strand.

Use even with casing advancing system.

2. Installation:
    - Re-inspect hole for presence of caving.
    - Do not allow Contractor to forcibly shove tiebacks down drilled hole.
    - Assure tieback installed to required depth.

G. Tieback Grouting:

1. Within corrugated sheathing:
    - Section 50-1.09 – 5 gals per sack of cement
2. Outside of corrugated sheathing:
    - Where holes exceed 8 inches in diameter special provisions allow fine aggregate added to grout mix.
    - Cement content is not less than 500 kg per cubic meter.
3. Post grouting (pressure grouting):
    - Section 50-1.09 requirements apply.
4. Grouting equipment:
    - Check equipment for wear.
    - Check plumbing for all required valves and gauges.
5. Water Pollution Control Program:
    - Address WPCP/Regulatory permit requirements.
    - Prevent run-off into drainage structures & natural courses.
6. Primary Grout Placement:
    - Within sheathing:
      Grout to end of sheathing.
      Continue until all air is expelled.
    - Outside of sheathing:
      Grouting via exterior tube with one-way valve:
      Two conditions:
        Holes less than 6 inches in diameter:
          Grout to within 6 inches of sleeve.
        Holes greater than 6 inches in diameter:
          Grout only the bonded length of the tieback.
          Difficult to verify – error on high side.
7. Post Grouting:
    - Involves pressure grouting in vicinity of bonded length.
    - At Contractor's discretion – not required by the Special Provisions.
    - Typically provided for on Contractor's tieback working drawings.
    - Grout pressure injected until specified pressure is achieved:
      Typically 300 to 500 psi.
    - Volume of grout injected and pressure recorded by Contractor.
    - Provides an indication as to the soil conditions within the bonded length.
8. Grout Volume Determinations:
    - Per Special Provisions, Contractor must record all primary grout volumes and furnish results to the Structures Representative – file within Category 41 for tiebacks.

- Have contractor pump one stroke of grout into a wheelbarrow or other suitable container and measure. Note number of pump strokes per nail grouted and compare with estimated volumes.

H. Bearing Plate/Trumpet Installation:
   1. Concrete waler construction:
      - Cast into waler structure.
      - Assure face of bearing plate is flush with face form.
   2. Dual piling system:
      - Installed prior to stressing.

I. Testing/Stressing:
   SAFETY:
      - Static nature of testing/stressing belies danger of forces involved
      - Hold a special safety meeting prior to testing including:
        All assistants and District personnel who may be working in the area
      - Topics to cover include:
        "OSC Code of Safe Practices – "POST TENSIONING OPERATIONS
      - Prestress Manual – "SAFETY"
      - Emphasize avoiding path of forces being applied
      - Inspect pumps and hoses for excessive wear
   1. Testing/stressing normally conducted against the permanent structures.
   2. Testing equipment:
      - Shims of varying thickness.
      - Hydraulic ram, pump, and pressure gauges:
        Must be calibrated by METS annually.
      - Device capable of measuring elastic/slip movement of strands to 0.025mm.
   3. Permanent stressing Devices:
      - Anchor head.
      - Seating wedges.
   4. Testing/stressing procedure:
      •Testing conducted on all tiebacks.
      - Threading anchor head onto strands:
        Assure proper arrangement of strands to achieve uniform loading of strands.
      - Placement of shims between anchor head and bearing plate.
      - Threading wedges onto strands and seating into anchor head.
      - Threading strands through ram and installing lock wedges and loading plate at upper end of ram.
      - Aligning ram along axis of tieback.
      - Apply alignment load to ram:
        Check for uniform bearing of anchor head on bearing plate and alignment of ram with strands.
      - Commence with testing.

5. Performance vs. Proof tests:
   - Performance testing:
     Cyclic loading with incremental increase in maximum load for each cycle up to 1.5*T.
   - Proof testing:
     Single cycle of loading to 1.5*T.
6. Performance Tests:
   - Special Provisions specify a required minimum number or test.
   - Locations per Structures Rep's observations:
     During plan review – highest design forces.
     During preliminary site investigation.
     During drilling operations.
     During post grouting operations.
7. Loading:
   - Each loading increment applied within 1 minute.
   - Each loading increment held for no more than 2 minutes.
   - Movement for each load increment is noted and recorded.
   - Test load hold at 1.5*T:
     Test load of 1.5*T held constant for 10 minutes.
8. Test acceptance criteria:
   - Performance tests:
     Elastic movement exceeds 80% of theoretical.
     Movement at 1.5*T test load hold less than 1 mm.
   - Proof tests:
     Elastic movements similar to performance tests.
     Movement at 1.5*T test load hold less than 1 mm.
9. Failed tests:
   - Elastic movement does not exceed 80% of theoretical.
   - Possible causes:
     Unbonded length insufficient – should be checked by visually inspecting prior to installation.
     Insufficient loading is being applied to the strands by the testing mechanism – verify accuracy of gauge pressure.
     Check load path from ram to strands for losses – check that wedges are properly seated within both the loading plate and permanent anchor head.
   - Contact Designer and Geotechnical Services contact for further direction.
   - Movement at test load hold exceeds 1 mm:
     Tieback is rejected.
     Test load held for additional 50 minutes.
     Movement recorded at 5 minute intervals.
     Deflection vs. time plotted & forwarded to Geotechnical Services.
10. Contractor's responsibility to address failed tiebacks:
    - Elastic movement does not exceed 80% of theoretical:
      If applied loading and load path are satisfactory, reject and replace tieback.

- Movement at test load hold exceeds 1 mm:
  Contractor normally repeats post grouting operation and retests after sufficient cure time.

11. File all testing results within Category 37 for tiebacks.

J. Lock-Off:

1. Lock-off results in relaxation of tieback force to 75% of Design Load.
   - 75% of design load is specified to achieve residual capacity within the tiebacks.
   - Lock-off conducted upon successful testing of tieback:
     Ram is backed off anchor head.
     Stands stressed to relax anchor head off shims.
     Shims between anchor head and bearing plate removed.
     Anchor head returned to bearing plate.
     Perform Lift-off test.
2. Lift-off test:
   - Verifies force in tieback.
   - Load re-applied to strands until anchor head lifts off of bearing plate.
   - Pressure/load at lift-off noted:
     Should be within 5% of required 75% of Design Force.
     Record final force in tieback upon test sheets.

   Potential Problems:

   Actual lift-off force exceeds 75% of Design Load in excess of 5% tolerance
   - Shim thickness used too thin
     Back strand wedges out of anchor head.
     Install thicker shim - 3mm increments.
     Re-stress to 1.5*T.
     Repeat lock-off & lift-off test.

   Actual lift-off force less than 75% of Design Load in excess of 5% tolerance:
   - Shim thickness used too thick.
     Back anchor head off of bearing plate.
     Install thinner shim - 3mm increments.
     Re-stress to 1.5*T.
     Repeat lock-off & lift-off.

K. Testing/Stressing Summary:

1. STRESSING/TESTING REQUIRES FULL-TIME, ATTENTIVE INSPECTION.
2. ASSURE RESIDUAL FORCE IN EACH TIEBACK IS PER CONTRACT DOCUMENTS WITHIN ALLOWED TOLERANCES.
3. BE SAFE AROUND TESTING/STRESSING OPERATIONS.

IX. PROJECT COMPLETION/AS-BUILTS

A. Bridge Construction Memo 9-4.0 applies:
   Do not forward post tensioning test results – maintain within the job files.
B. Note/draw any modifications on the as-built drawings on the number or location of tiebacks.

"Before". Facing east. Survey stakes placed. Slope is eroding (on the left). 12/31/03

"Before". Facing west. Survey stakes placed. Slope is eroding. Existing drainage pipe and creek on right.

"After". Facing east. Type 7 chain link fence. Type 27 modified concrete barrier on top of barrier slab. 6/24/04

"After". Facing west. Wall is complete. Two rows of concrete walers (with tieback anchors). Creek on the right. 6/24/04

Soldier Pile Construction Checklist

GENERAL OVERVIEW

The following construction checklist for soldier piles has been developed to assist field personnel in preparing documents and inspecting fieldwork to ensure compliance with contract requirements for the placement of soldier piles. It is important that the Structure Representative reviews the plans and specifications and conducts preconstruction meetings with the Contractor to lay out the procedures, identify potential field problems and solve those problems prior to the commencement of soldier pile installations.

Soldier piles are commonly used for temporary shoring and permanent timber lagged maintenance walls, with or without tiebacks. In permanent applications, soldier piles are simply CIDH piles with structural steel piling partially embedded within Class 3 concrete with the balance of the drilled hole filled with a sacrificial lean concrete backfill. For permanent soldier pile walls with tiebacks, this checklist should be integrated with the "TIEBACK CONSTRUCTION CHECKLIST".

The Structure Representative is encouraged to employ the following checklist for soldier pile installations. If a problem or situation is encountered this checklist does not address, you are encouraged to contact either your Senior Bridge Engineer or the Earth Retaining Systems Specialist.

I. SOURCES OF TECHNICAL INFORMATION

A. Trenching & Shoring Manual
   1. Chapter 10 – Soldier Piles

B. Foundation Manual:
   1. Chapter 6 – Cast-In-Drilled-Hole-Piles
      Foundation Manual does not currently address soldier piling
   2. Chapter 11 – Tiebacks, Tiedowns, and Soil Nails
   3. Appendix H – Tiebacks, Tiedowns, and Soil Nails
      - Construction Checklist

II. SOURCES OF PROJECT SPECIFIC INFORMATION

A. Structures Pending File
   1. Foundation Report
      Review recommended depth for soldier piles.
      Note any comments concerning anticipated constructability problems:

- Caving, presence of boulders, groundwater problems
- Structure Rep should verify Foundation Report comments regarding specifications or design features are incorporated into the Contract Documents.

2. Designer's notes
3. Engineer's Estimate quantity calculations for contract items related to Soldier Piling installations.

B. Information Handout:

1. Local, Regional, State, and Federal regulatory and permit specific requirements
   - Focus review on regulatory requirements/restrictions related to Structures contract items.

III. CONTRACTURAL REQUIREMENTS

A. Contract Special Provisions

1. Section 5. General
   - Section 5-1. Local Agency/State Parks/DFG/RWQCB/USACOE Requirements
     Review requirements for all local, regional, State, and Federal regulatory agencies having jurisdiction over the work and note potential impacts to structures related work
2. Section 8. Miscellaneous
   - Section 8-2. Concrete
     Class 3 Concrete (Backfill) requirements
   - Section 8-3. Welding Quality Control
     Note applicable shop and field welding to piles:
       Concrete anchors
       Shim plates – double pile systems
3. Section 10. Construction Details
   - Water Pollution Control Program - WPCP
   - Soldier Pile Earthwork
   - Piling
   - Steel Soldier Piling
   - Drilled Holes
   - Clean and Paint Steel Soldier Piling

B. Contract Plans

1. General Plan – Structures
   - Check curve data, profile grades and construction stationing for conformity with road plans.
2. Retaining Wall Elevations

- Check Superelevation Diagram from road plans and spot-check specified top of wall elevations within Pile Data Table.
- Note specified tip elevations & compare with LOTB for anticipated drilling/placement conditions. Pay particular attention to locations where caving may occur
- Note number of lagging at each pile location and determine top of Class 3 concrete elevations

3. Retaining Wall Sections
   - Note pay limits for soldier pile, clean and paint soldier pile, Class 3 concrete backfill
4. Retaining Wall Details
   - Note any required welding of/to soldier piling for Welding Quality Control Program requirements
   - Top of pile cut-off with respect to barrier rail slab
5. Log of Test Borings
   - Note groundwater surface (GWS) elevations – note time of year borings were taken – GWT level could seasonally fluctuate.
   - Focus review on soil structure in vicinity of drilled holes for soldier piles – be cognizant that plane of GWS could possibly tend uphill across varying elevations of soldier piles due to original ground surfaces beneath existing fills.

C. Standard Specifications
1. Section 19 – Earthwork
   - Section 19-3.062 – Slurry Cement Backfill
2. Section 49 – Piling
   - Section 49-4.03 Drilled Holes
   - Section 49-5 Steel Piles
3. Section 57 – Timber Structures
4. Section 58 – Preservative Treatment of Lumber, Timber, and Piling
5. Section 59 – Painting
6. Section 90 – Portland Cement Concrete
7. Section 91 – Paint

IV. JOB BOOKS SET-UP

A. Category 8 – Surveys
1. Soldier Pile cut sheets

B. Category 12 – Contractor's Submittals

NOTE: Contact Documents Unit ASAP!

e-mail Manjit Sandhu at the Documents Unit as soon as possible with your latest office address/location and telephone number to avoid any unnecessary delays in receiving submittals for review and comment back to the design engineer

1. Soldier Pile Earthwork Working Drawings
    - Required per Special Provisions section "Soldier Pile Earthwork"
2. Soldier Pile Placement Plan
    - Required per Section 49-1.01 of Standard Specifications

C. Category 20 – Contractor's Water Pollution Control Program
1. Drilled hole dewatering plan

D. Category 37 – Initial Tests and Acceptance Tests
1. Class 3 Concrete aggregate gradation and SE tests

E. Category 41 – Reports of Inspection of Material
1. Soldier Piling
2. Clean and Paint Soldier Pile

F. Category 42 – Welding Quality Control Program
1. Contractor's Welding Quality Control Program
2. CWI Inspection Reports

G. Category 43 – Concrete and Reinforcing Steel
1. Class 3 Concrete Mix Design
2. Lean Concrete Backfill Mix Design

V. PRE JOB DISCUSSION WITH DESIGN & GEOLOGY/GEOTECHNICAL

A. Establish contacts with Structures Design and Geotechnical Services, Foundations Branch –
1. Structures Design Contacts:
   Specifications – Look for RCE Stamp at beginning of Specials
   Plans – Engineer of record on Plans
2. Foundations Contact:
   Name and phone number should be at bottom of Foundation Report

B. Discuss/resolve any concerns developed either during review of project specific information, or as a result of preliminary site investigations.

VI. PRE CONSTRUCTION MEETING WITH CONTRACTOR

A. Remind Contractor of his responsibility to submit soldier pile earthwork drawings, soldier pile placement plans, Welding Quality Control Program, Water Pollution Control Program measures related to Drilled Hole activity, concrete mix designs, and proposed paint systems in a timely manner to allow sufficient time for review/comment/approval by the Structure Representative.

VII. SUBMITTAL REVIEWS

A. Soldier Pile Earthwork Drawings
   1. DSD responsibility to approve with Structures review/comment
   2. Likely to contain additional construction including lagging, tiebacks, etc
   3. Order of work should be defined in sufficient detail
   4. Special Provisions special requirements should be re-iterated

B. Soldier Pile Placement Plans
   1. DSD responsibility to approve with Structures review/comment
   2. Order of work should be defined in sufficient detail
   3. Proposed traffic control measures adhere to Special Provisions requirements
   4. Method of installation of soldier piles:
      - Proposed drilling equipment
      - Proposed de-watering methods – WPCP
   5. Contingency plans for difficult soil conditions
      - Very important to avoid claims by contractor
      - Casing methods
        Steel pipe casing
        Tremie seal:
        Placement of 3-sack lean concrete backfill and re-drill following day
        Drilling muds:
        Only if permitted within Special Provisions section "Drilled Holes"

C. Contractor's Water Pollution Control Program
   1. District responsibility to approve with Structures review/comment
   2. Verify regulatory/agency permit requirements w/in Section 5 of SPs' are addressed:
      - Pumping, treatment, and disposal of groundwater removed from drilled holes
      - Handling and disposal of drilling muds or chemical stabilizers – if permitted

D. Welding Quality Control Program
   1. BCM 180-9.0
   2. Submitted WQCP forwarded to appropriate regional METS office for review/approval
   3. Structures Representative's responsibility to approve with METS review/comment/input
   4. Assure all applicable requirements and submittals per Special Provisions Section 8 are met
      - Separate QCP for each Item of work

E. Paint
   1. Verify inorganic zinc primer on Department's list of approved brands
   2. Verify –finish coating systems are supplied by the same manufacturer of the primer system and are compatible with the primer system

VIII. CONSTRUCTION

A. Surveys
   1. Have Contractor submit staking request – review w/ Contractor prior to submittal
   2. Request surveys provide two points at stationing for each pile: one for grade at 6 feet offset from RWLOL and one for line approximately 6 feet beyond.
   3. Prepare cut sheets for each pile
      - Cut to required pile tip
      - Cut to top of Class 3 Concrete (Backfill)
      - Cut to top of pile cut-off

B. Delivery/Storage of Materials
   1. Assure piling is carefully handled when unloading and placed upon wooden sleepers
   2. Verify primer paint system has not been damaged due to shipment and handling – notify contractor of any required repairs to primer paint system
   3. Piece-mark each pile and field measure overall length for future reference when determining tip elevation and required cut-off

C. Equipment Mobilization
   1. Note and photograph all equipment mobilized to jobsite:
      - Drill rig, augers, clean out buckets, and core barrels
      - Hydraulic crane – check crane certifications
      - Casings - in event caving conditions encountered
      - Settling tank, pumps, hoses – condition of hoses
   2. Verify all equipment mobilized conforms to Contractors pile earthwork and placement submittals

D. Materials Inspection, Sampling, & Testing
   1. Soldier Piling
      - Locate and pull all TL-0624 (orange) inspection tags from each shipment of piles
      - Associate all TL-0624s' with TL-29s' transmitted from METS into Category 41
   2. Class 3 Concrete (Backfill)
      - Strength tests not required
      - Aggregate gradations, SE, & penetration daily

E. Drilling

SAFTY CONCERNS:

Stay away from perimeter of drilled hole

- unpredictable soil conditions at top of shaft
- could result in bank failure
- subsequent fall could result serious injury or death

Stay in visual contact with drill rig operator

- be constantly aware of operator's blind spots and direction of drill rig swing to clean auger of drill spoils
- when maneuvering around drill rig be aware of swing radius at back end of drill rig
- place caution flagging to warn personnel of swing radius

1. Full Time Inspection Required:
   - Drilled hole activity is highest potential for Differing Site Condition – DSC to occur.
   - Begin logging holes immediately – don't wait for Contractor to file a claim
   - Compare drill tailings (spoils) with information contained upon LOTBs'
   - Note productivity rates for drilling advancement, soil conditions, presence of groundwater at given drilling depth/elevation
   - Use high-powered spotlight or mirror to observe soil structure at face of drilled hole
   - Note depth ranges where changes in soil conditions occur, presence of caving occurs, depth to groundwater, etc.
   - Measure depth of hole with respect to 6 foot offset grade point to check clearance between bottom of hole and tip of pile and to estimate Class 3 concrete backfill quantities for hole.
   - Verify bottom of hole is cleaned of all loose drilling spoils prior to placement of soldier pile

Potential Problems

Caving

- Contractor's in driver's seat – refer to Contractor's Pile Placement Plan for contingencies
- Possible solutions:
  - Casing – difficult to retrieve during concrete placement with pile placed and plumbed in center of drilled hole
  - Tremie seal
    The drilled hole is advanced beyond the depth of caving and backfilled with typically a 3-sack slurry mix and allowed to set overnight. The hole is then re-drilled the following day through the 3-sack slurry, which essentially cases the hole. This procedure may need to be repeated within the same hole to achieve the necessary pile tip elevation.
  - Suspend drilling until the Class 3 concrete backfill arrives. The final drilling is then advanced, hole is cleaned, and the pile quickly placed,

plumbed, and secured, and then the hole is quickly backfilled with Class 3 concrete before caving continues. If the caving proceeds before the concrete can be placed, use of either a casing or a tremie seal will be necessary.

High groundwater infiltration

- Again, Contractor is in driver's seat
- Refer to Contractor's Pile Placement Plan for contingencies
- Verify settling tank is of sufficient capacity to contain water pumped from hole

F. Soldier Pile Installation

1. Lowering pile:
   - Assure soldier pile is slowly lowered into drilled hole
   - Prevent pile from hitting side of drilled hole causing caving
2. Setting pile:
   - Verify tip elevation is achieved
   - Verify pile clears bottom of drilled hole – 75 mm (3 inches) is adequate
   - Verify web of pile is aligned with reference points
   - Verify pile is centered on Retaining Wall Lay Out Line - RWLOL
3. Plumb pile:
   - Once the pile is properly placed vertically, the contractor normally rests two channels on the ground beside the drilled hole and clamps them to each flange of the pile to suspend the pile in the drilled hole during concrete placement

G. Class 3 Concrete (Backfill)

1. With pile set note top of pile elevation and determine cut to top of Class 3 Concrete (Backfill)
2. Verify solider piling remains plumb, aligned, and on RWLOL as concrete backfill is placed.
3. Carefully monitor level of Class 3 concrete backfill to prevent over-filling drilled hole beyond level required for bottom lagging course

NOPC WARNING!

Error on placing the Class 3 concrete backfill on the low side of the bottom lagging course elevation

- Class 3 Concrete (Backfill) too high?
- Extra work required to chip out the concrete
- The prime contractor may file a NOPC
- Therefore keep the Class 3 concrete backfill on the low side

H. Lean Concrete Backfill

1. Completely fill balance of drilled hole to prevent a safety hazard.

I. Welding

SAFETY

Avoid eye contact during stud welding/field welding operations

1. Concrete anchors:
    - Welder Qualification
        - Assure Contractor's personnel have been qualified by concrete anchor/welding equipment supplier for horizontal position for concrete anchor placement
    - CWI inspection required
2. Double pile systems:
    - Welding of shims takes place after placement and grouting of tiebacks.
    - CWI inspection required
3. Welding Quality Control Program
    - CWI need not be onsite during anchor placement, however must inspect and accept anchors prior to commencement with reinforcing steel placement
    - Structure Representative or Assistant should follow-up CWI's inspection – even a CWI has been prone to overlook an unacceptable anchor.
    - File all CWI inspection reports within Category 42

J. Painting
1. Shop Prime:
    - Inspected by METS in fabricator's facility
2. Finish Coats:
    - Verify temperature and humidity are acceptable for paint application
        - Refer to paint manufacturer's technical data sheets
    - Assure minimum dry film thickness will be achieved
        - Spot check wet film thickness as paint is applied
    - Application of second coat is delayed by over seven days:
        - Contractor required to pressure wash surface prior to second finish coat application

IX. PROJECT COMPLETION/ AS BUILTS

A. As-Built Drawings
1. Changes to pile tip elevations
2. Indicate upon log of test borings deviations from subsurface conditions indicated including presence of groundwater

B. Completion of Report of Completion - Bridges DS-OS C3
1. Soldier piling source and supplier
2. Stud connectors source and supplier
3. Paint system, manufacturer and supplier.

APPENDIX

Case Study- Soldier Pile Wall with Tieback Anchors

Contract No. 04-1S2724 04 - SM - Rte 84 - PM 10.2
Soldier Pile Wall with Tieback Anchors (7.7 km west of Rte 35).
Completed in June 2004. $850,000.00.

Hwy 84 runs through the Santa Cruz Mountains in San Mateo County. The forested hills form a ridge that separates the Pacific Ocean from San Francisco Bay and from Santa Clara Valley. Hwy 84, along with Hwy 1, 9, 17, and 35, are continuously being improved because of unstable slopes and erosion.

Project 04-1S2724 specified a soldier pile wall with ***tieback anchors***. The wall prevents further erosion of the slope adjacent to a creek. The design specified 44 soldier piles with timber lagging. Two rows of tieback anchors are connected to concrete walers. A concrete barrier slab and concrete barrier with chain link fence is on top of the wall.

Soldier Pile Wall with Tieback Anchors within two concrete walers. Date: June 2004.
Fiber rolls placed along slope for erosion control. Photo by D. Dait, P.E., Sr Br Engr

Soil Nail Construction Checklist

GENERAL OVERVIEW

Soil nails provide a means to reinforce and strengthen an existing soil structure in order to achieve a slope face steeper than the natural angle of repose. Soil nails provide tensile reinforcement for soils that typically exhibit low tensile strength. They are termed "passive inclusions" as they are not pre-tensioned but rather simply grouted in place along their full embedment into the ground. Tensile forces develop within soil nails as active failure planes develop in the soil mass reinforced by the soil nails. The soil nails are designed with sufficient embedment depths to adequately transfer the tensile stresses developed by the active soil mass pressures back into stable soil structures behind the active failure planes.

Common applications of soil nails include but are not limited to:

- Temporary shoring walls.
- Permanent walls into cut slopes and bridge abutment fill slopes for roadway widenings.
- Slope stabilization measures.

Construction of soil nail walls commonly follow or closely follow the following sequence:

Excavation by lifts from the top of slope, downward (i.e., "top-down" construction) to depths generally limited by some dimension below the horizontal level of each row of soil nails.

Drilling for, insertion of, and grouting of soil nails.

Placement of geocomposite drain material in between soil nails.

Placement of welded wire fabric across surface of excavated lift and continuous reinforcing steel above and below row of soil nails and application of shotcrete to construct a temporary shoring of the excavated embankment slope.

Embedment into the wet shotcrete of anchor plates upon soil nails to transfer lateral active soil pressures from the shotcrete facing into soil nails.

Completion of sub-drainage system at base of soil nail wall.

Construction of a permanent cast-in-place wall facing affixed to anchor plates embedded into the shotcrete temporary shoring.

I. SOURCES OF TECHNICAL INFORMATION

A. Foundation Manual:
   1. Chapter 11 – Tiebacks, Tiedowns, and Soil Nails.
   2. Appendix H – Tiebacks, Tiedowns, and Soil Nails.
      - Construction Checklist

B. "Soil Nailing Field Inspectors Manual":
   Federal Highway Administration Publication No. FHWA-SA-93-068
   Very extensive technical publication focusing on the inspection of soil nail wall construction and the resolution of common problems encountered in the process.

II. SOURCES OF PROJECT SPECIFIC INFORMATION

A. Structures Pending File:
   1. Foundation Report:
      - Review recommended lengths for each soil nail level beyond the Retaining Wall Layout Line - RWLOL.
      - Note any comments concerning anticipated soil nail constructability problems:
        Caving, presence of boulders, groundwater problems.
      - Structure Rep should verify Foundation Report comments regarding specifications or design features are incorporated into the Contract Documents.
   2. Designer's notes.

B. Information Handout:
   1. Local, Regional, State, and Federal regulatory and permit specific requirements:
      - Focus review on regulatory requirements/restrictions related to Structures contract items.

III. CONTRACTURAL REQUIREMENTS

A. Contract Special Provisions:
   1. Section 5. General:
      - Section 5-1._ Local Agency/State Parks/DFG/RWQCB/USACOE Requirements:
        Review requirements for all local, regional, State, and Federal regulatory agencies having jurisdiction over the work and note potential impacts to structures related work.
   2. Section 8. Miscellaneous:

- Section 8-2. Concrete:
  Shotcrete requirements – 375 kg/m^3 cementious material– Class 1.5 concrete
3. Section 10. Construction Details:
   - Water Pollution Control Program – WPCP.
   - Designated Waste Handling – if applicable.
   - Water Pollution Control.
   - Designated Waste Handling – if applicable:
     Of importance if potential for subsurface contaminated soils exists.
   - Soil Nail Wall Earthwork.
   - Soil Nail Assembly.
   - Geocomposite Drain.
   - Shotcrete.

B. Contract Plans:
1. General Plan & Elevation:
   - Check stationing, grades, and bearings with District layout plans.
2. Typical Section:
   - Note inclination angles for various soil nail levels.
   - Note offset distances to RWLOL.
3. General Notes:
   - Note Ultimate Bond Stress σ_b for pullout test load determination.
4. Structure Plans/Elevations:
   - Note top and bottom wall elevations.
   - Note/check dimensioning and stationing with General Plan.
   - Check continuity of grades and stations at match lines between sheets.
5. Foundation Plans:
   - Review against District layout, utility, and drainage plans.
   - Check for conflicts and required coordination with other agencies.
6. Soil Nail Details:
   - Note dimensions from RWLOL to face of shotcrete/wall excavation.
   - Test soil nail assembly detail:
     Note required embedment and bonded lengths.
   - Production soil nail assembly detail:
     Note required embedment lengths for various soil nail levels.
   - Drainage Details:
     Note geocomposite drain placement with respect to soil nail locations.
7. Soil Nail Layouts:
   - Note soil nail spacings and dimensions from top and bottom of finished wall.
     Combine with Structure Plans/Elevations to develop cut sheets for soil nail installations.
   - Note test soil nail assembly locations.
8. Log of Test Borings – LOTB:
   - Review LOTBs' with respect to wall layout line and soil nail locations/elevations – the data provided may be extrapolated over the full depth of the soil nails extending into the hillside.

C. Standard Specifications:
   1. Section 19: Earthwork:
      - Section 19-3 Structure Excavation and Backfill.
   2. Section 50: Prestressing Concrete:
      - Section 50-1.09 Bonding and Grouting.
   3. Section 52: Reinforcement.
   4. Section 53: Shotcrete.
   5. Section 88: Engineering Fabrics.

IV. JOB BOOKS SET-UP

A. Category 12 – Contractor's Submittals:

NOTE: Contact Documents Unit ASAP!

e-mail Manjit Sandhu at the Documents Unit as soon as possible with your latest office address/location and telephone number to avoid any unnecessary delays in receiving submittals for review and comment back to the design engineer.
   1. Soil Nail Earthwork Drawing Submittals.
   2. Soil Nail Working Drawing Submittals.

B. Category 20 – Contractor's Water Pollution Control Program.

C. Category 37 – Initial Tests and Acceptance Tests:
   1. Refer to BCM 4-5.4.
   2. Include:
      - Test Soil Nail testing results.
      - Grout tests.
      - Shotcrete tests.

D. Category 41 – Reports of Inspection of Material:
   1. Includes METS Source Inspections, Field Release of Materials, Certificates of Compliance
   2. Refer to BCM 4-5.6
   3. Include:
      - Soil Nails.
      - Bearing plates, couplers, beveled washers.
      - Geocomposite drain material.
      - Sub-drainage piping materials.
      - Welded wire fabric.
      - Reinforcing steel.

E. Category 43 – Concrete and Reinforcing Steel
   1. Include:
      - Grout mix designs.
      - Shotcrete mix designs.

V. PRE JOB DISCUSSION WITH DESIGN & GEOLOGY/GEOTECHNICAL

A. Establish contacts with Structures Design and Geotechnical Services:
   1. Structures Design (SD) Contacts:
      Specifications – Look for RCE Stamp at beginning of Specials
      Plans – Engineer of record on Plans.
   2. Geotechnical Services Contact:
      Name and phone number should be at bottom of Foundation Report.

B. Discuss/resolve any concerns developed either during review of project specific information, or as a result of preliminary site investigations.

VI. PRE CONSTRUCTION MEETING WITH CONTRACTOR

A. Remind Contractor of his responsibility to submit soil nail earthwork drawings, soil nail working drawings, Water Pollution Control Program measures related to soil nail installations, grout mix designs, and shotcrete mix designs in a timely manner to allow sufficient time for review/comment/approval by the Structure Representative.

VII. SUBMITTAL REVIEWS

A. Soil Nail Earthwork Drawings:
   1. SD responsibility to approve with Structures review/comment.
   2. Review with respect to soil nail installations.
   3. Verify excavation restrictions are specified.

B. Soil Nail Working Drawings:
   1. Memo to Designers 5-14, “Review of Working Drawings for Tieback Anchors” applies for soil nails as well:
      • “The responsibility for checking working drawings is shared by the Designer and the Structure Representative”.
   2. Verify:• Conformity with Construction Plan Soil Nail Details.
      • Stressing equipment calibrated by METS within 1 year of intended use on the project.
      • Ram proposed for testing is properly sized:
        Lower 10% of rated capacity not used for testing• Contingencies specified should difficult drilling/soil nail installations develop.
      • Testing loads specified agree with Plans/Special Provisions requirements.
      • Embedment lengths for production and test soil nails meet minimum lengths specified upon the Structures Soil Nail Details.
      • Bonded lengths for test soil nails meet minimum lengths specified upon the Structures Soil Nail Details.
      • Repair procedures for damaged epoxy coatings.
      • Centralizers and spacing are noted.

C. Grout Mix Designs:
   1. Section 50-1.09 of Standard Specifications.

D. Shotcrete Mix Designs:
   1. Section 53-1.02 of Standard specifications

E. Contractor's Water Pollution Control Program:
   1. District responsibility to approve with Structures review/comment.
   2. Verify regulatory/agency permit requirements w/in Section 5 of SPs' are addressed:
      - Containment of groundwater pumped from soil nail drilled holes.
      - Containment of grout during soil nail grouting operations.

VIII. CONSTRUCTION

A. Construction Sequence:
   1. Excavation by lifts from top of slope downward – i.e., "top-down" construction.
   2. Drilling, insertion, and grouting of production and test soil nails.
   3. Placement of geocomposite drains between soil nails.
   4. Placement of welded wire fabric over face of excavation.
   5. Placement of continuous reinforcing bars along soil nail row.
   6. Application of shotcrete for temporary shoring of slope.
   7. Embedment of bearing plate over soil nails into shotcrete.
   8. Testing of soil nails.9. Construction sequence repeats to bottom of wall.
   10. Completion of sub-drain collection system from geocomposite drains.
   11. Placement of permanent wall facing.

B. Layout:
   1. Review staking request with Contractor prior to submittal.
   2. Structures Representative responsible for initial line and grade for embankment excavation and soil nail installations.
   3. Establish additional references for horizontal and vertical as top-down construction progresses using:
      - Soil nails.
      - Re-bar embedded into shotcrete.

C. Delivery/Storage of Materials:
   1. Soil Nails:
      - Check epoxy coating and encapsulation for any damage.
      - Refer to repair procedure within the Contractor's Soil Nail Working Drawings.
   2. Centralizers:

- Verify centralizers adequately support soil nail bar in center of drilled hole.
- Verify centralizers for different drilled hole diameters are delivered if the Contractor's Soil Nail Working Drawings indicate a potential for different sized holes.

D. Equipment Mobilization:
1. Note and photograph all equipment mobilized to jobsite:
   - Drill rig, augers, and casings.
   - Grout plant, pumps, and compressors.
   - Testing equipment.
   - Settling tank, pumps, hoses – condition of hoses.
2. Verify all equipment mobilized conforms to Contractors soil nail working drawing submittal.

E. Materials Inspection, Sampling, & Testing:
1. Initial and acceptance tests – Category 37:
   - Grout tests:
     Verify <2% air per Ca Test 504.
     Verify initially 90 mm penetration per Ca Test 533.
   - Shotcrete tests:
     Gradations and SE – daily production.
     Compressive strength–Average of three cores:
     Every 30 m^2 of wall area placed.
2. Field Releases – Category 41:
   - Field inspect and release on form DH-OS C53, the following materials:
     Soil nails.
     Bearing plate assemblies.
   - Certificates of Compliance are required for the following items:
     Soil nails, encapsulation PVC, epoxy coatings.
     Bearing plate assemblies.
     Couplers.
     Beveled washers.
     Geocomposite drain.
     Sub-drain piping.
     Re-bar, welded wire fabric.
     Cement.
3. Materials Certifications – Category 41:
   - Mill certs required for:
     Soil nails.
     Fabricated steel assemblies.
     Couplers.
     Beveled washers.
     Re-bar and welded wire fabric.

F. Soil Excavations:SAFTY CONCERNS:
   Review OSC Code of Safe Practices – Drilling Tiebacks and Soil Nails:
   - Section IV – Excavations.
   - Section XVII – Earthwork:
     Stay away from vertical cut slopes.
   - Note signs of soil distress.
   - Note changes in soil structures.
   - Maintain eye contact with excavation equipment operator.

   1. Monitor embankment excavation noting:
      - Potential unstable soils:
        Cohesionless sands and gravels.
        Saturated soils.
      - Unstable soils may not remain stable over period of exposure prior to shotcrete placement.
      - Perched groundwater day lighting at face:
        Could affect curing of shotcrete.
      - Excavation limits below soil nail levels do not exceed depth permitted by Special Provisions.

   Potential Problems:
   Embankment collapse:
   - Caused by disturbed or poor soil conditions.
   - Solutions differ by circumstances:
     Flash excavated face with shotcrete:
       Not always practical.
     Segmental slot excavations.
     Cut back bank to stable slope.

   Groundwater day lighting at face of cut:
   - Solutions include:
     Place additional geocomposite drain material.

   Embankment collapse behind wall above:
   - Solutions differ by circumstances:
     Shotcrete up into cavity
       Not always practical
     Erect plywood forms and apply shotcrete against plywood
       Core shotcrete at cavity and pump controlled fill material

G. Drilling:
   1. Requires full-time inspection:
      - Begin logging holes immediately – don't wait for Contractor to file a claim.
      - Compare drill tailings (spoils) with information contained upon LOTBs'.
      - Note productivity rates for drilling advancement, soil conditions, presence of groundwater at given drilling depth/elevation.
      - Note locations exhibiting poor soils.
      - Use high-powered spotlight or mirror to observe soil structure over full depth of drilled hole.
      - Note any potential problem soils areas.

- Verify actual hole depth and actual drilled diameter to estimate grout volume to assure soil nails are being fully grouted.

2. Test Soil Nails:
    - Adjust location shown on plans or require additional test soil nails if necessary to test poor soil conditions.
    - Measure actual diameter for test load determination.
    - Verify adequate depth is achieved and hole is cleaned prior to soil nail installation.
    - Note hole depth and with actual drilled diameter, determine required grout volume and cut-off of grout to assure the specified bonded length for test nails is accurately achieved.

Potential Problems:

Rocks or boulders are encountered within drilled holes:

- Submitted/approved soil nail working drawings should address.
- Amend as necessary in writing prior to proceeding.
- Contractor is contractually on notice within "SOIL NAIL ASSEMBLY" of the Contract Special Provisions:
  On notice to anticipate and be prepared for difficult drilling conditions.
  For encountering rocks and boulders, down hole pneumatic hammer drill rigs and drill bits should be employed.

Drilled holes caving:

- Usually not detected until soil nail installation is attempted.
- Submitted/approved soil nail working drawings should address.
- Amend as necessary in writing prior to proceeding.
- Caving in dry holes:
  Casing system advancing with drilling auger.
- Caving in wet holes:
  Casing system, or tremie seal:
    3-sack slurry & re-drill 24 hours later. Differing Site Condition – DSC:
- High potential for DSC during soil excavation and drilled hole activity.
- Contractor is contractually on notice to anticipate and be prepared for difficult excavation and drilling conditions.
- Difficult conditions should be addressed within the submitted/approved soil nail earthwork and working drawing submittals:
    Amend as necessary in writing prior to proceeding

H. Soil Nail Installations:

1. Prior to installation:
    - Verify drilled hole depth and cleanliness.
    - Ensure centralizers are properly installed and secured to soil nails.
2. During installation:
    - Verify centralizers do not gouge into soil.

Potential Problem:

Centralizers rut bottom of drilled hole.

- Results in bar not being centered in hole.

Solution:

- Use 150 mm length of CHPEP cut in half as a trough to slide nail down hole – withdrawing trough cleans out remaining drilling spoils.

I. Soil Nail Grouting:
   1. Where holes exceed 150 mm in diameter special provisions allow fine aggregate added to grout mix:
      - Cement content is not less than 600 kg per cubic meter.
   2. Grout tests:
      - 2% air per Ca Test 504.
      - 90 mm penetration per Ca Test 533:
        Only on initial trial batch
   3. Grouting equipment:
      - Check equipment for wear.
      - Review Water Pollution Control Program for grouting operations:
        Address WPCP/Regulatory permit requirements.
        Prevent run-off into drainage structures & natural courses.
   4. Requires full-time inspection:
      - Verify grout tube is fully inserted into hole.
      - Ensure grout tube remains embedded within advancing grout to avoid air pockets.

   Potential Problems:
   Soil nails pushed into hole during grout tube insertion.
   Soil nail bending at face of drilled hole.
   Solution:
   - Tie soil nail to welded wire fabric prior to grouting.
   5. Test Soil Nails:
      - Verify actual limits of grout placement to determine actual bonded length for each individual test soil nail.
   6. Grout Volume Determinations:
      - Have contractor pump one stroke of grout into a wheelbarrow or other suitable container and measure. Note number of pump strokes per nail grouted and compare with estimated volumes.

J. Shotcrete Application:
   1. Shotcrete functions as temporary shoring prior to final facing.
   2. Method of application can affect performance:
      - Apply in horizontal lifts along length of exposed cut to avoid sloughing.
   3. "Wet set" bearing plate over soil nails to obtain flush bearing upon shotcrete.

K. Soil Nail Testing:
   SAFETY:
      - Static nature of testing/stressing belies danger of forces involved
      - Hold a special safety meeting prior to testing including:
        All assistants and District personnel who may be working in the area
      - Topics to cover include:
        "OSC Code of Safe Practices – "POST TENSIONING OPERATIONS.
      - Prestress Manual – "SAFETY."

- Emphasize avoiding path of forces being applied.
- Inspect pumps and hoses for excessive wear.

1. Testing is normally conducted against the shotcrete wall facing.
2. Testing equipment:
   - Hydraulic ram, pump, and pressure gauges:
     Must be calibrated by METS annually.
   - Device capable of measuring movement of soil nail to 0.025mm. (English)
   - Use temporary bearing yoke to avoid punching shear through shotcrete:
     Verify bearing area will not distress shotcrete.
3. Test Loads:
   - Pullout test loading is specified within the Special Provisions:
   - Commonly specified as: $M = k * \sigma_b * D$ where:
     M = the maximum pullout test load.
     k = a constant provided within the Specials.
     σ_b = ultimate bond stress provided within the General Notes within the Structures Construction Plans.
     D = actual diameter of drilled hole for test nail, in meters.
   - Verify the constant, k, agrees with the basic equation:
     $M = \pi * D * L_b * \sigma_b * F.S.$ where:
     L_b = bonded length as specified upon the plans.
     F.S. = factor of safety – typically 1.5 for pullout.
   - NOTE:
     Careful monitoring of the actual drilled hole diameter and embedment length is essential to accurately verify successful testing of the soil/cement bond. If a test soil nail embedment length is excessive, require of the Contractor to apply a test load determined by the basic equation.
4. Testing procedure:
   - Install temporary bearing yoke over nail against shotcrete
     Verify uniform bearing against shotcrete.
   - Place ram and loading plate over nail and install coupler and bring ram up to alignment load.
   - Ensure ram is aligned along axis of soil nail.
   - Commence with loading sequence.
5. Loading sequence:
   - Test load applied in 0.1 increments of M.
   - Load at 0.7M held for 10 minutes.
   - Each loading increment applied within 1 minute and held for no more than 2 minutes (except 0.7M loading).
   - Movement for each load increment is noted and recorded on Contractor's supplied test data sheet.
6. 10 minute load hold at 0.7M:
   - Movement recorded at 1,2,3,4,5,6, and 10 minutes.
   - Total movement < 2 mm @ 10 minutes?:
     Incremental test loading continues to M.
   - Total movement > 2 mm @ 10 minutes?:
     0.7M load maintained for additional 50 minutes.
     Movement recorded at 15,20,25,30, 45, and 60 minutes.

Incremental test loading continues to M unless failure occurs.

7. Test acceptance criteria:
   - Successful testing is achieved whenever the pullout test load M is achieved without:
     Continuous movement of test soil nail, and
     Movement in excess of 50mm.
   - Soil Nail Test Records:
     Contractor to complete.
     Forwards to Structures Representative.
     File within Category 37.
     Forward copy of results to Geotechnical Services – if requested to do so.

Potential Problems:

Test Soil Nail fails:

- Notify Geotechnical Services contact immediately, forwarding test results.
- Exhume test nail to asses cause of failure:
  Bond length not fully grouted:
  Install new test nail and re-test.

Solutions differ depending upon degree of failure:

- Design and Geotechnical Services should take lead on course of action.
- Augment wall with additional soil nails.
- Deepen existing nails:
  Use couplers to utilize existing nails:
  Not always practical – easement restrictions, poor soil conditions, etc.
- Perform a secondary fracture grouting around the existing bonded length:
  An alternative when bond values are lower than those estimated.
  Additional expense as extra work – requires a Contract Change Order.
  Requires drilling and installing grout tubes to inject the grout.

L. Onward and Downward:

1. Next level of excavation cannot proceed until wall area directly above is structurally complete:
   - Structurally completeness:
     Soil nail assemblies installed.
     Shotcrete cover has set.
     Testing has been satisfactorily completed and results have been furnished to Structures Representative.
2. Maintain a minimum 3 meter spacing between wall areas not structurally complete and subsequent lower level excavations:
   - Normally not a concern as Contractor typically completes one continuous level of wall at a time, returning to the opposite end of the wall for the next subsequent lower level.

IX. PROJECT COMPLETION/AS-BUILTS

A. As-Built Drawings:

- Indicate limits for any grout, controlled fill material, or shotcrete where backfilled behind shotcrete walls at slip-out areas.
- Indicate locations of actual and any additional test soil nails installed.
- Indicate locations of any additional production nails installed.

APPENDIX

Case Study - Soil Nail Wall

Contract No. 04-1123C4

04-SM - Rte 1- KP 61.2/62.2

Soil Nail Wall (South Rock Cut) near Devil's Slide.
Construction began in Spring 2005 and finished Spring 2006.

Description of Work:
The South Rock Cut Soil Nail Wall project, located in San Mateo County between the City of Pacifica and town of Montara, is part of the overall Devil's Slide Tunnel and Bridge work.

The large "rock-cut" at the Tunnel's south portal area is planned to align the highway and to provide adequate site distance. The face of the large rock-cut is designed to match the appearance of existing rock-cuts in the immediate view.

The South Rock Cut wall consists of a ***soil nail wall***. The soil nail wall is composed of two walls separated by a 25m concrete barrier. Total length of walls is 281m (RW No. 2 is 190m long, RW No. 1 is 91m long). The soil nail assembly pay item equals 18, 860 meters.

Retaining Wall #2. Shotcrete sculpting at northern end of wall.
Photo by Ann Meyer, P.E., Structure Representative. Photo Date: Feb.24, 2006.

Drilltech drilling top row of nails at Wall #2. Test nail installed between two nails. 8/09/05

Drilltech shooting next section of initial shotcrete at Wall #1. 8/11/05

Soil nail getting ready to be installed in a drilled hole. Grouting lower nail w/1" PVC grout tube. 8/04/05

Drilltech crew installed 12" geocomposite drain strips between nail columns.

GNB installing tieback walers at temporary shoring wall at Wall #1. 6/28/05

GNB begins hand excavation at Wall #1. 7/01/05

Drilltech begins drilling at Wall #2. 8/04/05

Drilltech installing soil nails with forklift at top row of Wall #2. 8/05/05

Drilltech workers shotcreting at Wall #1. 8/04/05

Northern end of Wall #2, sculpted. 03/21/06

Southern end of Wall #1, after staining complete.
Matches existing geology and color well. 04/18/06

Type 60D barrier being installed at Wall #2. 03/21/06

APPENDIX

I Cofferdams and Seal Courses

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BRIDGE DESIGN AIDS JUNE 1990

Caltrans

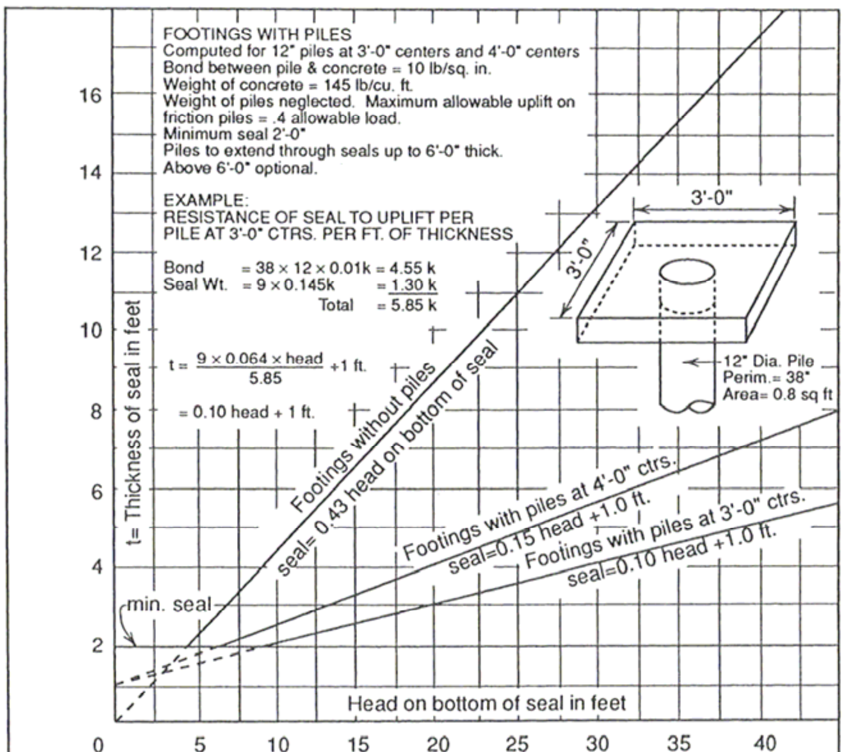
Seal Courses

A) General: The foundation report will give information relative to ground water conditions at the site and will indicate whether or not seal courses will be needed. If the foundation report indicates that seal courses will be needed the designer should show the seals on the plans, indicating the elevation of the bottoms of the seals and their thicknesses based on a careful consideration of foundation bearing value, anticipated hydrostatic head, and the permissible highest elevation of the top of the reinforced concrete footing.

B) Thickness: The figure shows the required thicknesses of seal courses for footings with and without piles.

Thickness of Seal Courses

Spread Footings and Friction Piles



NOTE:

For piles other than 12" diameter and for large footings the designer should make a special study.

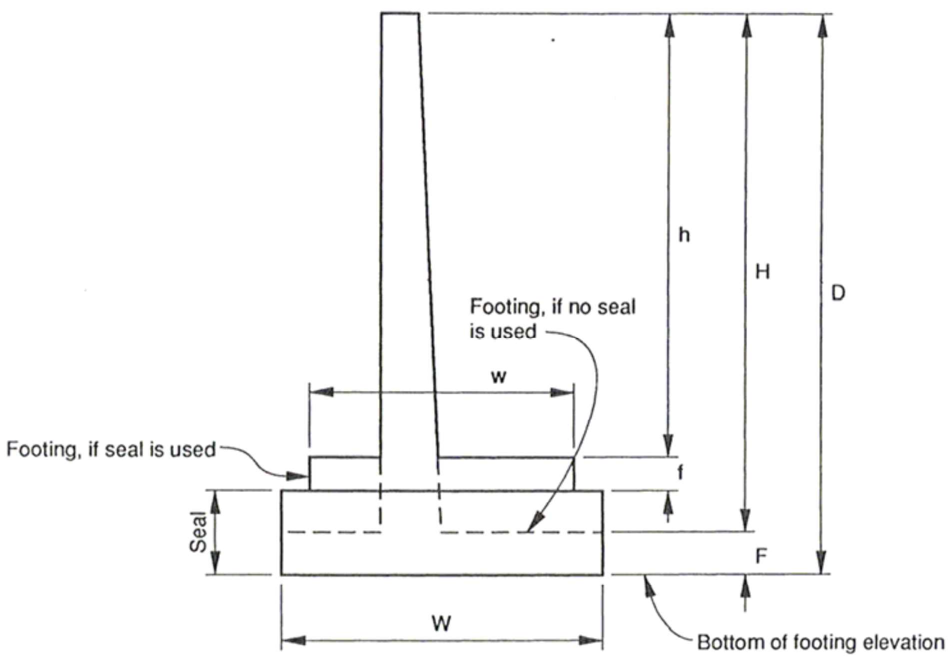
An additional 1 ft. of seal is provided for pile footings to allow for irregularity to top and bottom surfaces.

"Head" may be greater than the depth of standing water if footing is located in heavy viscous soil such as "Bay Mud"

2-6

C) Width of Seal Courses: In some cases, particularly in the case of reaining walls, the width of the footing is a function of the height (h) of the wall above the top of the footing. When seal courses are shown on the plans to be placed below spread footings of retaining walls, the width (w) of the seal shall be the same as would be used if the seal were omitted and the retaining wall footing constructed with its bottom at the elevation shown for the bottom of the seal. If the seal is used, the width (w) of the footing slab (as constructed on top of the seal) shall be a function of the height (h) of the wall above the top of the footing slab. The designer should indicate clearly on the plans the procedure to be followed in the field in the event the elevation of the bottom of the seal is changed from that shown. Except in special cases where extremely deep footings or great seal thicknesses would be required, the above method of establishing footing dimensions shall be used.

Below is a sketch showing graphically the intent of this article.



W = Width of footing for H
F = Thickness of footing for H
H = D – F (Dimension from top of wall to bottom of footing elevation minus F)

Widths of Seal Course

2-7

SEAL COURSE PROBLEM

Given : 14" square piles, Spacing 3'-6" by 4'-0" centers, Hydrostatic head of 15'-0".

Assume: Unit Wt. Concrete 145.0 pcf, Unit Wt. Water 64.0 pcf, Friction Pile/Seal = 10.0 psi, Friction Seal/Sheet Pile = 0.0 psi.

Calculate required thickness of concrete to resist uplift than add 1'-0"for seal course thickness.

Uplift Force = Wt. water X Head X Pile Spacing
= 64.0 X 15.0 X 3.5 x 4.0
= 13,440 #

Resisting Force = weight of concrete + friction (pile/seal)

Weight of concrete (1.0 foot thick) = Unit Wt. Conc. X Pile Spacing X 1.0
= 145.0 X 3.5 X 4.0 X 1.0
Concrete = 2,030.0 #

Friction on 1'section of pile = Perimeter X Height X 10.0 psi
= 14.0 X 4 X 12.0 X 10.0
Friction = 6,720.0 #

(Friction + Concrete) X Thickness = Uplift
(2,030.0 + 6,720.0) T = 13,440.0
T = 13,440.0/8,750.0
T = 1.54 feet

Seal Course Thickness is 1.51 + 1.0 = 2.5 feet > 2.0 OK

APPENDIX

J Micropiles/Alternative Piles

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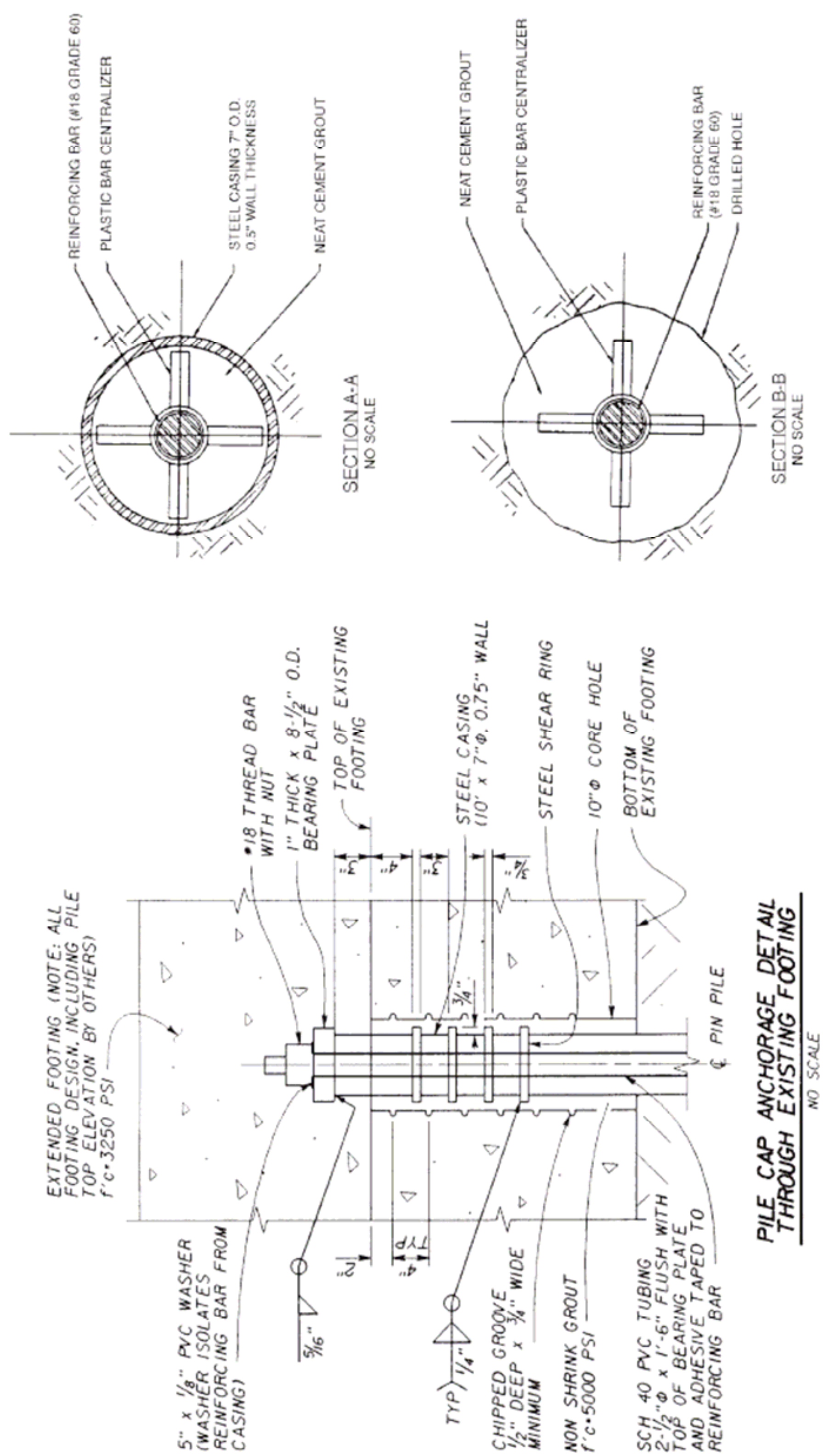
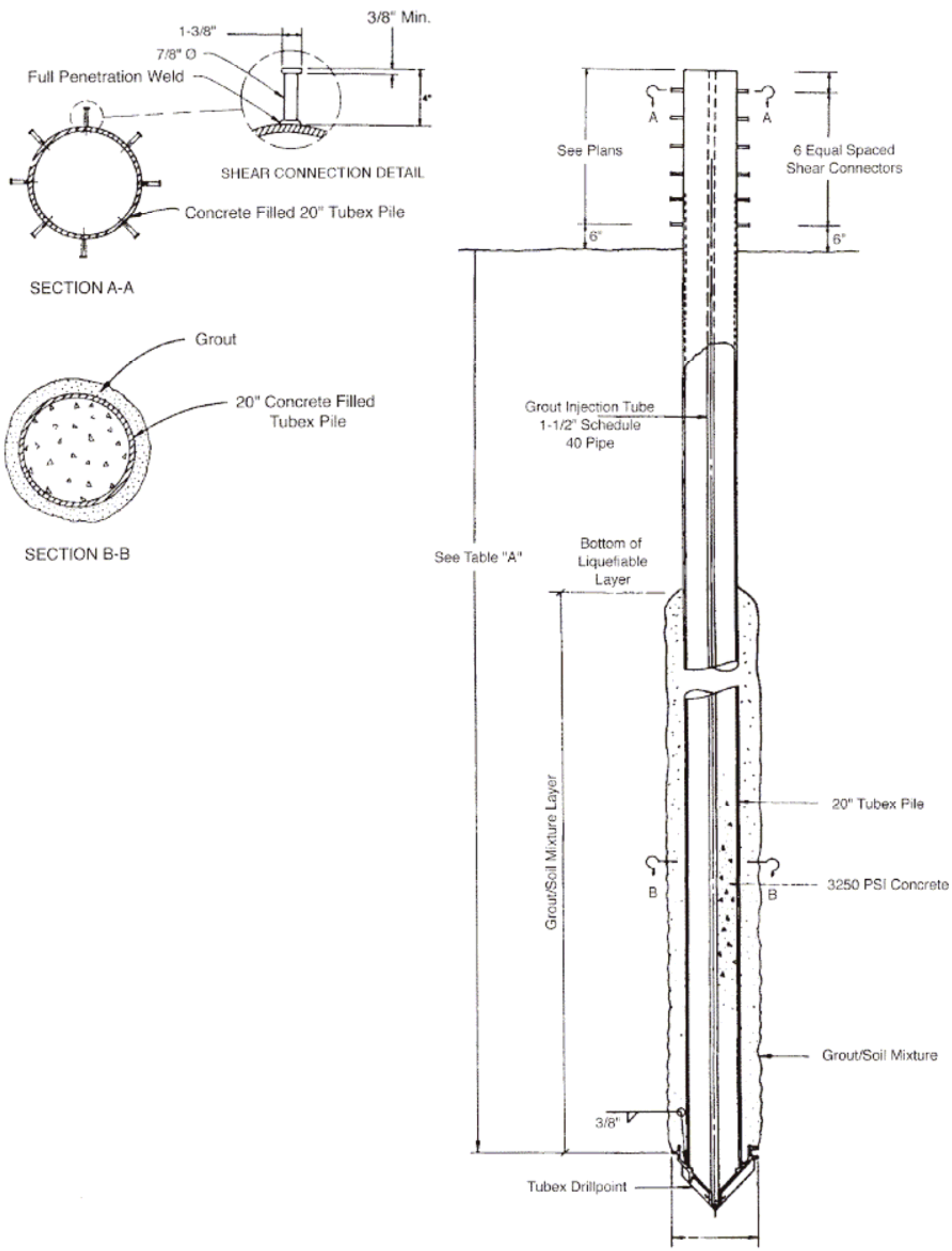
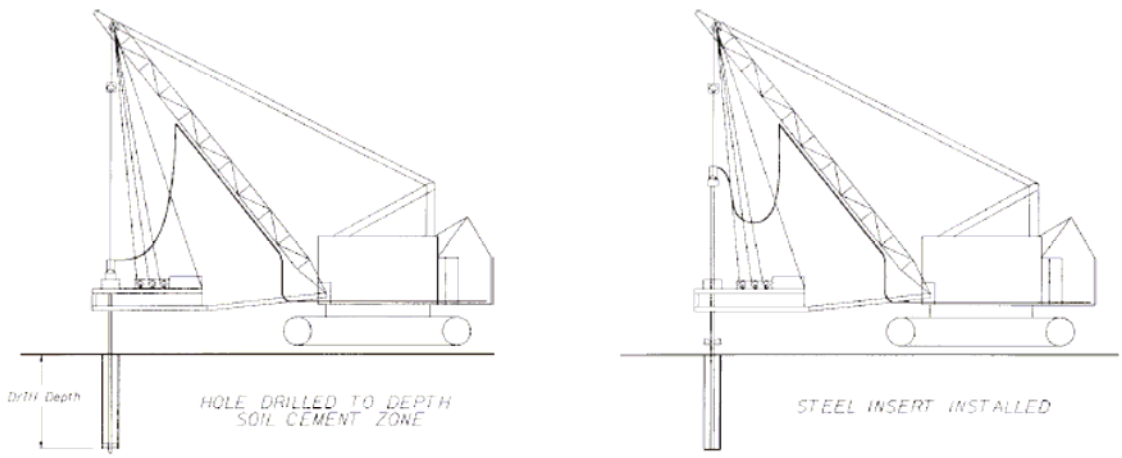
Nicholson Pin Pile

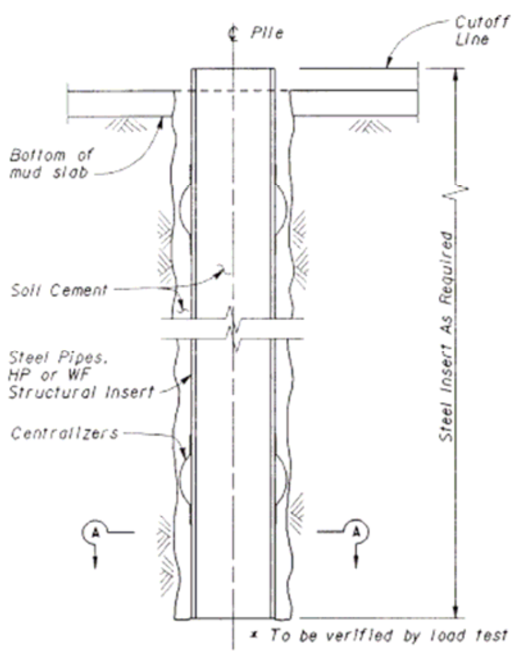


Tubex Grout Unit

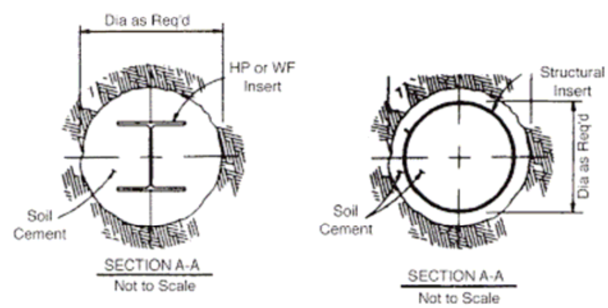


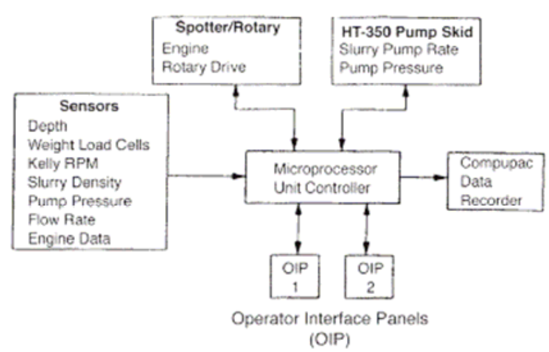
GeoJet Foundation Unit





GeoJet FOUNDATION MEMBER
NO SCALE





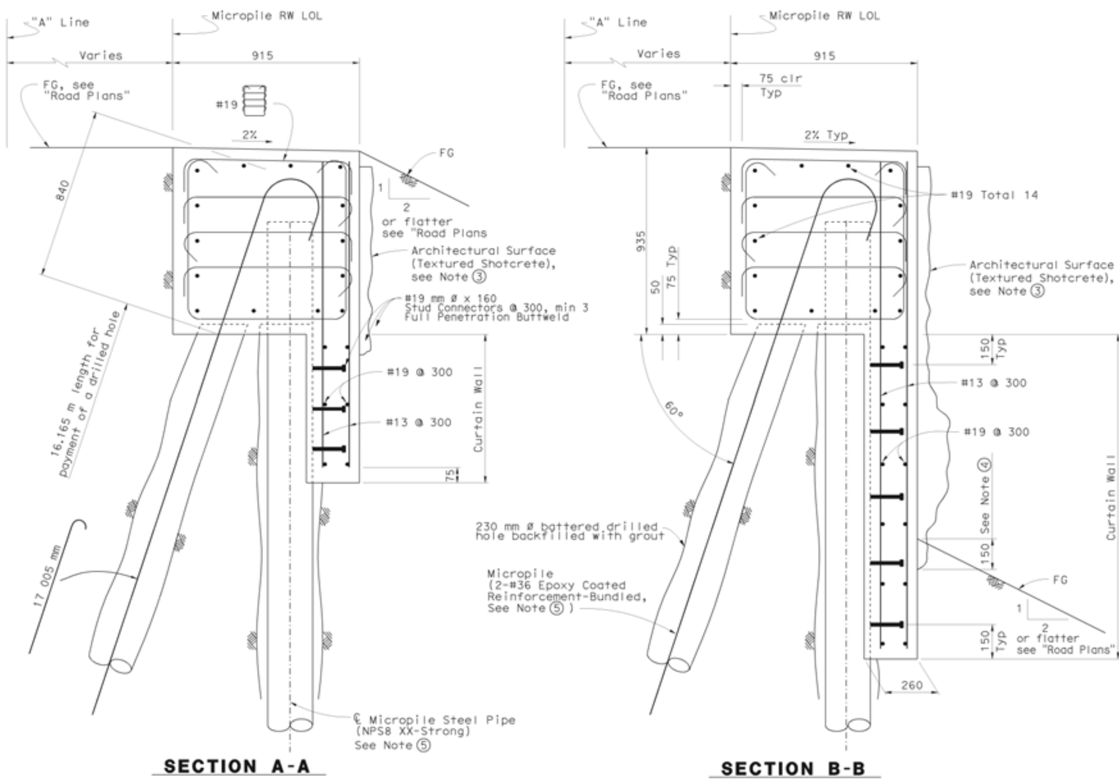
GeoJet Process Control Schematic

Specification Example - Micropiles for Earth Retention

Contract No. 04-1S2804
04 - SON - Rte116, PM 3.2
Duncan's Mills Retaining Wall
Construction completed in 2007.

Description of Work:
The micropile retaining wall was constructed along the eastbound shoulder of Highway 116 in Sonoma County and separates the roadway from the Russian River, which flows west approximately 15-ft below the road surface. The wall consists of a reinforced concrete cap beam and curtain wall supported on micropiles. The face of the curtain wall has an architectural surface (textured shotcrete). Type ST-30 bridge rail (modified) is on top of the wall. The length of the wall is approximately 300-ft long. The 100 micropiles are 12-inch diameter with steel pipes installed to a depth of 50-ft and spaced 3-ft on center with another set of 100 piles set at an angle to form a buttress to ***stabilize the soil and the roadway***. Inclinometers (slope indicators) were installed in six micropiles.

Construction Issues:
Pile production was slow at the western end of the wall due to the hard rock conditions. At another location along the wall, a loose sand and ground water contributed to the caving of the drilled hole during drilling and while waiting for the holes to be grouted.



Cap beam construction. Photo from Jim Cook, Sr Br Engr

Sample - CONTRACT SPECIAL PROVISIONS

10-1.32 PILING

GENERAL

Piling shall conform to the provisions in Section 49, "Piling," of the Standard Specifications, and these special provisions.

Unless otherwise specified, welding of any work performed in conformance with the provisions in Section 49, "Piling," of the Standard Specifications, shall be in conformance with the requirements in AWS D1.1.

Foundation recommendations are included in the "Information Handout" available to the Contractor as provided for in Section 2-1.03, "Examination of Plans, Specifications, Contract, and Site of Work," of the Standard Specifications.

Attention is directed to "Welding" of these special provisions.

Difficult pile installation is anticipated due to the presence caving soils, rocks, serpentine materials, tidal flow fluctuation, high ground water, the requirement of pile embedment into rock, sound control and traffic control.

MICROPILING

Micropiling consisting of steel pipe NPS 8 double extra strong and epoxy coated bar reinforcing steel that is grouted in place shall conform to the design requirements and layout shown on the plans and these special provisions.

Materials

Double extra strong steel pipe shall conform to the requirements of ASTM Designation: A53, Grade B. Galvanized pipe is not required.

The stud connectors shall conform to the provisions in Section 55, "Steel Structures," of the Standard Specifications and these special provisions.

Stud connectors shall be Type B as defined in AWS D1.5, Section 7.

Grout shall be non-shrink type. Grout shall conform to the provisions in Section 50-1.09, "Bonding and Grouting," of the Standard Specifications. Fine aggregate may be added to the grout mixture of Portland cement and water used outside of the grouted sheathing in drilled holes which are 200 mm or greater in diameter, but only to the extent that the cement content of the grout is not less than 500 kg per cubic meter of grout. Fine aggregate, if used, shall conform to the provisions in Section 90-2, "Materials," and Section 90-3, "Aggregate Gradings," of the Standard Specifications.

Epoxy-coated reinforcement shall conform to the provisions in Section 52, "Reinforcement," of the Standard Specifications.

Working Drawings

The Contractor shall submit complete project specific working drawings for the micropile system to the Office of Structure Design (OSD) in conformance with the provisions in Section 5-1.02, "Plans and Working Drawings," of the Standard Specifications. Working drawings for micropiling shall be 559 mm x 864 mm in size. For initial review, 10 sets of drawings shall be submitted. After review, between 6 and 12 sets, as requested by the Engineer, shall be submitted to (OSD) for final approval and use during construction. Within 3 weeks after final approval of the working drawings, one set of the corrected prints on 75-g/m sq. (minimum) good quality bond paper, 559 mm x 864 mm in size, prepared by the Contractor, shall be furnished to (OSD).

Working drawings for micropiling shall show the State assigned designations for the contract number, bridge number, full name of the structure as shown on the contract plans, and District-County-Route-Kilometer Post on each drawing and calculation sheet. The pile vendor company name, address, and phone number shall be shown on the working drawings. Each sheet shall be numbered in the lower right corner and shall contain a blank space in the upper right corner for future contract sheet numbers.

Working drawings for micropiles shall contain all information required for the construction and quality control of the piling, including the following:

A. *Information on space requirements for installation equipment that verify that the proposed equipment can perform at the site.*
B. *Step-by-step procedure describing all aspects of pile installation including personnel, testing, and equipment to assure quality control. This step-by-step procedure shall be shown on the working drawings in sufficient detail so that the Engineer can monitor the construction and quality of these micropiles.*
C. *Details for drilling a plumb and battered hole.*
D. *Details of centralizers.*
E. *Grout mix designs.*
F. *Details and procedures involved in testing components, including grout.*
G. *Pipe and reinforcement splice locations.*
H. *Details of equipment and operation for grouting. Details shall be included for monitoring grout quality, volume installed, and pressure during installation.*
I. *Information on the minimum cure time and strength requirements of the pile system.*
J. *Proposed method for casing installation and removal when necessary.*

A supplement to the working drawings shall include the following:

A. Construction details, structural details, and load test results from at least 3 previous successful installations by the proposed micropile vendor. The installations shall be from 3 separate test sites. The installations shall be similar to those proposed for this contract.
B. Methods of removal and disposal of excavation, slurry, and contaminated water, including removal rates.

The working drawings and supplement shall be stamped and signed by an engineer who is licensed as a Civil Engineer in the State of California. The Engineer will notify the Contractor in writing when the submitted working drawings and supplement have been determined to be complete. The Contractor shall allow the Engineer 30 working days to review the working drawing submittal after a complete set has been received.

No micropile shall be installed until the Engineer has approved, in writing, the working drawing submittal for micropiling.

Should the Engineer fail to review the complete working drawing submittal within the time specified, and if, in the opinion of the Engineer, the Contractor's controlling operation is delayed or interfered with by reason of the delay in reviewing the working drawing submittal, an extension of time commensurate with the delay in completion of the work thus caused will be granted in conformance with the provisions in Section 8-1.09, "Right of Way Delays," of the Standard Specifications.

Construction

Drill cuttings resulting from installing micropiling shall be disposed of in conformance with the provisions in Section 19-2.06, "Surplus Material," of the Standard Specifications. Material resulting from grouting micropiles shall be disposed of in conformance with the provisions in Section 7-1.13, "Disposal of Material Outside the Highway Right of Way," of the Standard Specifications, unless otherwise permitted in writing by the Engineer.

Drilling mud or chemical stabilizers shall not be used.

Foreign material dislodged or drawn into the hole during construction of the micropiles shall be removed. Loose material existing at the bottom of the hole after drilling operations are complete shall be removed prior to placing grout.

Steel pipe NPS 8 double extra strong and epoxy coated bar reinforcing steel shall be installed using centralizers as shown on the plans.

The pipe shall be placed vertically and grouted in place. Grout shall be injected at the bottom of the pile and may be placed before or after placing the steel pipe.

A positive means of support shall be provided for maintaining the position of the steel pipe NPS 8 double extra strong and epoxy coated bar reinforcing steel until the grout has set.

INCLINOMETER- MONITORING SYSTEM

General

The Contractor shall furnish and install an inclinometer monitoring system consisting of slope inclinometer casing at the location shown on the plans. The Contractor shall use a specialist to design and oversee installation of the instrumentation system.

The Contractor shall submit to the engineer working drawings and a construction sequence for the proposed method of installation of the micropile monitoring system construction for the site, at least 10 weeks prior to the planned micropile installation. The drawings shall include all product documentation and specifications for the proposed slope inclinometer casings, strain gauges, wiring, conduit and data logging system, including the name of the manufacturer's design and oversight specialist or representative. The drawings shall conform to the provisions in Section 5-1.02, "Plans and Working Drawings," of the Standard Specifications. One set of the drawings, construction sequence, and product specifications shall be furnished to the Engineer. The working drawings and construction sequence shall include, but not be limited to, defining order of work, traffic control, method of installation of inclinometer casings, strain gauges and data logging system, method of testing system prior to pile installation, and method of grouting the pile. The Contractor shall allow six weeks after complete drawings and all support data are submitted for the review and approval of the proposed method of micropile wall construction.

Should the Engineer fail to complete the review and approval within the time allowance and if, in the opinion of the Engineer, the Contractor's controlling operation is delayed or interfered with by reason of the delay in working drawing and construction sequence plan review and approval for the micropile wall, the delay will be considered a right of way delay in conformance with the provisions in Section 8-1.09, "Right of Way Delays," of the Standard Specifications.

Inclinometer Casings

The Contractor shall furnish and install a total of 6 vertical slope inclinometer casings complete with caps centered inside locations shown on the plans. The inclinometer casings shall be non-metallic with an outside diameter of approximately 70 millimeters. Centralizers shall be used to position the casing along the center axis of each micropile. The casing shall be installed prior to the grouting of the micropiles. The grout shall conform to the requirements specified in "Micropiling" elsewhere in these special provisions. At the Contractor's option, inclinometer casings *may be of the type manufactured by either of the following companies or equal:*

Geokon, Inc.
48 Spencer Street
Lebanon, NH 03766
(603) 448-1562
www.geokon.com

Roctest
P.O. Box 3556
Champlain, NY 2919-3568
(877) 762-8378
www.roctest.com

Durham Geo Slope Indicator
2175 West Park Court
Stone Mountain, GA 30087
(770) 465-7557

These inclinometers, once installed, shall be monitored and data processed by the CalTrans Instrumentation unit.

The Contractor shall allow a maximum of 7 working days for the Engineer to install the inclinometer system, in the location shown on the plans, immediately prior to pouring concrete at the location.

MEASUREMENT AND PAYMENT (PILING)

Measurement and payment for the various types and classes of piles shall conform to the provisions in Sections 49-6.01, "Measurement," and 49-6.02, "Payment," of the Standard Specifications and these special provisions.

Micropiles will be measured and paid for by the meter.

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 drilling, providing temporary casings, double extra strong pipe, grout, epoxy coated bar reinforcement, 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 these special provisions, and as directed by the Engineer.

No payment will be made for micropiles which 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.

The contract lump sum price paid for the inclinometer monitoring system, shall include full compensation for furnishing and installing slope inclinometer casings, including drilling, grout, providing temporary casings, installing studs, disposal of material, and any additional required appurtenances; and for all labor, materials, tools, equipment, and *incidentals and for doing all the work involved in submitting the construction sequence plans, as shown on the plans, as specified in the Standard Specifications and these special provisions, and as directed by the Engineer.*

Case Study – Micropile Retaining Wall Foundation

Contract No. 12-043214
12-ORA-74 PM 13.3/16.6
Route 74 Widening Project (Anchored Walls)
Construction began in 2007.

Description of Work:
The structure work to be done consists, in general, of constructing thirteen anchored shotcrete retaining walls ***founded on micropiles***. The anchored shotcrete walls are founded on steel pipe micropiles and capped with concrete barrier slabs and concrete barriers. Architectural treatment applied includes sculptured shotcrete at various walls and stain application at all walls.

The project site is located on Route 74 (Ortega Highway), between the Orange/Riverside county line and San Juan Creek Bridge. Route 74 is a two-lane highway cut into the side of the Santa Ana Mountains along the San Juan Creek valley. The existing roadway consists of substandard 3.05-meter (10-ft.) lanes and no shoulders.

The purpose of the project is to bring the lanes to the standard 3.66-meter (12-ft.) width with 1.2-meter (4-ft.) shoulders on each side and to increase the sight distance for this 5.3 kilometers of roadway. Since the existing roadway is cut into the mountains, it is necessary to cut further into the mountains, build viaducts, or add retaining walls on the downhill (north) side of the road in many locations. A total of 20 structures (13 anchored retaining walls, 3 sidehill viaducts, and 4 retaining walls) are planned throughout the project limits. ***The anchor walls will be supported on micropiles.***

Structure Representative Comments:
The drilling operation and drilling conditions are difficult, however, the drilling is being completed rapidly. The solid rock is between 9,000 and 15,000 psi, the fractured rock is even more difficult to drill because it has a tendency to cave in and jam the drill stem. The time required to drill a 50-ft deep (6-inch) anchor is approximately 1 hour. The time needed to drill a 21-ft deep, 12-inch diameter micropile is about 1.25 hrs.

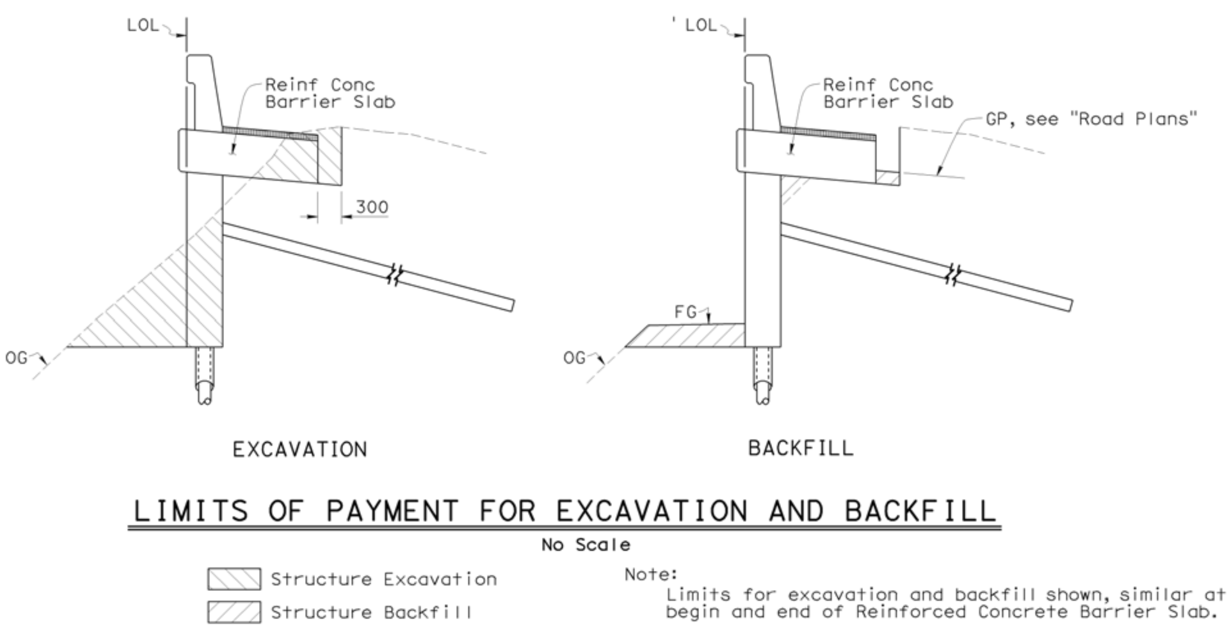
There are several factors affecting the anchored wall (rock anchor and micropile) drilling operation.

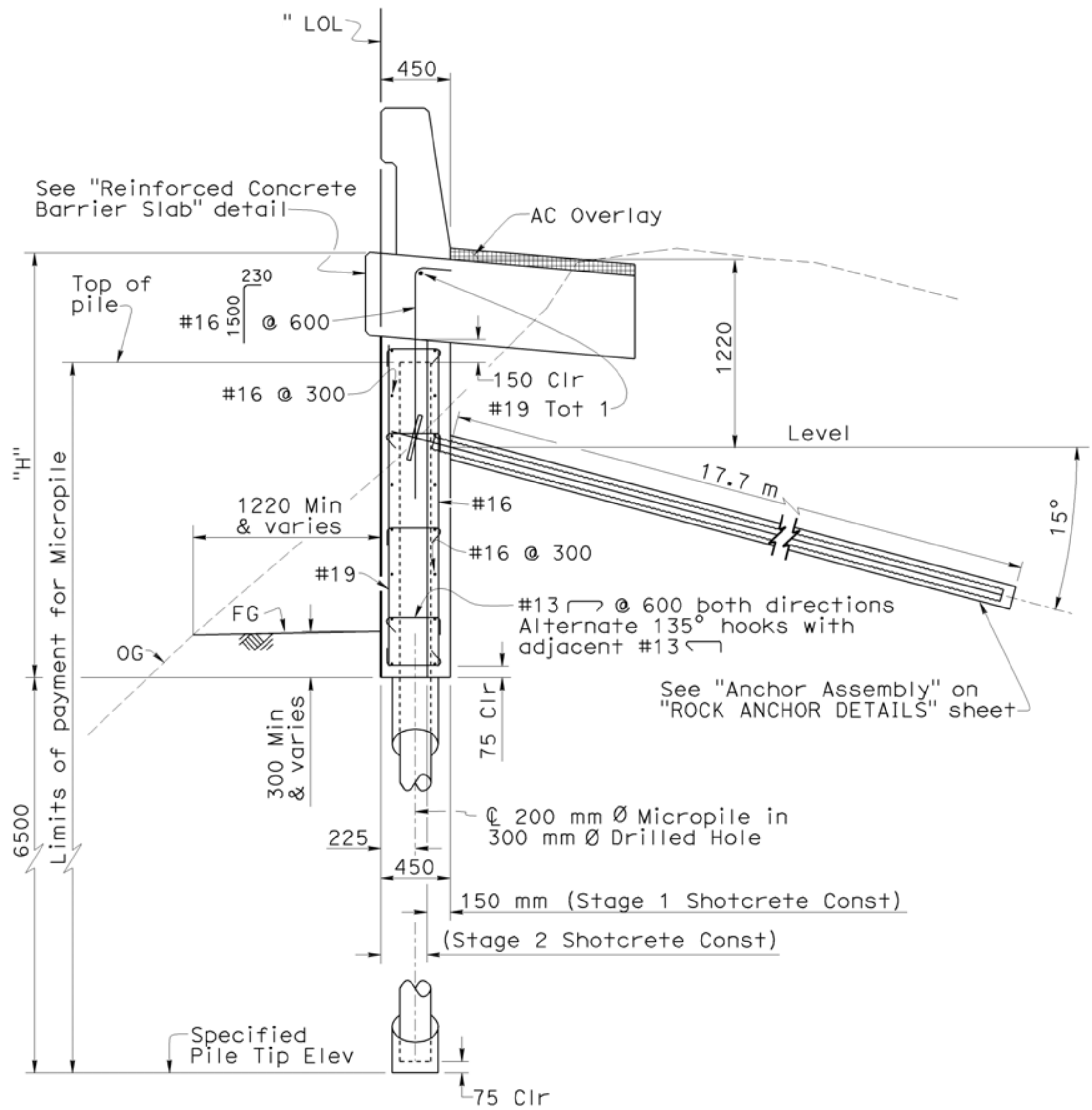
1) The experience of the drilling contractor.
2) The suitability of the equipment used.
3) The material characteristics of the earth at the site.

Drilling has been difficult. The "specialty" drilling subcontractor, required by the Contract Special Provisions (documentation of 3 previous similar and successful installations), was directed to leave the job due to lack of performance. The project special provisions also required the drilling to be done with minimal deleterious effects (airborne drilling dust) to the sensitive "environmental area" and endangered species (Arroyo Toad) in the creek 50 feet from the wall construction area. The constraints of the work area, the requirement to maintain the road open to traffic, requiring the drilling subcontractor to work at night, combined with the need to capture all dust, caused the drilling subcontractor to throw in the towel and cease operations. The drilling subcontractor had equipment that may or may not have been able to complete job.

The prime contractor is currently performing the drilling and had never done any drilling prior to this project. The contractor purchased an Austrian made Triton drilling machine that was designed to drill vertical blast holes for mining operations and redesigned and modified it to drill horizontally. The machine creates a hole using a pneumatic hammer and has the capability of capturing drill cuttings as well as using water to minimize dust. The rig is used for installing both the 6-inch anchor holes 50 feet deep into hard and fractured rock and the 12-inch micropile holes.

(Comments and project photos from Victor S. Francis, P.E.)





TYPICAL SECTION
("H" LESS THAN OR EQUAL TO 4 METERS)

1:25

Notes:

- Equivalent size wire mesh reinforcement steel may be substituted in Stage 1 Shotcrete Construction upon approval of the Engineer.

- Anchor length is based on a design bonded length of 6.7 meters.

Micropile (NPS 8-XX Strong Steel Pipe) in a 300-mm dia drilled hole.
On the ground - Sections of Rock Anchors to be installed later. Date: 2007

Total wall length = 753 ft. The area is mostly comprised of very hard rock croppings. The road, Rte 74, is open to traffic. Date: 2007

Case Study - Micropile Seismic Retrofit

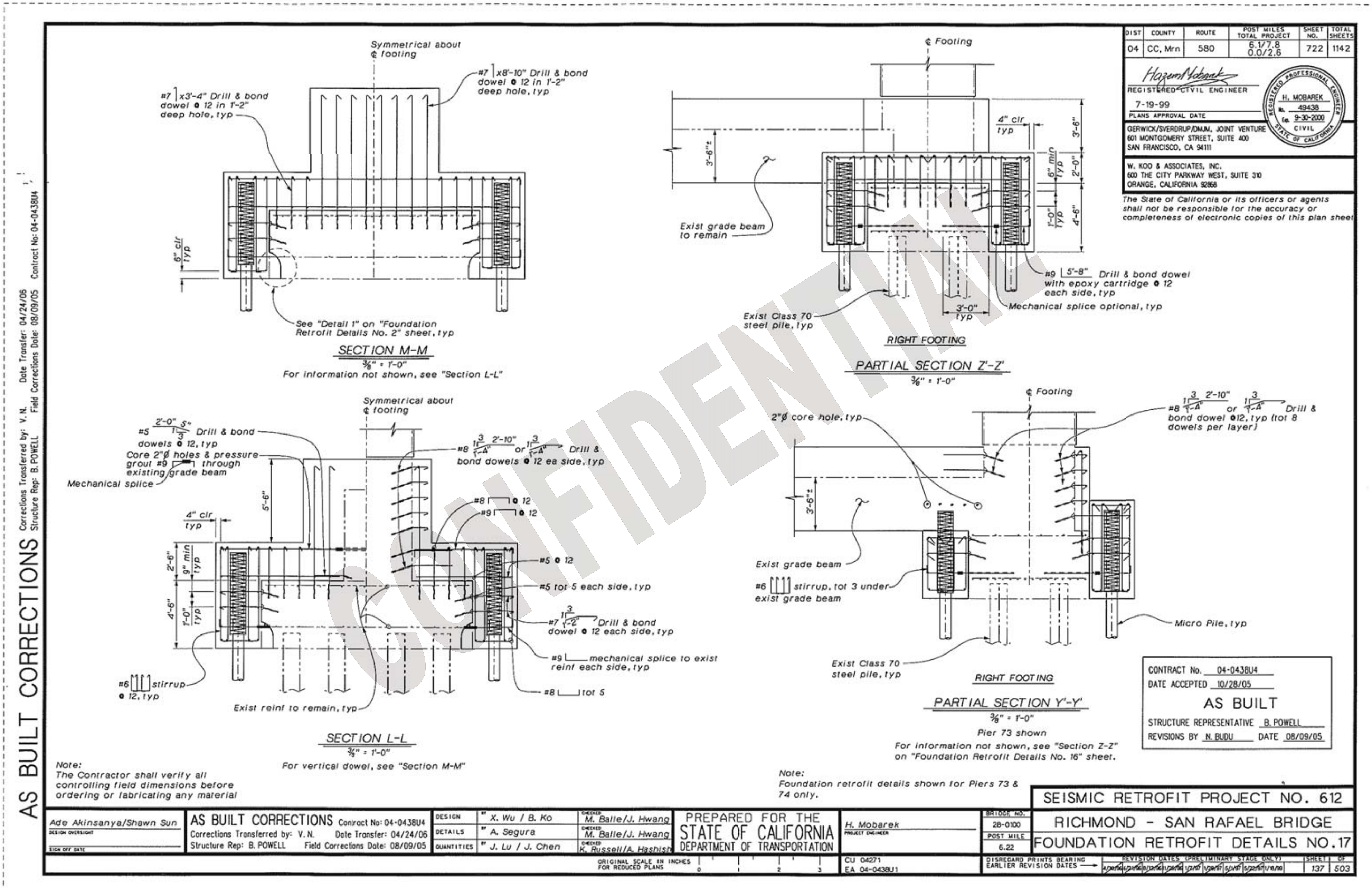
Contract No. 04-0438U4 04-CC,Mrn-580-6.1/7.8,0.0/2.6
Seismic Retrofit of the Richmond-San Rafael Bridge (Br. No. 28-0100)
Work started August 2001; work completed February 2004.

Description of Work:
The Richmond-San Rafael Bridge is one of the toll bridges in the San Francisco Bay Area. The Richmond-San Rafael Bridge includes two single deck reinforced concrete approach trestle, two steel plate girder approach structures which convert from single-deck to double deck at each end of the bridge, two variable-depth, double-deck, cantilever-truss-type structures and 38 constant-depth 289 foot span, double-deck trusses which span between the two cantilever spans and between the cantilever spans and the approach structures. The structure has a combined length of approximately 21,335 feet.

The bridge work on this project consisted of, in general, the replacement of the concrete trestle portion and the seismic retrofit on the rest of the structure. ***The seismic retrofit included constructing 481 micropiles in the substructure.*** The micropiles were driven underwater.

Per the contract special provisions, micropiles (substructure) were specified to consist of small diameter steel pipe reinforcement grouted in place and conforming to the design requirements and layout shown on the plans and the special provisions.

(Photo from Caltrans Office of Geotechnical West Photo Gallery)



AS BUILT CORRECTIONS
Corrections Transferred by: V. N. Date Transfer: 04/24/06 Contract No: 04-0438U4
Structure Rep: B. POWELL Field Corrections Date: 08/09/05
Symmetrical about ¢ footing
#7 x3'-4" Drill & bond dowel @ 12 in 1'-2" deep hole, typ
#7 x8'-10" Drill & bond dowel @ 12 in 1'-2" deep hole, typ
6" clr typ
See "Detail 1" on "Foundation Retrofit Details No. 2" sheet, typ
SECTION M-M
3/8" = 1'-0"
For information not shown, see "Section L-L"
¢ Footing
4" clr typ
3'-6"
2'-0"
4'-6"
6" min typ
1'-0" typ
3'-6"±
Exist grade beam to remain
#9 5'-8" Drill & bond dowel with epoxy cartridge @ 12 each side, typ
Mechanical splice optional, typ
3'-0" typ
Exist Class 70 steel pile, typ
RIGHT FOOTING
PARTIAL SECTION Z'-Z'
3/8" = 1'-0"
Symmetrical about ¢ footing
#5 2'-0" 5" Drill & bond dowels @ 12, typ
Core 2"Ø holes & pressure grout #9 through existing grade beam
Mechanical splice
#8 2'-10" or Drill & bond dowels @ 12 ea side, typ
#8 @ 12
#9 @ 12
4" clr typ
5'-6"
2'-6"
9" min typ
4'-6"
1'-0" typ
#5 @ 12
#5 tot 5 each side, typ
#7 Drill & bond dowel @ 12 each side, typ
#9 mechanical splice to exist reinf each side, typ
#6 stirrup @ 12, typ
#8 tot 5
Exist reinf to remain, typ
SECTION L-L
3/8" = 1'-0"
For vertical dowel, see "Section M-M"
Note:
The Contractor shall verify all controlling field dimensions before ordering or fabricating any material
¢ Footing
2"Ø core hole, typ
#8 2'-10" or Drill & bond dowel @12, typ (tot 8 dowels per layer)
3'-6"±
Exist grade beam
#6 stirrup, tot 3 under exist grade beam
Micro Pile, typ
Exist Class 70 steel pile, typ
RIGHT FOOTING
PARTIAL SECTION Y'-Y'
3/8" = 1'-0"
Pier 73 shown
For information not shown, see "Section Z-Z" on "Foundation Retrofit Details No. 16" sheet.
Note:
Foundation retrofit details shown for Piers 73 & 74 only.
DIST 04 COUNTY CC, Mrn ROUTE 580 POST MILES TOTAL PROJECT 6.1/7.8 0.0/2.6 SHEET NO. 722 TOTAL SHEETS 1142
REGISTERED CIVIL ENGINEER
7-19-99
PLANS APPROVAL DATE
H. MOBAREK No. 49438 Exp. 9-30-2000 CIVIL STATE OF CALIFORNIA
GERWICK/SVERDRUP/DMJM, JOINT VENTURE
601 MONTGOMERY STREET, SUITE 400
SAN FRANCISCO, CA 94111
W. KOO & ASSOCIATES, INC.
600 THE CITY PARKWAY WEST, SUITE 310
ORANGE, CALIFORNIA 92868
The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet
CONTRACT No. 04-0438U4
DATE ACCEPTED 10/28/05
AS BUILT
STRUCTURE REPRESENTATIVE B. POWELL
REVISIONS BY N. BUDU DATE 08/09/05
SEISMIC RETROFIT PROJECT NO. 612
Ade Akinsanya/Shawn Sun
DESIGN OVERSIGHT
AS BUILT CORRECTIONS Contract No: 04-0438U4
Corrections Transferred by: V. N. Date Transfer: 04/24/06
Structure Rep: B. POWELL Field Corrections Date: 08/09/05
DESIGN X. Wu / B. Ko CHECKED M. Balle/J. Hwang
DETAILS A. Segura CHECKED M. Balle/J. Hwang
QUANTITIES J. Lu / J. Chen CHECKED K. Russell/A. Hashish
PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION
H. Mobarek PROJECT ENGINEER
BRIDGE NO. 28-0100 POST MILE 6.22
RICHMOND - SAN RAFAEL BRIDGE
FOUNDATION RETROFIT DETAILS NO. 17
ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3
CU 04271
EA 04-0438U1
DISREGARD PRINTS BEARING EARLIER REVISION DATES
SHEET 137 OF 503
CONFIDENTIAL

California Department of Transportation
Division of Maintenance

Structure Maintenance and Investigations

BRIDGE

INSPECTION

RECORDS

INFORMATION

SYSTEM

The requested documents have been generated by BIRIS.

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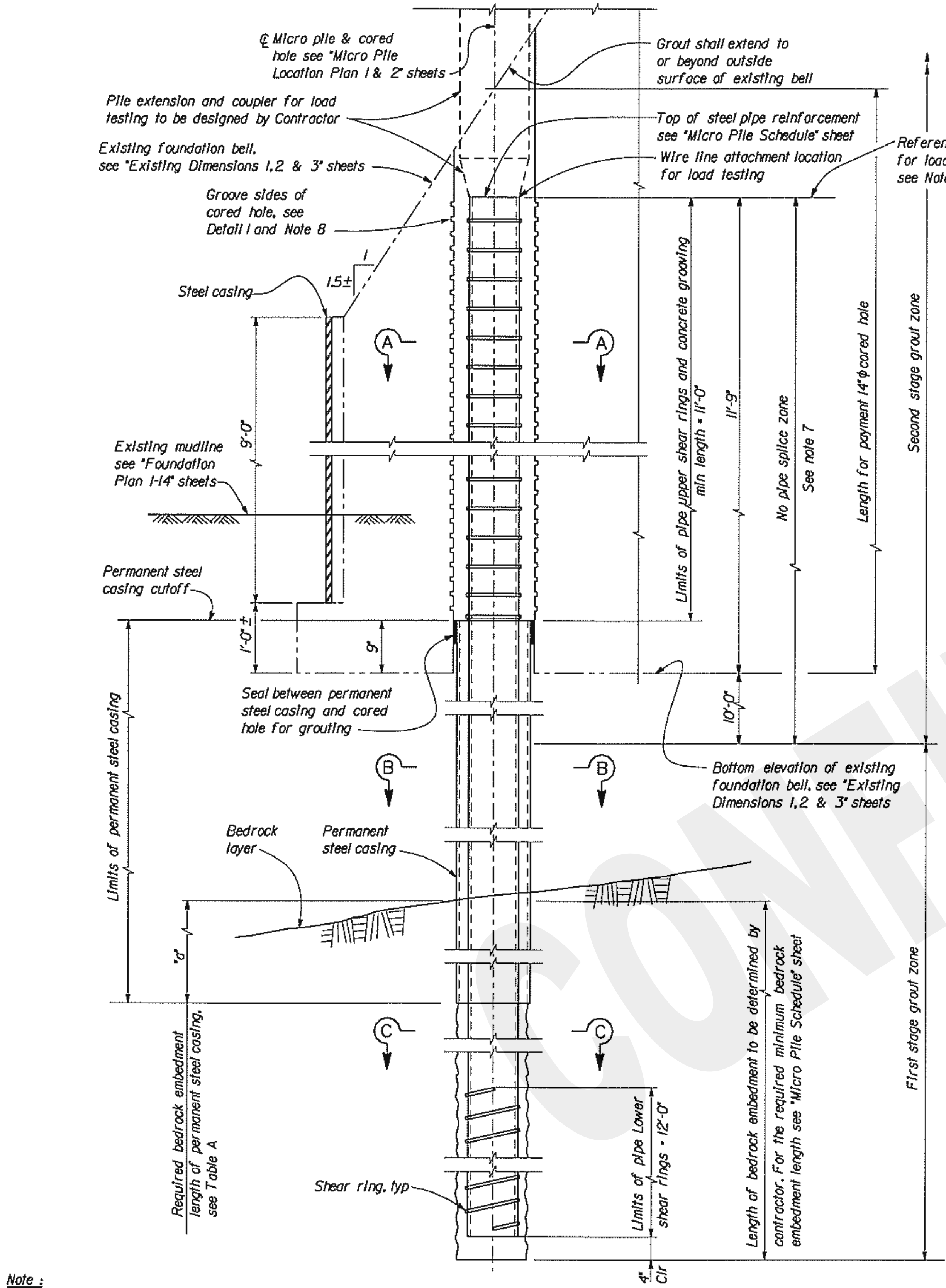
Records for "Confidential" bridges may only be released outside the Department of Transportation upon execution of a confidentiality agreement.

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	CC, Mrn	580	6.1/7.8 0.0/2.6	270	1142

REGISTERED CIVIL ENGINEER
7-19-99
PLANS APPROVAL DATE
REGISTERED PROFESSIONAL ENGINEER J. M. VINCENT No. 45355 Exp. 9/30/02 CIVIL STATE OF CALIFORNIA

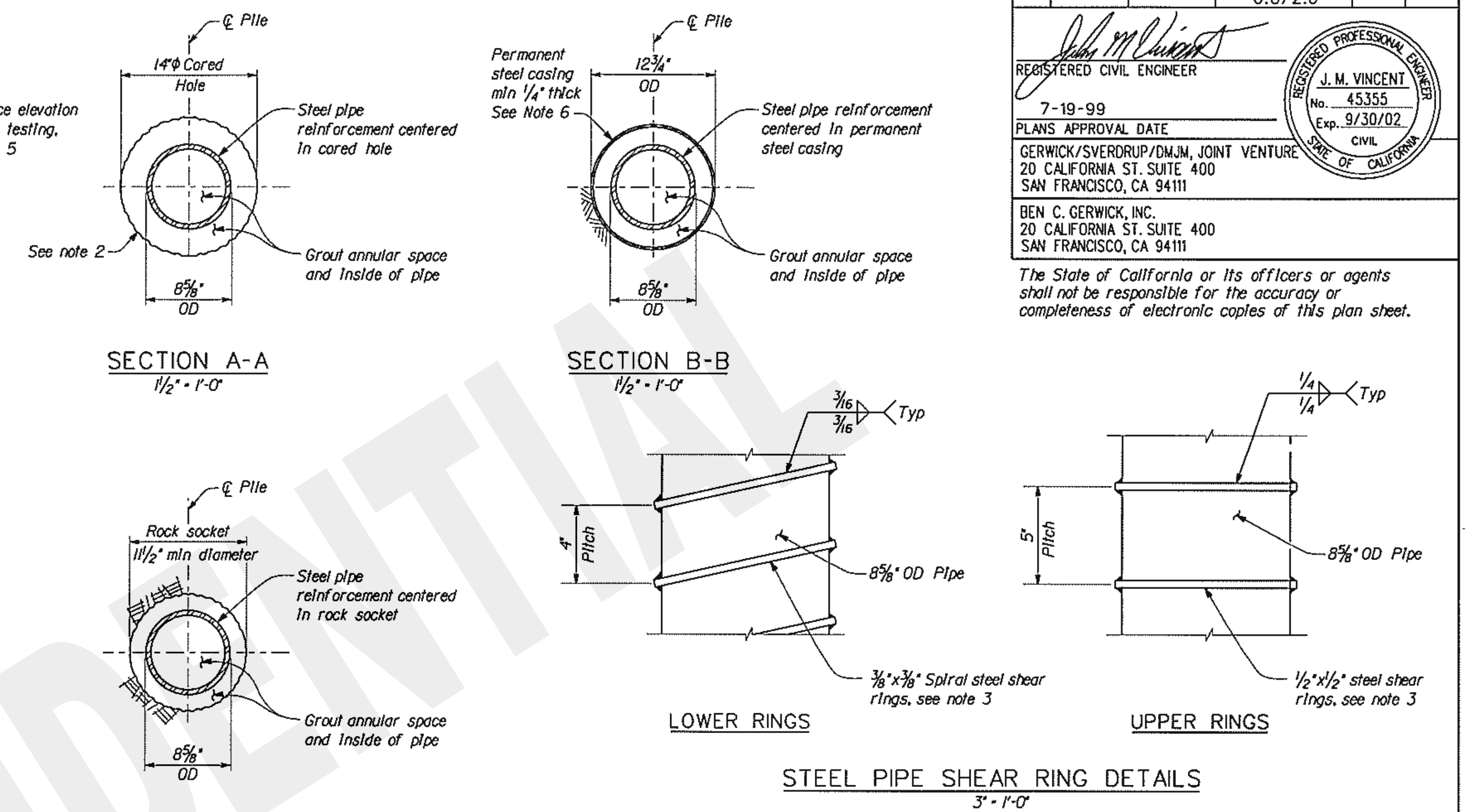
GERWICK/SVERDRUP/DMJM, JOINT VENTURE
20 CALIFORNIA ST. SUITE 400
SAN FRANCISCO, CA 94111

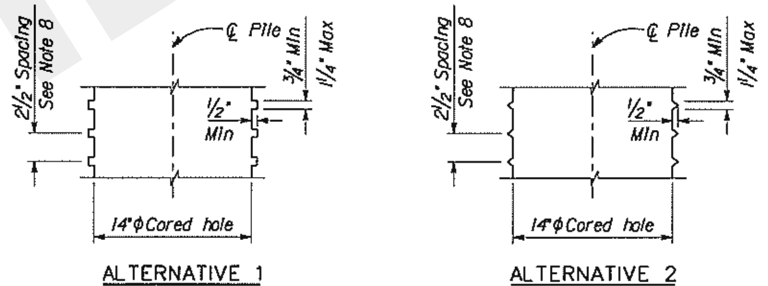
BEN C. GERWICK, INC.
20 CALIFORNIA ST. SUITE 400
SAN FRANCISCO, CA 94111



MICRO PILE ELEVATION
1" = 1'-0"

Note :
The Contractor shall verify all controlling field dimensions before ordering or fabricating any material.





DETAIL 1
1½" = 1'-0"

TABLE A	
Micro Pile Condition Designation ×	Dimension "d" (Min)
A	0'-0" (Firm seating only)
B	10'-0"

× For Micro Pile condition designation see "Micro Pile Location Plan - 1 & 2" sheets

CONTRACT No. 04-0438U4
DATE ACCEPTED 10/28/05
NO AS BUILT CHANGES
STRUCTURE REPRESENTATIVE B. POWELL
REVISIONS BY N/A DATE N/A

Notes:
1. For Pile Data see "Micro Pile Schedule" sheet
2. See "General Notes" sheet for material requirements
3. Shear rings may be slotted for grout tubes
4. Grout pile from bottom up
5. Elevation at which the allowable vertical displacement during performance and proof tests is measured, see "Micro Pile Schedule" sheet.
6. Permanent steel casing wall thickness shown is minimum. Contractor shall increase wall thickness as needed to facilitate selected construction method and actual site conditions.
7. This "No Pipe Splice Zone" requirement does not apply to piers A through 4. At Piers A through 4 no splices are allowed 3'-0" above and below the bottom elevation of existing foundation bell.
8. Concrete grooves may be continuous maintaining a 2½" pitch.

PIERS A-18, 39, 40, 50-54, 56-60

NO AS BUILT CORRECTIONS
Corrections Transferred by: M. B. Date Transfer: 02/27/06
Structure Rep: B. POWELL Field Corrections Date: N/A
Contract No: 04-0438U4

ADE AKINSANYA/SHAWN SUN
DESIGN OVERSIGHT
SIGN-OFF DATE

NO AS BUILT CORRECTIONS Contract No: 04-0438U4
Corrections Transferred by: M. B. Date Transfer: 02/27/06
Structure Rep: B. POWELL Field Corrections Date: N/A

DESIGN BY P. DURNAL CHECKED T. DAHLGREN
DETAILS BY M. SMITH CHECKED T. DAHLGREN
QUANTITIES BY T. DAHLGREN CHECKED P. DURNAL

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

SUBSTRUCTURE PLAN
T. DAHLGREN
PROJECT ENGINEER
BRIDGE NO. 28-0100
POST MILE 6.22

SEISMIC RETROFIT PROJECT NO. 612
RICHMOND-SAN RAFAEL BRIDGE
MICRO PILE DETAILS

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3
CU 04105
EA 0438U1
DISREGARD PRINTS BEARING EARLIER REVISION DATES
REVISION DATES (PRELIMINARY STAGE ONLY)
SHEET 134 OF 414

Plot Scale: 12

Case Study – Micropile Retaining Wall Foundation (Devil's Slide)

Contract No. 04-1123U4 04-SM-1 KP 61.2/64.9
South Portal Retaining Wall No.1 (retaining wall on micropiles) was completed in 2007.

Description of Work:
On Hwy 1, San Mateo County near the City of Pacifica in the San Francisco Bay Area, construction was completed in 2007 on the South Portal Retaining Wall No. 1, a ***retaining wall supported on micropiles.*** The retaining wall is on a steep cliff facing the Pacific Ocean. On one portion of the wall, the micropiles are battered in opposite directions providing lateral support. The retaining wall is also supported laterally with tieback anchors and with anchor bars connected to an anchor beam. On top of the wall is a concrete railing with chain link fence. A pedestrian sidewalk runs parallel to the concrete railing.

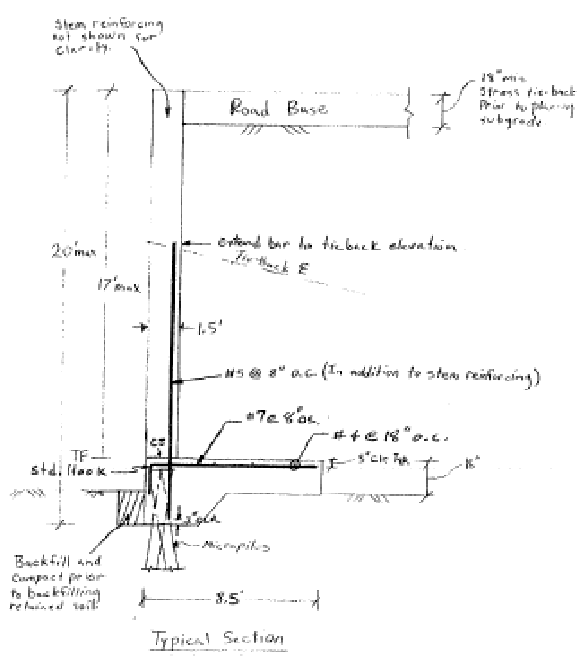
The South Portal Retaining Wall No. 1 is part of the overall work to re-align Route 1 at the south portal of the Devil's Slide Tunnel. The micropile wall was placed to provide a future parking lot and a turn-around when the tunnel is complete. In addition, the wall provides valuable work space for construction (i.e., haul road and construction yard) without closing Hwy 1 during the tunnel construction.

Total length of wall:	103-meters.
Total micropiles:	144 piles
Length of pile:	7.5m (piles 1 thru 36); 10.0m (piles 37 thru 144);

Construction Issues / Comments:
Comments from Peter Lam, P.E., Assistant Structure Representative:
- The micropiles were ConTech Titan System piles.
- The micropile contractor was Condon-Johnson & Associates.
- Specs required non-shrink grout, but normal grout was allowed.
- CT Foundation Testing Branch (FTB) specified pull tests into zones. Testing was by FTB. The specs required non-shrink grout, which hydrates quicker and cost 2 to 3 times more than regular grout. Regular grout is the industry standard for micropile installation. Initially, the CT Geotechnical designer felt comfortable waving the load test requirement if non-shrink grout was used. However since regular grout was used, load testing was required. The test results came out great with little or no movement. The CT Geotechnical designer speculated that a grout beam was created below grade due to the piles being spaced so closely.
- In some areas, soft soil caused grout bubbling through adjacent piles; the excess grout probably formed a grout curtain.

- Micropile operation is very messy operation; proper SWPPP measures are needed.
- Pile production/installation was approximately 1 pile per 30 to 40 minutes

Comments from Jeremy Light, Assistant Structure Representative:
- The original wall design did not provide enough embedment in the retaining wall for wind load stability. The revised design specified a spread "L" footing that provided the proper stability.
- The addition of the footing to the structure satisfied the wind load requirements and enabled the Contractor to backfill the wall prior to anchor rod (Sta. 1+00 to 1+36) & tie-back installation (Sta. 1+36 to 2+03). Tiebacks were installed from the outside of the wall with a reach-over drill rig. The plans called for temporary supports (Sta 1+36 to 2+03) to temporarily retain the wall during backfill operations and the footing satisfied this. Installing the tiebacks from behind the wall and using them for temporary supports was considered but tieback testing and working around the exposed tendons during the backfill operation proved to be an inefficient method of construction. The designer initially wanted tie-backs installed & tested behind the wall but it was brought up that the tendons would be compromised by 'bite' marks from the wedges as well as the exposure of the tendons during the construction operations (a temporary waler was called out in the specs to achieve this; impractical with the geometry of the site). Following this, Design proposed installing three sacrificial tendons for testing, but this proved to be a problem with again, the issue of providing a temporary waler to support the tieback loads. This was the main construction issue of this project…"How do we build it?". The addition of the footing, at a cost to the State in this case, proved to be a good solution.



Sketch -Revised Wall Design – "L" Footing.

Site photo, facing south. South Rock Cut Retaining Wall (soil nail wall with sculptured face) on the upper left.

Foreground: battered micropiles, facing south. Background: Pacific Ocean.

Drilling/installing micropiles. Pacific Ocean in the background.

Battered micropiles. Facing north.

Lifting micropile pipe into place.

Drilling and flushing.

Adding another section of pipe (hollow treaded bar with coupler). Grout hose is in front of the man on the left.

Checking top of pile elevation with a laser level (grout hose is blocking view of the survey rod).

Assembled micropile components (Con-Tech Titan System): drill head, pipe (TITAN bar), pipe coupler.

Laborer assembling the micropile components (Con-Tech Titan System): drill head, pipe (Titan bar), & pipe

Holes are used for grout flushing. When each micropile has been drilled to depth, it is withdrawn back up then redrilled in a reaming motion to flush out the drill cuttings and increase scouring of the grout flush. Scouring creates a very rough, irregular shaped grout body with a much greater mechanical connection to the soil, providing greater pull-out resistance and lower settlement characteristics. (Source: Ischebeck Titan brochure)

Drill bit / drill head with holes.

Short section of pipe (Titan hollow threaded bar).

Left: Crawler mounted drill rig w/hydraulic rotary percussive head; hose grout hose on left. Right: excavator.

Grout mixer. Grout is pumped to the micropile drill rig.

Forklift. Background: South Rock Cut Retaining Wall (soil nail wall with sculptured face)

Cement sacks covered with plastic.

Micropile load test performed by
Foundation Testing Branch.

Hydraulic jack and measuring device.

Hydraulic jack and caliper. (rotated view)

Recording measurements using auto-level.

Storm Water Pollution Prevention Plan (SWPPP). Right: grout settlement container. Left: mobile tank and pump.

SWPPP: grout settlement container / concrete washout container.

SWPPP: laborer is creating a check dam at each drill location to prevent grout and drill cuttings from covering next pile location.

Portable pump used to pump excess grout and drill cuttings into settlement containers. Behind pump is silt fence and rolled straw.