State of California
Department of Transportation
Engineering Service Center
Division of Structures

# California Foundation Manual

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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 judgement 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.

CHAPTER

# 1 Foundation Investigations

## Introduction

The ultimate strength and longevity of any structure depends on the adequacy of its foundation. Structure Representatives administering projects for the Office 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 useful life.

It is essential that Structure Representatives and all personnel working for the Office of Structure Construction commit themselves to learning the provisions within the *Standard Specifications*, Standard Plans, Contract Plans, Special Provisions and all relevant documents related to each contract they are working on. 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 who to contact for further information or for advice on solving project problems.

This chapter will give an overview of the foundation investigation process. It 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 that assists 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 Engineering Service Center are normally performed by one of the two Foundation Investigation sections of the Office of Structural Foundations. The southern section in Los Angeles handles the investigations for projects to be constructed in Districts 7, 8, 9, 11, and 12. The northern section handles investigations in the remainder of the twelve Districts in the northern half of the State.

At times, the design of structure projects have oversight provided by the Office of External Liaison and Support through either the Consultant Contracts Management Branch, the Externally Financed Projects Branch, or the Local Assistance and Program Section. Foundation investigations for these projects are produced by consultant geotechnical companies. In these types of projects, the Log of Test Borings and Foundation Report for structures are reviewed by the Geotechnical Design Oversight Section of the Office of Structural Foundations.

## Office of Structural Foundations Geotechnical Support

Personnel from the Office of Structural Foundations are available to provide support to Office of Structure Construction employees throughout the life of a construction project.

Structure Representatives are encouraged in Bridge Construction Memo 2-2.0 to schedule pre-construction meetings with personnel from the appropriate Foundation Investigation Section (Northern or Southern). The primary purpose of the pre-construction meeting would be to forge a good relationship with the Engineering Geologist that performed the foundation investigation, and to discuss the Foundation Report, Log of Test Borings, and any potential foundation problem areas in detail. This meeting may be invaluable to Structure Representatives in their efforts to recognize and discuss any potential foundation problems that may need extra attention during the foundation work on the project.

Once construction projects are under way, personnel from the Foundation Investigation Section lend their expertise when problems occur during foundation work. Engineering Geologists advise Structure Representatives over the phone and often visit projects to evaluate problems and recommend solutions. Structure Representatives are encouraged to inform the Foundation Investigation Section of any problems with structure foundations as early as possible. Early notification often gives the Engineering Geologists the best chance of resolving foundation problems with the most economical solution.

Structure Representatives who have problems related to foundations on projects with oversight provided by the Office of External Liaison and Support; Externally Financed Projects Branch, Consultant Contracts Management Branch, or Local Assistance and Programs Section should contact the Liaison 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 Designer sends in a Foundation Investigation Request to the appropriate Foundation Investigation Section. At that point an Engineering Geologist is assigned to perform the foundation investigation.

The Engineering Geologist 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 Engineering Geologist an idea of what to look for during their field work.

Once an Engineering Geologist collects all of the preliminary information, the Engineering Geologist lays out a boring pattern in relation to the structure's proposed foundation locations. The main goal in establishing a boring pattern 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.

Once the boring layout is established, the Engineering Geologist 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 by the Engineering Geologist 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 Designer. The Log of Test Borings is included as the last portion of the structure plans for the project.

After the Log of Test Borings is completed, the Engineering Geologist analyzes all the subsurface information collected and designs a recommended foundation for the structure. The recommended foundation type as well as other important pieces of information are compiled and included within the Foundation Report for the structure. This Foundation Report is also transmitted to the Project Designer.

Once the Log of Test Borings and Foundation Report are sent to the Project Designer, the design of the structure is completed using the foundation recommendations included in the Foundation Report. The Log of Test Borings is included in the project plan sheets, and the Foundation Report is included in the RE Pending File.

## Subsurface Drilling Operation

The most important aspect of a foundation investigation is the subsurface drilling operation. Foundation drilling crews, led by the Engineering Geologist, 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 assists the Engineering Geologist in the development of a soil/rock profile. A soil/rock profile is a visual representation of the subsurface conditions interpreted from the subsurface investigations and laboratory testing. The soil/rock profile is included within the Log of Test Borings.

During the subsurface drilling operation, the Engineering Geologist 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 Engineering Geologist will describe the soil or rock samples within the field logs. During the drilling operation, elevations of significant changes in material are noted and soil samples are usually taken from each different soil layer 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 the Engineering Geologist to estimate the strengths of the soil or rock layers. 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. The Engineering Geologist also determines whether or not the groundwater encountered is "perched" or in an "artesian" condition. These observations along with the various field and laboratory testing assist in the development of the soil/rock profile.

Two of the most 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 which 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 tests and strength tests. Examples of these tests are direct shear, triaxial shear, and unconfined compression tests. The strength tests provide shear strength design parameters which are used in static analysis for pile foundations. Consolidation tests provide parameters needed to estimate settlements of spread footings or pile groups.

The most common method of retrieving a disturbed soil sample is with the split spoon sampler. The split spoon sampler is used for the Standard Penetration Test. As previously stated, disturbed samples, such as those retrieved from the split spoon sampler, are mainly used to assist in the soil classification and final identification of the soil.

Several types of soil samplers are used to retrieve undisturbed samples during subsurface investigations. Types include the California Sampler (which is used by the Office of Structural Foundations), the Shelby Tube, the Piston Sampler, and the Hydraulic Piston Sampler. Undisturbed soil samples provide the Engineering Geologist the best opportunity to evaluate the soil in its natural undisturbed state. This type of sampling usually provides the most accurate soil parameters once tests are performed.

In-situ tests are needed to provide soil parameters for the design of structure foundations, especially when standard drilling and sampling methods cannot be used to obtain high quality undisturbed samples. Undisturbed samples from non-cohesive soils are difficult to obtain, trim, and test in the laboratory. Soft saturated clays, saturated sands and intermixed deposits of soil and gravel are difficult to sample and test in the laboratory. To overcome these difficulties, in-situ test methods must be used to measure soil parameters.

The most common in-situ test used during a subsurface investigation is the Standard Penetration Test (SPT). The test results in a penetration resistance value, "N". The "N" value can be used to estimate the angle of friction of a cohesionless soil, the unconfined compressive strength of a cohesive soil, and the unit weight of a soil (refer to Appendix A). Other in-situ tests are the static cone test, pressure meter test, vane shear test, and the borehole shear test. In-situ tests, such as the vane shear and Iowa borehole shear tests, provide soil shear strength parameters, such as cohesion, angle of internal friction, and shear strength.

These design parameters are used for static analytical design procedures for pile foundations and may also provide valuable information to a Structure Representative during the course of a construction project.

## Log of Test Borings

After the subsurface investigation is complete, the Engineering Geologist develops the Log of Test Borings (Refer to Appendix A for examples). 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 (refer to Appendix A for a standard Log of Test Borings legend).

## Foundation Report

Once the Log of Test Borings is complete, the Engineering Geologist performing the foundation investigation develops the Foundation Report (refer to Appendix A for an example of a Foundation Report). The foundation report is basically a compilation of all the information retrieved during the foundation investigation and provides the Project Designer 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 possibility of liquefaction occurring at the site. The report gives recommendations to the Project Designer as to the type of foundation that should be used for the proposed structure and also recommends the seismic data, such as peak horizontal bedrock acceleration, that should be used for the seismic analysis to be performed. The report includes the recommended elevations for spread footings and pile type and tip elevations.

Most reports include special comments regarding anticipated constructability problems, 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 within the contract Special Provisions. The Structure Representative should pay particular attention to these comments. Advance knowledge of potential problems during foundation work allows for more effective problem mitigation.

The Foundation Report is normally included in the RE Pending File. Structure Representatives should contact the Office of Structure Construction in Sacramento if they do not receive a copy of the Foundation Report for any project assigned to them.

In the early stages of every project, 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. Personnel from the Foundation Investigations Section and the Project Designer should be consulted if there are any discrepancies.

If there are special comments regarding constructability problems within the Foundation Report, these issues should be discussed with the Contractor as early as possible. Once the Contractor begins work, the Structure Representative should observe if and how the Contractor makes preparations to deal with the constructability problems that they were informed of. 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 employees, to be aware of how the *Standard Specifications* interpret 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 how these documents should be viewed by all contractors performing work for the State.

Section 2-1.03 of the *Standard Specifications* states that the Contractor has an obligation to examine the site of the work, the plans, and the specifications. From this examination, contractors are to make their own determination as to what conditions they may encounter while doing the proposed work. This section also states that bidders can inspect any records compiled by Caltrans, but the use of this information is only for the purpose of study and design and that they are not part of the contract documents. Caltrans assumes no responsibility as to the sufficiency or adequacy of any of its investigations and this section states that there is no warranty or guaranty that any conditions indicated by a subsurface investigation performed by Caltrans are representative of the conditions throughout a construction site. This section also describes the Log of Test Borings and any other geotechnical data obtained as being Caltrans' opinion as to the character of the materials encountered during its investigations at the site of the work.

Section 2-1.03 also very clearly states that contractors should make whatever independent investigations that they feel are necessary to inform themselves of the conditions to be encountered. Contractors should only be using the Log of Test Borings and any other records of subsurface investigations compiled by Caltrans to supplement their own site investigations.

## 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.

There are a number of soil classification systems used in the engineering industry. Most of these systems are based on those properties which are most important in the phase of engineering for which the classification was developed.

The Office of Structural Foundations has adopted a classification system based on the Unified Soil Classification System. All Foundation Reports and Log of Test Borings now follow this classification system.

| 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. |
| Gravel | Particles of rock that will pass a 3-inch sieve but will be retained on a No. 4 sieve. |
| Sand | Particles of rock that will pass a No. 4 sieve but will be retained on a No. 200 sieve. |
| Silt | Soil passing a No. 200 sieve that is nonplastic or very slightly plastic and exhibits little or no strength when air dried. |
| Clay | Soil passing a No. 200 sieve that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and that exhibits considerable strength when air dried. |
| 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 decompostion. 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. |

Engineering Geologists often describe soils with a series of descriptive adjectives before the noun. An example of this would be:

"Slightly compact, dark gray, micaceous Clayey Sand."

This statement describes a material that is predominantly made up of sand, but has enough clay within it to make it a little plastic when handled.

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 tests utilizing measures of settling, plasticity, dry strength, and permeability characteristics of the soil permit a more accurate classification of these soils.

Once a soil is immersed 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, being non-plastic, 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 permeability of a soil is shown by its reaction to shaking or patting. When a small amount of silt is subjected to this type of movement, water appears on the surface, which assumes a ‘‘livery’’ appearance. Upon shaking or patting clay, this reaction occurs slowly or not at all.

## Geotechnical Drilling and Sampling Equipment

Many different pieces of equipment are used by foundation drilling crews and engineering geologists 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 pieces of equipment used to perform the drilling operation have different levels of reliability. The reliability of the equipment used during the subsurface investigation is an important factor for the Structure Representative to take into consideration during the early phases of a project.

The following is a brief description of the various pieces of equipment used by the Office of Structural Foundations as well as consultant geotechnical companies.

| EQUIPMENT | DESCRIPTION |
|---|---|
| $2^1/_4$-Inch Cone Penetrometer | The $2^1/_4$-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.<br><br>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.<br><br>The blows per foot are recorded by the Engineering Geologist in a manner similar to the Cone Penetrometer test.<br><br>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.<br><br>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.<br><br>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 $2^1/_4$-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 in common one fundamental characteristic; that is, they provide a means whereby service loads are transmitted into the supporting medium.

For new construction, the types of structure foundations that can be used are generally limited by geologic conditions. For bridge widenings, bridge rehabilitation, and seismic retrofits, the types of structure foundations that can be used may also be limited by site accessibility, overhead clearance, superstructure requirements, existing utilities, and noise restrictions.

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 pier columns, tiebacks, soil nails, and tiedowns.

The primary source of information for structure foundation decisions is the Foundation Report prepared by the Office of Structural Foundations of the Engineering Service Center. The Project Designer selects the appropriate foundation type based upon data and recommendations contained in the Foundation Report.

For pile-supported foundations, it is the Project Designer's responsibility to select the type of pile consistent with the Foundation Report's recommendations. Additionally, the selected pile type should fulfill the requirements for economy, competitive bidding, and availability for the particular conditions prevailing at the site.

Instead of a specific recommendation as to foundation type, the Engineering Geologist may provide the Project Designer with engineering data for both footing and pile foundations. In this case, existing field conditions and/or economics will generally determine the foundation type.

While it is true that the foundation type is determined primarily by the geological nature of the foundation material itself, non-geological features are considered in the selection and design of structure foundations.

All available site data is reviewed by the Engineering Geologist and the Project Designer to determine if there are existing conditions or proposed changes that would restrict or exclude certain foundation types.

Seal courses are frequently specified as a foundation aid when water problems are anticipated. Seal course concrete is placed under water, the general purpose being to seal the bottom of a tight cofferdam against hydrostatic pressure. This enables dewatering of the cofferdam and construction of the footing "in the dry."

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 a Cast-In-Drilled Hole (CIDH) concrete pile. Pile types are precast concrete, steel structural sections, steel pipe, and timber. Geologic considerations include the soil profile, driving difficulties, and corrosive soil problems. Non-geologic considerations include adjacent structures, existing utilities, required pile length, restricted overhead clearances, accessibility, and noise restrictions. |
| Non-Driven Piles | …consist of 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 the location of the water table and potential fluctuation, 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 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, for seismic retrofitting of existing footings where overturning must be prevented. |

Generally, footing foundations are more economical than pile supported foundations.

CIDH concrete piles are the most economical pile-supported foundation with steel piles generally being the most expensive.

# CHAPTER 3 Contract Administration

Section 5-1.01 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."

Contract Administration may be defined as the sum total of all those 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 will include, but not be limited to: (1) enforcement and interpretation of the plans and specifications, (2) ensuring compliance with applicable Caltrans policies, (3) objective and subjective decisions, 4) sampling, testing and inspection of the work, (5) problem solving, and (6) proper documentation.

It should be pointed out that a well administered contract does not always produce excellent results. Although it is the Contractor's contractual obligation to construct and complete the project in accordance with the contract documents, the best results are generally obtained when the Contractor's attitude is one of cooperation, rather than one which is antagonistic in nature. On current contracts Caltrans will promote 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 at the lowest possible level.

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.

A detailed study must be made of the plans and specifications. This would include the Log of Test Borings, which, although attached to the contract plans, is not considered a part of the contract documents (refer to Section 2-1.03 of the *Standard Specifications*). The Engineer must become completely familiar with the contract plans and their requirements. 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. Note any conflicts or potential problems.

Other documents to be reviewed are:

| DOCUMENT | DESCRIPTION |
|---|---|
| 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. |
| Foundation Report | Prepared by the Office of Structural Foundations and is a part of the Preliminary Report. This report will contain a description of the area geology, a soil profile for selected locations and the Engineering Geologists' recommendations of foundation types. This report is very informative and should be thoroughly reviewed. A foundation review is made by the Project Designer and the Engineering Geologist prior to design |

The contract plans and specifications, the aforementioned reports 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 in the work.

It is advisable to meet with the Project Designer and the Engineering Geologist to discuss foundation details (refer to Bridge Construction Memo 130-1.0). If an on-site meeting is impractical, the meeting should be held by telephone. Clarify and resolve any questions or problems regarding foundations and foundation material. *Now* would 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 with the Project Designer and Engineering Geologist have been held, the Engineer should have a firm grasp of the contract requirements for the project and the foundation conditions to be encountered at various locations. Special attention should be given to those locations requiring extreme care in performing the work and any remaining problems concerning utilities. These should be presented at the pre-construction conference(s) to be held with the Contractor and other interested agencies.

Pre-construction conferences are usually held at about the time the Contractor begins mobilizing at the site, but well before work actually starts on the job. Four general subjects are normally covered: (1) safety, (2) labor compliance and affirmative action, (3) utilities, and (4) matters related to the performance of the work itself. Depending on District policy and the complexity of the project, very often more than one meeting is desirable in order to limit the scope and the number of individuals present. From this meeting should come a common understanding of the proposed work and the problems and possible solutions which may be expected during the life of the contract.

The pre-construction conference presents an excellent time to focus on inherent project problems and specifications which 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 which might apply. However, the following list covers some of the areas which should be considered:

| ITEM | REFERENCE |
|---|---|
| Test Boring Information | *Standard Specifications*, Section 2-1.03 |
| Excavation Plans | *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 |
| Sound Control Requirements | *Standard Specifications*, Section7-1.01I |
| Water Pollution | *Standard Specifications*, Section7-1.01G |
| Public Safety | *Standard Specification*s, Section 7-1.09 |
| Preservation of Property | *Standard Specifications*, Section 7-1.11 |
| Responsibility for Damage | *Standard Specifications*, Section 7-1.12 |
| Protection of Utilities | *Standard Specifications*, Section 8-1.10 |
| Cofferdams | *Standard Specifications*, Section 19-3.03 |
| 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 | — |

All utility locations shown on the plans should be verified with the utility representative. The Contractor should 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 utilities must also be discussed. Problems in this area could result in serious delays. Hence, if they are not solved at the pre-construction conference, they should be resolved at the earliest possible time.

The Contractor's proposed methods of performing foundation work adjacent to utilities should also be covered at the pre-construction conference. All present should be advised of any proposed change orders affecting 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 the Project Designer and the Engineering Geologist.

Certain revisions in excavation limits, footing elevations and sizes, and revisions to or elimination of seal course concrete are not considered contract changes. Written direction can be given to the Contractor to implement various changes without the immediate need for a change order. However, this situation is limited to instances where only contract items are affected. As most items are final pay items, a change order will ultimately be needed in order to allow the quantity change (refer to Bridge Construction Memo 2-9.0).

In actual practice, change orders are almost always issued to cover footing revisions. 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, advising of the following (refer to Bridge Construction Memo 2-9.0):

| 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 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. |

For pile-supported foundations, the plans and specifications will almost always specify both a bearing value and a specified tip elevation for driven piles and a tip elevation for non-driven piles.

In accordance with Section 49-1.08 of the *Standard Specifications*, driven piles must penetrate to the specified tip elevation unless otherwise permitted in writing by the Engineer. On those occasions when the required bearing value is obtained at the specified tip elevation, but pile tips penetrate below the specified tip elevation, no additional payment will be made for the additional length of pile below the specified tip elevation unless ordered in writing by the Engineer.

In order to avoid the cost of cutting off piles, the Contractor may elect to drive the pile head to the required 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.

Frequently, driven piles must penetrate below the specified tip elevation in order to obtain the required bearing value. In this case, the Contractor will be compensated for the additional length of pile between the specified tip elevation and the tip elevation where bearing was obtained, as determined by the Engineer. Compensation will be at contract item price for furnishing piling.

When this problem occurs and the specified pile type is steel "H" piles, the Engineer should consider using lugs in order to reduce the additional pile length required. When lugs are ordered by the Engineer, the cost of furnishing and welding steel lugs to piles is paid for by extra work at force account or agreed price (refer to Bridge Construction Memo 130-5.0).

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 accordance with Section 49-4.03 of the *Standard Specifications*, the Contractor has the option to submit a proposal to increase the diameter and revise the tip elevation of CIDH piling. In this instance, the Contractor is paid for the theoretical length of the specified pile to the specified tip elevation.

Bridge Construction Memo 9-1.0 covers 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 are often not shown on the As-Built plans and become a problem during the construction of footing seismic retrofits. Other problems have resulted from existing shoring and utilities that are moved or left in place. These have added to the cost of projects involving improvements to existing structures.

Currently many contractors are submitting claims regarding Differing Site Conditions. These claims are usually the result of problems with foundation work. Accurate As-Built plans can sometimes help to prevent such claims.

According to Section 5-1.116 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. Be familiar with the information discussed in Chapter 1 of this manual and you may be able to avoid such claims. Remember that the Contractor is not the only person who can find differing site conditions.

CHAPTER

# 4 Footing Foundations

## General

Footing foundations transmit design loads into the underlying soil mass through direct contact with the soil immediately beneath the footing, in contrast to pile-supported foundations which transmit design loads into the adjacent soil mass through pile friction, end bearing, or both.

Since the load bearing capacity of most soils is quite low, about 2 to 5 Tons per Square Foot (TSF), footing areas will be large in relation to the cross section of the supported member, particularly when the supported member is a column.

Each individual footing foundation must be sized so that the maximum soil bearing pressure does not exceed the allowable soil bearing capacity of the underlying soil mass. In addition, footing settlement 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.

## 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.”

Although not a separate category, seismic retrofits of pre-1973 spread footings are now quite common. Designs of spread footing seismic retrofits typically include adding a top mat of rebar so that any seismic uplift force, which would produce tension in the top of the footing, can be resisted. In some cases footing dimensions are increased and/or perimeter piles added, which create a resisting couple required to provide additional restraint against rotation. Typical spread footings seismic retrofits are shown in Figures 4-1 and 4-2.

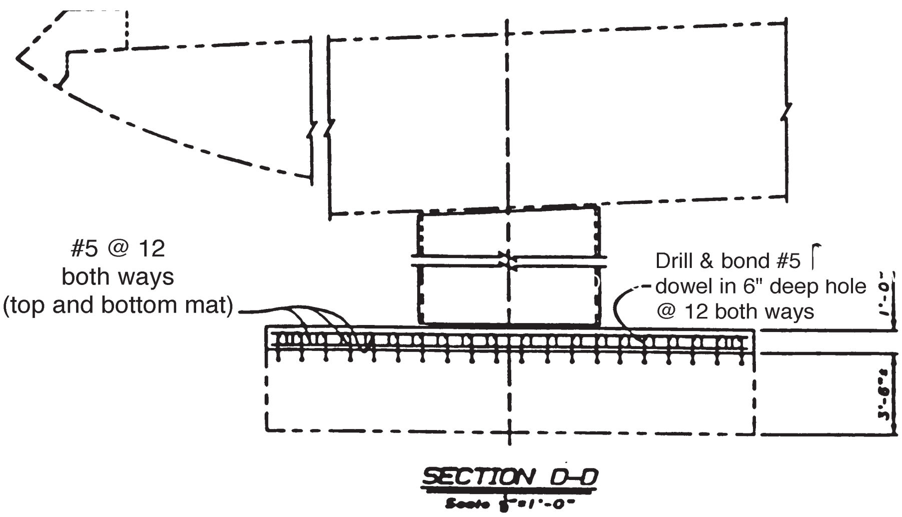



**Figure 4-1: Footing Retrofit (add top mat of rebar)**

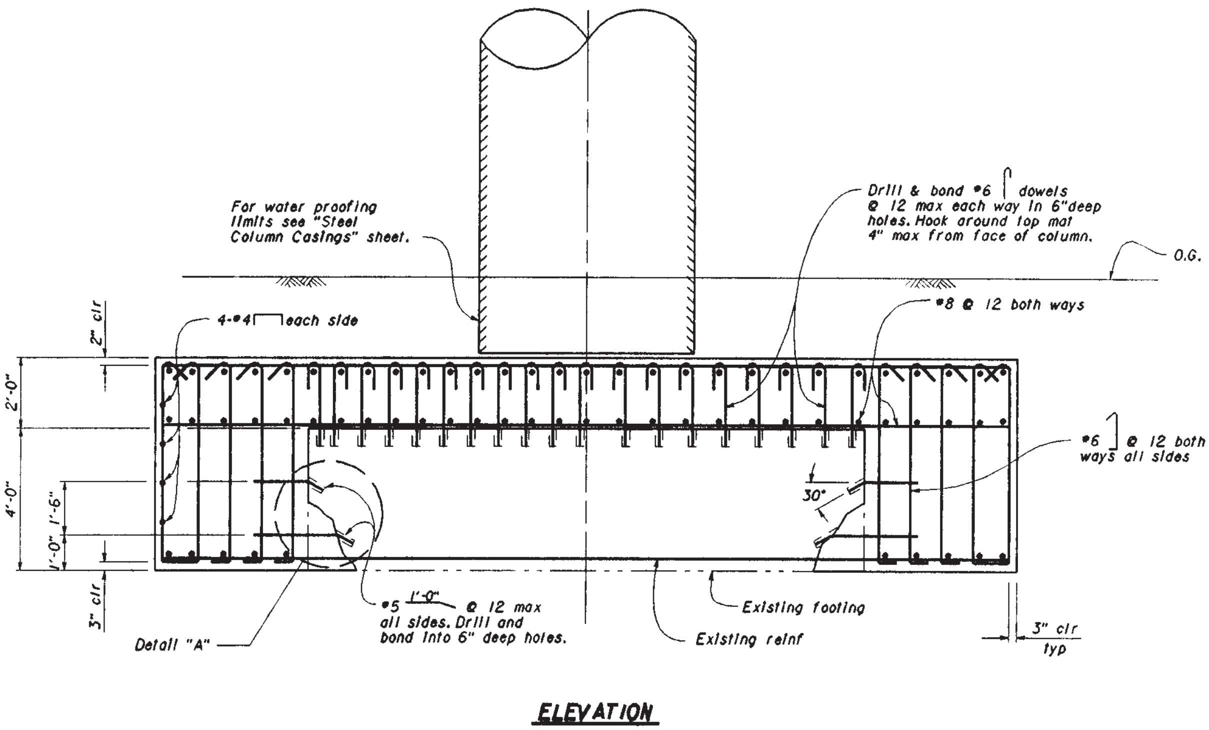



**Figure 4-2: Footing Retrofit (increase footing size and add top mat of rebar)**

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.

Combined footings are generally required when loading conditions (magnitude and location of load) are such that single column footings create undesirable engineering problems, are impractical, or uneconomical. For example, locating a column at or near a footing edge will invariably result in a soil bearing pressure that exceeds the allowable bearing capacity of the soil mass. Other potential engineering problems associated with edge-loaded footings are excessive settlement and/or footing rotation. The results of footing rotation on soil bearing pressures can be seen in Figure 4-3.

These problems can be eliminated, or at least minimized, by combining an edge-loaded footing with an adjacent single column footing. This is generally accomplished by one of two methods. In the first method, two footings are combined to form a single rectangular or trapezoidal footing. This type is referred to as a combined footing. In the other method, two spread footings (one edge-loaded) are structurally connected by a narrow concrete beam. This type is referred to as a cantilever or strap footing.

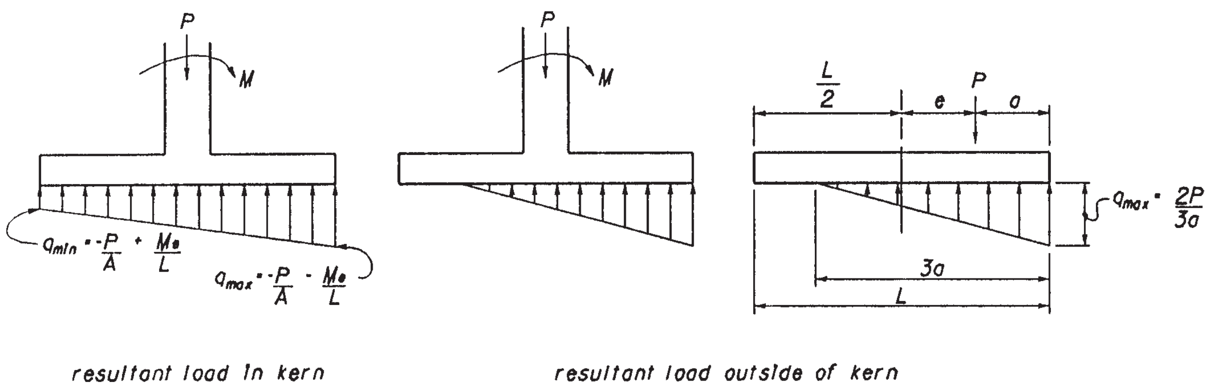



**Figure 4-3: Loaded Footing with Moment**

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.

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 retrofit projects have incorporated designs which have attached 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.

At present, ultimate bearing capacity solutions are based primarily on the Theory of Plasticity; that is, the soil mass is incompressible (does not deform) prior to shear failure. After failure, deformation (plastic flow) occurs with no increase in shear.

The implication of the foregoing 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 are used in bearing capacity analyses but are modified to provide for variations in soil characteristics. These modifications are based primarily on data obtained empirically and through small-scale testing.

## Failure Modes

Shear failure of a soil mass supporting a footing foundation will occur in one of three modes: (1) general shear, (2) punching shear, or (3) local shear. The general shear failure mode can be theoretically described by the Theory of Plasticity. The other two failure modes, punching and local shear, have, as yet, no theoretical solutions.

General shear failure is shown in Figure 4-4 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.

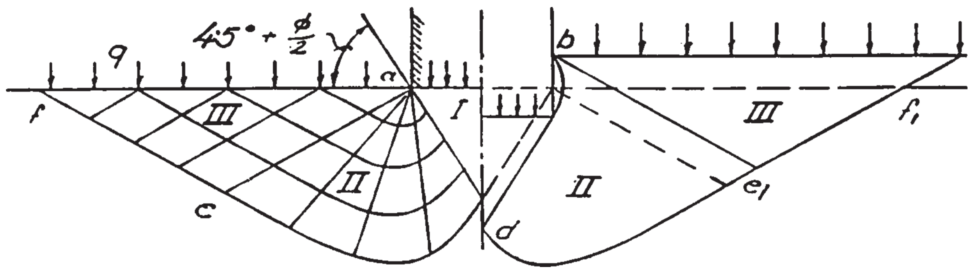



**Figure 4-4: General Shear Failure Concept**

General shear failure for the most part is sudden and catastrophic. Although bulging of the ground surface may be observed on both sides of the footing at a stress level below failure, failure usually occurs on one side of the footing. For example, an isolated structure may tilt substantially or completely overturn. A footing restrained from rotation by the structure will increase structure moments (stresses) and may lead to collapse or excessive settlement.

Punching shear failure (Figure 4-5) 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 which may or may not be observed, i.e., movement may be occurring in small increments. Footing stability (no rotation) is usually maintained throughout failure.

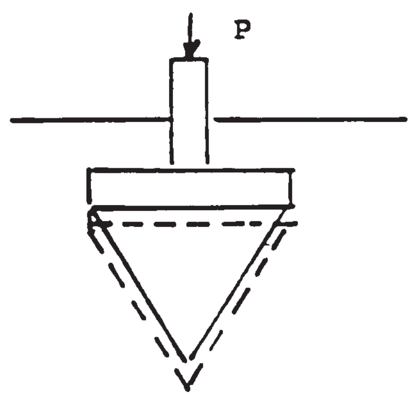



**Figure 4-5: Punching Shear Failure**

Local shear failure (Figure 4-6) may exhibit both general and punching shear characteristics, soil compression beneath the footing, and possible ground surface bulging.

Refer to Figure 4-7 for photographs of actual test failures using a small steel rectangular plate (about 6 inches wide) and sand of different densities.

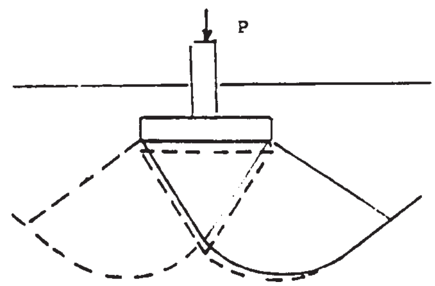



**Figure 4-6: Local Shear Failure**

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 4-7: Failure Modes**

The failure mode to be expected for a given soil profile cannot be predicted. The statement can be made, however, that the mode of failure depends substantially on the compressibility or incompressibility of the soil mass. This is not to imply that soil type alone determines failure mode. For example, a shallow footing supported on a very dense sand will usually fail in general shear, but the same footing supported on a very dense sand which is underlain by a soft clay layer may fail in punching shear.

The ultimate bearing capacity of a given soil mass under spread footings for permanent construction 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

$\gamma$ = soil unit weight

B = foundation width

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

c = soil cohesion, which for the 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, $N_\gamma$, $N_c$, and $N_q$ are dimensionless bearing capacity factors that are functions of the angle of internal friction. The term containing factor $N_\gamma$ shows the influence of soil weight and foundation width, that of $N_c$ shows the influence of the soil cohesion, and that of $N_q$ shows the influence of the surcharge.

## Factors Affecting Bearing Capacity

When the supporting soil is a cohesionless material, the most important soil characteristic in determining the bearing capacity is the relative density of the material. An increase in the relative density is accompanied by an increase in the bearing capacity. Relative density is a function of both ø and $\gamma$, the angle of internal friction and unit weight, respectively.

For cohesive soils, the unconfined compressive strength $q_u$, which is a function of clay consistency, is the soil characteristic affecting bearing capacity. The bearing capacity increases with an increase in $q_u$ values.

The location of the water table surface is another factor having a significant impact on the bearing capacity of both sand and clay.

When the depth of the water table from the bottom of the footing is greater than or equal to the width of the footing B, the full 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, the submerged unit weight, $\gamma_{sub} = \gamma_{sat} - \gamma_w$, is used in the first term of the bearing capacity equation. The result, when the water table is at the bottom of the footing, is to reduce the first term of the equation by approximately 50%. If the water table is above the bottom of the foundation, the surcharge unit weight is also affected, and the submerged unit weight must be used in the third term of the equation.

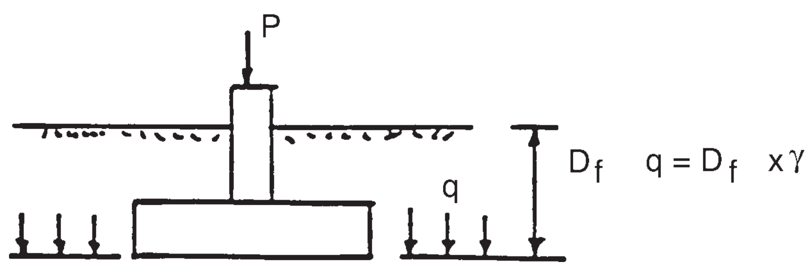



**Figure 4-8: Surcharge Load on Soil**

Moisture reduces the apparent cohesion of clay and therefore, the shear strength. The unit weight of clay is also reduced when submerged in water (saturated). In saturated clays in undrained shear, the foundation width has little effect on bearing capacity.

It is apparent that bearing capacity of both cohesionless and cohesive soils will be reduced by rising water tables. This can be seen in the general bearing capacity formula when the lighter submerged unit weight of soil is substituted for the dry unit weight. Therefore, the effects of a rising water table on the bearing capacity of the footing soil mass, at any time during construction, must be considered.

Structure related factors affecting the bearing capacity are the depth of footing below ground line ($D_f$) and the footing shape.

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 4-8). 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.

Theoretical solutions for ultimate bearing capacity are limited to continuous footings (LENGTH/WIDTH $\geq$ 10). Shape factors for footings other than continuous footings have been determined primarily through semi-empirical methods. In general, the ultimate bearing capacity of a foundation material supporting a square or rectangular footing is greater than for a continuous footing when the supporting material is clay and less when the supporting material is sand.

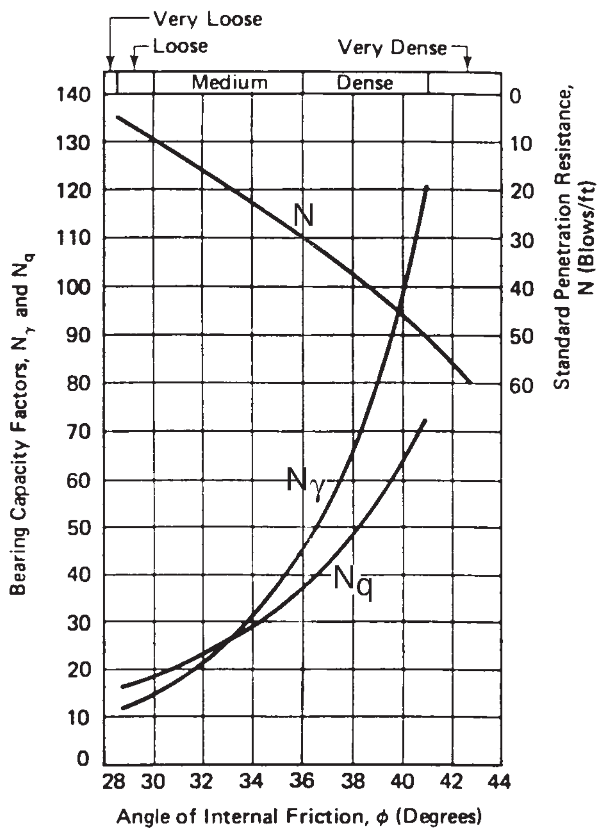



**Figure 4-9: Bearing Capacity Factors for Granular Soils**

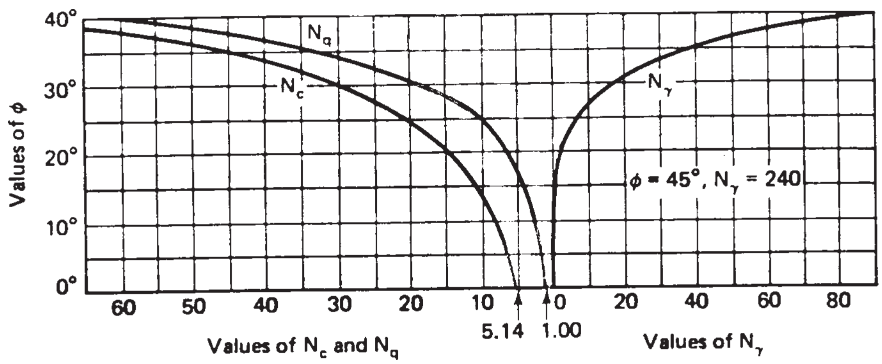



**Figure 4-10: Bearing Capacity Factors for Cohesive Soil**

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 granular soils, a relationship between the standard penetration resistance, N, and the bearing capacity factors, $N_\gamma$ and $N_q$, is shown in Figure 4-9. For cohesive soils, the relationship between N and the angle of internal friction, ø, is shown in Figure 4-9. The value of ø determined from Figure 4-9 can then be used to determine the bearing capacity factors, $N_\gamma$, $N_c$, and $N_q$, as shown in Figure 4-10. Values for ø, $q_u$, N, and $\gamma$ can be approximately determined using the tables for granular and cohesive soils shown in Appendix A. Values for N can also be determined from the Log of Test Borings.

# Settlement

The allowable bearing capacity for granular soils will almost always be governed by tolerable settlement. When the Structure Foundations Branch of the Office of Structural Foundations recommends spread footings for bridge structures, they specify net allowable bearing pressures that will produce no more than $^1/_2$ inch of maximum settlement or $^1/_2$ inch of differential settlement between footings. These allowable bearing pressures are generally 25% to 33% of the ultimate bearing capacity as determined by the general bearing capacity formula.

Frequently bridge fills are constructed on unstable foundation material. When this occurs, the Structure Foundations Branch will specify that the foundation area be pre-loaded with a surcharge for a specified length of time, known as a settlement period, prior to the start of structure construction. The loading consists of an embankment constructed to specified limits. The Structure Foundations Branch will determine the need to pre-load 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 in excess of 30 days are specified, settlement platforms will usually be required. The Foundations Testing and Instrumentation Section will furnish and supervise the installation of the settlement platforms (Refer to Appendix C for California Test 112 - Method for Installation and Use of Embankment Settlement Devices). A change order written by the Engineer to compensate the Contractor for the initial installation of the settlement platforms will be required.

For settlement periods of less than 30 days, the Structure Representative may install settlement hubs in the top of the bridge embankments. The Structure Representative can then monitor and record the hub elevations weekly. Because the Structure Representative is responsible for terminating a settlement period, data from the hub elevations can be used to determine when this should take place. At the end of the 30 day settlement period, if settlement is still taking place, the settlement period should be extended until settlement has ceased. However, if no settlement has occurred during the last week or two of the settlement period, the Structure Representative should terminate the settlement period at the end of the 30 day period or may alternatively elect to shorten the length of the settlement period. The Contractor should be notified of this decision in writing.

## Construction and Inspection

In order to anticipate possible future foundation problems and then formulate some possible solutions before construction begins, the Structure Representative should review and have a complete understanding of all contract documents. As soon as practical, the Contractor should also be reminded that footing elevations shown on the plans are approximate only and foundation modifications are possible. The Structure Representative should draft 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).

The Structure Representative should then review the following documents:

**Contract Plans**. . .Check footing elevations and adequate cover, design bearing pressures, special treatment of foundations, proximity of utilities, existing structures, highways and railroads, etc.

**Special Provisions**...Review sections on earthwork, concrete structures, order of work, etc.

**Log of Test Borings**...Check soil profile, groundwater, etc.

**Foundation Report**

**Standard Specifications**...Review the appropriate sections of the *Standard Specifications* relating to construction methods for spread footings.

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 are 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 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 footing. The Contractor is responsible for costs incurred due to fabrication of materials or other work prior to final determination of footing elevations. The Contractor should be notified in writing of the possibility of foundation changes prior to commencing foundation excavation operations (refer to Bridge Construction Memo 2-9.0).

Section 19-3.07: When the Engineer determines that it is necessary to increase the depth or width of the footing beyond that which is shown on the plans, for a depth of up to 2 feet below the planned footing elevation or for a width of up to 3 times the planned footing width, increased structure excavation quantities will be paid for at the contract price per cubic yard for structure excavation.

Section 19-3.05: The Contractor shall notify the Engineer when the footing excavation is substantially complete and is ready for inspection. No concrete shall be placed until the footing has been approved by the Engineer.

Section 19-5.03: Relative Compaction of not less than 95% is required for embankments under the bridge or retaining wall footings not supported on piles.

Section 19-3.04: Discusses acceptable methods for removing water from excavations where seal course concrete is specified (or not specified). For footings supported on an excavated surface other than rock, suitable foundation material encountered at the planned footing elevation which has been disturbed or removed by the Contractor shall 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. If groundwater is encountered during excavation, dewatering shall be commenced and shall precede in advance of or concurrent with further excavation. When unsuitable foundation material is encountered at planned footing elevation, the corrective work will be as directed by the Engineer. Payment for additional work will be at contract prices (preferred) and/or extra work, as determined by the Engineer. When footing concrete or masonry is to rest upon rock, removal shall be as required to expose sound rock. Bearing surfaces shall be roughly leveled or cut to steps and roughened. Seams shall be pressure-grouted or treated as directed by the Engineer and the cost of such work will be paid for as extra work.

Section 51-1.04: Pumping from foundation enclosures shall be done in such a manner as to preclude 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.

Review the appropriate sections of the *Standard Specifications* relating to forms, rebar, concrete, etc..

Section 51: Concrete Structures

Section 52: Reinforcement

Section 90: Portland Cement Concrete

## Excavations

An obvious hazard associated with footing foundations is the open excavation. Worker safety must be provided for during excavation operations and/or shoring construction. The Division of Occupational Safety and Health (DOSH) requires that each employee in an excavation be protected from cave-ins by an adequate protective system. The protective system can consist of either 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 (refer to the Caltrans *Trenching and Shoring Manual* for DOSH Standards when reviewing a Contractor's excavation safety plan).

Surcharge loads 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 4-11.

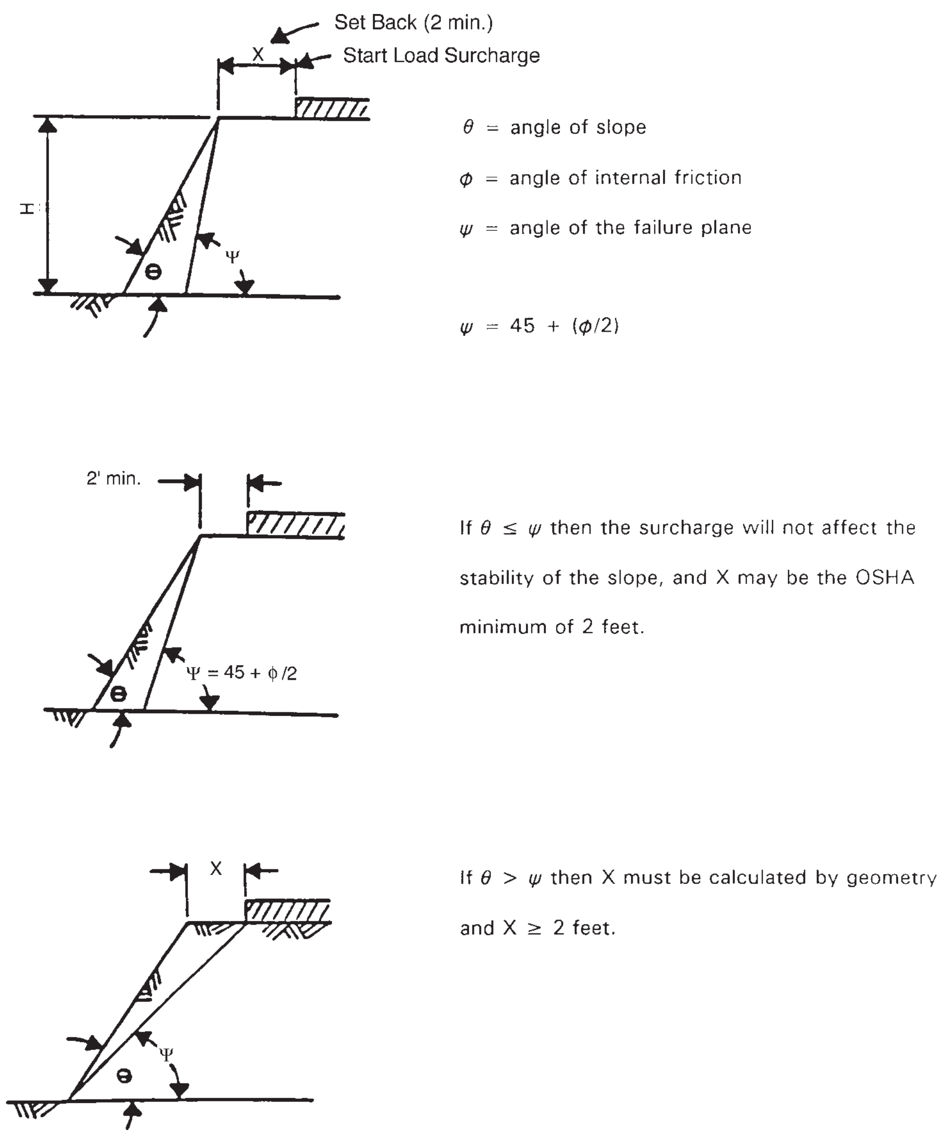
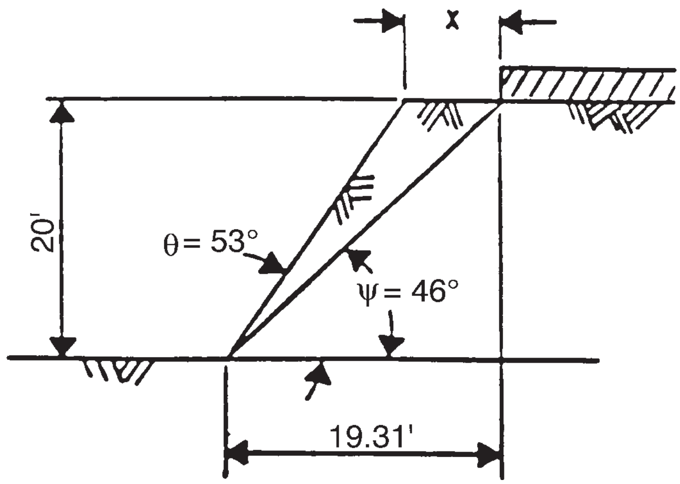



**Figure 4-11: Slope Setback**

Example:




$H = 20'$

$\theta = 53^\circ$ ($^3/_4$:1 maximum slope allowed by OSHA)

$\psi = 46^\circ$ (Material has little internal friction)

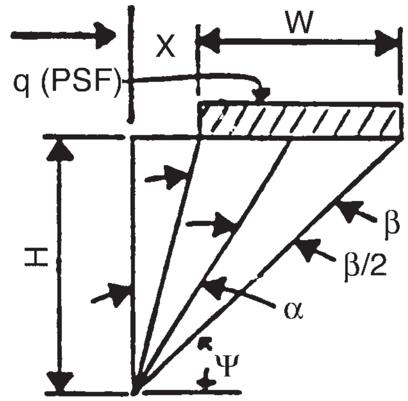
$20'/\tan 46^\circ = 19.31'$

$20'/\tan 53^\circ = 15.07'$

$X = 20' - 15.07' = 4.24'$

For shored excavations, the minimum setback in level ground is equal to the depth of excavation unless surcharge loads are considered in the shoring design (Figure 4-12).

No setback of the surcharge load is required if the earth support system is designed for the summation of lateral pressures due to the surcharge and earth pressures and a barrier with a minimum height of 18 inches is provided to prevent any debris or other material from entering the excavation.




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

At Depth "H",

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

where $\beta_r$ is in radians

At full height H,

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

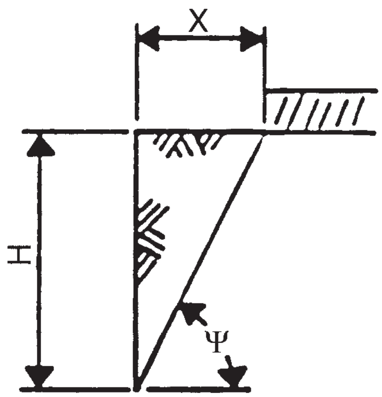
$$\alpha = \arctan\frac{X}{H} + \frac{\beta}{2}$$

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

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

**Figure 4-12: 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, which is calculated as shown in Figure 4-13.




Let X = setback of surcharge load

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

$$\phi = \angle \text{ of internal friction}$$

$$\psi = \angle \text{ of failure plane} = 45 + \frac{\phi}{2}$$

For most soils, $\psi$ is about 55°

**Figure 4-13: Setback Calculation for Shored Excavations**

Refer to the Caltrans *Trenching and Shoring Manual* for information regarding shoring design and construction.

## Wet Excavations

Sump pumps are frequently used to remove surface water and a small infiltration of groundwater.

Sumps and connecting interceptor ditches should be located well outside the footing area and below the bottom of footing so the groundwater is not allowed to disturb the foundation bearing surface.

In granular soils, it is important that the fine particles not be carried away by pumping. Loss of fines may impair the bearing capacity and cause settlement of existing structures. 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 prolonged pumping is required, the sump(s) should be lined with a filter material to prevent or minimize loss of fines.

When it becomes necessary to lower the water table, one commonly used method is the single well point system (Figure 4-14).

A well point is a section of perforated pipe 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. Well points are connected to 2 to 3 inch diameter riser pipes and are inserted into the ground by driving and/or jetting. The riser pipes, which are spaced at 2 to 5 foot centers, are connected to a header pipe which is connected to a pump.

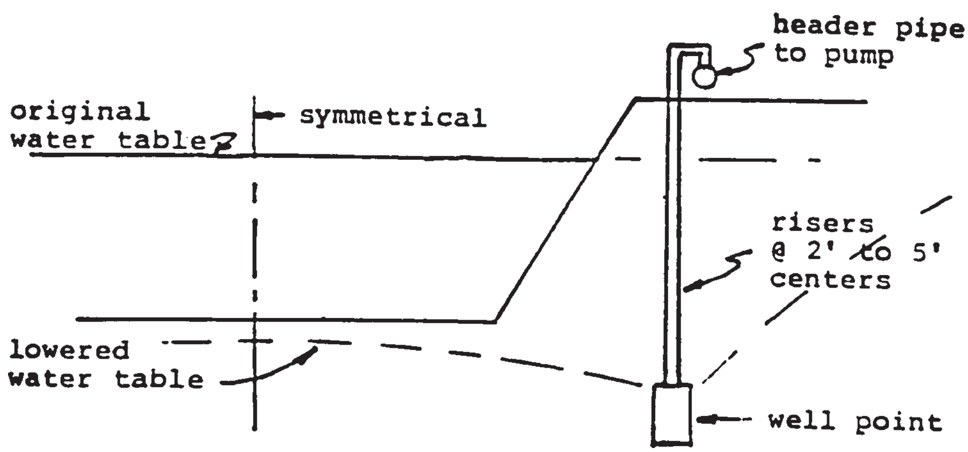



**Figure 4-14: Single Stage Well Point System**

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 are 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.

If a soft clay strata overlying sand is encountered and dewatering is contemplated, the Structure Representative is cautioned that lowering the water table by pumping from underlying layers of sand may cause large progressive settlement of the clay strata in the surrounding area. This is due to consolidation of the saturated clay below the lowered water table caused by 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 4-15).

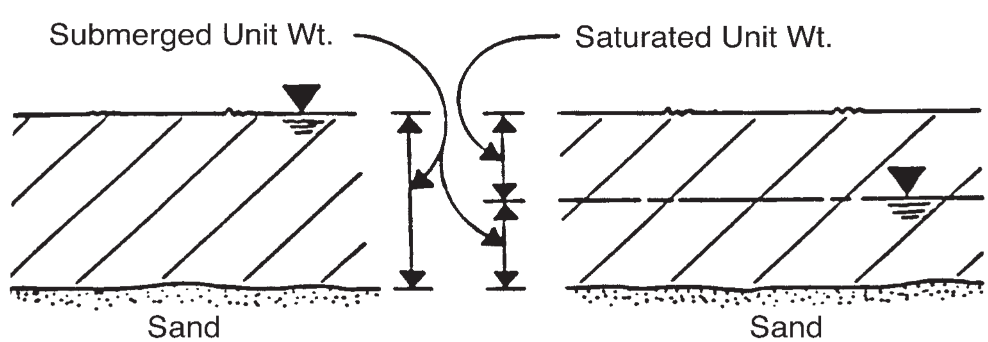



**Figure 4-15: Saturated vs. Submerged Unit Weight**

## Bottom of Excavation Stability

Heave and piping are problems associated with bottom of excavation stability.

Heave is the phenomena whereby the "head" of the surrounding material causes the upward movement of the material in the bottom of the excavation with a corresponding settlement of the surrounding material. Heave generally occurs in soft clays when the hydrostatic head, 62.4(h + z), is greater than the weight of the overburden at the bottom of the excavation, $\gamma z$ (Figure 4-16).

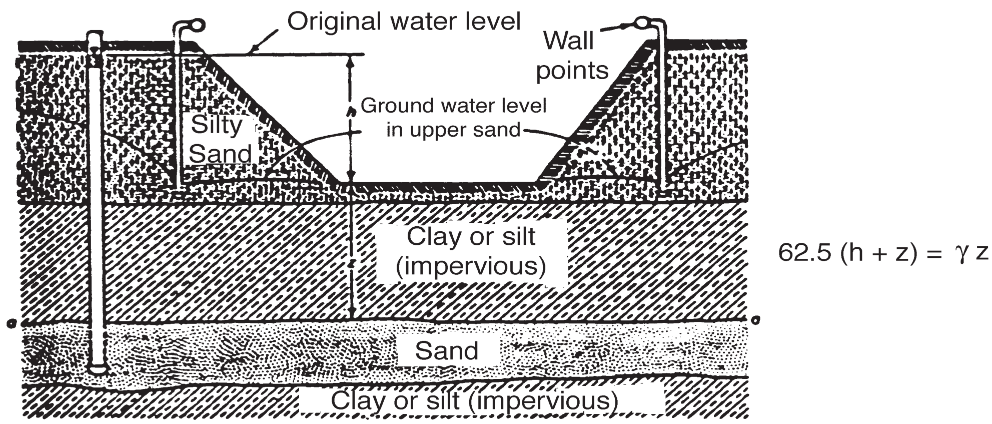



**Figure 4-16: Bottom of Excavation Stability Problems due to Excess Hydrostatic Head Against an Impervious Layer**

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 suspected at the bottom of excavations.

## Foundation Inspection

Inspection should include determination of the following:

1) Stability of slopes and sides of excavations conform with DOSH requirements.
2) Conformity of the foundation material with 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 which may require shoring or underpinning.

5) Forms conform with layout, depth, dimensions, and pour grade of plans. Forms are mortar tight.

6) Reinforcing steel 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 (2 inch minimum rebar clearance to wood forms, 3 inch minimum for neat formed excavation).

7) Concrete placing operations: proper mix number, truck revolutions, back-up alarm, concrete temperature. 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.

Footing forms are either built-up 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.

Often, the footings are excavated "neat," (excavated to the footing dimensions) and the concrete placed against the sides of the excavation, eliminating the need for footing forms. Top of footing grades must be clearly delineated with stakes or flagged spikes driven into the sides of the excavation. A bench of sufficient width to prevent sloughing or cave-in should be provided around the excavation for access and work area. Ensure that "neat" excavations conform to the planned footing dimensions, or if not, place the exact footing dimensions as constructed on the "As-Built" drawings. On recent seismic retrofit projects, several costly Contract Change Orders have had to be written to correct past undocumented footing overpours.

Whether footings are formed or excavated "neat", a template should be constructed for positioning vertical reinforcing steel cast in the footing to prevent displacement of the vertical rebar during the pour.

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 (the rebar weight can cause settlement in the form panels affecting pour grades). Top reinforcing steel mats supported by chairs should be blocked to the forms or sides of the excavation. The bottom reinforcing steel mat which supports the vertical column steel should be blocked to prevent any settlement. It is required that all reinforcing steel dowels be tied in place prior to concrete placement and not "stuck in" during or after concrete placement.

The effective depth of reinforcing steel is critical and must always be checked. 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 vertical column reinforcement is supported by the bottom mat 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 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.

Immediately prior to placing concrete, all material which has sloughed into the excavation must be removed. Check again the clearance between the bottom of the excavation and the bottom reinforcing steel mat. The foundation material should be wet down but not saturated. To avoid segregation of the concrete, the ends of the concrete pour chutes should be equipped with a hopper and length of tremie tube to prevent free fall of concrete in excess of 8 feet.

## Foundation Problems and Solutions

It is mandatory that the Engineer inspect the excavated surface at the planned footing elevation after the excavation is completed (Section 19-3.05 of the *Standard Specifications* requires the Contractor to notify the Engineer after the excavation is completed). Only by visual inspection can the Engineer determine if the foundation material is suitable, disturbed and/or contaminated, or unsuitable.

## Suitable Foundation Material

If the foundation material encountered at planned footing elevation is suitable, the Contractor should be notified in writing of the Engineer's decision.

## Disturbed and/or Contaminated Material

A suitable foundation material encountered at planned footing elevation but disturbed or contaminated is unacceptable and must be corrected. Disturbance of the foundation bearing surface is invariably caused by the Contractor's choice of excavation methods. Often the bearing surface is disturbed simply by excavating below the footing elevation, and occasionally the bearing surface is disturbed at grade by the teeth on the excavator bucket. Contamination is usually due to water 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 Structure Representative.

Acceptable restoration methods include:

1) Maintain top of footing as planned and overform footing depth. With few exceptions, the Contractor will choose this method when the restoration depth is about one foot or less.
2) Replace the material removed (to planned bottom of footing elevation) with Class C (4 sack) concrete.
3) Footings having a design bearing pressure of not more than 3 TSF and where the depth of the material removed does not exceed one foot, the bottom of footing may be restored with structure backfill material compacted to 95% relative compaction. Structure backfill material must meet the requirements of Section 19-3.06 of the *Standard Specifications*.

It cannot be over-emphasized that the restored foundation must be at least equal to the undisturbed foundation as determined by the Engineer.

It is recommended that the following precautionary measures be taken during excavation and construction in order to avoid or minimize disturbance and/or contamination of the foundation surface:

1) Under-excavate with mechanical equipment and hand excavate to bottom of footing.
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 Foundation Investigations Section should be contacted when the Engineer determines that the undisturbed original material encountered at planned footing elevation is either unsuitable or of a questionable nature.

If the Structure Representative is absolutely certain that the material encountered at the planned footing elevation is unsuitable, then hand-excavating a small exploratory hole would be advisable prior to contacting the Engineering Geologist. First, review the Log of Test Borings to determine if the unsuitable material is shown but at a higher elevation. If so, the anticipated suitable material may well be just below the excavated surface.

Simple tests that can be performed in the field are:

1) Penetration tests - granular soils
2) Finger tests - cohesive soils
3) Pocket penetrometer - cohesive soils
4) Rodding and probing

Note that these simple and expeditious tests give only an approximate evaluation of the soil at or immediately below the surface. If the bearing capacity of the foundation material is questionable for any reason, consult the Engineering Geologist.

The depth of any hand excavation should not exceed 2 feet, however, because lowering a footing 2 feet or less is not considered a change in the plans or specifications. In any event, the information obtained from the exploratory excavation will be useful in determining the required footing modifications. The required footing modifications will be determined by the Engineer in consultation with the Engineering Geologist, the Bridge Construction Engineer, and the Project Designer.

Footing modifications normally entail one or more of the following procedures:

1) Excavate to a stratum that has sufficient bearing capacity, replace the removed unsuitable material with Class C concrete, and then construct the footing at the planned footing elevation.
2) If the over-excavation is relatively shallow, about one foot or so, replace the removed unsuitable material with footing concrete placed monolithically with the footing.
3) Lower the footing 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.
4) 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. In addition, settlement must not exceed tolerable limits.

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.

The preferred method for compensating the Contractor for the cost of the corrective work is by contract items at contract unit prices and is the specified method of payment for the following revisions:

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.
3) Increasing or decreasing the thickness, or elimination of the entire seal course.

For other revisions, agreed price or force account methods should be used when the above method is unsatisfactory as determined by the Engineer.

## Safety

Any excavation in which there is a potential hazard of cave-in or moving ground requires a protective earth retaining plan. Section 5-1.02 of the *Standard Specifications* requires the Contractor to furnish a temporary earth retaining system plan to the Engineer for approval prior to starting excavation. Also prior to beginning any excavation 5 feet or more in depth into which a person is required to descend, the Contractor must first obtain a DOSH excavation permit.

Regardless of the worker protection system used, the Contractor's Shoring Plan or Excavation Safety Plan should be inspected to ensure compliance with DOSH requirements.

Daily inspections (or after any hazard-increasing occurrence) 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 railing must be located at 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 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.

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.

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 DOSH-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 illegal.
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 the level of 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. 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 Cal/OSHA approved design is on file in the job records prior to their use.

CHAPTER

# 5 Pile Foundations – General

## Specifications

The specifications for piling and pile driving are contained in Section 49 of the Standard Specifications. Unusual requirements or options for particular projects are contained in the Special Provisions. The project plans and Standard Plans are additional contract documents needed for pile work. As-Built plans should be obtained for widening and seismic retrofit projects.

The normal format would be to describe the intended pile type, specified tip elevation, and, in the case of driven piles, the minimum bearing value in the project plans or Special Provisions or both. These documents would also include specific requirements for embankment predrilling, load testing and other items peculiar to the project. For example, if driving difficulty is anticipated, the Project Designer may allow substitution of steel "H" piling for 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.). If the specification allowing the option is not included in the contract, changes from one pile type to another cannot be made without the benefit of a change order.

Structure details for the piles are found in the Standard Plans. The Standard Plans also provide alternative details for certain pile types. This is not to be confused with the option to substitute or change pile type as mentioned in the previous paragraph. If, for example, Class 70C concrete piles are specified, they can be either of the alternatives shown in the Standard Plans for Class 70C concrete piles. However, the alternatives that may be used do not include the alternatives shown in the Standard Plans for Class 70 concrete piles. Occasionally, the Project Designer may decide not to allow all of the alternatives for a given pile type. In this situation, the alternatives to be excluded would be noted in the Special Provisions or project 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.

## Non-Driven Piles

Cast-In-Drilled-Hole (CIDH) piles consist of concrete cast in holes drilled in the ground to a specified tip elevation. Diameters range from 12 to 126 inches and lengths range from 10 feet to over 120 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 driver might damage adjacent structures such as pipelines, etc. For obvious reasons the ground formations into which the holes are drilled should be capable of retaining their shape during drilling and concrete placing operations and no ground water should be present. If ground water is present, 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.

## 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 18 inches in diameter for heavy walled pipe which 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.<br><br>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.<br><br>Although steel piling is relatively expensive on a per foot furnish basis, it has a number of advantages. They come in sizes varying from HP 8×36 to HP 14×117 rolled shapes or may consist of structural steel plates welded together. They are available in high strength and corrosion-resistant steels. They can penetrate to bedrock where other piles would be destroyed by driving. However, even with "H" piles, care must be taken when long duration hard driving is encountered as the pile tips can be damaged or the intended penetration path of the pile can be drastically deflected. Some of this type of damage can be prevented by using a reinforced point on the pile. Due to the light weight and 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.<br><br>Splice details are shown on the Standard Plans or project plans for contracts that permit the use of steel piling. Pile welding work requires special attention and various methods can be used to prequalify welders who will be doing the work.<br><br>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 new alternative pile types which are being reviewed and tested by the Office of Structure Construction and the Office of Structural Foundations. These are designed and used on a site-specific basis. The three types now being reviewed and tested are the GeoJet Foundation Unit, the Tubex Grout Injection Unit, and the Nicholson Pin Pile. The GeoJet Foundation Unit consists of a structural member inserted into an augured soil-cement column. The Tubex Grout Injection Unit is a steel pipe pile with a special cast-iron tip filled with concrete surrounded by an injected grout/soil mixture layer. The Nicholson Pin Pile is a bored cast-in-place pile with a reinforcing bar in a grouted hole.

Refer to Appendix D for drawings and schematics of the various alternative piles.

# CHAPTER 6 Cast-In-Drilled-Hole 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.

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. Nor are they recommended where groundwater is present, unless dewatering can be done without unreasonable effort and unless concrete can be placed without a casing having to remain in place. 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.

## Specifications

The Standard Specifications describe two different types of CIDH pile. The first type is the cast-in-drilled-hole pile. 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 material within the steel shell is then removed and the steel shell is filled with reinforced or non-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 method and hole drilling. Section 52 contains information on pile bar reinforcement. Section 90 contains information on the concrete mix design, transportation of concrete, and curing of concrete used for CIDH piles.

The Special Provisions contain job-specific requirements and revised specifications. Because the CIDH pile specifications are continually updated, it is very important that the Structure Representative 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 piece of equipment 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 which 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

**Figure 6-1: Auger – Short Section**

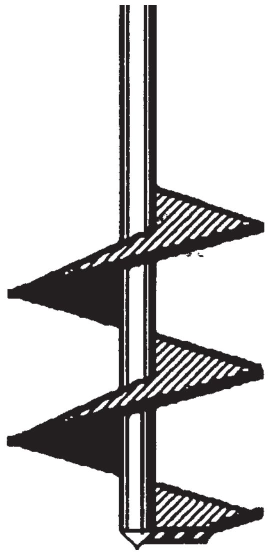


**Figure 6-2: Auger – Single Flight**

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 or for predrilling for driven piles. They may also be used where overhead clearance is not a problem.

Short flight augers are powered by "Kelly Bar" units fixed to the drill rig. The length 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.

There are also a variety of different types of augers 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.

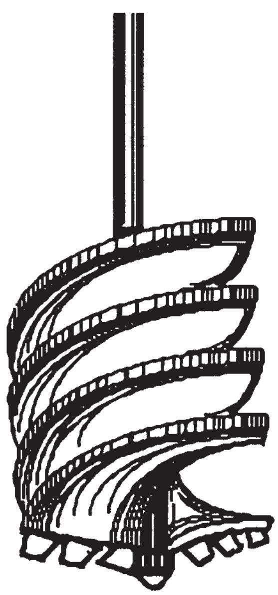


**Figure 6-3: Auger – Double Flight**

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 where the soil will not stay on the flights during auger extraction. They may also not work well in highly cohesive materials where the auger may become clogged.

Rock augers 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. 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**

Drilling buckets (Figure 6-4) are 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 which forces material into the bucket during rotation. When the drilling bucket is full, the bucket is spun in the direction opposite of drilling to close the built-in flaps, which prevent 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 of the drilled hole. This allows the tip of the pile to be founded on a firm flat surface. These buckets have no cutting teeth but are similar to drilling buckets in other aspects. 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 6-5 shows the difference between the cleanout bucket and the drilling bucket.

**Figure 6-5: Drilling Bucket/Cleanout Bucket Comparison**

**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.

Drilling is performed almost exclusively with portable drilling rigs. These units can be self-propelled (Figure 6-7), truck-mounted (Figure 6-8), or crane-mounted (Figure 6-9).


**Figure 6-7: Drill Rig – Crawler Mounted**

**Figure 6-8: Drill Rig – Truck Mounted**

**Figure 6-9: Drill Rig – Crane Mounted**

**Figure 6-10: Steel Casing**

Steel casings (Figure 6-10) are used to support drilled holes when unstable conditions are encountered. Various methods have been used to advance steel casings, 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.

## Drilling Methods

Various other materials are used to supplement the drilling work. Water 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 which 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 of 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. |
| 3 | If permitted by the contract Special Provisions, use of a 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 placing (keeping bottom of casing below the concrete surface). |
| 5 | Dewatering the entire area using well points, deep wells, etc. This should be thoroughly discussed with the Bridge Construction Engineer and the Engineering Geologist. |
| 6 | By contract change order, substitute an alternative type of piling. Again, this should be discussed with the Project Designer, the Engineering Geologist, and the Bridge Construction Engineer. |

Operations should proceed with caution when drilling near utilities known or thought to be in close proximity. The Contractor should contact the utility company and have the utility located. It is also advised that the Contractor 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 Designer, the Engineering Geologist, and the Bridge Construction Engineer.

In the case of problems with groundwater, cave-ins or obstructions at the lower portion of the hole and under certain conditions, the Standard Specifications allow the Contractor to propose increasing the pile diameter in order to raise the pile tip. Before allowing this, the Structure Representative should consult with the Project Designer and Engineering Geologist 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 problems would stimulate the Contractor's action and a change would be proposed to the Structure Representative.

## Inspection

Before drilling begins, it is advisable to 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, the equipment to be used, the plan for avoiding existing utilities (if any), and safety precautions to be taken during the work.

The Structure Representative 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 the soil materials and groundwater level shown on the Log of Test Borings, the Project Designer should be contacted for clarification.

Due to recent changes in design practice, it is possible that the pile may not be designed for end bearing alone. The Project Designer should be contacted to determine whether skin friction has a role in the capacity of the pile. This is especially true of seismic retrofit projects, where pile tensile capacity may be a requirement.

The Contractor is required to lay out the pile locations at the site prior to drilling. This layout should be checked by the Structure Representative prior to drilling. The Structure Representative should also set reference elevations in the area so pile lengths and pile cutoff can be ascertained.

During the drilling operation, the Structure Representative should verify that the piles are being drilled in the correct location and are plumb. Usually, the Contractor will check the Kelly bar with a carpenter's level during the drilling operation. The Structure Representative should also evaluate the material encountered and compare it to that shown on the Log of Test Borings. If the material at specified tip differs from that anticipated, a change may be needed. It may be advisable to keep a written record of the drilling progress and the record utilized to investigate any differing site conditions claims submitted by the Contractor.

When the hole has been drilled down to the specified tip elevation, the Contractor should always use a cleanout bucket to remove any loose materials and to produce a firm flat surface at the bottom of the drilled hole.

After drilling, the depth, diameter and straightness of the drilled hole must be checked. The drilled hole should be checked using a suitable light, furnished by the Contractor, or a mirror. At this time, the Structure Representative should measure and record the length of each pile. Unless the Structure Representative 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 Structure Representative or the Engineering Geologist to inspect the bearing surface at the bottom of the drilled hole. All pertinent requirements of the Construction Safety Orders shall be met before anyone enters the drilled hole.

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. Large amounts of water may need to be pumped out. It may be necessary to remove the rebar cage to accomplish this. Steel reinforcement cage clearances and blocking should also be checked at this time. In addition, the reinforcing cage must be adequately supported and some means must be devised to ensure concrete placement to the proper pile cutoff elevation.

Concrete placement warrants continuous inspection. This subject is covered comprehensively in the contract specifications. 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/PILE DEFECT |
|---|---|
| 1 | The Contractor does not clean off the bottom of the drilled hole with a cleanout bucket. 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 6-11. |
| 2 | The Contractor uses a tapered auger to advance the drilled hole to the specified tip elevation and does not flatten the bottom of the hole with a cleanout bucket. This can result in concrete crushing 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 6-11. |
| 3 | The drilling operation smears drill cuttings on the sides of the drilled hole. 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 6-12. |

These problems are preventable. Adherence to the contract specifications and timely inspection will eliminate most of these problems.

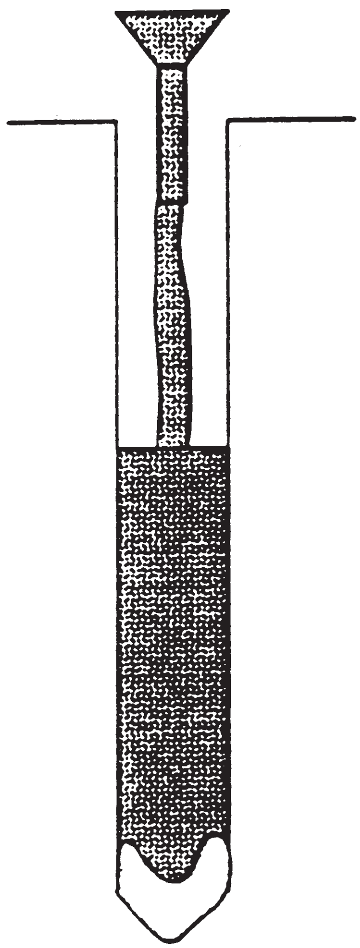


Figure 6-11: Pile Defects
No Cleanout, Tapered Bottom of Hole

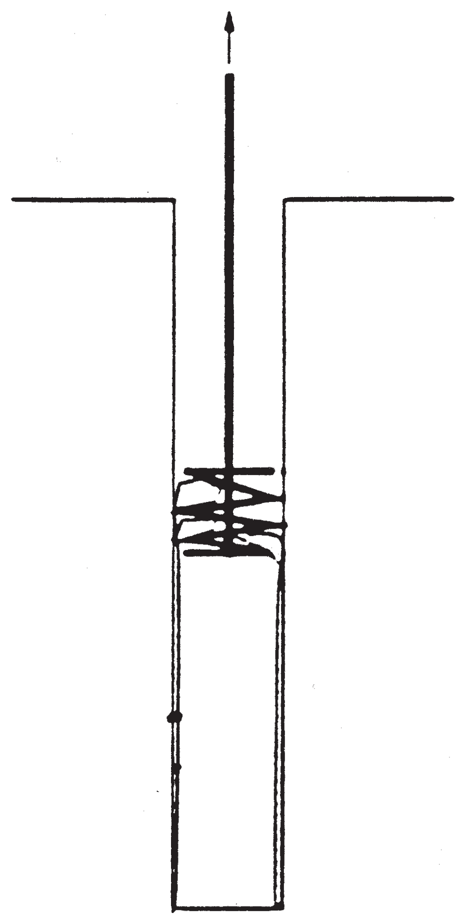


Figure 6-12: Pile Defects
Smeared Drill Cuttings

The following concrete placement problems can cause pile defects:

| ITEM | CONCRETE PLACEMENT PROBLEM/PILE 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. This can result in degraded concrete at the location, thus reducing the capacity of the pile. This defect is shown in Figure 6-13. |
| 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 rebar cage or supporting bracing and segregates. This can result in defective concrete, thus reducing the capacity of the pile. This defect is shown in Figure 6-14. |
| 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. This can result in the sidewall blowout of freshly poured piled into the adjacent drilled hole. This would probably cause the rebar cage to buckle. This defect is shown in Figure 6-15. |
| 4 | The Contractor does not remove groundwater from the drilled hole. This groundwater mingles with the concrete placed and may result in defective concrete at the bottom of the pile. If the pile is designed for end bearing, the capacity would be reduced. This defect is shown in Figure 6-16. |

Most of these 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 rebar cage and concrete, and then start over.

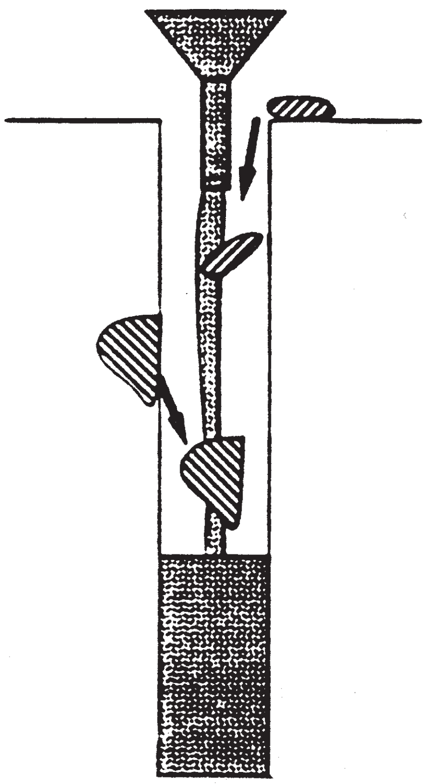


**Figure 6-13: Pile Defects**
**Cave In**

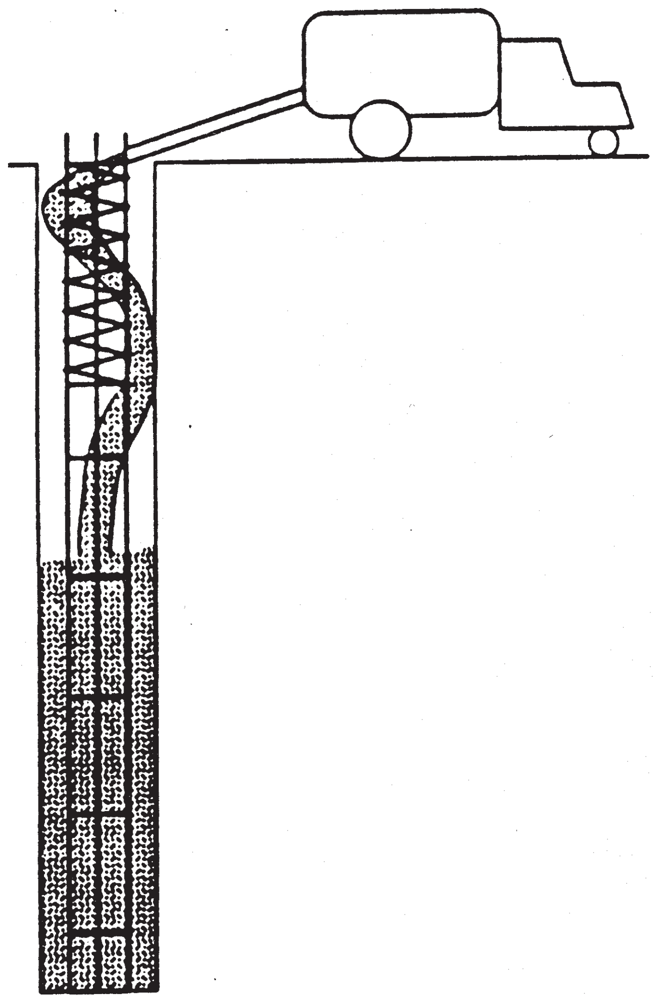


**Figure 6-14: Pile Defects**
**Concrete Segregation**

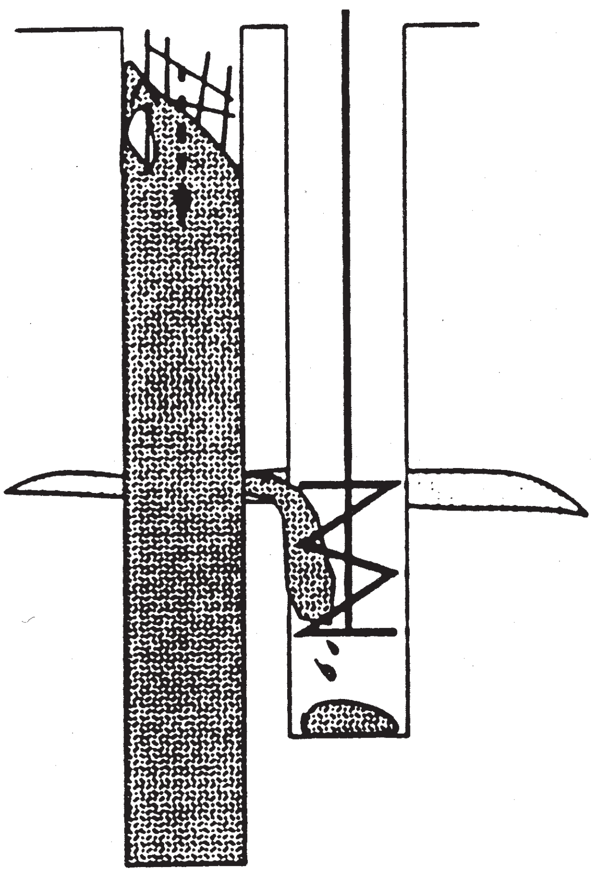


**Figure 6-15: Pile Defects**
**Adjacent Hole Blowout**

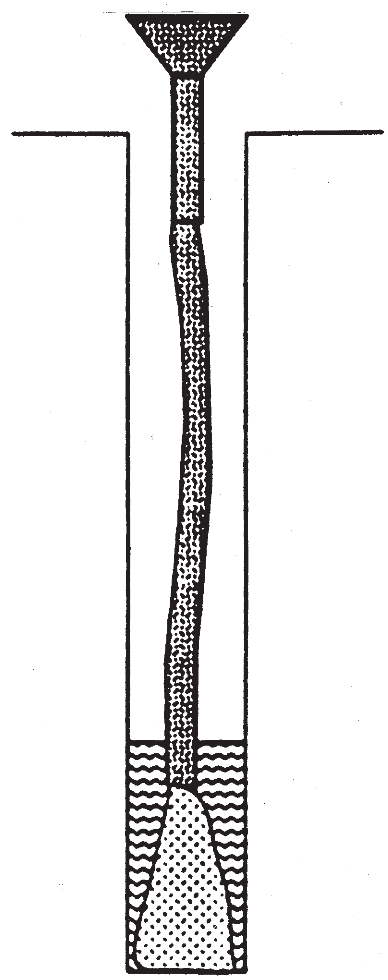
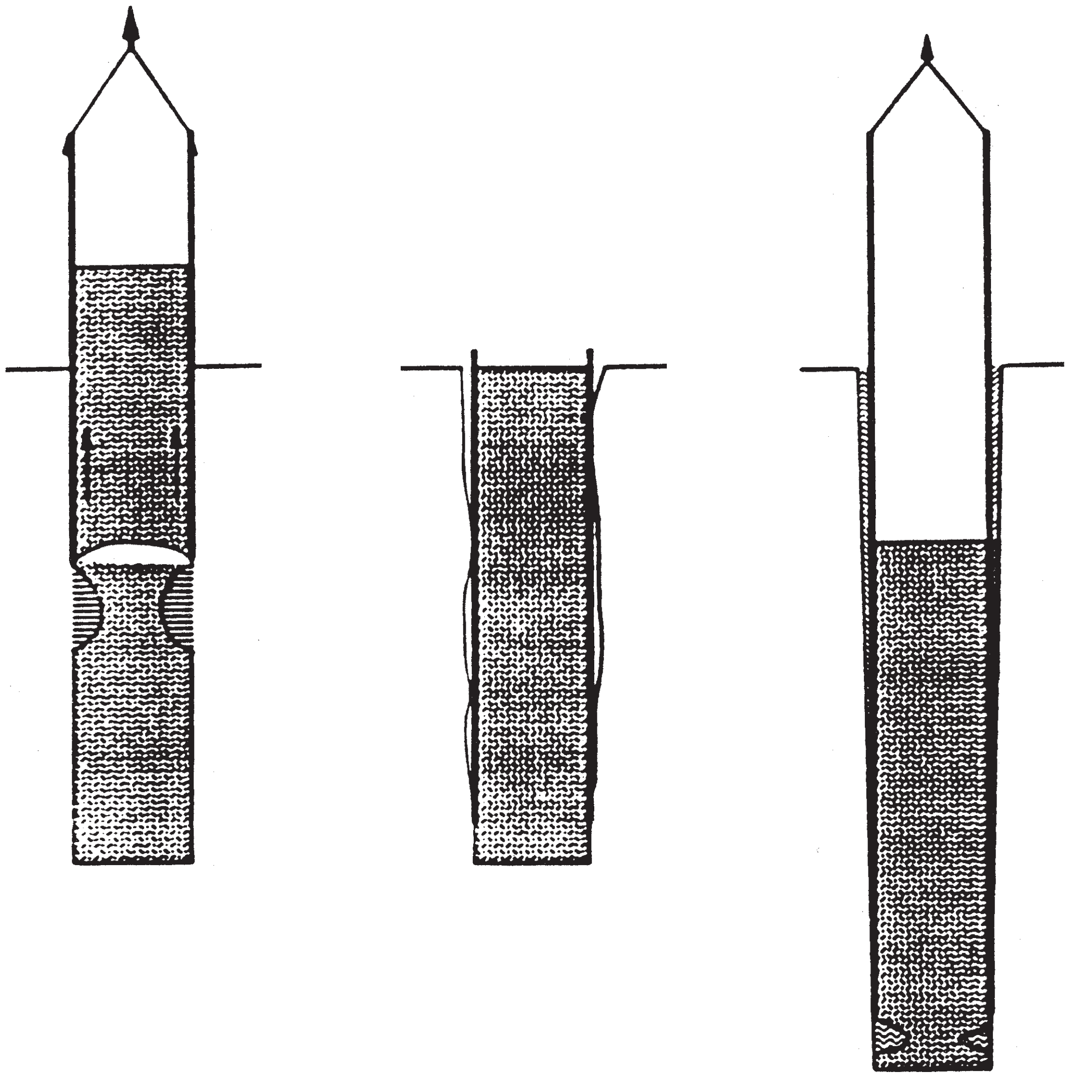


**Figure 6-16: Pile Defects**
**Water in the Hole**

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 6-17(a), (2) the concrete sets casing cannot be removed as shown in Figure 6-17(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 6-17(c). The first problem may result in a void being formed in the pile at the bottom of the casing. It is possible that the suction created may cause a cave-in at this location. The second and third problems result in the loss of the pile's capacity to transfer skin friction to the ground. |

Historically, problems with casings have produced the largest number and the worst type of CIDH pile defects. However, these problems are preventable. Adherence to the contract specifications and timely inspection will prevent most of these problems. It is recommended to allow 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. To further prevent CIDH pile defect problems when casings are used in construction the CIDH pile contract specifications have been revised so that all CIDH piles constructed with the use of temporary casings will be tested for structural adequacy 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 6-17: Pile Defects**
**Casing Problems**

## Safety

As with all construction activities, the Structure Representative should be aware of safety considerations associated with the operation. As a minimum, the Structure Representative 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. 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.

# CHAPTER 7 Driven Piles

## History

Piles are braced, structural columns that are driven, pushed or otherwise forced into the soil. Early primitive man found that pile foundations were very useful in that they allowed construction of a home high above the water or the land and out of reach of marauding animals and warring neighbors.

Piles can be classified as friction piles in either compression or tension (or both), end bearing piles, or a combination of the two. Piles can also be used to generate lateral stability in foundations. Largely by the expensive method of trial and error, early builders discovered that when soil strata immediately beneath a structure were weak and compressible, the foundation should be lowered until more suitable soils were reached. It has also been discovered that, in some cases, there may be a need to develop a hold-down force through the piling. This is accomplished by driving piles that resist uplifting by utilizing tension forces developed between the soil and the pile.

Two types of foundations were developed through the ages to meet the need of supporting structures on deep soil; piles and piers. Piles, by far, are the more commonly used.

The City of Venice was built in the marshy delta of the Poe River because the early Italians wanted to live in safety from the warring Huns of Central Europe. The buildings of Venice are supported on timber piles, driven centuries ago, through the soft mud onto a layer of boulders below. When the bell tower of St. Mark's, built in 900 A.D., fell in 1902, the timber piles in the foundation were found to be in such a good state of preservation that they were used to support the reconstructed tower.

For centuries, timber was normally used for piles. The first concrete piles were introduced in Europe in 1897, and the first concrete piles were driven in America in 1904 by the Raymond Pile Company. Timber piles were usually driven to under 25 Tons bearing, but the new concrete piles were designed for 30 Tons and over. This meant that fewer piles and smaller footings could be utilized for the same imposed loads. Technological advances in the cement and concrete industries made concrete piles cost competitive and, because of this, their use became prevalent.

Pile driving is the operation of forcing a pile into the ground without previous excavation. Historically, the oldest method of driving a pile, and the method most often used today, is by a hammer. No doubt, the earliest bearing piles were driven by hand using a wooden mallet of some sort.

For thousands of years the Chinese and other oriental builders used a stone block as a hammer. It was lifted by ropes and stretched taut by human beings, who were arranged in a star pattern about the pile head. The rhythmic pulling and stretching of the ropes flipped the stone block up and guided the downward blow upon the pile head.

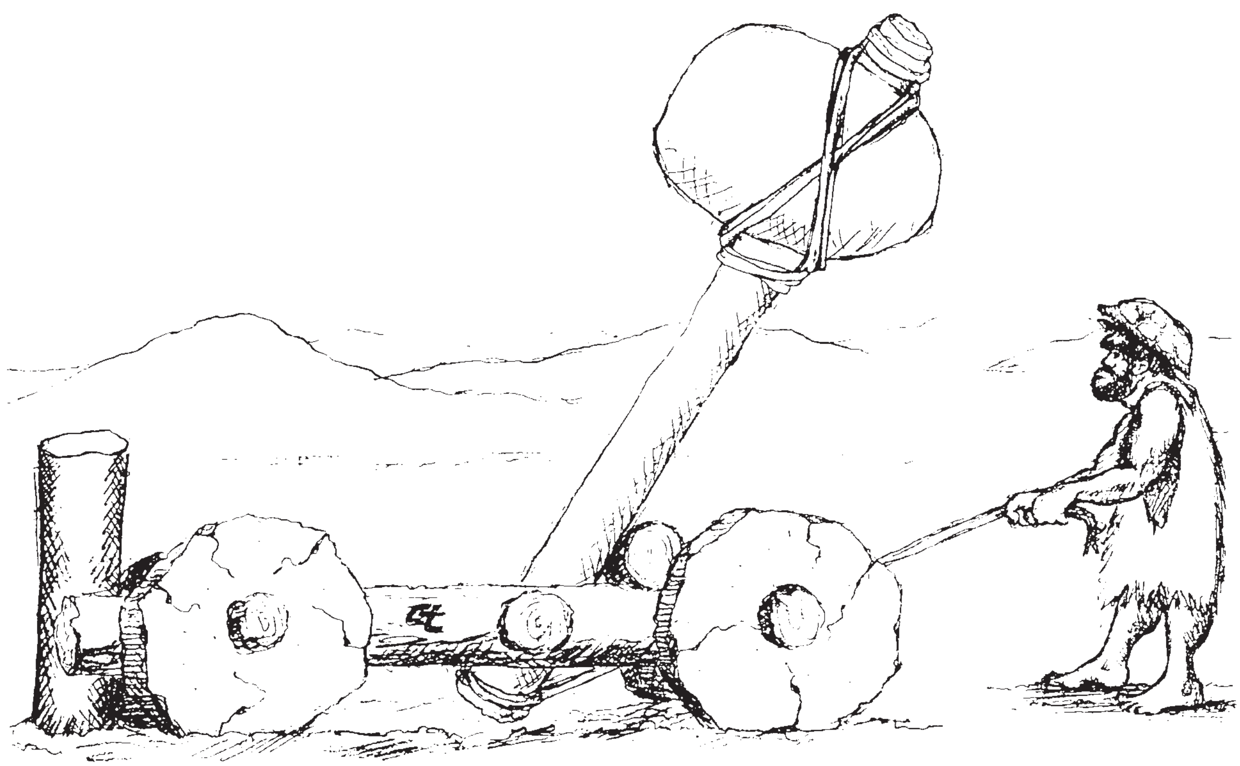


**Figure 7-1: Early Pile Hammer**

The Romans also used a stone block as a hammer. It was hoisted, first by humans then later by horses, by a rope over a pulley. The downward blow was guided by vertical guides that are similar to pile leads used today. This method continued until the invention of the steam engine which was then used to pull the rope. Further development resulted in steam, air, diesel and hydraulic powered impact hammers plus vibratory and sonic hammers.

Modern day requirements for construction have 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.

## General Specifications

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 1992 *Standard Specifications* and should be reviewed as applicable:

Section 19-6.01: When bridge footings are to be constructed in an 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. Rocks, broken concrete or other solid materials larger than 0.33 foot are not allowed in fill where piles are to be driven.

Section 19-6.025: The embankment shall remain in place for the required settlement period before driving foundation piles.

Section 49-1.03: The Contractor is responsible for furnishing piling of sufficient length to develop the specified bearing value, to obtain specified penetration, and to extend into the cap or footing block as indicated on the plans or specified in the Special Provisions. (Estimated tip elevations shown on the plans are used only for estimating the quantity of piling needed for the structure.)

Section 49-1.05: Pile hammers shall be approved steam, air, or diesel hammers that develop sufficient energy to drive piles at a penetration rate of not less than $^1/_8$-inch per blow at the design bearing value. Steam or air hammers shall be furnished with boiler or air capacity 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).

Section 49-1.08: Except for piles to be load tested, driven piles shall be driven to a bearing value of not less than the design loading shown on the plans or specified in the Special Provisions.

Section 49-1.08: When a pile tip elevation is specified, driven piles shall penetrate at least to the specified tip elevations.

Both of the preceding specifications are to indicate that there are in fact two different pile driving requirements: (1) A specific pile tip penetration, and (2) a prescribed bearing value.

## 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 which 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 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 which 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. |
| Hammer Energy | The amount of potential energy available to be transmitted from the hammer to the pile. Usually measured in foot-pounds. |
| Leads | A wooden or steel frame with one two parallel members for guiding the hammer and piles in the correct alignment. There are three basic types of leads:<br>• *Fixed*, which are fixed to the pile rig at the top and bottom. Refer to Figure 7-4.<br>• *Swinging*, which are supported at the top by a cable attached to the crane. Refer to Figure 7-5.<br>• *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 for cast-in-place concrete piles. This assists in maintaining pile alignment and preventing the shell from collapsing. It is removed after driving is completed. |
| Moonbeam | A device attached to the end of a lead brace which will allow 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 which 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 part 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 a hard ground strata to permit pile driving. Used extensively in the driving of timber piles. |
| Striker Plate | A steel plate place 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. |

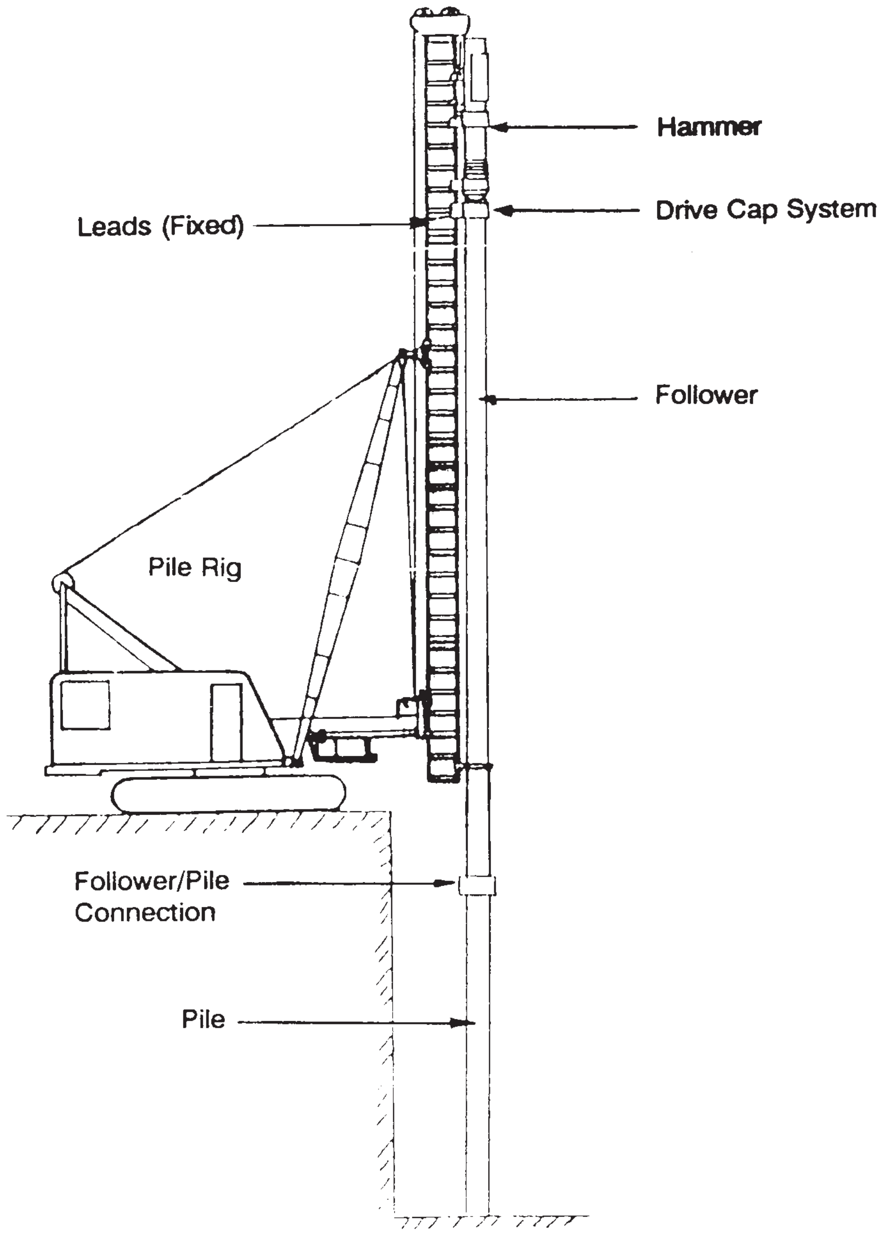



**Figure 7-2: Typical Pile Rig Configuration**

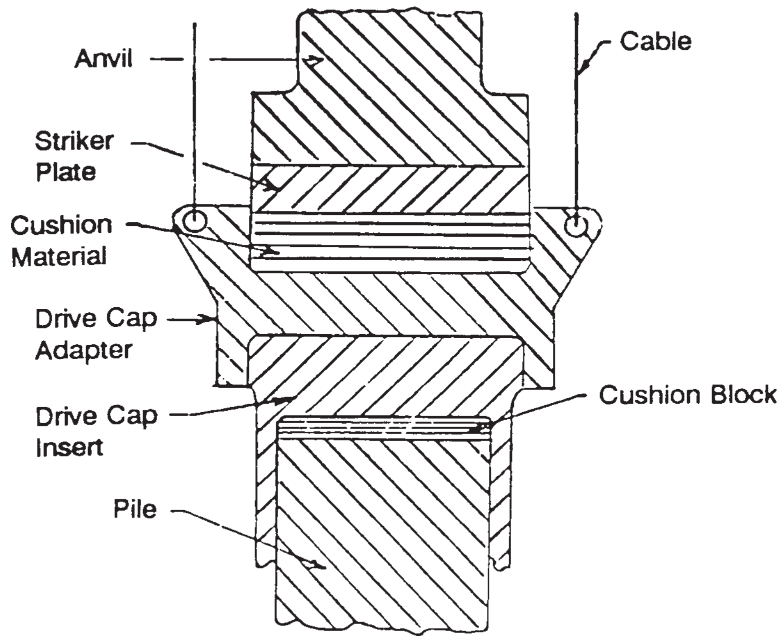



**Figure 7-3: Drive Cap System**

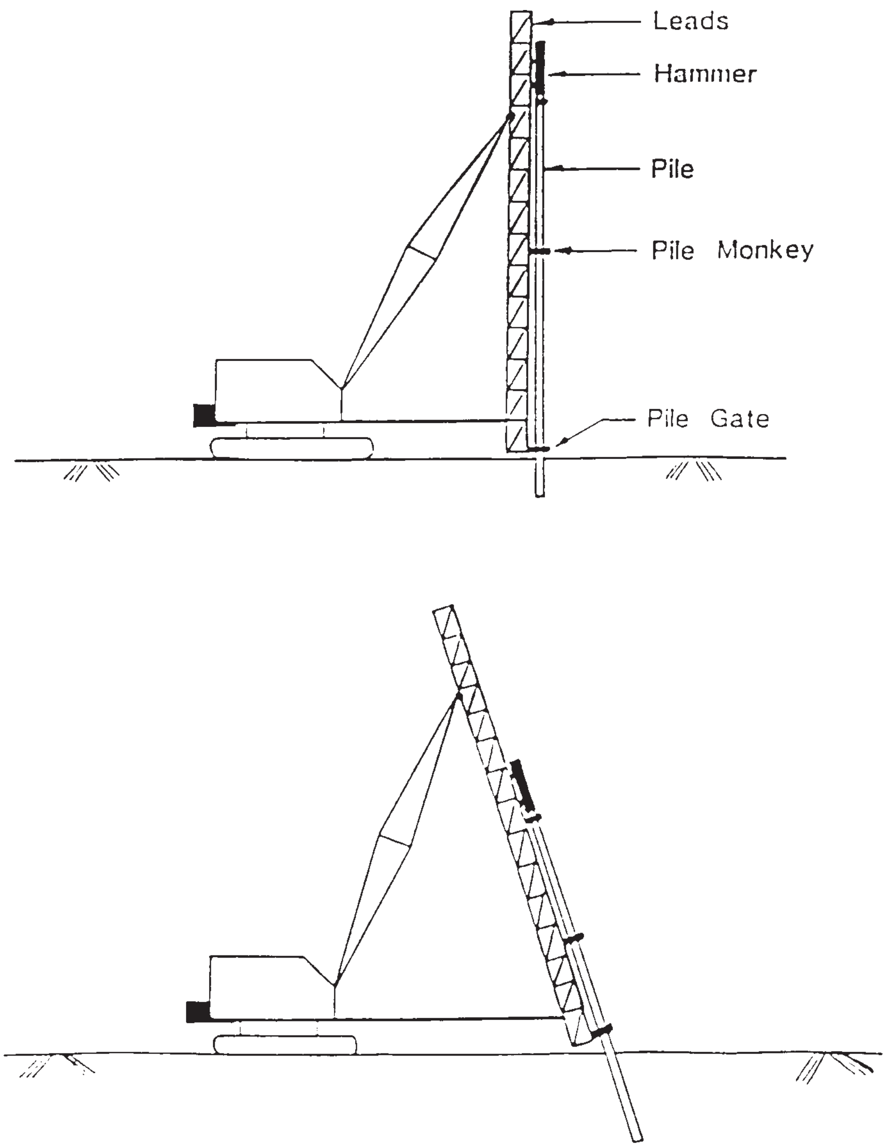



**Figure 7-4: Fixed Lead System**

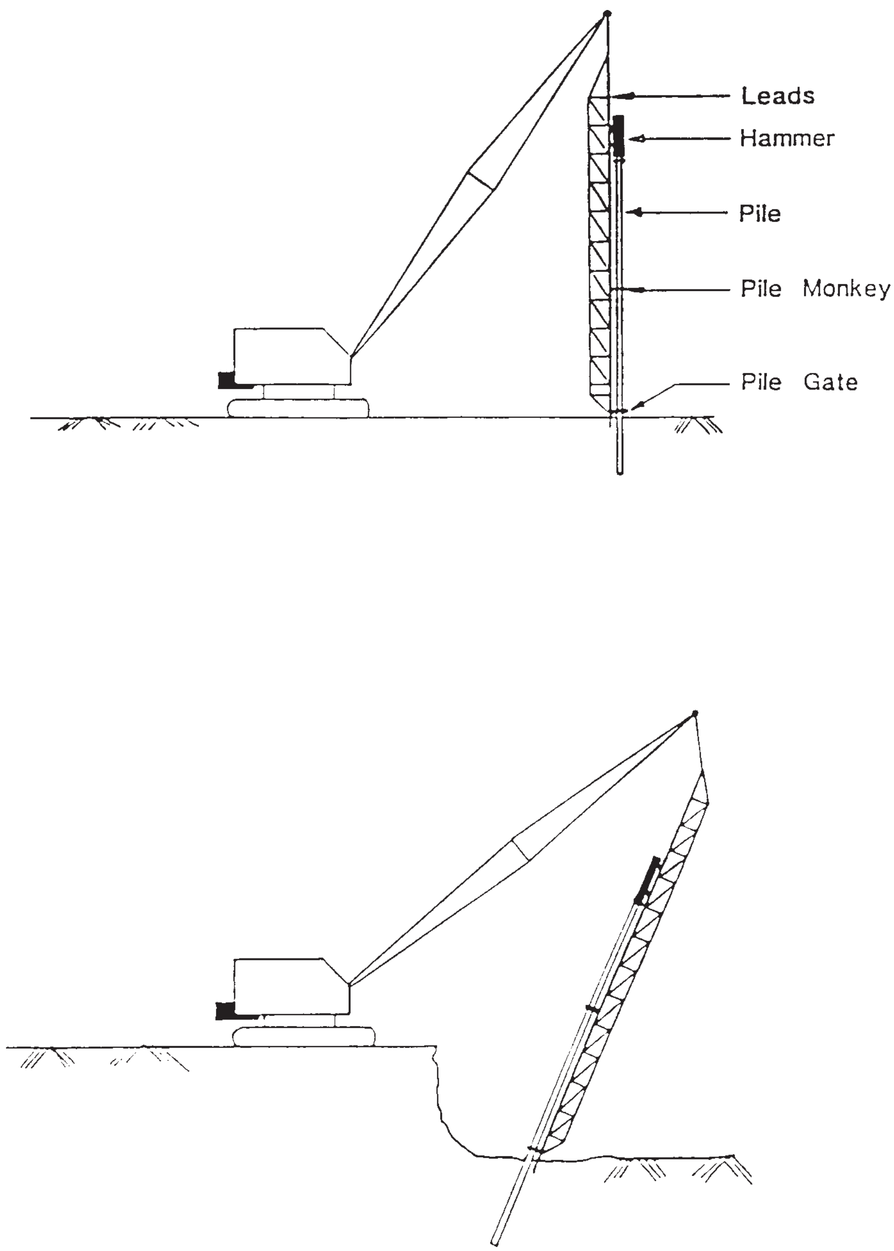



**Figure 7-5: Swinging Lead System**

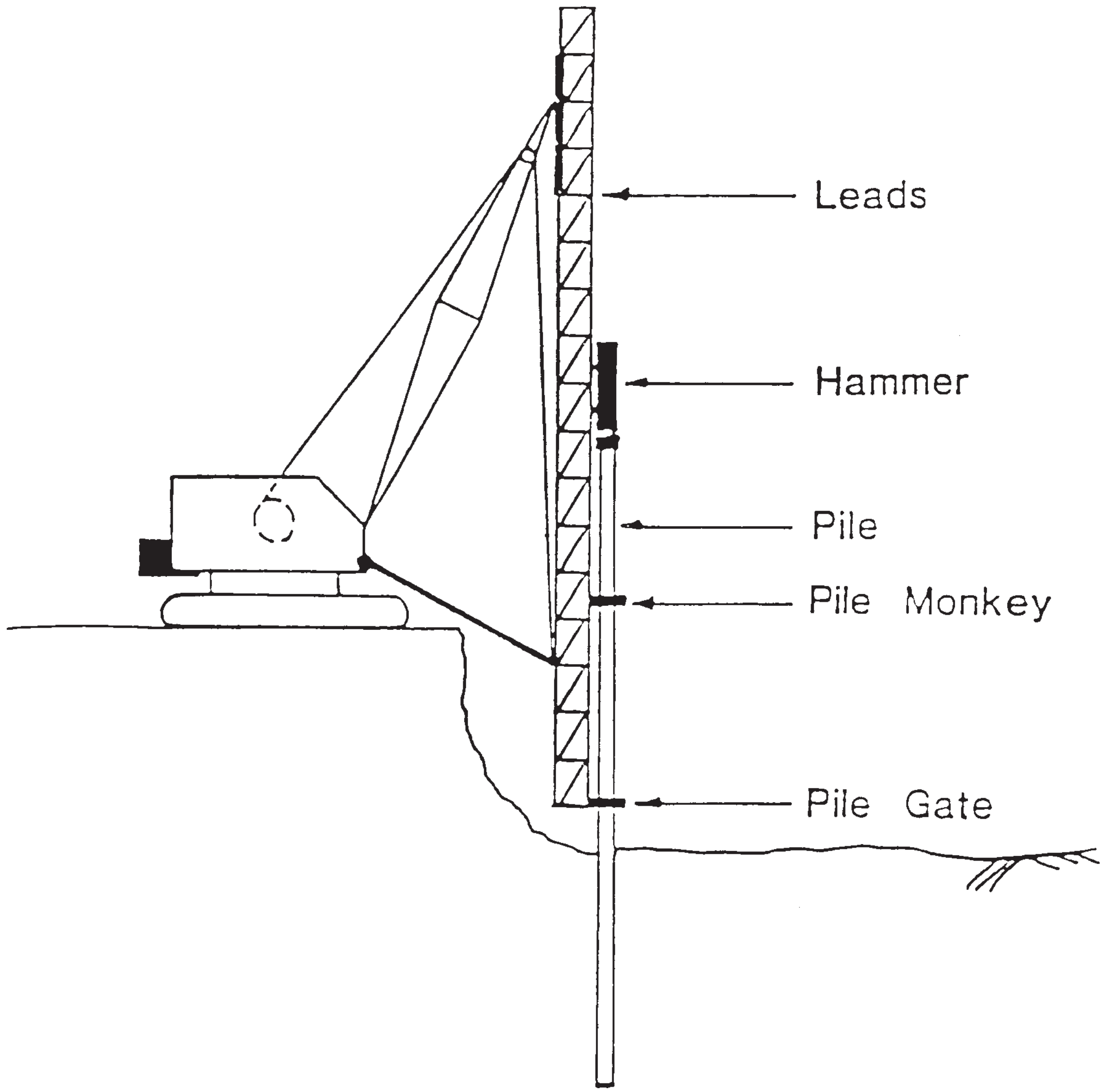



**Figure 7-6: Semi-Fixed Lead System**

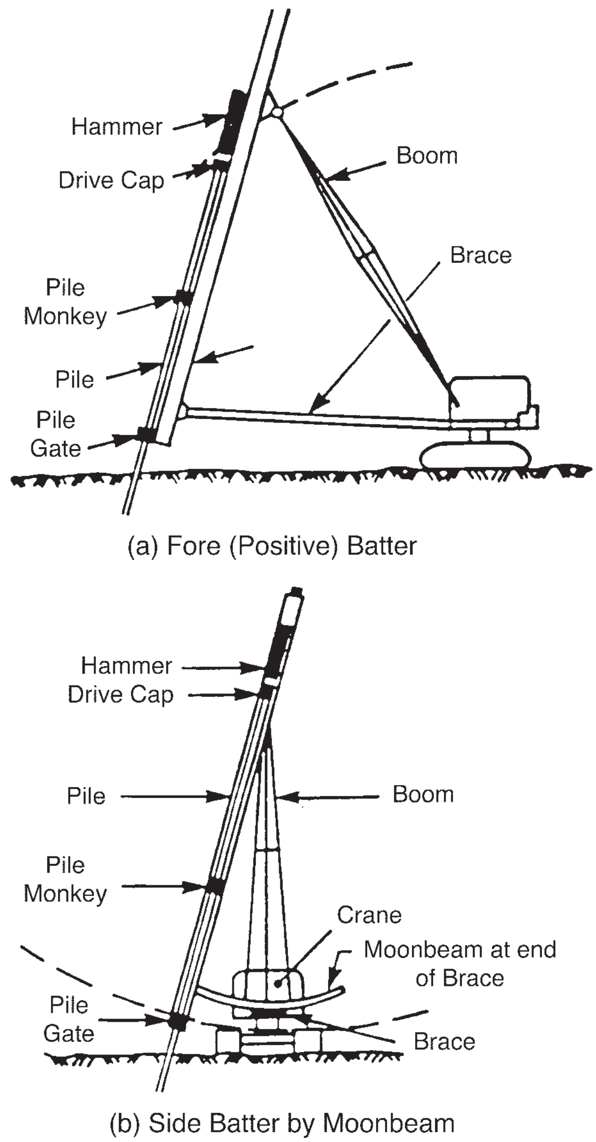



**Figure 7-7: Lead Configurations for Battered Piles**

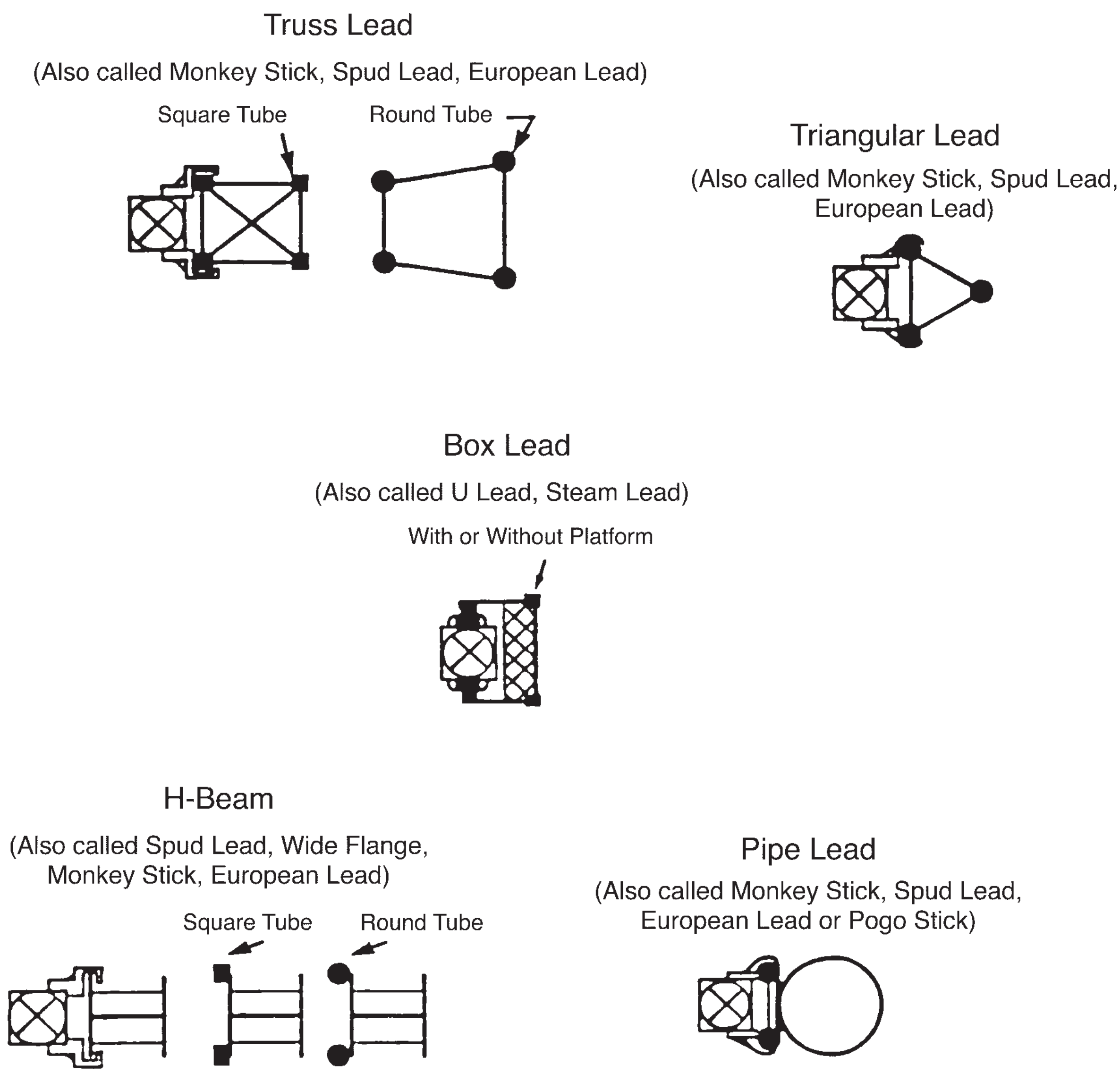



**Figure 7-8: Lead Types**

# Hammer Types

Many different hammer types are used in the pile industry today. In the very recent past one could expect to see, predominantly, the single acting diesel hammer in use on most jobs. 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. Specific job requirements, 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. 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 drop, striking a pile cap block. The available potential energy is calculated by multiplying the known weight of the hammer times its height of fall.

One variation of the drop hammer currently finding its way to the job site is one which requires only a minimal amount of head room. The idea is one which utilizes a pipe pile with a large enough diameter to allow the pile hammer to move up and down 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.).

This type of hammer is limited to use 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*.

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. |

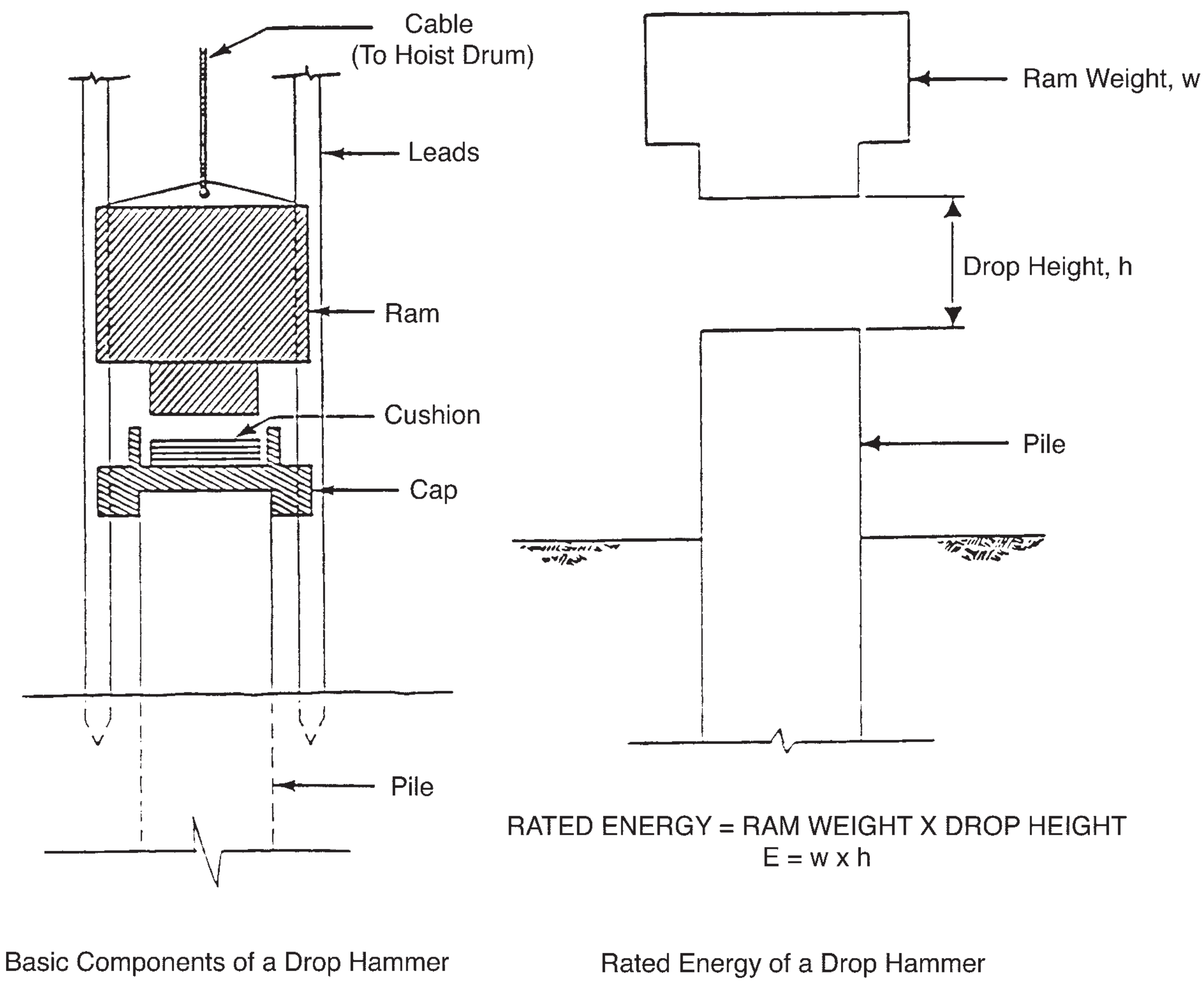



**Figure 7-9: Drop Hammer**

## 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 allowed to fall 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 stroke.

These hammers have a stroke of between 30 and 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.

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. |

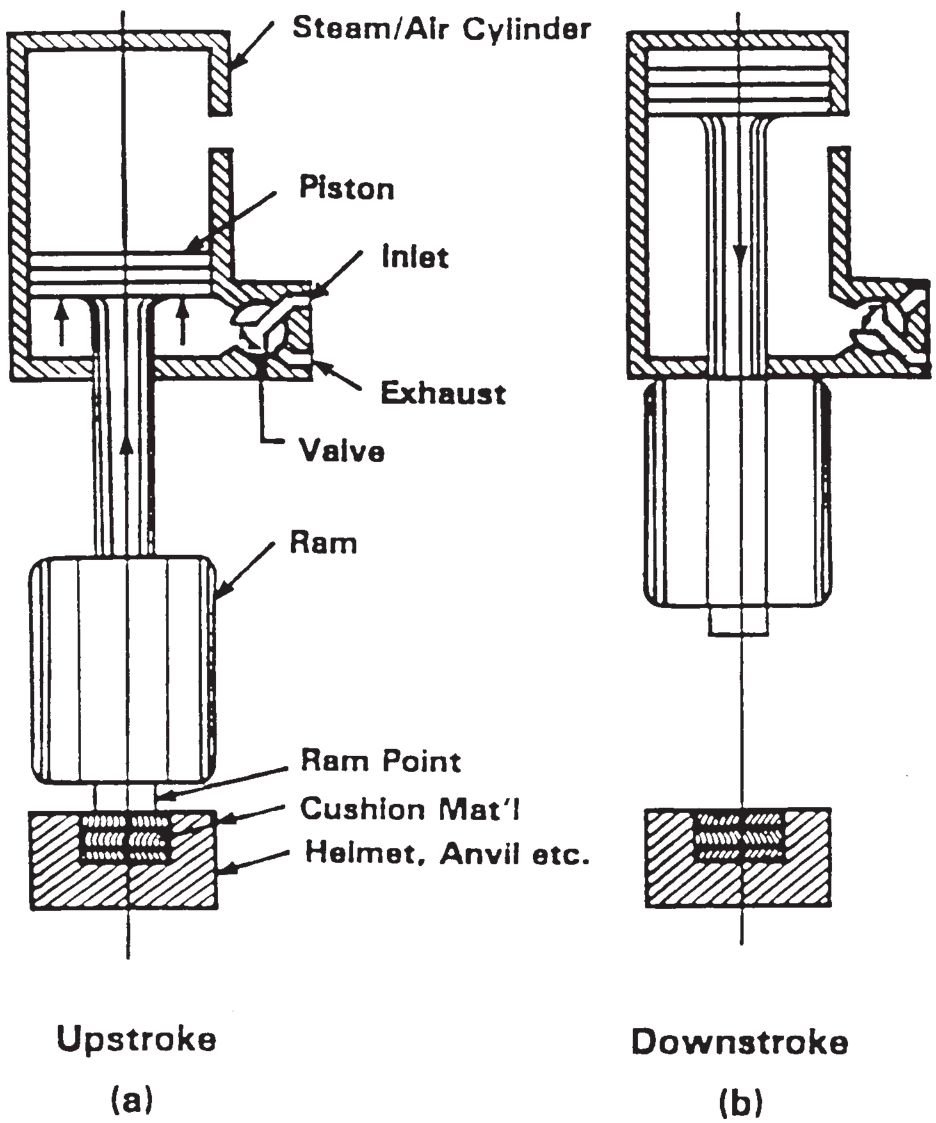



**Figure 7-10: Single Acting Steam/Air Hammer**

## Double Acting Steam/Air Hammers

The single acting steam/air hammer employs steam or compressed air to lift the ram. Once at the top of its stroke, the hammer free falls. With the double acting steam/air hammer, the hammer employs steam or air to not only lift the piston to the top of its stroke, but also to accelerate the piston downward. The energy put into the downward stroke by the compressed air or steam is in addition to the gravitational force utilized by the single acting hammer. Refer to Figure 7-11.

Thus the need for the longer stroke is eliminated with the double acting hammer. It is the reduction of the required stroke that makes these hammers useful when operating with limited overhead clearances. The stroke typically ranges from 10 to 20 inches. The blow rate is more rapid than the single acting hammer, somewhere between 120 and 240 blows per minute.

Some double acting steam/air hammers are entirely enclosed and can be operated submerged in water.

The rated available energy of the double acting steam/air hammer is calculated by multiplying the ram weight times the length of stroke and the effective pressure of the fluid acting down upon the piston head. With this type hammer it is essential that the hammer is operating at the manufacturer's specifications. This requires that the pressure used to drive the hammer be known and that a table be available depicting rated impact energy to the operating speed of the hammer. This type of hammer does not use a cushion block between the ram and the anvil block.

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. |

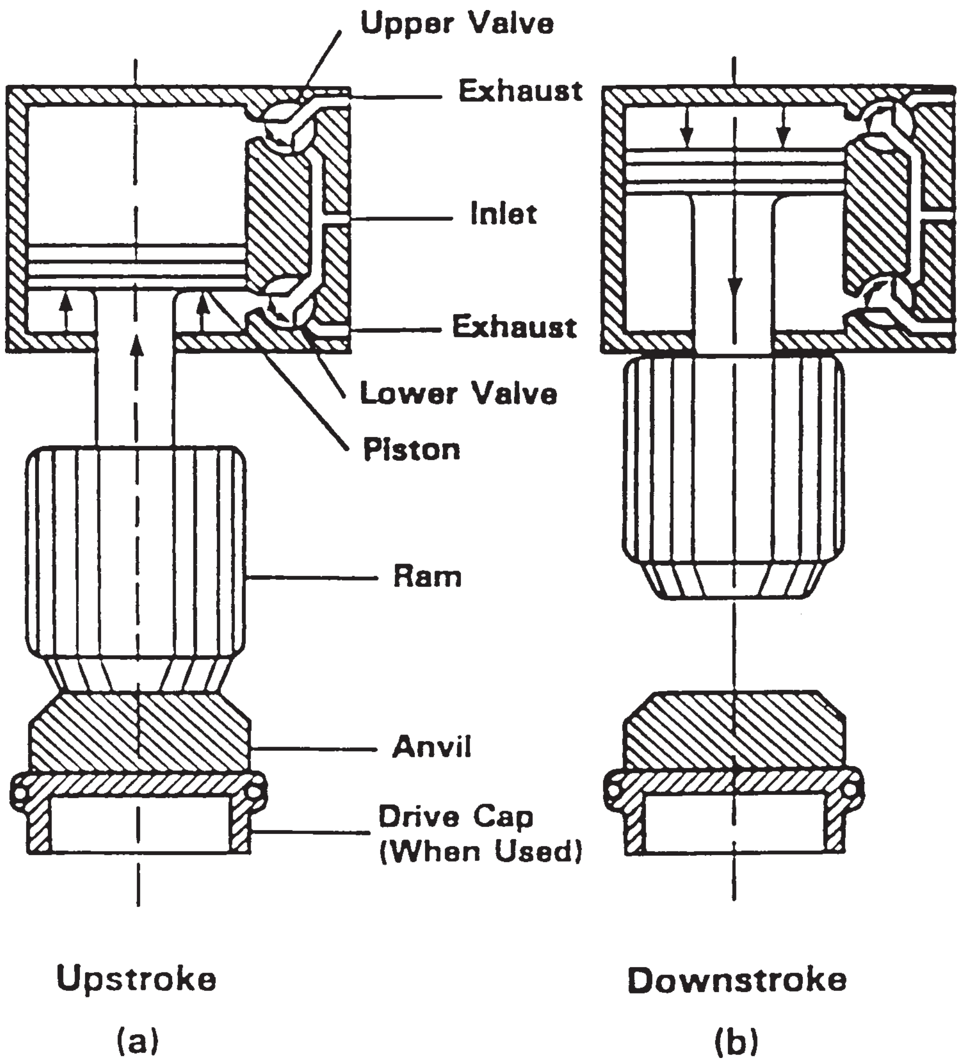



Figure 7-11: Double Acting Steam/Air Hammer

### Differential Acting Steam/Air Hammer

The differential acting steam/air hammer is similar to a double acting hammer. Compressed air or steam (motive fluid) is introduced between large and small piston heads to lift the ram to the top of its stroke. Motive fluid is then introduced over the large piston head to accelerate the ram in its down stroke. Refer to Figure 7-12.

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.

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. |

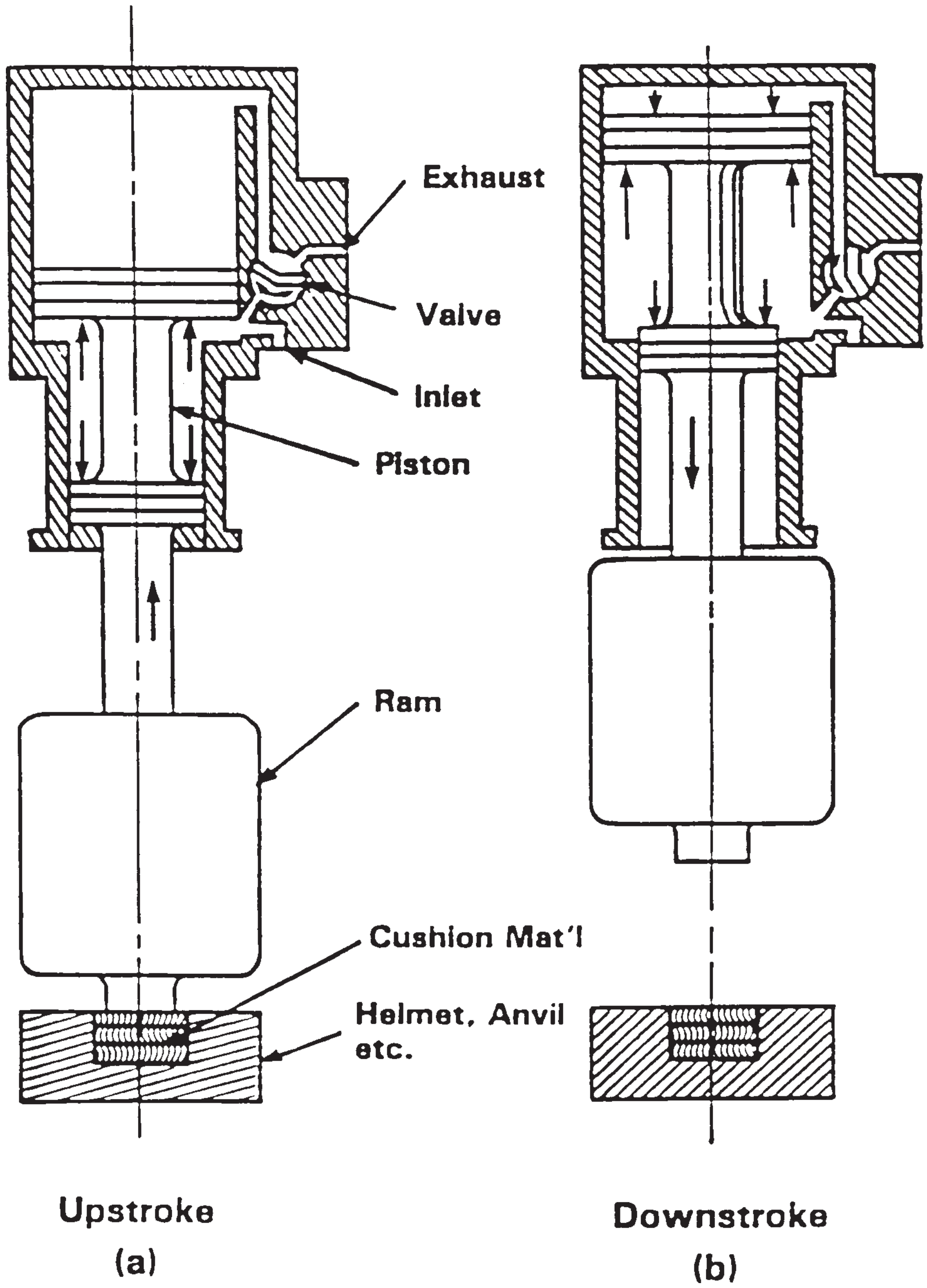



**Figure 7-12: Differential Acting Steam/Air Hammer**

## 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 and 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 which regulates the same amount of fuel to each cycle. New models added a variable fuel metering system which 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 which 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 continue 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. As most of the energy is absorbed by movement of the pile downward, little remains to lift the ram high enough to create sufficient compression in the next downstroke to ignite the fuel. To resume operation, the ram must again be raised by the cable hoist.

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 which acts downward. In production pile driving, the stroke is a function of the driving resistance, the pile rebound, and the combustion chamber pressure. The combustion chamber pressure, in turn, will be 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.

Diesel hammers are very versatile. They may be connected to almost any leads. Since they do not require an additional energy source, such as steam or air, the size of the pile crew can be reduced. On occasion, piles are driven with as few as three workers, including the crane operator.

These hammers typically operate within a speed of 40 to 60 blows per minute and 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.

Diesel hammers are noisy and they tend to spew oil and grease throughout. They also emit unsightly exhaust.

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 | Be aware of the actual stroke of the hammer during driving and that it will vary depending on soil resistance. |

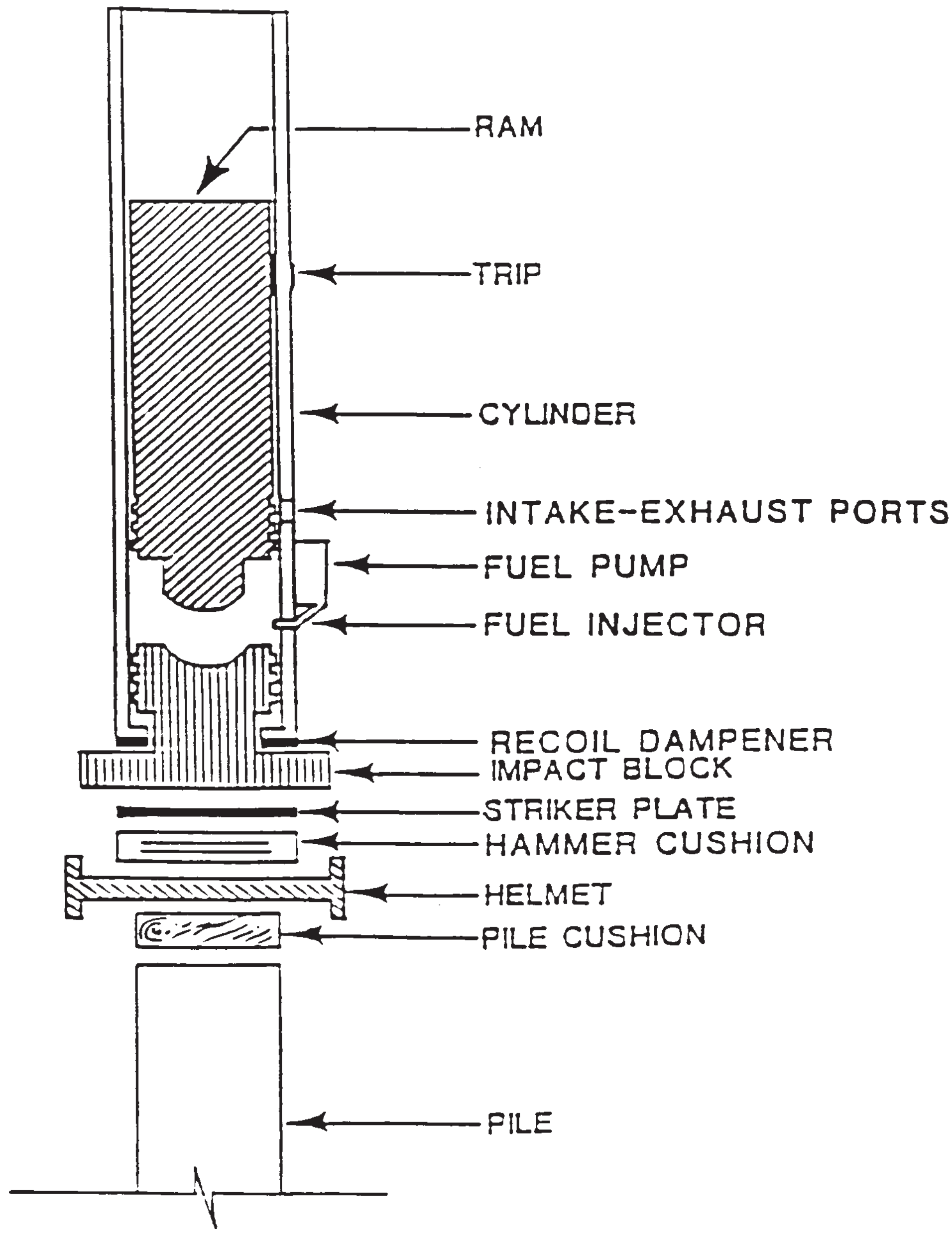



**Figure 7-13: Single Acting Diesel Hammer**

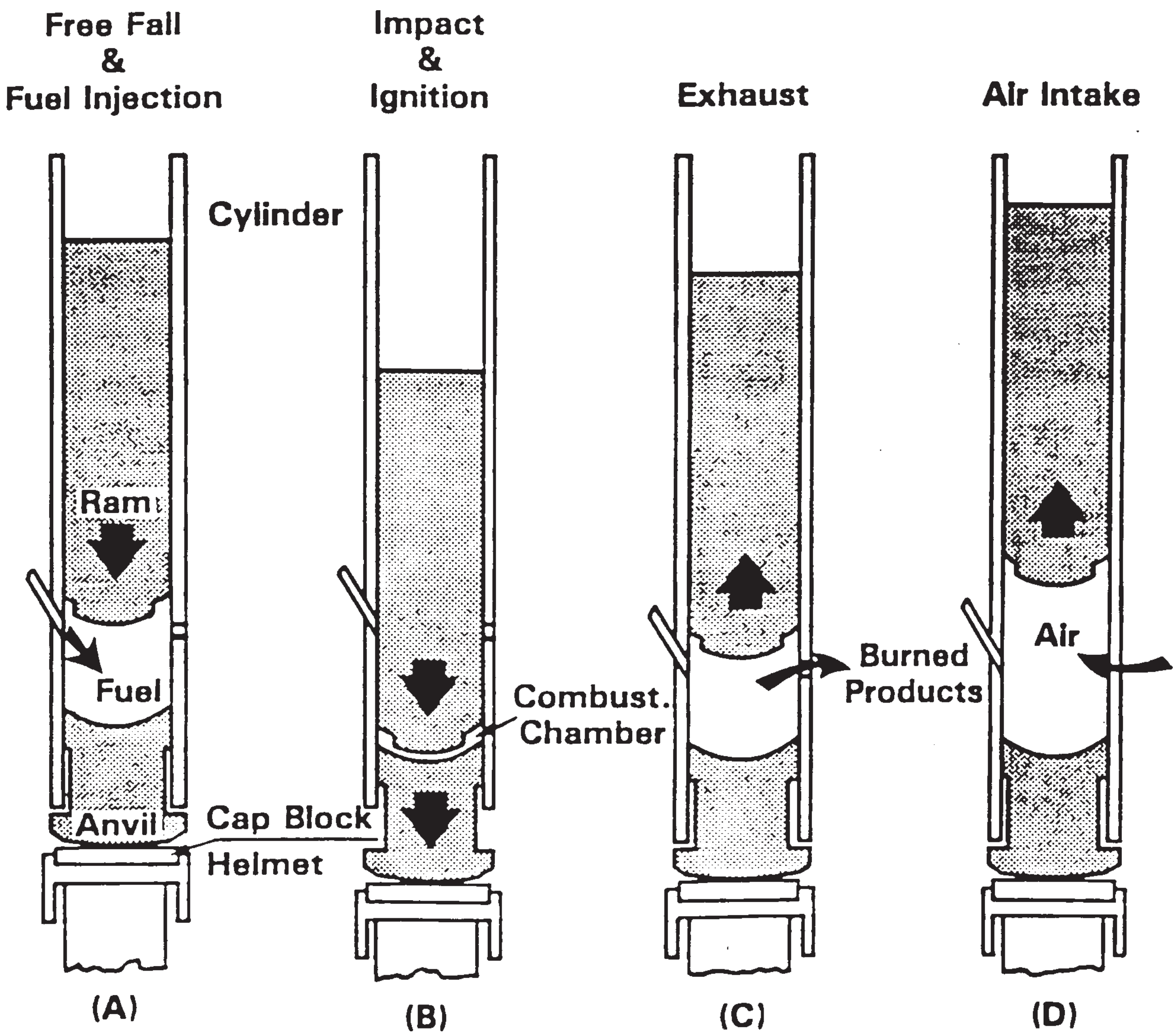



Figure 7-14: Operational Cycle for Single Acting Diesel Hammer

**The Double Acting Diesel Hammer.** The double acting diesel hammer is similar in its operations to other double acting hammers. The downward stroke is not just a function of gravity. The top of the cylinder is capped. As the ram nears the top of its upward stroke, the air is compressed in a "bounce chamber", which halts the upward flight of the ram. The downstroke energy is a function of both gravity and the internal pressure generated in the bounce chamber. The stroke of these hammers is around 3 to 4 feet and they operate at a much higher speed as 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 of unknown quantity.

The rated energy of a closed end diesel hammer is computed from a formula incorporating the length of the free fall downstroke of the ram times the sum of its weight and 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.

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. |

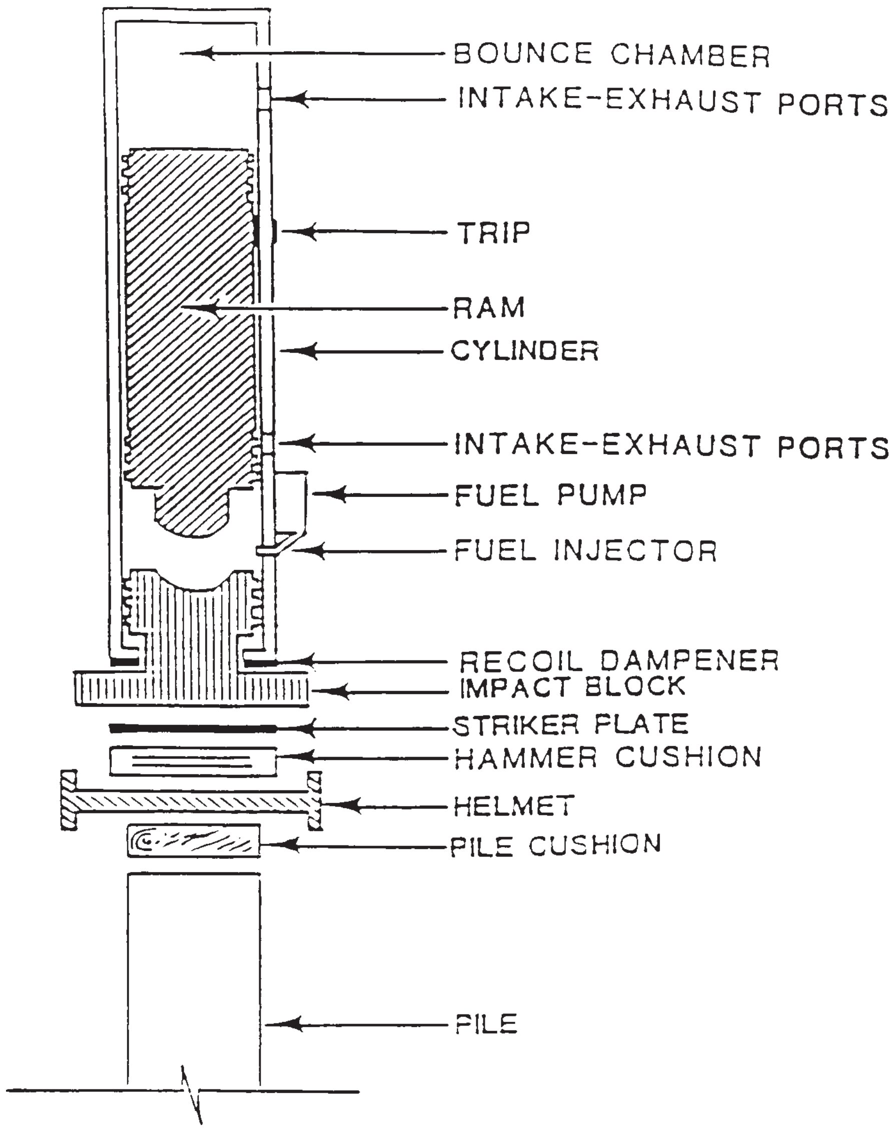



**Figure 7-15: Double Acting Diesel Hammer**

## Vibratory Driver/Extractor

Because there is no way to determine the amount of energy delivered to the pile, Section 49-1.05 of the Standard Specifications prohibits the use of the vibratory hammer for driving permanent contract piles. However, it is frequently used by contractors for items of a temporary nature (i.e. placing and extracting sheet piles, etc.). They are also typically used as a means of extracting piles.

Vibratory pile drivers/extractors could be likened to ministroke, 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.

Heavier piles mean higher vibratory weight which tends to reduce amplitude. So as piles get larger, it is necessary to use drivers with larger eccentric moments. The nonvibratory 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 ground vibration as normal pile driving, thereby making the vibratory hammer desirable when possible damage to adjacent structures could occur.

The vibratory hammer has been permitted to drive a bearing pile to a point which would be a specified distance from expected final penetration. An accepted impact hammer has then been placed upon the pile to take it to acceptable bearing and final penetration in the normal fashion. Situations where this is of some use 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. 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 as might be stated in the Special Provisions and the Standard Specifications. |
| 2 | Discuss the proposal with the Bridge Construction Engineer, the Project Designer, and the Engineering Geologist. |

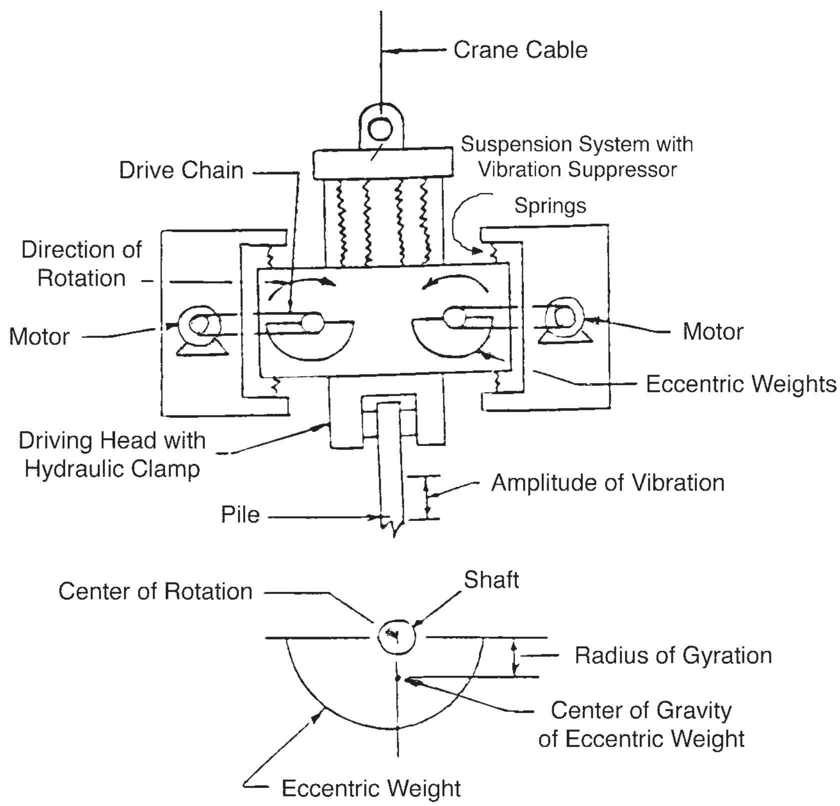



**Figure 7-16: Vibratory Driver/Extractor**

## Hydraulic Hammers

A hydraulic hammer is one that incorporates the use of an external energy source to lift the hammer to the top of its stroke. For the single acting hydraulic hammer, the actual energy induced into the pile is developed by the free-falling piston, much the same power stroke as a drop hammer or a single acting diesel hammer. The rated energy for the differential acting hydraulic hammer is found by similar means as other differential acting hammers.

Some will say that a differential acting hydraulic hammer is no different than any other differential hammer.

One particular hydraulic hammer manufacturer utilizes a ram made of composite material. In this case the ram is made of lead wrapped in steel. The stroke is relatively short for a single acting hammer, generally about 4 feet. The theory behind this particular hammer is that the lead ram produces a blow with a longer impact duration. This longer impact duration produces a compression wave which 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. With the control box the stroke can be varied, finitely (reported to be in the centimeter range), such that the stroke can be optimized to the point of matching the dynamic spring constant of the hammer and pile. Manufacturers have stated that the ability to vary the stroke and frequency is what makes the difference.

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. The dynamic spring constant is also known as the acoustic impedance. It is believed that, the closer the dynamic spring constant is between the pile and the hammer, the higher the energy transmission will be through the pile and the lower the internal stress will be in the pile, since all the hammer energy goes into penetration of the pile. If the hammer impedance is the same as the pile impedance, a pile cushion would be unnecessary and driving would be optimized.

The manufacturers of these types of hammers claim the following:

| NO. | ITEM DESCRIPTION |
|---|---|
| 1 | Hammer efficiencies in the range of 80% to 98%, while claiming diesel hammers to have an 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. |

A hydraulic hammer has been tested on a Caltrans project. The hammer was tested in conjunction with a single acting diesel hammer. Piles were driven with both hammers. Bearing of the hydraulic driven piles were verified by means of a retap, using the diesel hammer.

The results indicated that the diesel hammer required twice as much energy to move the pile one foot when compared with the energy of the hydraulic hammer.

Currently the hydraulic hammer is not considered an approved impact hammer for determining the bearing value of a driven pile. The problem with the use of such hammers is the verification of energy induced. The question remaining is to the relationship of the hydraulic hammer versus the Engineering News-Record (ENR) formula.

Although we do not allow the hydraulic hammer to be used with the ENR formula, the Wave Equation Analysis has been used on this type of hammer successfully.

## 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.

Economics often dictate hammer size selection by contractors. Large hammers with increased energy will reduce driving time. They will also help achieve penetration where hard driving is encountered, thus enabling completion of the work without the need of supplemental measures (jetting or predrilling). On the other hand, heavy hammers require heavy leads and heavy cranes; the result being decreased mobility and increased equipment costs. Ideally the Contractor will come up with a "mid-range" selection. Another consideration is that a larger hammer will deliver more usable energy to the pile. Hence, the probability of pile damage (heavy spalling or other) is increased as hammer size is increased. 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. At constant driving energy, the driving stress on the pile will decrease as the ram weight increases.

## Bearing Capacity

In lieu of a static load test, the typical method for determining load bearing capacity of a pile depends on knowledge of the energy used to drive the pile. This was stated earlier in the chapter by saying, "not only is the pile hammer the production tool for the Contractor, it is also a measuring device for the Engineer". Various methods and procedures are available when using the known driving energy to determine the bearing capacity of the pile. These procedures can be categorized into three areas: (1) Pile driving formulas, (2) wave equation analysis of pile driving, and (3) dynamic pile driving analysis.

Pile driving formulas all utilize the energy delivered per blow, the resistance to the movement of the pile per blow, pile penetration, and some acknowledgement of the unknown produced by all the components which act to drive 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 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.

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.

Section 49-1.08 of the *Standard Specifications* requires that the bearing value of driven piles be determined using the ENR formula.

This formula was developed prior to the year 1895 by Arthur Mellon Wellington, the general manager and part owner of Engineering News-Record. Wellington developed it for timber piles driven with a drop hammer. The formula was modified at a later date with the advent of steel and concrete piles and more sophisticated hammers. Both the original and the updated versions are given in the *Standard Specifications*. The ENR formula is the most commonly used in this country.

Section 49-1.08 of the *Standard Specifications* gives two different variations of the ENR formula. One of these is used for drop hammers and the other is used for single or double acting hammers or diesel hammers (Refer to Appendix E for a derivation). Of these two, the most commonly used is the diesel hammer formula and is as follows:

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

where:

$P$ = safe load in pounds
$E$ = manufacturer's rating for foot-pounds energy developed by the hammer
$s$ = penetration per blow in inches

Inspection of the diesel hammer formula shows that it is an energy conservation equation with an additional 0.1 inch term and a reduction factor of 6. The intention of the 0.1 inch term is to account for elastic rebound and to encompass energy losses in the piling system. The reduction factor of 6 relates the result of the ENR formula to a safe (working stress) load. Ideally, this would mean a safety factor of 6. Comparisons of safe pile loads predicted with the ENR formula and measured capacities from statically load-tested piles have shown the factor of safety to vary greatly. Actual factors of safety have been shown to range between 0.8 and 30 when this formula is used.

Why do we use the ENR formula in the field?

1. Simplicity – Relatively short calculation needed for determination.
2. Simplicity – Certain factors in other formulas are subjective in nature and could be difficult to determine.
3. Simplicity – Subjective evaluations required by other formulas could precipitate contractual problems.
4. Dependability – Over the years, our experience with the ENR formula has been good.

It must be remembered that driving formulas are simply tools used to aid the evaluation of a pile's capacity. Experience and engineering judgment are of immense value in this work.

Field measurements of bearing capacity on large displacement piles, in excess of 100 Tons, have shown that the ENR formula is not necessarily reliable. It is believed that a significant amount of the driving energy is being used just to get the large pile mass moving.

Although the Standard Specifications do require the use of the ENR equation, research is showing that the use of the wave equation analysis provides a more reliable correlation to static pile load test results than does the ENR formula. The enhanced accuracy is achieved by providing a more detailed accounting of energy losses.

## Dynamic Analysis by Wave Equation

Wave equation analysis of a pile driving operation is a one dimensional finite difference method which models the transmission of a hammer's impact wave down a pile and into the soil. The method has been around for more than two decades. Several versions of the program are available, some of which can be run on a personal computer. One of the most widely known versions is called Wave Equation Analysis of Piles (WEAP).

What a wave equation analysis does is model the pile and the driving system as a series of masses and springs. The relative sizes of the springs and masses depends on the actual pile and driving system characteristics. The soil is represented by a series of elastic plastic springs and linear dashpots. After modeling the pile, the driving system and the soil, the ram elements are given a velocity equal to the hammer impact velocity and a dynamic analysis is performed.

The wave type analysis is relatively new to Caltrans. To date it has varying uses. Among them are: (1) Driveability Study, (2) Hammer Acceptance Study, and (3) Acceptance Curves.

**Driveability Study.** In the Driveability Study, the wave equation analysis would be used in the design phase. For this study, the user (as of now this would be someone from the Office of Structural Foundations) will input a driving system model. The input information consists of a typical hammer, cushion, soil characteristics, and length of pile; all of which is generally referred to as the pile driving system.

The output of the analysis will tell the user if the pile is driveable. The output information will include the internal stresses of the pile as it travels through the varying strata and as it approaches a particular tip elevation. The output will also give driving rates for specific hammers at specific elevations. The hammer input information is within a range for a "typical hammer" that might be used. The soil information is based on information gained from the Log of Test Borings for the particular site. All of the above input information is refined for further uses in the wave equation. As a summary for this use, the Driveability Study is one which is used by the Project Designer to help in choosing a foundation type.

**Hammer Acceptance Study.** The Hammer Acceptance Study would typically be done after the contract is bid. This type of study requires information regarding all of the components that a Contractor would propose to use for the installation of contract piles. This information would be submitted to the user. The user inputs all of the submitted information and, from the output of the program, determines if the hammer can drive the pile, what the driving rate would be, and the user will be able to predict the internal stresses of the driven pile. From this information, the Engineer can decide if the Contractor's hammer will drive the pile to tip without overstressing the pile. From the output information, the Contractor might discover that the chosen hammer would not be efficient to use in a given situation. The Contractor might choose to use a different hammer that might be more efficient. The new hammer's characteristics would be re-input and the output would be studied for the previous stated reasons.

**Acceptance Curve Study.** For the Acceptance Curve study, the previous two studies are completed. After the Contractor's hammer is known, modeled and accepted, a pile would be driven in the field. The driving of this pile would be monitored using Pile Dynamic Analysis (PDA) equipment, which records a dynamic analysis using monitors attached to the driven pile. After the pile is driven a static load test might be performed. Using information gained from the PDA, the static load test and information from other analysis programs, the input information for the WAVE analysis is refined to more closely model the hammer and soil system characteristics. From the refined input, the WAVE output can be used to develop acceptance curves, based on stroke, blow count and driving rate relative to a specific footing location. The output would be used by the Engineer in the field to determine acceptance of a driven pile. Refer to Appendix E for samples of acceptance curves.

The previous method, as outlined, is quite comprehensive. This type of analysis would normally be done only on a large project. Where this type of analysis has proven useful is as follows. Piles were driven in Bay Mud type material. Take-up curves were developed using the static load tests on test piles over a given period of time. These take-up curves measured the gain in soil resistance realized over time. All of the information was put into a WAVE analysis. The output predicted what the blow rate and stroke should be at the time of driving for a particular driving system. What happened in the field is that the Engineer accepted a pile driven to specific blow rates and stroke length. The accepted blow rate and stroke were determined using the knowledge of the increased bearing value, over time, characteristics of the piles. If the piles had been driven using only the ENR formula, the Contractor might still be driving them. The WAVE program allowed the user to utilize the take-up information to model the soil characteristics more accurately. In this case it save the owner (Caltrans) lots of money.

The advantage of wave equation analysis over pile driving formulas is that it accounts for the physical characteristics of the pile driving system and soil resistance. A WEAP analysis has its greatest advantage over driving formulas where the pile is driven through layers of soft soil and a significant portion of the pile capacity is derived from skin friction. Where a pile is driven into firm strata or bedrock, the pile often derives most of its static capacity from end bearing. For predominantly end bearing piles, WEAP analyses for the purpose of generating penetration/capacity relationships may not be necessary. Nevertheless, a WEAP analysis has the capability of being able to predict potential pile overstressing during hard driving.

Where a pile is advanced through material which can not be relied on for contributing skin resistance, WEAP analysis provides a means of establishing pile driving criteria consistent with design procedures. Situations where this is a concern include river channels where scourable material is present, and sites having down-drag or negative skin friction forces generated by soil consolidation. Because the ENR formula predicts safe rather than ultimate capacity, and has an unknown safety factor, it does not provide a logical means of considering the driving resistance contributed by these layers. WEAP is based on ultimate pile capacity and thereby provides this capability.

## Manufacturer's Energy Ratings

Generally each manufacturer publishes a catalog or brochure for their hammer. Within the printed information they typically will outline operating specifications, including any specific equipment which will be required. Specifications of importance to those involved in monitoring driven pile installation are ram weight, stroke, blows per minute, energy rates and required steam or air pressure.

Accurate determination of a given hammer's available energy is difficult. Manufacturers calculate hammer energy differently. McKiernan-Terry uses ram weight times stroke. At one time, Delmag calculated a hammer's energy as a function of the amount of fuel injected. They now use the weight of the hammer times the stroke. Link-Belt considers stroke, fuel, and the effect of the bounce chamber. Keep in mind that a hammer's rated maximum energy is typically rated when the pile hammer is operating at or near refusal.

The Standard Specifications state that "E" is the manufacturer's rating in foot-pounds of energy developed by the hammer. It is used in the ENR formula. When single acting diesel hammers are used, it is necessary to determine energy by multiplying ram weight times the actual stroke. This requires field observation of the actual stroke. This approach is both simple, conservative and almost universally accepted. The manufacturer's given energy rating should not be used "blindly" in the ENR equation. Energy output should be verified by measuring the stroke of single acting diesel hammers or by comparing the operations of the hammer with the manufacturer's operating specifications.

One sometimes hears discussion about the "efficiency of a hammer". The efficiency is merely the net kinetic energy normally delivered by a particular hammer to the pile cap block divided by the potential energy of the hammer at the beginning of its stroke; or, in other words, the percentage of the total potential hammer energy available at the top of the stroke which is actually delivered by the ram of the hammer at impact. As has been mentioned above, engineers and inspectors use the rated energy of hammers as an indication of the driving capability of the hammer. Manufacturers do not specify the amount of kinetic energy delivered by a hammer at the head of the pile after undergoing energy losses. These losses occur in transfer of energy through the driving system. Because all pile load bearing formula are applied with safety factors, these energy losses, common to all hammers, are usually accounted for in the load bearing formulas or are ignored. What you, as the Engineer, must then ensure is 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).

There are many factors that contribute to the loss of efficiency, such as wear, improper adjustment of valve gear, poor lubrication, unusually long hoses, minor hose leaks, binding in guides, and minor drops in steam or air pressure. All of the preceding illustrates the necessity for the Engineer to have a working knowledge of hammer operations.

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 well advised to look into the mechanical aspects of the pile operation when the Contractor starts assembling the equipment and driving begins.

Even though it is usually very small, the Engineer should also be aware of the energy adjustment needed when battered piles are driven. Since the path of the ram will follow the slope of the pile, the stroke used to compute delivered energy must be adjusted to reflect the vertical fall of the ram. This is simple to determine for single acting air, steam or diesel hammers. For example, a 70 Ton pile driven with a Delmag 30 hammer will normally require 14 blows per foot. If the pile is driven on a 1:3 batter the minimum blow count would be increased to 15 blows per foot ((3.162/3) ¥ 14). Refer to Appendix E for an example of this.

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

## Preparing to Drive Piles

Pile driving techniques (including solutions to problems) are normally developed with time and experience. It is the intent here to provide some insight into the areas where problems do exist and where they can develop, so that as many as possible can be eliminated or resolved before they occur.

The following material is essentially a check list of what the Engineer should look for before driving begins and while driving is underway. This list is by no means complete as new and different problems will develop with each and every project.

Prior to going out in the field:

| NO. | ITEM DESCRIPTION |
|---|---|
| 1 | Review the Plans, Special Provisions and Standard Specifications 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 for welder certification requirements. |
| 4 | Prepare the pile layout sheet. |
| 5 | Prepare the pile log forms. |
| 6 | Advance preparation of a chart, table or graph which 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 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 lay out 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 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

Some method of field verification of driving energy must be available to the Engineer. 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 some manufacturers disagree with this method, this remains the best method available in the field. For diesel hammers, measure the depth of ram below the top of the cylinder before driving and add the height of ram visible during driving. This may require the painting of one foot marks on the trip carriage.

Some hammers have rams with identifiable rings which 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 130-2.0.

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 should also be available to check required steam and air pressures.

Check for proper hose size of steam and air hammers. The hoses should comply with the manufacturer's specifications. All hoses should be in good condition (no leaks).

## Materials Checklist

### Pipe Piles

| 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. If piles are to be spliced, the welder must be prequalified. Refer to Section 49-5.02 of the *Standard Specifications*. |

### 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 for 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.

$$\text{Allowable Stress} = 5\sqrt{f_c'} \text{ PSI tension}$$

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

With concrete piles, make sure the cushion blocks are maintained in good condition. If the driving is hard, they may have to be changed once or twice per pile.

## Steel Piles

If the piles are to be spliced, the Contractor must have a prequalified welder do the work. If the welder has been performing similar type work on other Caltrans jobs and 12 months have not elapsed since this work, the welder does not have to run a prequalification test. Verification of the welder's work record can be obtained by calling the Office of Materials Engineering and Testing Services (METS) in Sacramento with the welder's name and social security number.

Some welders who are not listed with METS 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.

If the welder does not meet either of the above requirements, a prequalification test will have to be made. A review of the ANSI/AASHTO/AWS D1.1 Welding Code will outline the requirements.

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. It should also be noted that once the welding work has been completed on the job by the certified welder, the Engineer should forward a copy of the work record to the Office of Materials Engineering and Testing Services. This will help to maintain current records and hopefully save someone some time later on. Also keep in mind that just because a person holds a welding certification, it does not mean you do not have to look at the welding work.

Early contact with Transportation Laboratory representatives in either Los Angeles, Berkeley, 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.

## 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 1¼ 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 retap to prove bearing. If all the piles drove in a similar manner, it might be possible to retap as few as 10% of the piles that did not originally achieve bearing. If the piles all drove differently, a retap of all of the piles may be required. The following is a discussion of factors affecting pile log data.

Typically when 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 ENR 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. When logging piles or determining final blow count, pick a fixed reference point as close to the pile as practical. This can be accomplished several ways: (1) Mark the lower part of the leads with one foot marks and observe passage of a fixed point of the pile, or (2) 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). 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 which are damaged should be extracted. However, this is not always possible. Frequently this problem is solved by driving a ''replacement'' pile next to the rejected one. When this is done the effect of the change could impact the footing design.

When driving hollow pipe piles in water, be aware of the water level in the pile. Water hammer developed during driving could split the pile. This may require that the Contractor seat the pile and stop driving long enough to pump it dry before continuing the drive. Beware if water gets close to the pile top! Another problem 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 pile. There are many implications if this happens. Among the possibilities include the possible overstressing of a pile as well as misleading blow counts.

## Driving Problems

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 either 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.

## Hard Driving

Hard driving occurs when either the soil is too dense to accept the pile or the hammer cannot produce enough energy to drive the pile. A review of the Log of Test Borings may be an indication of the type of driving that can be expected. Current specifications provide measures to be used when these conditions exist.

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."

When it appears that driving through dense or rocky soil could damage the tips of driven piles, the Standard Specifications require the Contractor to provide special driving tips, heavier pile sections, or other measures as approved by the Engineer, to assist in driving a pile through a hard layer of material.

The Engineer should consult with the Engineering Geologist if hard driving is a problem and the Contractor is considering either jetting or predrilling. There may be limitations on the use of jetting. Predrilling and predrilling depths should also be discussed since there may be certain limitations.

Care must be exercised when jetting is 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.

Spudding is another method used to assist penetration. This is where an "H" pile or similar section is driven to break or cut through hard material. Contract piles should not be used for spudding unless the pile has been specifically designed for this type of action.

The term "hard driving" is subject to much individual interpretation and there are no set guidelines in the specifications, save certain requirements for timber piles. When the blow count for timber piles reaches specified limits, the Contractor is required to take prescribed measures of assistance. This could include predrilling, jetting or a change to a larger low velocity hammer.

In the case of steel or concrete piles, no measures are 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 which will not result in damage to the pile.

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 well advised to investigate pile stresses. In the book *Pile Foundations*, Chellis covers this subject fairly well and it is recommended as reference material.

The subject of pile refusal has to be included in discussions about hard driving. Unfortunately, there are nearly as many definitions for the term "refusal" as there are those who attempt to define it. 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. Some contractors would like us to think of refusal as something in the double the required blow count range. These contractors also feel that Caltrans labors under the concept of refusal as that point just prior to self destruction of the hammer.

Is any specific definition valid? Not really. Refusal should be viewed as very little or no penetration of the pile with a reasonable amount of delivered hammer energy. In determining a reasonable amount of hammer energy one should be satisfied that the hammer is (1) operating efficiently, (2) operating at maximum energy, and (3) sized properly.

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 of the relative weights of the hammer and the pile. Certain "light" hammers will meet our minimum penetration or energy requirements and, at the same time, they can be dwarfed by the size of the pile. This can result in a situation analogous to driving a large spike with a tack hammer.

Hard driving can also be the result of a pressure bulb developed near the pile tip. This can occur in saturated sandy materials. Driving in stages is a suggested remedy for this situation.

Soil consolidation due to cluster driving of displacement piles frequently causes hard driving problems. A revised driving sequence often will alleviate this problem. This can often be a trial and error process. Driving from one side of the footing in a uniform heading helps. Driving from the center in a uniform outward pattern also can be helpful.

There are many factors which could contribute to hard driving and there are many solutions.

Because of the many variables involved, each hard driving problem must be evaluated on its own merit. There is no substitute for engineering judgement in this area. It should also be remembered that these are not infrequent problems and there is a broad base of experience to draw from within the Office of Structure Construction.

Occasionally, pile penetration to the specified tip elevation may not always be accomplished, despite the Contractor's best efforts. When this situation occurs, the Engineer may consider accepting piles that are not driven to specified tip. This solution, while it may solve the construction problem, may present administrative problems that will require resolution. Situations have been experienced where the foundation design was changed, during construction, from a driven pile foundation to another type of footing. Prior to making a decision to accept piles that are not driven to specified tip, or initiating a change in footing design, discuss the problem in detail with the Bridge Construction Engineer and, depending on the results of the discussion, with the Engineering Geologist and the Project Designer.

## Soft Piles and Retap

In this situation, the pile has been driven to the specified tip elevation but the specified bearing value, as determined by the ENR formula, has not been obtained. The Standard Specifications require the Contractor to satisfy both requirements.

Following are some ground conditions that may produce soft driving:

| CONDITION | DESCRIPTION |
|---|---|
| 1 | Loose submerged fine uniform sand. Driving temporarily produces a quick condition. Retap will probably not indicate capacity. |
| 2 | Cohesive soil. Driving temporarily breaks down the soil structure, causing it to lose a part of it's compressive strength and shear value. Retap should indicate increased capacity. |
| 3 | Saturated coarse grained pervious material. May display high driving resistance, but on retap will lose capacity as compared to the initial driving. This could be due to changes in pore water pressure within the soil mass. |

The *Standard Specifications* provide that the "*s*" value (inches/blow averaged over the last few blows) in the ENR formula can be measured when the pile is retapped after a set period.

As in the case of attempting to define the term "refusal", there are as many interpretations of acceptable retap criteria as there are interpreters. Interpretations vary from no pile movement with maximum hammer energy to once or twice the required blow count in one foot of retap to the minimum blow count in 1 to 3 inches of retap.

When acceptable retap criteria is defined within the context of the Special Provisions, it will be apparent that the last of the above interpretations (minimum blow count measured over the initial several inches of retap) complies with the intent of the specifications. This statement is based on the specification requirement that application of the ENR formula is the basis of acceptance and, in this formula, pile penetration is measured over the last few blows. This is not to suggest that all pile inspection be directed toward measuring pile penetration this way. In fact most engineers prefer to use the more conservative approach and determine the penetration by counting the number of blows per foot or half foot.

The point of the above is to emphasize the purpose of the retap, which is to measure the ground "take-up" that has taken place over a given period. Hence, the effort needed to get the pile moving is of prime importance and should be the prime consideration when determining acceptability of the piles. Most engineers will argue that once the "take-up" resistance is overcome (the pile "breaks loose"), the pile will display characteristics identical to those when it was driven initially.

The *Standard Specifications* do not specify elapsed time before attempting a retap. Hence, trial and error methods have to be employed. On contracts where soft driving in clay materials is anticipated, specific retap guidelines are frequently given in the Special Provisions. The period is usually set at a minimum of 12 hours unless bearing is obtained sooner. In addition, only a fixed percentage of the piles are retapped (10% or a minimum of

2 per footing). However, when retap requirements are not listed in the Special Provisions, it is up to the Engineer to determine what criteria will be used to determine pile acceptability. At times piles will not attain minimum bearing at specified tip, even if retapped. In these cases the Contractor is obligated to furnish longer piles to accomplish the work. While this situation rarely happens with precast concrete piles, prudent contractors will drive "test" piles at various locations on the job to confirm lengths prior to ordering piles. This should be suggested to the Contractor before work starts. In the case of steel "H" piles, this situation happens frequently. If 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 problem in detail.

### Alignment of Piles

Watch the alignment of each pile. This is extremely important if swinging leads are used. Immediate correction should be made if the pile begins to move out of plumb. Driving may have to be stopped and the pile may have to be pulled and redriven.

The *Standard Specifications* state that piles materially out of line will be rejected. This brings up the question as to what is "materially out of line". Some contracts have a specific tolerance in the Special Provisions as to alignment and/or plumbness of the piles. This is usually due to special considerations in the design of the structure. 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 some circumstances. However, no payment is allowed for the additional length driven below the specified tip elevation. This subject is covered by Bridge Construction Memo 130-6.0.

## Safety

The potential for an accident around pile driving is probably greater than for any other construction operation. The pile rig with a set of heavy leads and hammer is unwieldy enough; add a long pile and a high potential for danger exists. Add a hammer in operation with its moving parts, perhaps a steam or high pressure line, and the need for awareness is obvious.

Following are some of the items that individuals inspecting piles should be aware of, especially new personnel:

| 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 to do the work. |
| 3 | Keep clear of any steam or air 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 be off 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 inspecting this work. |

CHAPTER

# 8 Static Pile Load Testing and Dynamic Pile Monitoring

## Introduction

Chapter 1 of this manual explained how a foundation investigation for all new structures, widenings, strengthenings or seismic retrofits is performed by the Office of Structural Foundations. Under normal circumstances, the Engineering Geologist assigned to perform the investigation is able to gather enough information to allow them to recommend a pile type and tip elevation that they firmly believe is capable of supporting the required loads on the recommended pile foundation.

On the other hand, there are many situations when the Engineering Geologist is not able to gather the necessary subsurface information that allows them to comfortably recommend a pile foundation that they have complete confidence in. Whenever this occurs, the Engineering Geologist may recommend a Static Pile Load Test or Dynamic Pile Monitoring operation to ensure that the piles are going to perform in the manner predicted in the design of the foundation.

Personnel from the Foundation Testing and Instrumentation Section of the Office of Structural Foundations perform Static Load Testing and Dynamic Pile Monitoring on Caltrans projects. Once the testing is completed, written reports summarizing the findings are transmitted to the Structure Representative.

## Reasons For Static Load Testing and Dynamic Pile Monitoring

In order to determine the capacity of foundation piles, the Foundation Testing and Instrumentation Section performs Static Pile Load Testing for all types of foundation piles, and Dynamic Pile Monitoring of driven piles.

Static Load Tests are performed to determine the ultimate failure load of a foundation pile and to determine the pile's capability of supporting a load without excessive or continuous displacement. The purpose of such tests is to verify that the allowable loads used for the design of a pile are appropriate and that the installation procedure is satisfactory. Static load tests also allow for the use of a lower factor of safety and a more "rational" foundation design.

Static load tests may be recommended when foundation piles are being installed in certain locations or types of material where ordinary methods of determining safe pile loads are not considered reliable. They are often recommended for Cast-In-Drilled-Hole (CIDH) piles installed in unproven ground formations. Static Load Testing is also performed where it is desired to demonstrate that friction piles may be safely loaded beyond the indicated safe loads obtained by the application of the Engineering News-Record (ENR) formula. Static Load Tests are also recommended for piles designed to carry loads in tension, particularly those foundation piles used on seismic retrofit projects.

Dynamic Pile Monitoring of driven piles is used to verify and/or predict pile capacities and to gather various pieces of useful information during the installation of driven piles.

Obtaining a successful pile foundation, which meets the design objectives, depends largely on relating the static analysis results presented on the plans to the dynamic methods of field installation.

## Static Pile Load Tests

When Static Pile Load Testing is performed on a project, personnel from the Foundation Testing and Instrumentation Section follow 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 require approximately 4 to 8 hours to complete.

The static pile load tests are made using a reaction method. The test procedure involves applying an axial load to the top of the test pile with the use of one or more hydraulic jacks. The reaction force is provided by subjecting the anchor piles to tension, in the case of a static load test in compression; or by subjecting the anchor piles to compression, in the case of a static load test in tension. Various forms of instrumentation are installed onto the test and anchor piles to measure the displacement of the test pile. 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 (Refer to Appendix F). 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 usually 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 load causes the pile 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.

The Foundation Testing and Instrumentation Section work crews have reaction trusses for static load testing up to 2,000 Tons.

In general, a pile is considered to have failed when the total displacement exceeds ½ inch under load. An acceptable pile is one that reaches double the design load without exceeding the maximum displacement.

The purpose of a Static Pile Load Test is to cause a failure along the soil/pile interface. This failure generally occurs well before the 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 the pile load test is the downward displacement of the test pile. The same effect would be achieved if a pile hammer drove the pile the additional distance.

Once the pile load testing is completed, personnel from the Foundation Testing and Instrumentation Section 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 Structure Representative, along with any recommended changes or modifications, if necessary.

## Dynamic Pile Monitoring

Along with Static Load Testing, personnel from the Foundation Testing and Instrumentation Section are assigned the responsibility for performing Dynamic Pile Monitoring on Caltrans projects. In most cases Dynamic Pile Monitoring is performed in conjunction with Static Pile Load Testing.

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

Dynamic monitoring of piles is usually performed during the installation of the test and/or anchor piles to be used for the Static Load Test. Piles to be monitored, using the PDA, are usually driven to a predetermined distance above the specified tip before the monitoring begins. At that time, the Contractor must stop driving and allow personnel from the Foundation Testing and Instrumentation Section to attach the necessary instrumentation to the pile. This instrumentation is attached approximately 1½ pile diameters from the top of the pile. The Contractor then resumes driving the pile but only for a few blows. This allows the PDA Operator to ensure that the instrumentation is attached correctly and that the data is being transmitted to the PDA computer. The Contractor then resumes driving the pile until the tip of the pile is one foot above the specified tip elevation. Once all the monitored piles are driven to this elevation, they are then allowed to "set-up", usually overnight. On the next day the instrumentation is once again attached for a retap. Before the retap, the pile must be marked over a one foot length, in increments of one tenth of a foot. Again, the pile is hit for a few blows to ensure proper instrumentation connection. The pile is then driven for the remainder of the one foot length.

Before the pile monitoring begins, the PDA operator inputs parameters related to the physical characteristics of the pile. Data is also entered to describe the surrounding soil and its damping resistance.

The PDA is capable of analyzing the stress wave produced 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. Information retrieved by the PDA is used to predict a pile's static load capacity.

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 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.

During installation, damage to a pile can be detected by the PDA. The data retrieved during the monitoring can be used to locate the depth to cracking in concrete piles and to the point of buckling in steel piles.

Data retrieved from the PDA during the retap of a pile yields valuable information about the soils interaction with the driven pile during a pile's "set-up" period.

Dynamic Pile Monitoring is believed to be very reliable for piles driven in granular soils. For finer grained soils, such as silts and clays, this method is less reliable because these soils offer significantly larger damping resistance to piles during driving that are not yet accurately modeled.

Under normal circumstances, dynamic monitoring is used in conjunction with static load testing to determine the adequacy of foundation piles.

## Contract Administration of Static Pile Load Testing and Dynamic Pile Monitoring

At the beginning of any project requiring Static Pile Load Testing and/or Dynamic Pile Monitoring, the Structure Representative should do a thorough of review the project plans, Special Provisions, *Standard Specifications* (Section 49-1.10 in particular), and Bridge Construction Memo 130-1.0 to make themselves aware of the contract requirements.

It is the Structure Representative's responsibility to coordinate the set-up for the Static Pile Load Testing and Dynamic Pile Monitoring. Early contact and good communication with the Foundation Testing and Instrumentation Section is important to 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 and Instrumentation Section. Details relating to the logistical needs of the testing work crew should also be discussed with the Foundation Testing and Instrumentation Section and the necessary information relayed to the Contractor.

Section 49-1.10 of the *Standard Specifications* requires that a change order be written to compensate the Contractor for the various forms of assistance they will need to provide for the set-up and performance of the Static Pile Load Testing and/or the Dynamic Pile Monitoring. It should be noted that cutoff of the piles to grade, furnishing additional reinforcing steel and load test anchorages are usually paid for by the contract item for piling.

The Contractor should be notified as early as possible of the equipment and personnel that they will need to provide during a Static Pile Load Test or Dynamic Pile Monitoring operation. The assistance they provide is normally covered by the contract change order that is to be written for the operation.

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 off of the State transport trailers, and for returning the equipment to the trailer once the testing is complete. The crane will need to be capable of lifting and placing a 17,000 pound beam atop the pile test groups. Occasionally, a 54,000 pound beam is used for load testing. The actual beam size to be used should be confirmed with the Foundation Testing and Instrumentation Section. All necessary rigging will be supplied by the Foundation Testing and Instrumentation Section. 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 and Instrumentation Section to ensure that efficient coordination of the test set-up is accomplished.

The Structure Representative needs to ensure that the area of the Static Load Testing and/or Dynamic Pile Monitoring 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.

Section 49-1.10 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.

The *Standard Specifications* also state that Static Pile Load Testing on concrete piles may begin when the concrete reaches a compressive strength of 2,000 Pounds per Square Inch (PSI), except for precast concrete piles, which cannot be driven until 14 days after casting. Additional cement or Type III (high early) cement may be used at the Contractor's expense.

The *Standard Specifications* also state that the Engineer will not require more than 5 working days to perform each test unless otherwise provided in the Special Provisions.

## Inspection Requirements During Static Load Testing and Dynamic Pile Monitoring

It is very important that the Structure Representative ensure that all piles to be used for Static Pile Load Testing and Dynamic Pile Monitoring be driven or constructed in accordance with the contract plans and specifications. If the contract plans do not adequately describe the test pile set-up, the Structure Representative should discuss the set-up with the Foundation Testing and Instrumentation Section.

Test piles must be installed plumb and to the specified tip elevation shown on the plans. All piles (anchor and test piles) within each test group should be logged for the full length of driving. For drilled piles, a soil classification record should be kept for the full length of all piles. If any of the driven piles have an extremely low bearing value at the specified tip elevation (less than 50% required), the Structure Representative should contact the Foundation Testing and Instrumentation Section to see if a revision of the specified tip elevation is warranted. Changes to the specified tip elevation on test and/or anchor piles will necessitate the issuance of a contract change order.

For CIDH piles, additional reinforcement is required in the anchor and test piles. This additional reinforcement should be shown on the plans. If it is not shown on the plans or if the details are unclear, contact the Project Designer and/or the Foundation Testing and Instrumentation Section. The top of CIDH test piles must be level and troweled smooth.

The contract plans or Special Provisions usually require that the anchor piles be constructed 5 to 10 feet longer than the test piles to prevent pulling. If this is not shown, the Structure Representative should discuss this issue with the Foundation Testing and Instrumentation Section to determine whether or not the lengths of the anchor piles should be revised. Here again, any changes to the lengths of the piles may warrant a contract change order.

If a construction project includes Dynamic Pile Monitoring, the Special Provisions for the project will state when the piles to be used for the monitoring are to be made available for State personnel to make the necessary preparations before these piles are driven. A technician from the Foundation Testing and Instrumentation Section will need access to the piles to prepare them for the attachment of the necessary instrumentation. The Structure Representative may need to ensure that the Contractor provide assistance to the technician to maneuver the piles.

Once the load testing crew arrives on the jobsite, the Structure Representative 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.

Once the Static Pile Load Testing and/or Dynamic Pile Monitoring is completed on a project, the Foundation Testing and Instrumentation Section will recommend changes to the foundation piles if necessary. These changes are normally made without requiring additional load tests. If an additional test is required, the Structure Representative 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) 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 Structure Representatives set up a good line of communication between themselves and the Foundation Testing and Instrumentation Section 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 to allow the static load testing work crews from the Foundation Testing and Instrumentation Section 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 cave-in 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 is a specific construction method for the construction of CIDH piles, the reader is advised to review Chapter 6 of this manual. Chapter 6 contains information about inspection duties and responsibilities of the Engineer for construction of CIDH piles. This chapter contains modifications of inspection duties and responsibilities of the Engineer as 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 the drilled hole, 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.

Because 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 polymer drilling slurries. These drilling slurries are less hazardous to the environment and are easier to dispose of.

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, most of which 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 the University of Houston. Caltrans has also conducted research on several contracts in recent years, which has lead to the development of revised contract specifications for use of the slurry displacement method of CIDH pile construction.

Caltrans first used the slurry displacement method on a construction contract in 1984 and has used this method sporadically since then. However, 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 an urban environment with restricted access and noise limitations have also led towards the expanded use of CIDH piles. Because of these factors, in 1994 Caltrans started inserting the slurry displacement method specifications into all contracts with CIDH piles.

## 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 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 some cases, the site conditions are known to be wet or unstable. These conditions may be 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.

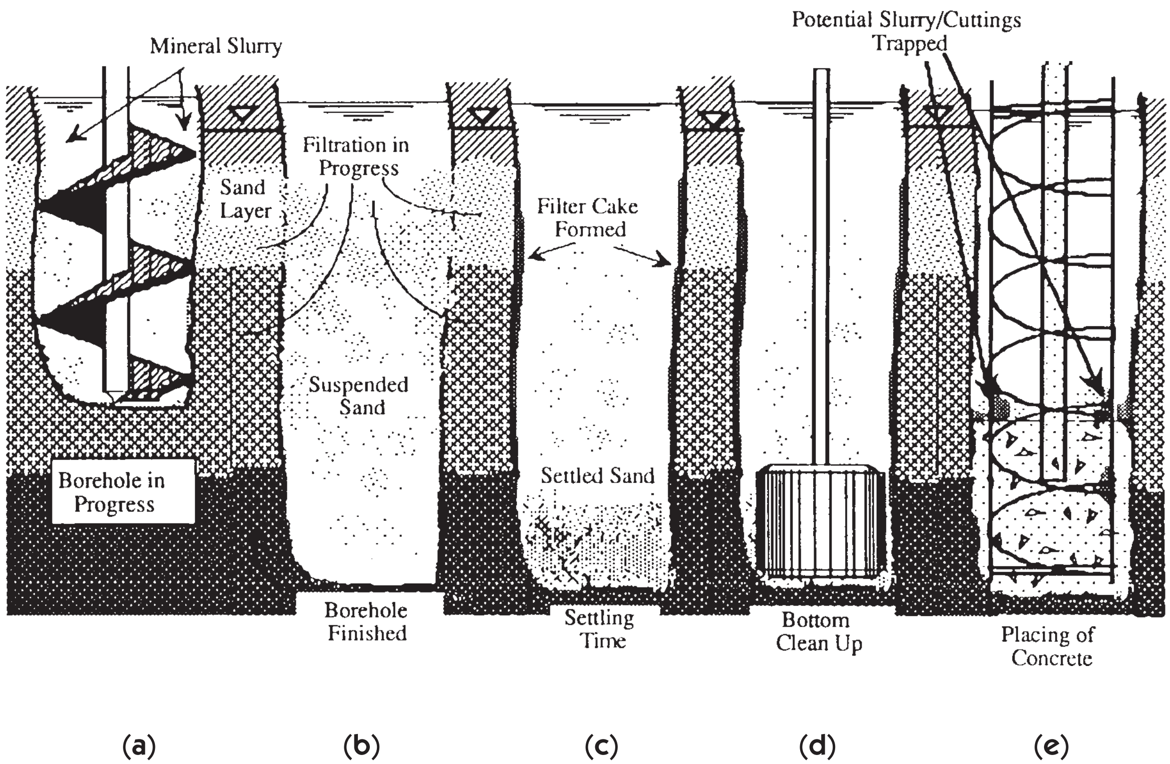



**Figure 9-1: Slurry Displacement Method**

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. Depending on the type of drilling slurry used, a process called filtration may also take place. Filtration results in the formation of a filter cake along the sides and bottom of the drilled hole. Figures 9-1(b) and 9-1(c) show the process of filtration and the cleaning of the soil cuttings from the drilling slurry. If the drilling slurry is cleaned such that it's 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 and prior to concrete placement, the drilling slurry is retested to make sure its properties are within the specified limits. Once the drilling slurry is ready, the rebar cage may be placed. Concrete is then 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.

## Principles of Slurry Usage

One of the ways drilling slurries function is by what is known as the "positive effective stress" principle (Figure 9-2). Essentially, this means that the drilling slurry produces stress on the sides of the drilled hole due to fluid pressure applied by differential head. This induced stress is produced by maintaining the level of the drilling slurry as high as possible (usually at least 5 feet) above the groundwater level in the drilled hole. In cases where the groundwater level is very close to the ground surface, use of a surface casing may be necessary to ensure positive effective stress is developed on the sides of the drilled hole.

Another way drilling slurries function is by the "filtration" principle. When drilling slurry is applying fluid pressure to the sides of the drilled hole, some of the drilling slurry and soil cuttings bonded to the drilling slurry may be forced 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, referred to as "mudcakes" if a mineral slurry is used, or "gelcakes" if a polymer slurry is used, on the sides of the drilled hole. These filter cakes help to temporarily stabilize the sides of the drilled hole.

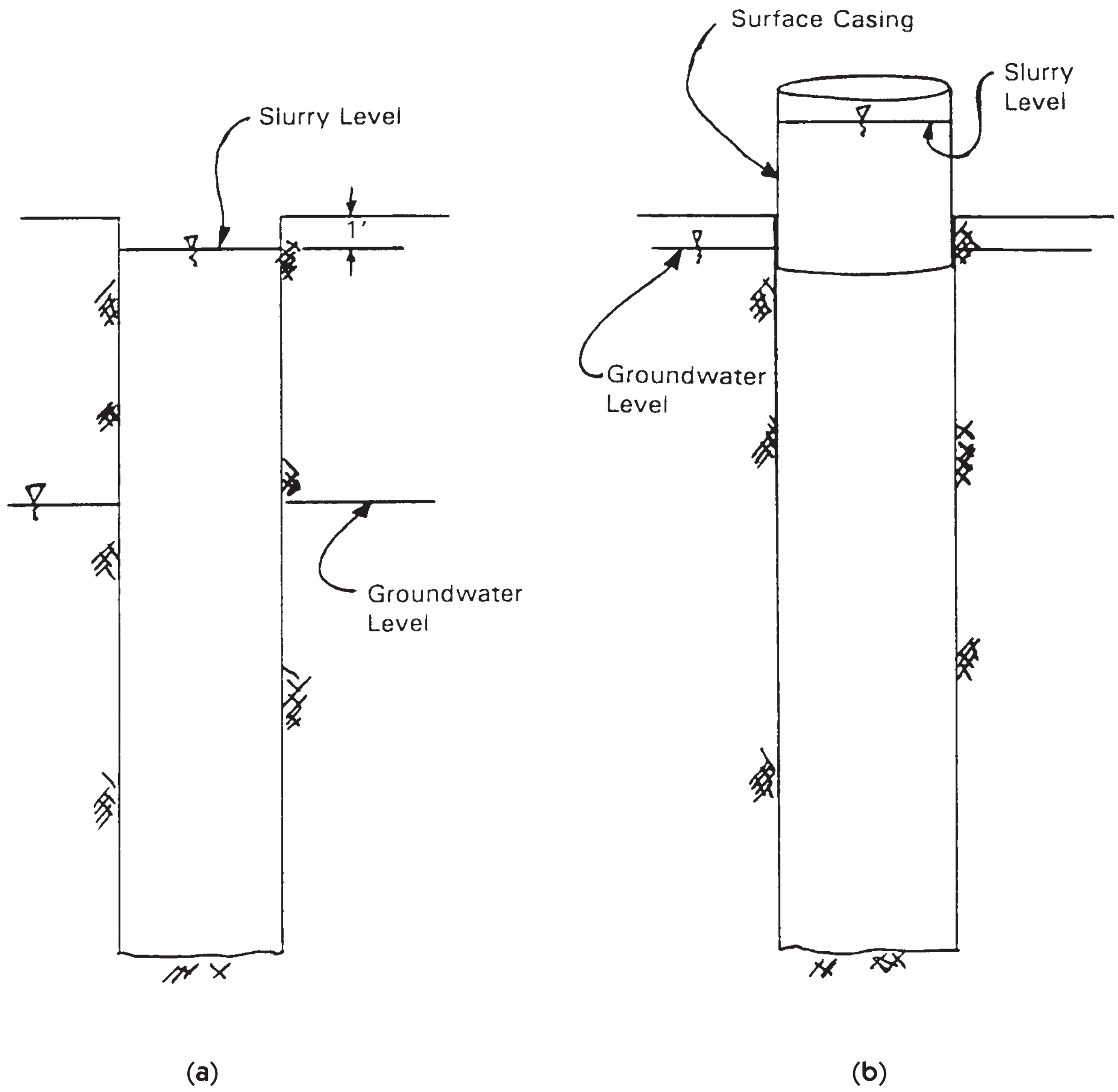



**Figure 9-2: Positive Effective Stress**

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

The nature of the ground formation has an effect on the thickness of the filter cake that develops on the sides of the drilled hole. In general, thicker cakes will form in looser ground formations, such as open sands and gravels. Because the pore spaces between the individual soil grains are larger in looser ground formations, the drilling slurry particles that are driven into the ground formation by positive effective stress tend to flow past the soil grains (Figure 9-3(a)). However, the drilling slurry particles will build up against the exposed faces of the soil grains. This build-up is capable of forming a thick filter cake on the sides of the drilled hole in a short period of time. 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 (Figure 9-3(b)). Once the pore spaces are filled, drilling slurry cannot be forced into the ground formation by positive effective stress. This causes the build-up of filter cake to cease, resulting in a thinner filter cake build-up than would be observed in looser ground formations.

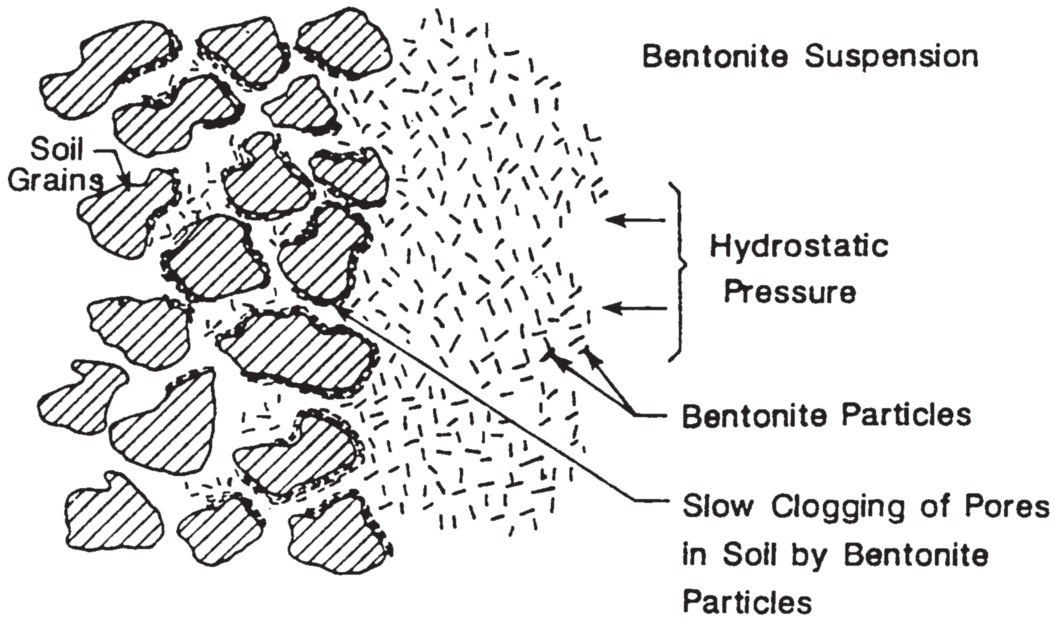



**Figure 9-3(a): Filtration – Loose Ground Formation**

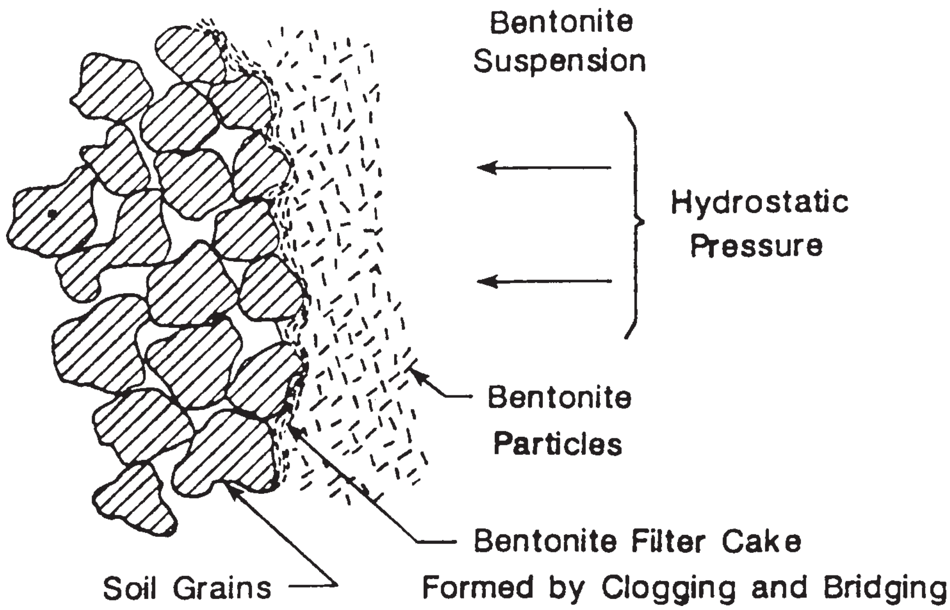



**Figure 9-3(b): Filtration – Tight Ground Formation**

The amount of time that the drilling slurry is in the drilled hole has a direct effect on the thickness of the filter cake that develops on the sides of the drilled hole. Since positive effective stress is a continuous phenomenon, the build-up of filter cake will continue so long as the pore spaces between the soil grains have not filled in. 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 build-up, which must be removed before concrete can be placed in the drilled hole.

The important thing to remember about filtration is that the 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.

## Sampling and Testing Drilling Slurry

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

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 at a different location as the sampler is removed from 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.

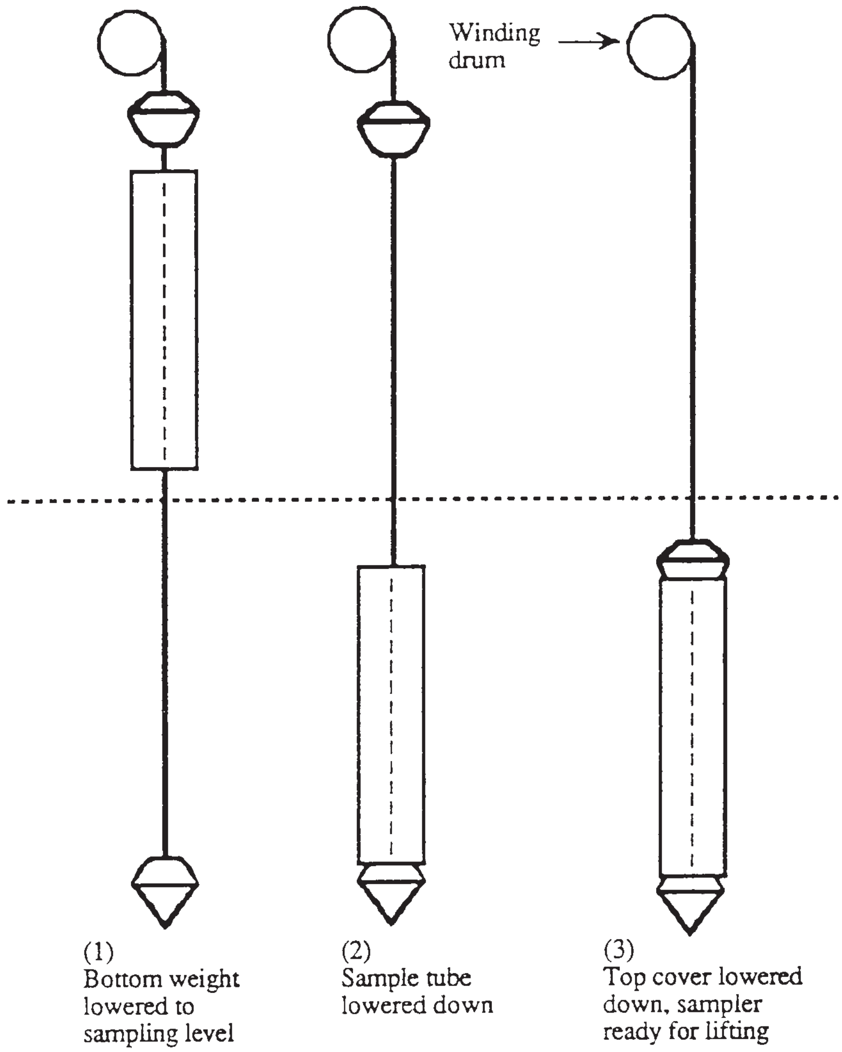



**Figure 9-4: Slurry Sampler Schematic**

One of the responsibilities of the Contractor is to verify that the sampler used seals properly. The Structure Representative 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 rebar 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. These are described in the following paragraphs.

## 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 polymer slurries and water, which do not hold solids in suspension as well. The density of the drilling slurry may be affected by its viscosity 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.

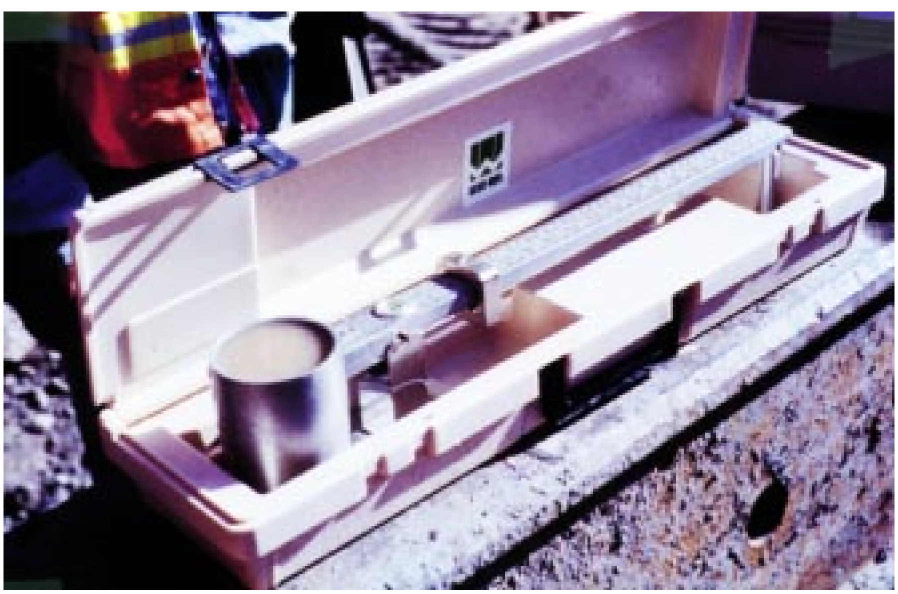


**Figure 9-5: Density Test Kit**

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 is included in Appendix G.

## 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 polymer 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, which is not desirable.

**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 is included in Appendix G.

## pH Value

The pH value of a drilling slurry is important to ensure its proper functioning. Mineral slurries which have pH values outside the allowable range will not fully hydrate the clay mineral and will not develop the expected viscosity. Polymer slurries which are mixed in water having pH values outside the allowable range may not become viscous at all. It cannot be assumed that drilling slurries that are mixed in a controlled environment (such as in a mixing tank) will not be affected by acids and organic material from the groundwater or the soil. Mineral slurries may deflocculate and fail to form a filter cake if the slurry becomes too acidic or too alkaline. Polymer 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 a 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 prevent the drilling slurry from becoming too thick and suspending more solids than permitted, which would affect the density and sand content values. On the other hand, a drilling slurry may require 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. A thinner drilling slurry would tend 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 a 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 is included in Appendix G.

## Types of Slurry

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

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

Because most drilling slurries are difficult and expensive to dispose of, most drilling contractors will want to reuse the drilling slurries. Occasionally, contractors will want to reuse the drilling slurry on another pile after completion of the previous pile. Some contractors may want to reuse the drilling slurry on another contract.

The contract specifications do not prohibit the reuse of drilling slurry. However, the drilling slurry 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 a drilling slurry from a different contract, the Structure Representative may want to have the physical properties of the drilling slurry tested prior to placement in the drilled hole.

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 polymer slurries, this is accomplished by allowing the contaminants to settle out.

The types of drilling slurries that are permitted for use by Caltrans (as of 1994) are detailed in the following sections. Three types of drilling slurries are permitted: water, mineral, and synthetic polymer.

## Water

Water may be a suitable drilling slurry under the right conditions. Most drilling contractors will try to use water as a drilling slurry if the ground conditions are right because it is inexpensive. However, use of water as a 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 5 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 a drilling slurry when temporary casing is used for the entire length of the drilled hole. Although water was permitted to be used as a drilling slurry in the recent past by the contract specifications, history has shown that water was inappropriately chosen as a drilling slurry for use in holes drilled in unstable ground formations by some contractors for economic reasons. 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 a 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 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 quick conditions at the bottom of the drilled hole and to be able to place concrete.

The physical properties of water used as a 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 is not important and water will not typically become more viscous, 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 a drilling slurry can be easily disposed of on site after settlement of all suspended materials has occurred unless the water has been contaminated by hazardous materials.

## 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 a rock composed of clay minerals, named after Fort Benton, Wyoming, where this particular type of rock was first found. It is processed from the clay mineral Sodium 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 salt 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 salt water is present. Due to the expense of transport and the relative rarity of use of this type of drilling slurry in California, it is unlikely that this type of mineral slurry will be encountered on Caltrans projects.

Mineral slurries stabilize the sides of the drilled hole by positive effective stress 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 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. Office of Structure Construction policy is that "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 is dependent upon 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, current Office of Structure Construction policy 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 judgement should be applied before implementing this policy. There are other indicators that can be used to assist the Engineer in making a judgement 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 build-up 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 overboring 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.

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 to 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 by the Engineer that it will usually take 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 the drilled hole. 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

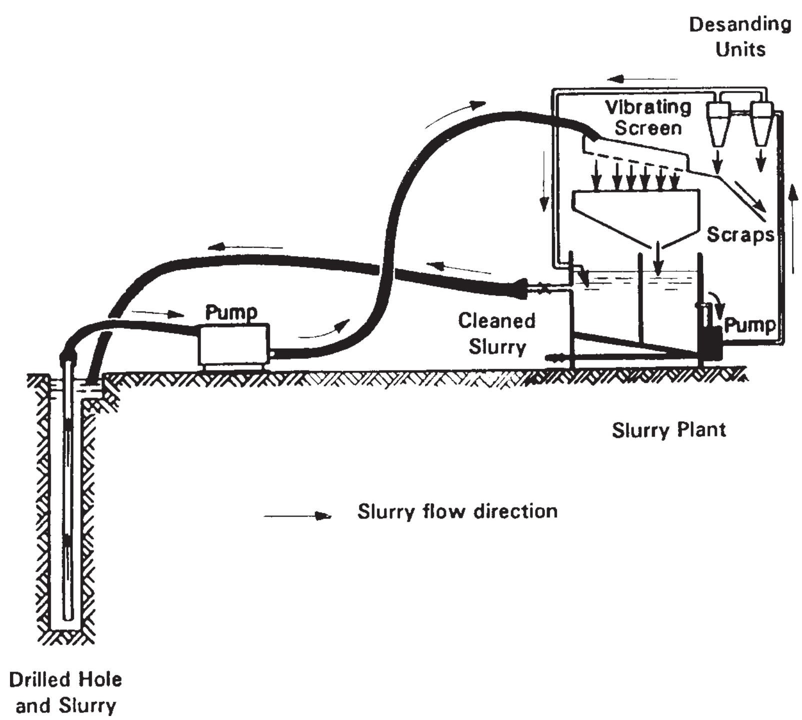



Figure 9-11: Recirculation and Cleaning Schematic

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 which 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, there are many chemical additives available that can reduce the filter cake thickness, lower 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 and lignosulfonates. Additives that lower the mineral slurry's pH include soda ash and pyrophosphate acid. Additives that decrease the mineral slurry's viscosity include tanins and polyphosphates. Caltrans has little experience with chemical additives and their use should be discussed with the Office of Structural Foundations and the Office of Structure Construction in Sacramento. In general, modifying the pH of the mineral slurry with chemical additives is not a problem.

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, 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 which 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.

## Polymer

Polymer drilling slurries have been gaining wide acceptance in the construction industry over the last 10 years. The main advantage of polymer 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. Polymer slurries are grouped into three groups: (1) naturally occurring polymers, (2) semi-synthetic polymers, and (3) synthetic polymers. The synthetic polymers currently consist of two types: (1) emulsified Partially Hydrolyzed Polyacrylamide, Polyacrylate (PHPA) polymers, and (2) dry vinyl polymers.

Research has been performed at the University of Houston, by the various polymer slurry manufacturers, and by Caltrans on the effect of polymer slurries on rebar bond, pile skin friction, and concrete contamination. Although incomplete, this research has resulted in the approval of some polymer slurries for use on Caltrans contracts. As of 1994, the contract specifications permit the use of two brands of synthetic polymer slurries. These are Super Mud, manufactured by PDS Company and SlurryPro CDP™, manufactured by KB Technologies, LTD.

Super Mud is an emulsified PHPA polymer type of synthetic polymer slurry. Emulsified PHPA polymers rely on the positive effective stress principle because they do not control filtration in open ground formations and will not form a filter cake on the sides of the drilled hole. However, the polymer chains in the drilling slurry do enter the ground formation and bond to the individual soil grains. This helps to stabilize the sides of the drilled hole. Continuous addition of drilling slurry may be required in order to stabilize the level of the drilling slurry in the drilled hole. Because a filter cake is not formed, maintaining the level of the drilling slurry well above the groundwater level is essential. A liquid form of Super Mud is currently approved for use on Caltrans projects. No other form is approved.


**Figure 9-12: *SuperMUD* Container**

SlurryPro CDP™ is a dry vinyl polymer type of synthetic polymer slurry. Dry vinyl polymers also rely on the positive effective stress principle. However, they also control filtration and will form a filter cake in the form of a thin gel membrane on the sides of the drilled hole. Removal of this gel membrane is not required because it is not considered to be "caked slurry". Unlike the filter cake formed by mineral slurries, according to the manufacturer, the gel membrane may actually improve concrete bonding to the sides of the drilled hole. Enough research has not been done to validate this claim. A dry granular form of SlurryPro CDP™ is currently approved for use on Caltrans projects. No other form is approved.


**Figure 9-13:** ***SlurryPro CDP*** **Container**

Polymer slurries must be thoroughly mixed but do not require additional time to hydrate. This is because polymer slurries do not hydrate in water. Water used to mix with the emulsion PHPA polymer must have a pH in the range of 9 to 11 in order to properly disperse the polymer. A more acidic pH will cause the polymer slurry to flocculate and become ineffective. A mixing tank is usually required in order to regulate the water. The manufacturer of Super Mud recommends tank mixing, but mixing directly into the drilled hole by introducing the Super Mud fluid into the flow of water is also acceptable to the manufacturer. Dry vinyl polymer in granular form is not as sensitive to the pH of water and will disperse in a more acidic environment and therefore can be mixed directly in the drilled hole. The manufacturer of SlurryPro CDP™ also recommends tank mixing, but mixing directly into the drilled hole by sprinkling the dry granules into the flow of water entering the drilled hole is also acceptable to the manufacturer.

The physical properties of polymer slurries should be carefully monitored during drilling of the hole and before concrete placement. Because polymer 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 polymer 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 and SlurryPro CDP™ due to the extensive research that had been done by the manufacturers during the Caltrans approval process.

The polymer slurry's viscosity value has a higher level of importance than that of mineral slurry. The polymer slurry's 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, polymer 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 polymer 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 and SlurryPro CDP™. This is due to the extensive research that had been done by the manufacturers during the Caltrans approval process. In general, polymer slurries with very high viscosity values (> 70) are not approved for use during concrete placement because not enough research has been done to determine the effect of polymer slurries with such high viscosities on the perimeter load transfer capabilities between the pile concrete and the ground formation.

In general, polymer slurries will break down when they come in contact with concrete. This is advantageous as long as the polymer slurry is clean and the rising head of concrete is the only concrete in contact with the polymer slurry. However, if concrete is allowed to intermingle with the polymer slurry, the polymer 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 polymer slurry is introduced into the drilled hole. The manufacturer's representative must be approved by the Engineer. Assistance on approval of a manufacturer's representative may be obtained from the Office of Structure Construction in Sacramento. The manufacturer's representative can provide assistance with polymer slurry property testing, can test the water to be used for contaminants that may adversely affect the properties of the polymer slurry and the stability of the drilled hole, and can give advice in the proper disposal of the polymer slurry. The manufacturer's representative may also recommend the use of chemical additives to adjust the polymer slurry to the existing ground conditions. Caltrans has little experience with chemical additives and their use should be discussed with the Office of Structural Foundations and the Office of Structure Construction in Sacramento before approval is given for their use. In general, modifying the pH of the polymer slurry with chemical additives is not a problem. The contract specifications also require the manufacturer representative's presence until the Engineer is confident that the Contractor has a good working knowledge of how to use the product, after which the manufacturer's representative can be released. This can usually be accomplished with the completion of one pile.

Polymer drilling slurries can be used in most types of ground formations. However, the contract specifications state that polymer slurries shall not be used in soils classified as "soft" or "very soft" cohesive soils. There are two reasons for this. First, polymer 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 instead of water to wash the fines through the #200 mesh screen during the sand content test. This will break up the encapsulated clay particles so they will wash through the #200 mesh screen. Second, as of 1994, the polymer 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 polymer slurries is somewhat easier than disposal of mineral slurries. The manufacturers of the approved polymer 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 polymer 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. Polymer 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 is not much different than that used to construct CIDH piles by ordinary means. However, there are some differences in the drilling tools, drilling techniques, cleaning techniques, and use of casing.

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 effective stress 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 9-14). 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 overbore 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 overbore 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.

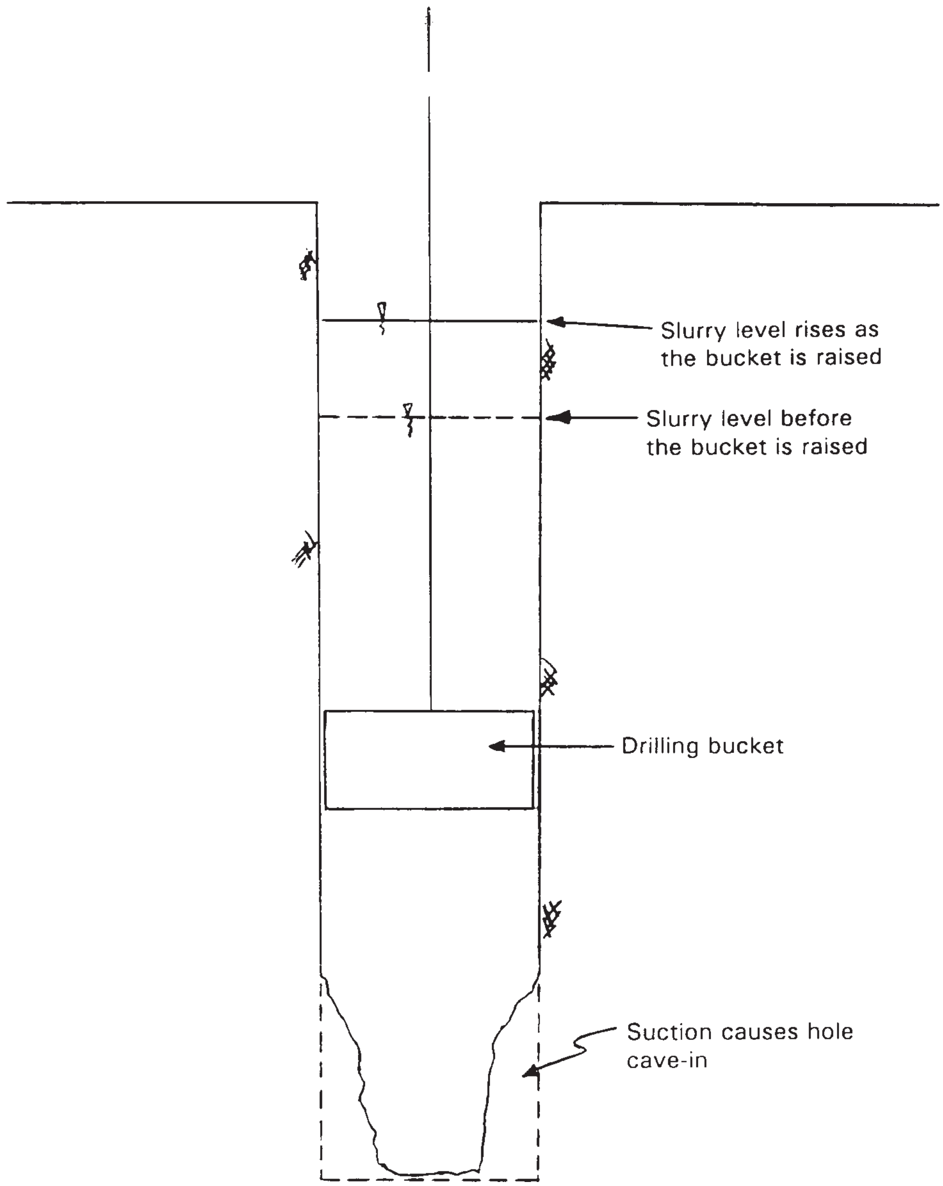



**Figure 9-14: Hole Collapse induced by Pressure Changes**

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 9-15). 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 9-16). 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 9-15: Cleanout Bucket**

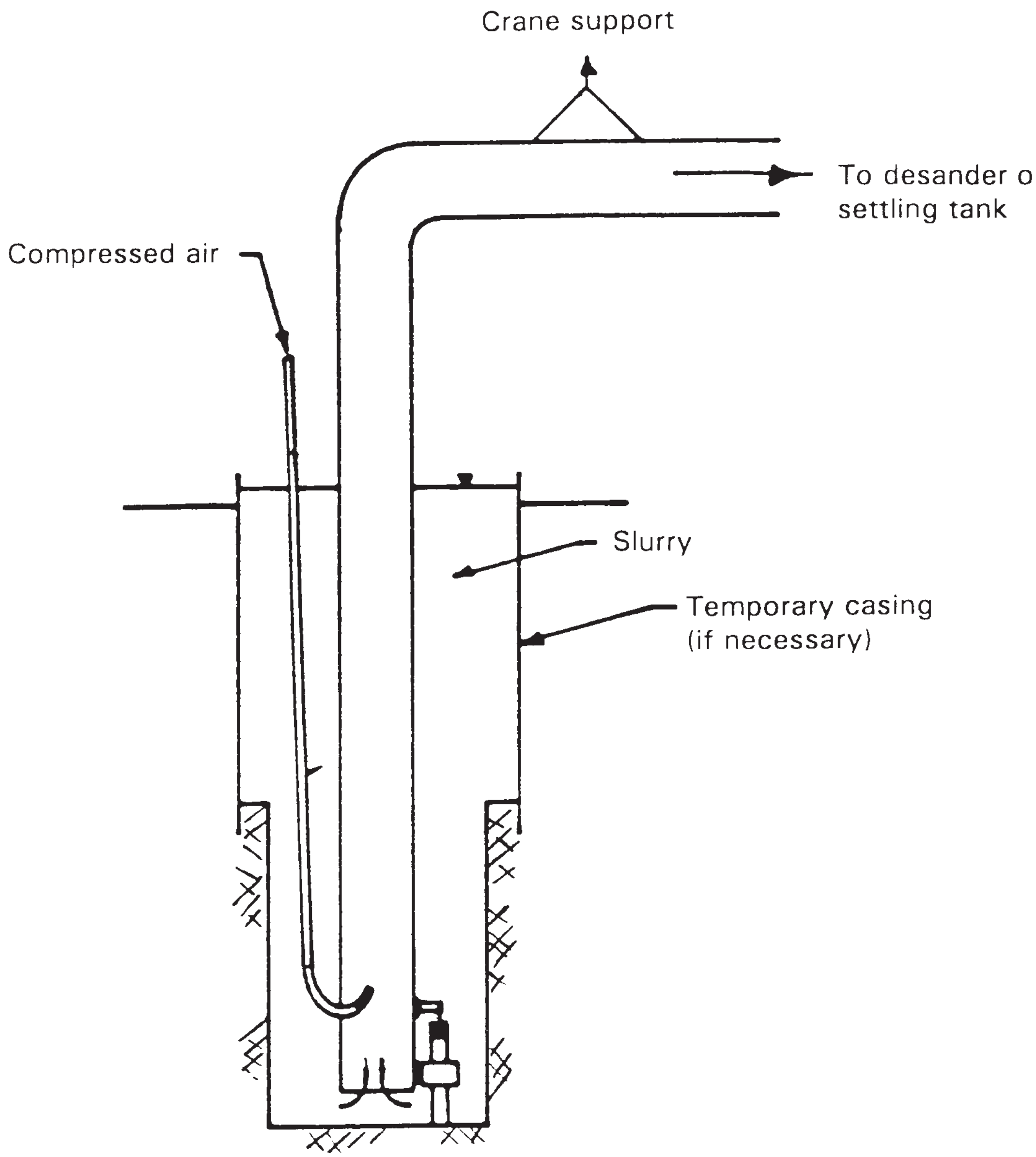



**Figure 9-16: Airlift Schematic**

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 strata or a loose material strata with flowing groundwater are encountered during drilling (Figure 9-17). 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 polymer drilling slurries to stabilize the remainder of the drilled hole. Another option is to place full length 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.

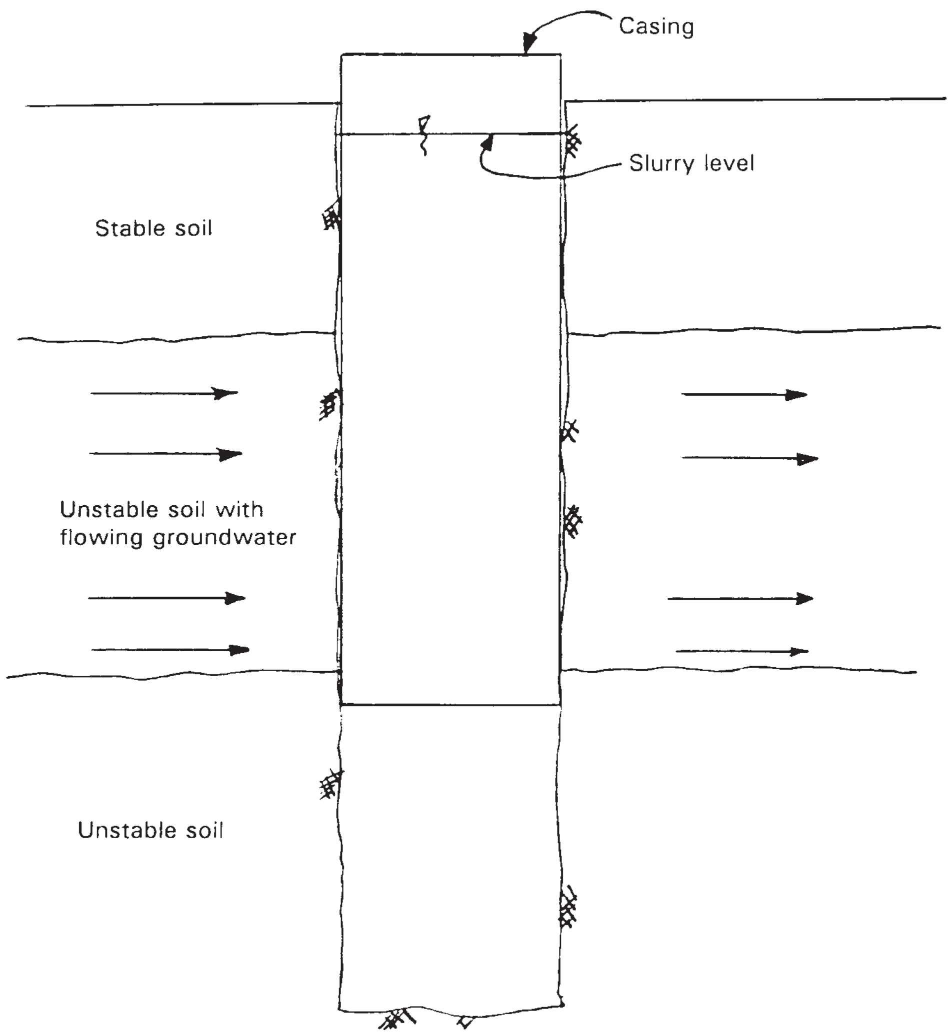



**Figure 9-17: Use of Casing**

## Concrete Placement

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 that the concrete mix design meet the trial batch requirements for compressive strength concrete. These requirements are described in Section 90-9 of the *Standard Specifications*. The concrete mix must contain at least 658 pounds of cement 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 times the maximum aggregate size. 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 bleedwater 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. Once 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 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 a minimum of 10 feet below the rising head of concrete 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. After this has been achieved, the concrete within 5 feet of the top of the pile is vibrated to consolidate the concrete at the top of the pile. Deeper vibration of the pile concrete is not necessary because concrete with high fluidity self-consolidates under the high hydrostatic pressure provided.

The contract specifications also require that the Contractor keep a concrete placement log for the concrete placement operation for each pile. The concrete placement log should contain information on the pile location, tip elevation, dates of excavation and concrete placement, quantity of concrete deposited, length and tip elevation of any temporary casing used, and details of any hole stabilization method that is used. In addition, the log shall include a graph showing the amount of concrete placed versus the depth of the hole filled. This means the Contractor must have a way of accurately measuring the volume of concrete being placed in the pile. For large piles, counting concrete trucks may be sufficient. For small piles, the Contractor may have to use some other means, such as determining the volume of concrete delivered per pump stroke and counting the number of pump strokes. Measuring the depth of the hole filled is usually accomplished using a weighted tape measure to locate the head of concrete within the pile. These readings must be taken at maximum intervals of 5 feet of pile depth.

The purpose of the concrete placement log is to provide the Engineer and the Contractor with a record of the concrete placement operation. It can be used to identify potential problem locations within the pile. Figure 9-18 shows a situation where several hole cave-ins took place. Using the concrete placement log, the Engineer can determine the approximate location of potential problem areas within the pile. Pile testing can then be used to determine whether there is a problem at the suspect area.

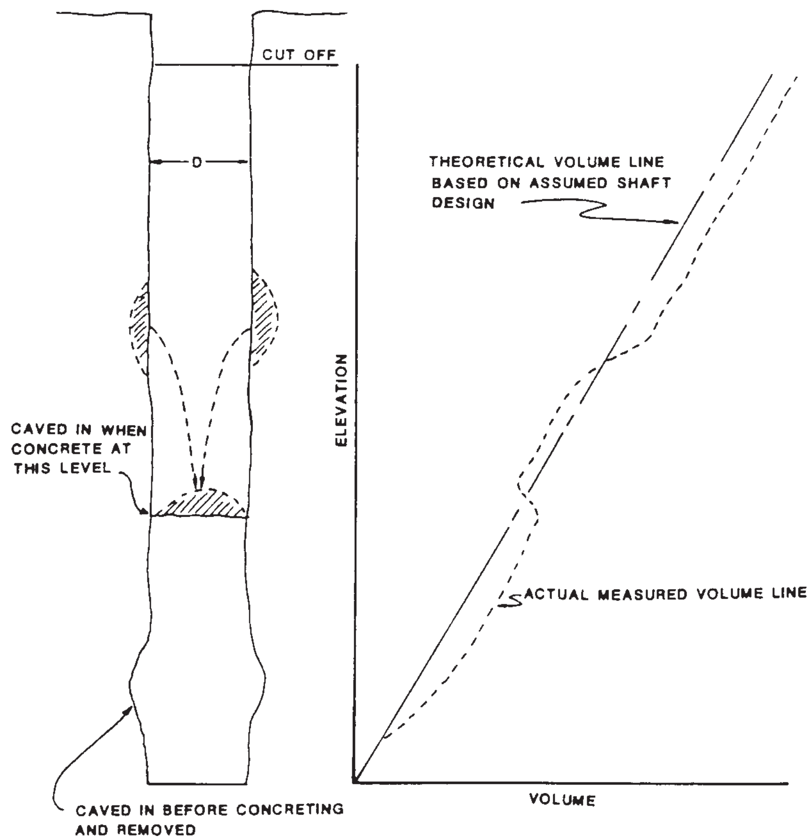



**Figure 9-18: Concrete Pile Log Graph**

# Pile Testing

In order to facilitate pile testing, the contract specifications require the installation of inspection tubes (Figure 9-19). These tubes must be installed inside the spiral or hoop reinforcement of the rebar cage before concrete is placed. Figure 9-20 shows a typical inspection tube layout and spacing pattern within the rebar cage. These tubes must be placed in a straight alignment, securely fastened in place, and be watertight. They permit the insertion of a testing probe that measures the density of the pile concrete. The most commonly used type of test probe is 1.87 inches in diameter and 22.7 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.

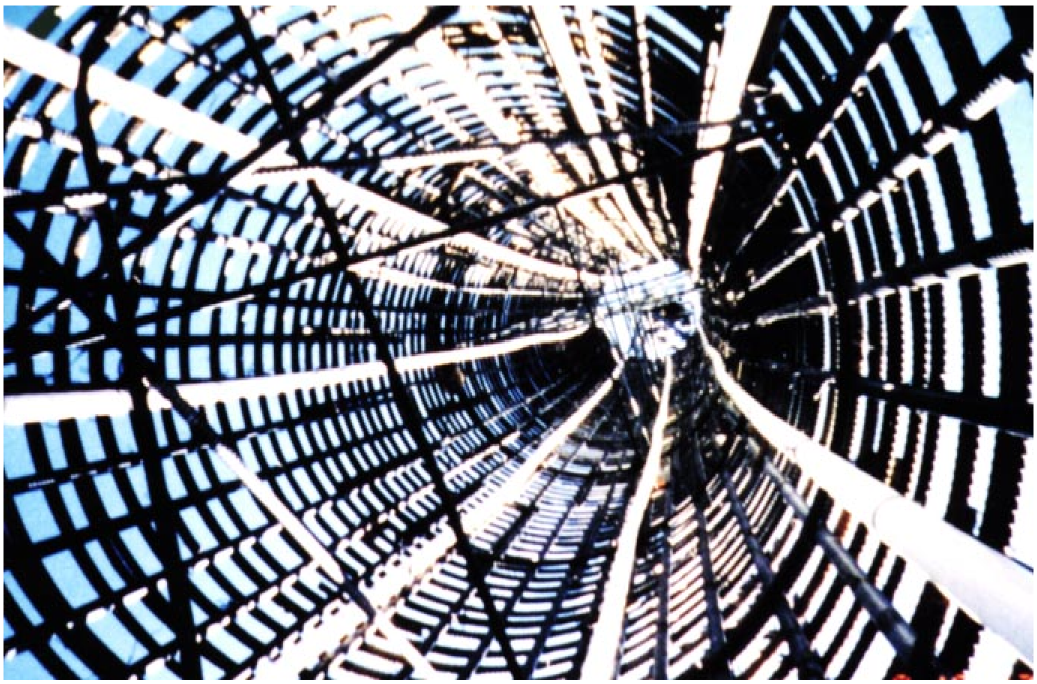


**Figure 9-19: Inspection Tubes**

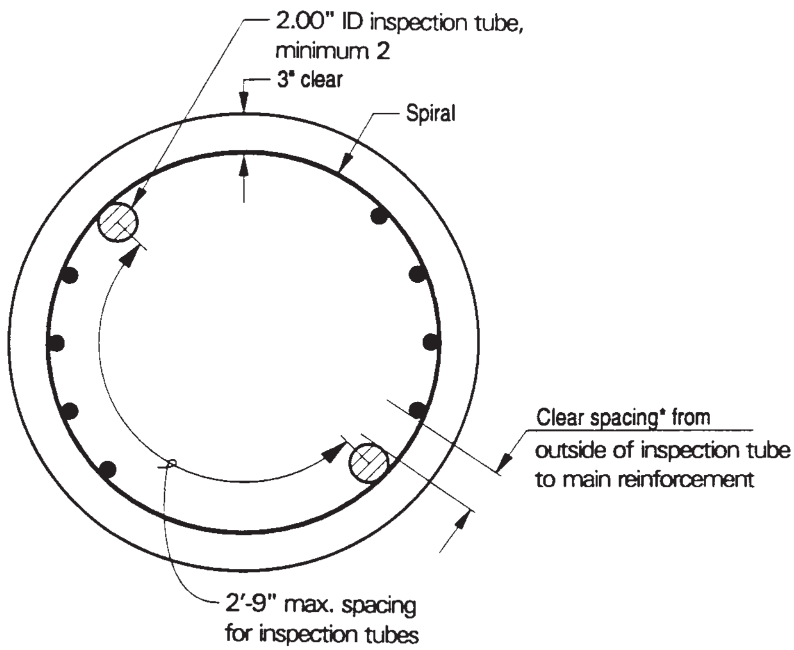



**Figure 9-20: Location of Inspection Tubes within the Pile**

The Contractor has the responsibility for placement of the inspection tubes. After concrete placement and before testing, the tubes shall be checked for blockages and straightness with a dummy probe which is the same size and shape as the test probe. Tubes that cannot accept the dummy probe shall be replaced with a 2 inch diameter cored hole the full length of the pile.

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 rebar cage prior to concrete placement. Another method uses an acoustical technique, which is commonly referred to as cross-hole 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. A fourth method uses a radiographic technique called gamma ray scattering (Figure 9-21).

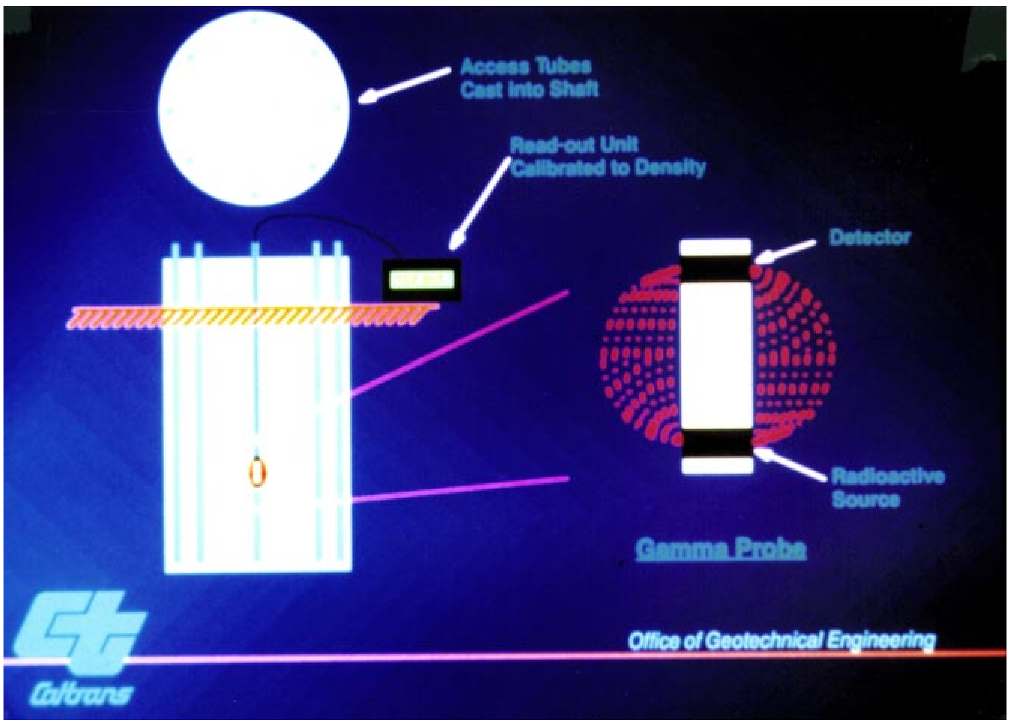



**Figure 9-21: Gamma Ray Scattering Test Schematic**

As of 1994, the contract specifications state that the gamma ray scattering method of testing piles constructed using the slurry displacement method will be used to determine acceptance of the pile. Other methods may be used in the future as the Office of Structural Foundations develops equipment and expertise.

In gamma ray 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 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 2 to 3 inch radius of concrete around the tip of the probe. 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 30 second counts taken at 6 inch increments for the whole length of the pile. This procedure requires about 2-$^1/_2$ hours for a single inspection tube in a 100 foot length pile. Even at this rate, a 48 inch diameter pile containing four inspection tubes would still require 10 hours to check out, assuming there are no access problems in the tubes.

Pile testing is performed by Caltrans personnel from the Office of Structural Foundations of the Engineering Service Center and the results, which include a recommendation of acceptance or rejection, are reported to the Structure Representative in writing. An example of these results can be found in Appendix G.

Occasionally, the Office of Structural Foundations will experience a staffing shortage and may not be able to test the piles. It is important for the Structure Representative to contact the Office of Structural Foundations well in advance of the need for testing so that the Office of Structural Foundations can determine whether they will have the staffing necessary to test the piles. If the Office of Structural Foundations is unable to test the piles, the Structure Representative shall make arrangements for pile testing through the current service contract. The Office of Structural Foundations can provide information on the current service contractor.

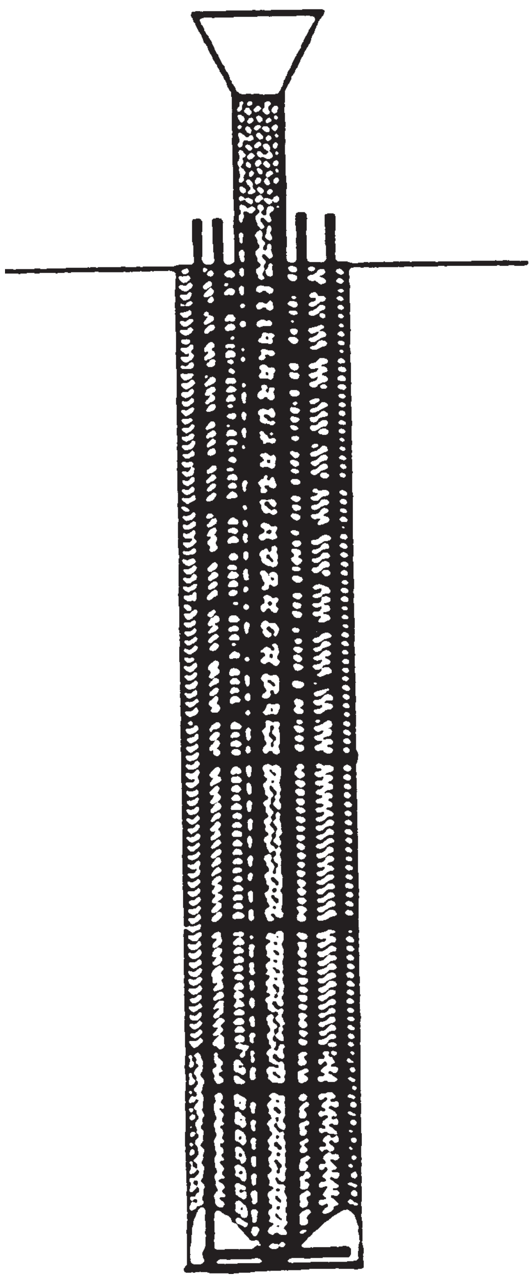
The Structure Representative has the responsibility for accepting or rejecting a pile based on the recommendations of the Office of Structural Foundations. If the pile is accepted, the inspection tubes may be cleaned and grouted, and the pile is complete.

## Defective Piles

If the Office of Structural Foundations or the service contractor determines that a pile is defective and the Structure Representative 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.

What causes piles constructed by the slurry displacement method to be defective? One of the primary reasons for pile defects is problems caused by settled materials. 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 9-22(a) or they can be enveloped and lifted by the fluid concrete only to become caught by the rebar cage or against the sides of the drilled hole and not be displaced by the fluid concrete as shown on Figure 9-22(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 rebar 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 polymer 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 polymer 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 rebar cage and not be removed by concrete placement as is shown in Figure 9-23. 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 9-24. Mineral and polymer 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.



(a) (b)

**Figure 9-22: Defects from Settled Materials**

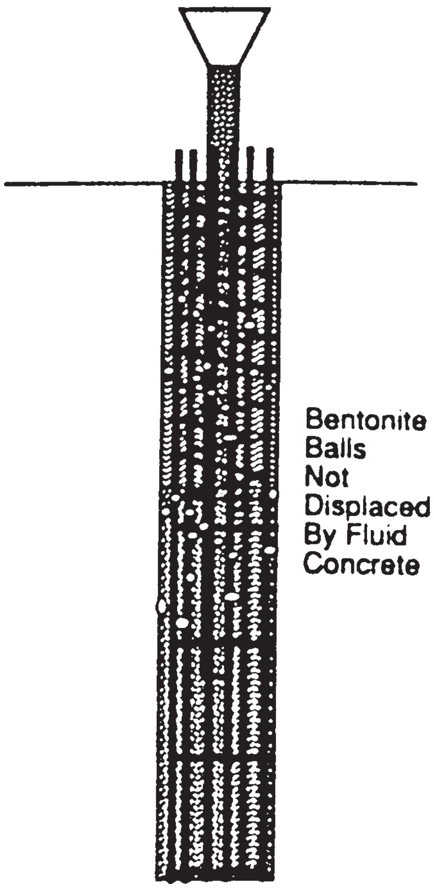



**Figure 9-23: Defect from Improperly Mixed Mineral Slurry**

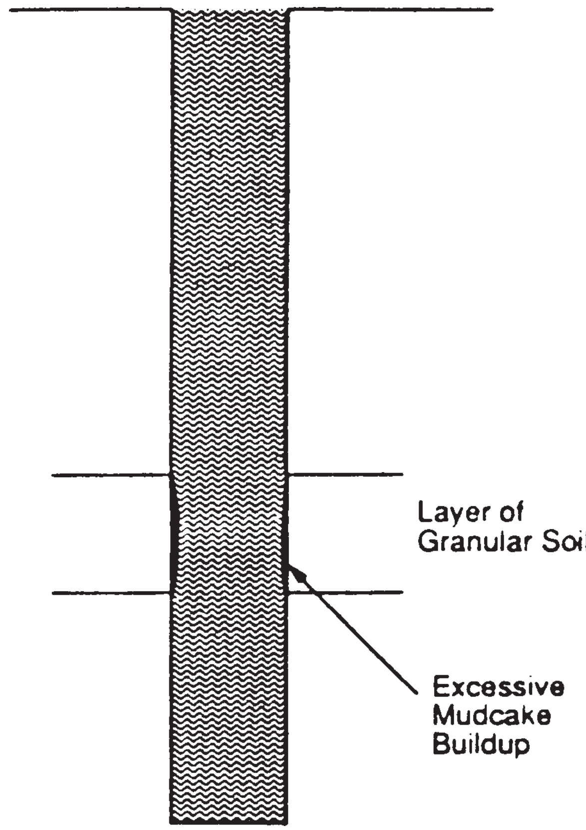



**Figure 9-24: Defect from Excess Filter Cake Buildup**

A third reason for pile defects is concrete mix design and placement problems. Whenever the seal between the head of concrete and the drilling slurry is lost, a defect is very likely to result. This is because entrapment of drilling slurry within the concrete is almost inevitable under this circumstance (Figure 9-25). 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. If the concrete head begins to set, the concrete can "fold" over as it is rising through the rebar cage and entrap drilling slurry and any settled materials as previously described. 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 bleedwater 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 polymer 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 rebar cage and cause pile defects.

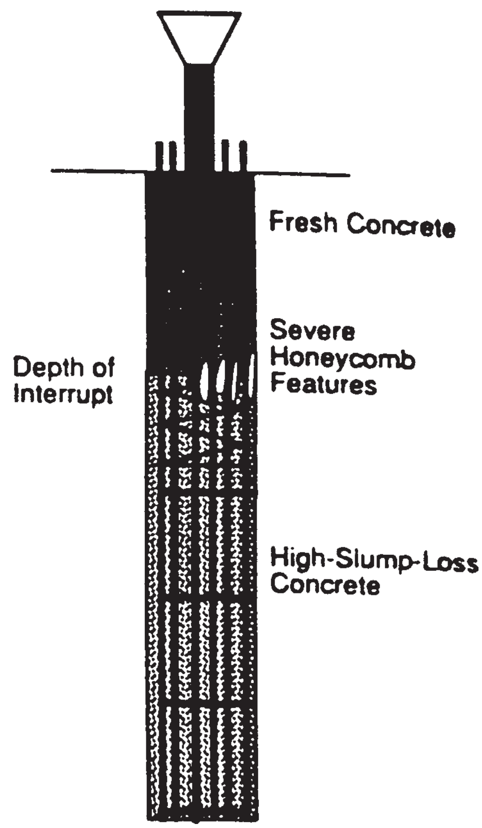



**Figure 9-25: Defect from Concrete Placement Problems**

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.

Once a pile has been determined to be defective, 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. For whatever solution the Contractor proposes, additional investigation will be necessary to determine the nature and extent of the defect. Other methods of pile testing, such as cross-hole sonic logging or coring, may be used to perform this investigation. An investigation of the ground formation will also be necessary since potential pile rebar corrosion will be a major concern. These investigations are to be performed by the Contractor at the Contractor's expense. With the additional information, the Contractor can propose a solution that may not require access to the defect or replacement of the pile. Methods of repair that would not require the defect to be exposed include the placement of supplemental piles and the use of pressure grouting. If a supplemental pile is proposed, the Engineer should keep in mind that corrosion of the existing pile rebar that may be exposed to the soil may be a concern. The Contractor must investigate the corrosive nature of the soil at the site of the defect before the existing pile can be assumed to have any capacity in combination with a supplemental pile. Pressure grouting is usually most effective as a means of improving the quality of the concrete at the tip of the pile. Any proposal made by the Contractor should be investigated by the Office of Structure Design and the Office of Structural Foundations to ensure that the pile will have sufficient structural and geotechnical capacity before approval is given.

## 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, which 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 sheets 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 the safety precautions will be adhered to by Caltrans employees, the Contractor's employees, and any manufacturer's representatives that may be present.

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

# 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, which were revised in 1994, 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 may not contain enough room for the rebar 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 rebar cage would have to be increased from the original size in order to accommodate the items mentioned above.

## Pile Placing Plan Requirements

Before any pile construction work using the slurry displacement method can begin, the Contractor shall submit a detailed placing plan to the Engineer for approval. The placing plan is necessary to show that the Contractor has thought out what needs to be done during the construction process, has a plan for addressing *all* aspects of the work, and can show that the pile can be constructed in a timely manner. Because the contract specifications state that the Contractor shall place the concrete for a single pile within a two hour period, the Contractor must be able to show that the equipment, including concrete pumps and delivery tubes, will be adequate to meet this requirement. The Contractor must also show that the delivery rate from the concrete supplier will be adequate to meet this requirement. The intent of this specification is to limit the amount of time in which suspended materials can settle out of the drilling slurry during concrete placement and to make sure the concrete retains high fluidity throughout the length of the pour. The amount of detail required for the Contractor's submittal will vary based on the size and depth of the drilled hole.

### 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 and a penetration of at least 2 inches after a period of four hours has passed from the time of placement. 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 rebar 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.

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 workshift. 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 midheight 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 midheight 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 rebar 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 midheight 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.

For polymer slurries, sampling for testing shall be conducted as necessary to control the physical properties of the polymer slurry. Samples shall be taken at the midheight and at the bottom of the drilled hole. Samples for testing shall be taken as necessary to verify the properties of the polymer slurry during the drilling operation. Once the drilling operation has been completed, samples for testing shall be taken. When the polymer 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.

Polymer slurries are cleaned by allowing for an unagitated settlement period, usually of about 30 minutes in length. Because polymer 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 polymer slurry to be recleaned so it meets the requirements of the contract specifications. Several iterations may be required before both the polymer slurry and the bottom of the drilled hole are clean. To verify the cleanliness of the polymer slurry, the contract specifications require additional samples to be taken for testing. Samples shall be taken at the midheight and at the bottom of the drilled hole. Once the test samples show the polymer 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 rebar 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 polymer slurry. Samples shall be taken at the midheight 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 polymer slurries at different levels is to make sure the polymer 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 which may settle during placement of the rebar cage and concrete.

## Pile Testing Access Requirements

During pile construction work, the contract specifications require the installation of inspection tubes at specific intervals around the perimeter of the bar reinforcement cage. This is necessary to provide access for structural integrity 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 intent of these specifications is to prevent the concrete placement tubes from allowing the concrete and drilling slurry to intermingle during concrete placement.

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. They are primarily a Cast-In-Drilled-Hole (CIDH) pile, except the means of excavation is something other than the conventional drilling method.

## Specifications

The Special Provisions will contain a great deal of information regarding mined 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 pre-construction 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 to the Contractor that care must be used in constructing the access road and/or work area so that the excavation does not extend below the top of the neat line areas.

## 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 stream beds, 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 before 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.

## 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 can also be used for pier construction or incorporated into control points for pier construction.

Soft material can be excavated with conventional methods, such as a Gradall, flight auger, clambucket and hand work. Hard material encountered in otherwise soft material requires other means. Since blasting is the most common method, it will be discussed the most throughout the rest of this chapter.

An air-track compressor type drill rig is commonly used for line drilling operations. The drill bits are 2½ to 5 inches in diameter and come in 20 foot lengths with screw-on attachments for greater depths. Another method is a rotary drill attached to a rotary table and Kelly bar.

The first phase of pier column excavation is to line drill the perimeter of the shaft at the neat line dimensions (the Contractor may elect to line drill slightly outside the neat line dimensions). Holes are usually drilled on 12-inch centers with additional holes placed inside the perimeter if needed. Lined holes should be blown out and filled with sand or pea gravel to facilitate blasting at different levels.

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.

| 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. 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). |
| 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. Protection must be provided for existing facilities, utilities, etc. A galvanometer should be used to check for shorts in the wiring prior to blasting. Blasting mats, tires, dirt, etc. should be used to prevent flyrock from being scattered beyond expected limits. Proper warning signs should be provided along highways and roads near the blast site. No explosive material should be left in the area overnight. If it cannot be avoided, leave a guard overnight in the area. During the blast, guards should be placed at selected locations to prevent individuals from entering the blast area. Beware of "misfires." In general, this operation is not our responsibility. If you have any questions on the responsibility of Caltrans in regards to blasting, contact the Caltrans Headquarters Construction Safety Officer. |
| 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 area, but the general public as well, since the potential for serious injury is ever present.

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

CHAPTER

# 11 Tiebacks, Tiedowns, and Soil Nails

## Tiebacks

Tiebacks are used in both temporary and permanent structures. The use of tiebacks with sheet pile or soldier beam shoring permits higher walls and deeper excavations than are possible with cantilever type construction—up to 35 feet or so versus 15 feet for cantilever construction. Walls higher than 35 feet can be built by using high strength sheet pile or soldier beams with additional tiers of tiebacks.

### Components

A tieback consists of the following components as shown in Figure 11-1:

| COMPONENT | DESCRIPTION |
|---|---|
| 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 which is free to elongate elastically and transmit the resisting force from the bond length to the wall face. |
| Prestressing Steel – Support Member | This transfers load from the wall reaction to the anchor zone and is generally a prestress rod or strand. |
| 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. |

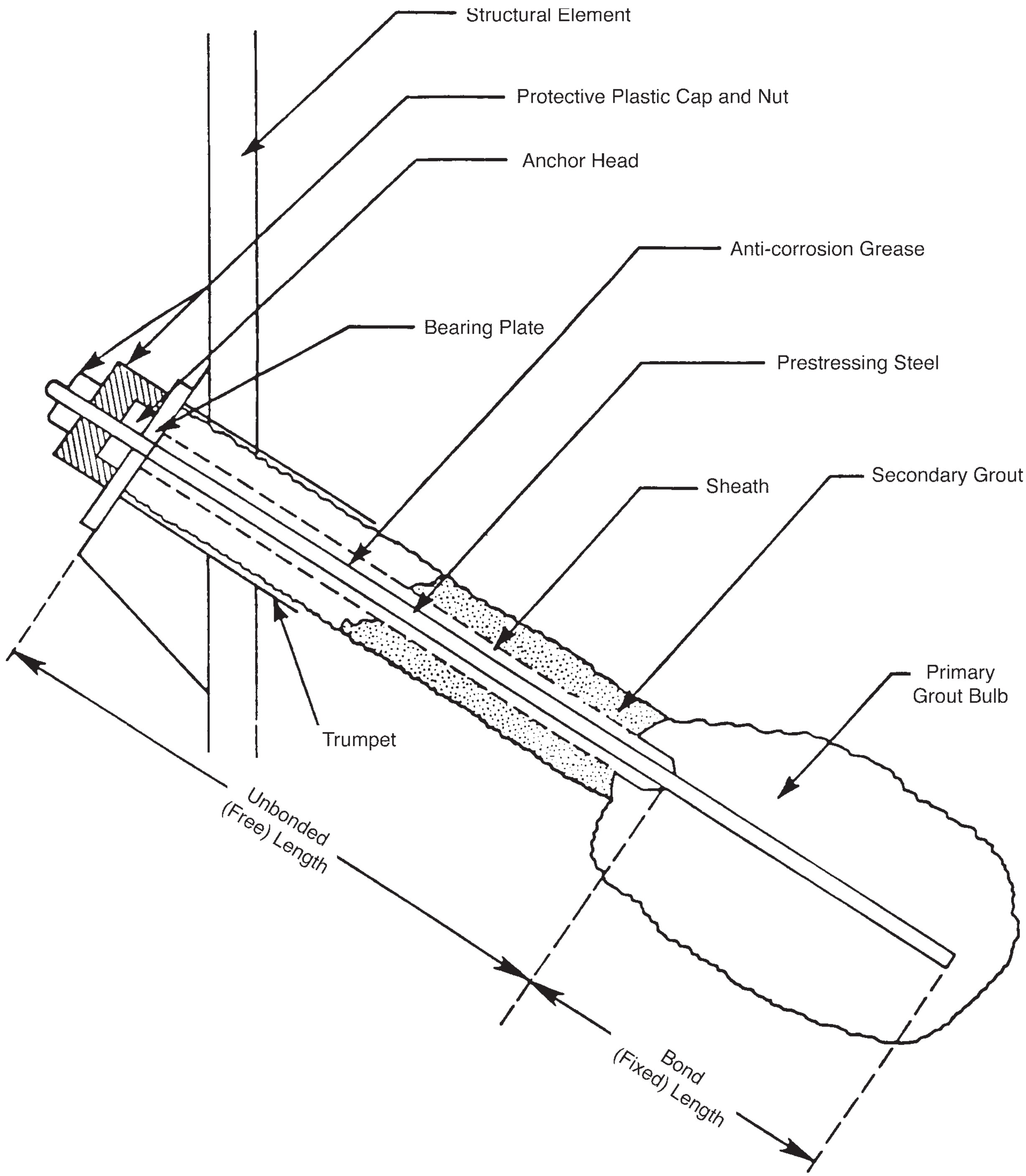



**Figure 11-1: Tieback Detail**

In addition to allowing higher wall designs, tiebacks serve another useful purpose. The only part of the system that projects beyond the wall into the excavation is the relatively small anchoring device. Hence, the system provides an open unrestricted work area in the excavation.

Tieback shoring designs require sophisticated engineering techniques and the calculations submitted by contractors and consultants can therefore be very complex.

For permanent structures, the Contractor is responsible for providing the tieback system which conforms to the design requirements shown on the plans and the testing requirements specified in the contract documents. The Contractor has the option of choosing which system will be installed. The record of readings from the Performance and Proof tests shall be documented by the Contractor and provided to the Engineer.

A Tieback Technical Specialist is available for consultation and is located in the Office of Structure Construction in Sacramento. In addition, the Staff Specialist for Earth Retaining Systems in the Office of Structure Design in Sacramento can be helpful in answering any questions that may arise.

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.

## Sequence of Construction

Sequence of tieback construction is as follows:

| SEQUENCE | DESCRIPTION |
|---|---|
| 1 | Drill the holes the required length 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 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.

## Tiedowns

Tiedowns are commonly used in footings on seismic retrofit projects. They are used to provide additional restraint against rotation of the footings and can be installed in both soil and rock. Tiedowns and tiebacks are constructed similarly but differ in their angle of reference. Tiedowns are installed with reference to a vertical zero angle, whereas tiebacks are referenced to a horizontal zero angle.

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

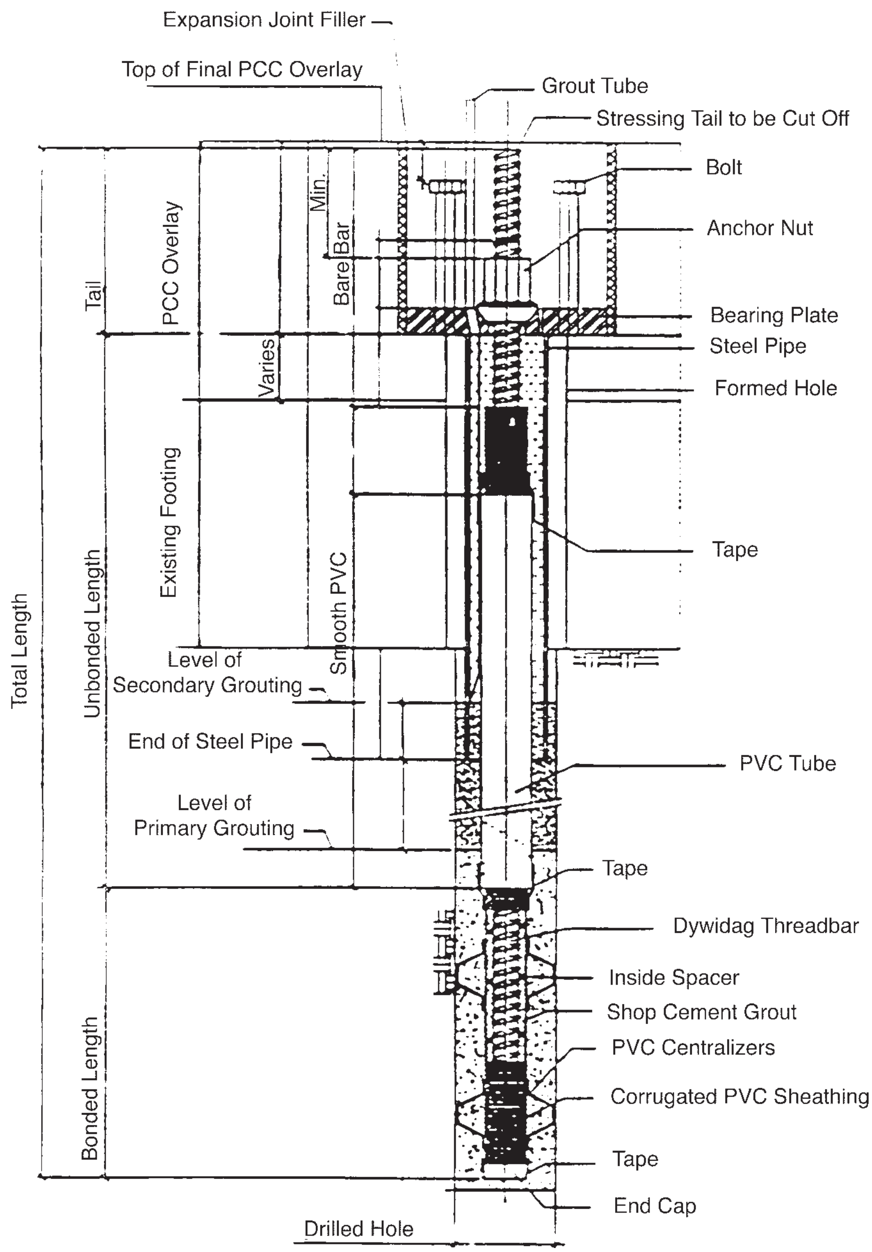



**Figure 11-2: Tiedown Anchor**

The Contractor is responsible for providing the tiedown anchor system which conforms to the design requirements shown on the plans and the testing requirements specified in the contract documents. The option of choosing which system will be installed is left to the Contractor. The record of readings from the Performance and Proof tests shall be documented by the Contractor and provided to the Engineer.

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 the 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.

## Testing of Tieback and Tiedown Anchors

The specific requirements for all testing will usually be provided in the contract Special Provisions, but the following is a general explanation of the required tests.

For both tiedowns and tiebacks, Performance tests are performed on a predetermined number of anchors, and Proof tests are done on all of the anchors. If the systems should fail, the contract Special Provisions provide for additional monitoring requirements. If they do not pass at that point, the Project Designer should be consulted.

## Performance Tests

A Performance test involves incremental loading and unloading of a production anchor to accurately verify that the design load may be safely carried, that the free length is as specified, and that the residual movement is within tolerable limits. As a minimum, the first two production anchors installed should be Performance tested. Do not wait until many anchors are installed before testing the first two anchors. The purpose of these tests is to verify the installation procedure selected by the Contractor before a large number of anchors are 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 was stated earlier, the contract plans and/or Special Provisions should specify the number of Performance tests be performed 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 maximum load value is reached. Each load shall be held constant for at least one minute or until measured deflection is negligible.

## General Acceptance Criteria

| CRITERIA | DESCRIPTION |
|---|---|
| 1 | The deflection at the anchor head should exceed 80% of the theoretical free length elongation for any test load. |
| 2 | The total deflection measured at the maximum test load does not exceed the theoretical elongation of a tendon length. |
| 3 | The creep movement does not exceed 0.08 inch during the final log cycle of time. |

### General Construction Control

| ITEM | DESCRIPTION |
|---|---|
| 1 | Mill certs should be provided for the steel tendons.<br>a) Check the steel for damage.<br>b) Ensure that grease completely fills the free length plastic tube.<br>c) Securely tape the bottom of the free length.<br>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). 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 top down. The nail bars are not pre-tensioned when they are installed. They are forced into tension as the ground deforms laterally in response to the loss of support caused by continued 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.

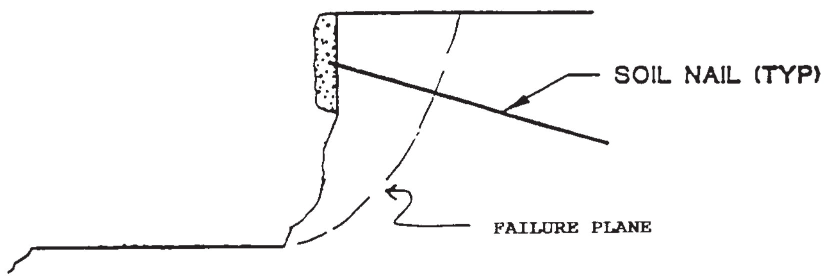



**Figure 11-3: Soil Nail Schematic**

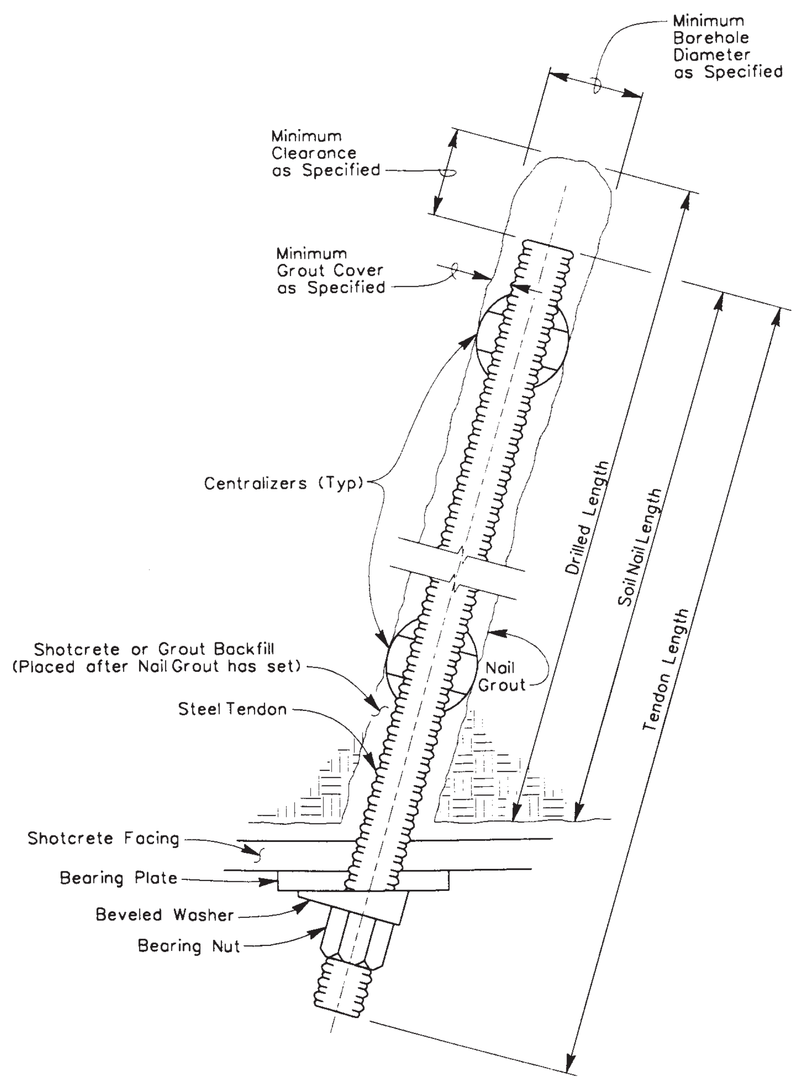



**Figure 11-4: Soil Nail Detail**

Soil nailing is a cost-effective alternative to conventional retaining wall structures when used in situations with ground formations suitable for nailing.

Common 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 that can maintain an open drilled hole for at least several hours.

## Construction Sequence

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 Engineer is responsible for reviewing and approving the Contractor's submittal of construction details. The Structure Foundations Branch of the

Office of Structural Foundations is available to assist the Structure Representative in the review of the Contractor's submittal. Prior to construction, the planned alignment, depth, and layout of the soil nails shall be checked in the field for any possible discrepancies.

## 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 to the Engineer for approval 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.

## Test Nails

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. Pullout 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 of the test nail is filled with grout. The location of test nails is determined by the Project Designer and shown on the plans. Refer to Figure 11-5 for a test nail detail.

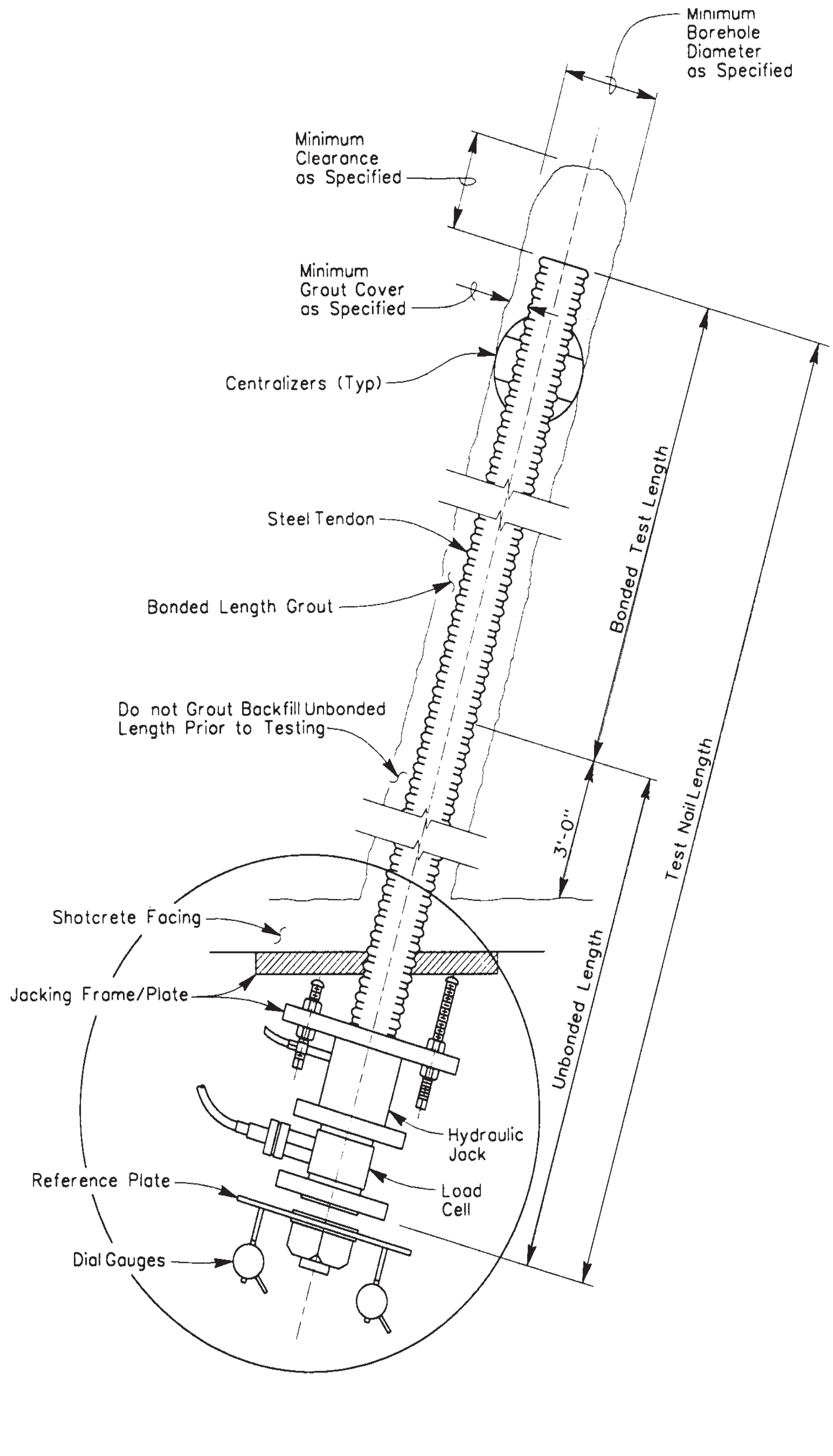



**Figure 11-5: Test Nail Detail**

## Testing

You should refer to your contract documents for specific test requirements for your project. The following is a general description of the required tests.

A pullout test consists of incrementally loading the test soil nail assembly to the maximum test load or the failure point, whichever occurs first. The failure point is the point where the movement of the test soil nail continues without an increase in the load or when the soil nail creep rate exceeds 0.08 inch between 6 and 60 minutes. 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 pullout test measuring the test load is applied to the test soil nail and measures the test soil nail end movement at each load. Each increment of load shall be applied in less than one minute and held for at least one minute but not more than two minutes. During the 10 minute load hold, the movement of the end of the soil nail shall be measured at 1, 2, 3, 4, 5, 6, and 10 minutes. After the 0.08 inch creep rate is established, the load is reduced to the final alignment load, and then the soil nail is unloaded. If a test soil nail fails to attain the maximum test load, one or all of the following procedures may be performed: (1) install and test additional test soil nails if the initial test results are believed to be in error, (2) determine if the cause is due to a variation of the soil conditions, installation procedure or materials, or (3) reevaluate the soil properties if differing soil conditions are encountered and redesign the wall if necessary. Any changes necessitated by failing tests shall be approved by the Project Designer.

## 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.

CHAPTER

# 12 Cofferdams and Seal Courses

## General

A cofferdam is a retaining structure, usually temporary in nature, which is used to support the sides of deep excavations where water is present. These structures generally consist of: (1) vertical sheet piling, (2) a bracing system composed of either wales and struts or prestressed tiebacks, and (3) a bottom seal course if required to seal out water.

Cofferdams are used in situations where adjacent ground must be supported against settlement or slides, and in construction of bridge piers and abutments in relatively shallow water.

## Contractor's Responsibility

Cofferdams usually fall under the category of temporary features 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 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 (Section 1811) and the provisions of Section 6705 of the Labor Code.

## Engineer's Responsibility

The Engineer is responsible for checking and approving the Contractor's drawings, and for making the decision as to whether a seal course should or should not be used. If the thickness of the seal course is not shown on the plans, the Engineer must determine the thickness of seal course concrete 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, 51-1.10, 51-1.22; and the following Bridge Construction Memos: 2-9.0 and 130-4.0.

## 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.

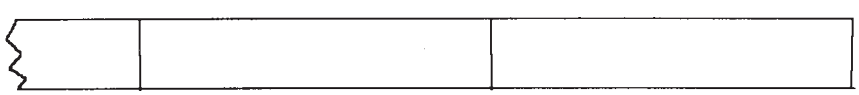


**Figure 12-1: Single Sheet Piling**

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.

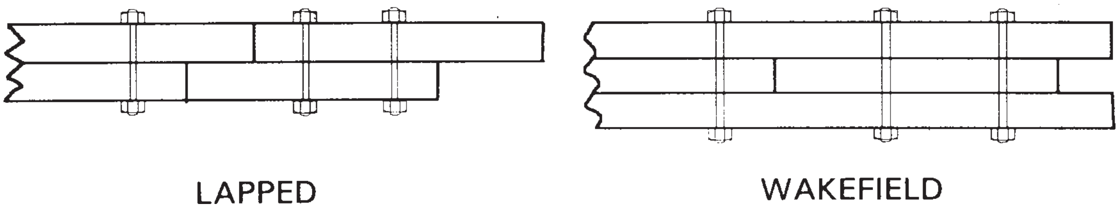



**Figure 12-2: Lapped and Wakefield Sheet Piling**

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

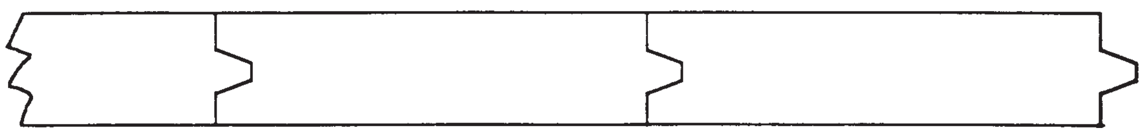


**Figure 12-3: Tongue and Groove Wood Sheet Piling**

Precast concrete sheet piles are normally used in a situation where the precast members are going to be incorporated into the final structure or are going to remain in place after they fulfill their purpose. For structure work, we normally do not encounter precast sheet piling. Precast concrete sheet piling 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 bars and hoops in much the same way as precast concrete bearing piles. This type of sheeting is not always perfectly watertight, but the spaces between the piles can be grouted.

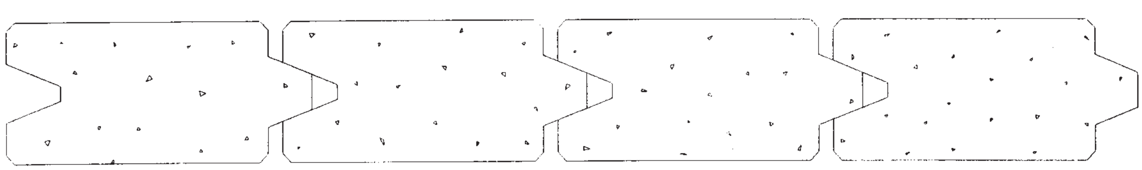


**Figure 12-4: Concrete Sheet Piling**

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 embedded in the pile.

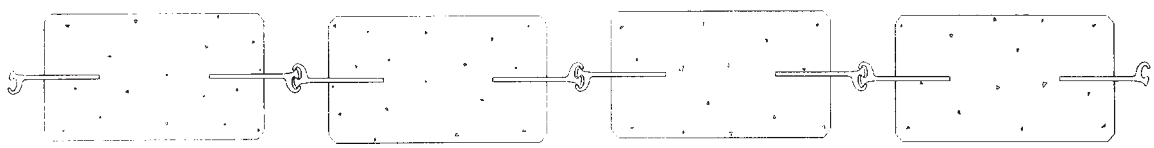


**Figure 12-5: Concrete Sheet Piling with Steel Interlocks**

Steel sheet piling is most commonly used and is available in a number of different shapes. The shape provides for bending strength and the interlock (connection between sheets) provides for alignment. Each steel company that manufactures sheet piling has its own form of interlock. The simplest shape is that known as the "straight-web" type. 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 areas where the horizontal thrusts are large.

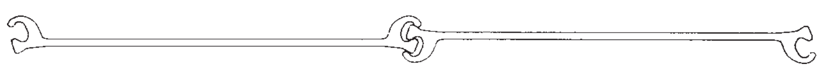


**Figure 12-6: Straight Section Steel Sheet Piling**

In order to provide a pile with a greater resistance to bending, the steel companies have developed a type known as the "arch-web" section, in which the center of the web is offset so as to provide a greater moment of inertia in the cross section. To provide for even greater stiffness, there is a "deep-arch" section. This is similar to the arch-web except that the offset in the web is increased considerably. A type known as the Z- Section has a stiffness considerably greater than that of the "deep-arch".

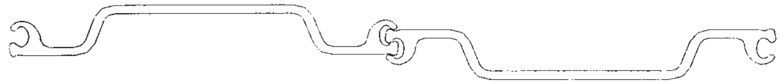


**Figure 12-7: Arch-Web Steel Sheet Piling**

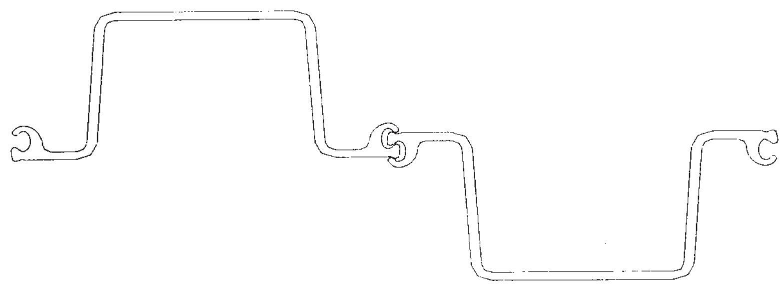


**Figure 12-8: Deep-Arch Steel Sheet Piling**

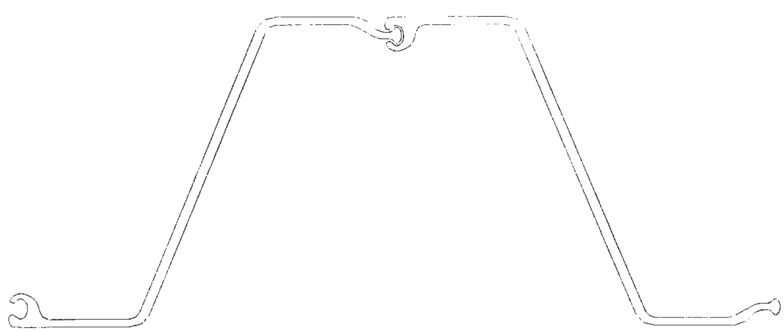


**Figure 12-9: Z-Section Steel Sheet Piling**

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 is intended to be used. The straight-web is comparatively flexible so that it requires a considerable amount of bracing when subject to a large horizontal thrust. However, its size allows it be used in close quarters, where a deep-arch or Z-Section will not fit.

The composition of the bracing system inside the cofferdam will be dependent upon the forces that are present, the availability of materials, and the costs connected with the system. Wood and steel are the normal materials used. Prestressed tiebacks are often used in large land cofferdams where a system of cross-bracing is impractical.

## Excavation

Section 19-3.04 of the *Standard Specifications* states, "…that excavations shall be completed to bottom of footings before driving piles". As in many other areas of our work, there are times when engineering judgement should be used depending on type of soil, amount of excavation required, type of pile, and depth below water surface. Normally, excavation would be by submerged clamshell, with the elevations being checked by sounding. In the case of pile foundations, it is often advisable to over-excavate a predetermined amount to compensate for heave of the material caused by pile driving displacement. This is done to eliminate the need for excavation after driving. If excavation is needed, care is required not to damage any of the driven piles.

## Seal Course

Following the installation of the cofferdam, the footing can be excavated and piles driven. Usually the footing area must also be dewatered. Depending on the volume of water present, this can be achieved by pumping. Otherwise, a seal course may be necessary. 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*.

As the name implies, a properly constructed seal course seals the entire bottom of a cofferdam and prevents subsurface water from entering the cofferdam. In so doing, it permits construction of footings and columns or other facilities in the dry. The seal course is a concrete slab placed underwater and constructed thick enough so that it's weight is sufficient to resist uplift from hydrostatic forces. In terms of it's importance to the designed structure, the seal course normally has no structural significance.

Information pertaining to the seal course for a project may be found in the contract plans. Additional information may be found in the RE Pending File. The decision as to thickness of seal course required, or whether the seal course is to be eliminated, rests with the Structure Representative. This decision is based on conditions encountered on the jobsite. The contract plans will also contain provisions for adjusting footing elevations if seal courses are eliminated. In usual field practice, this decision is not a difficult one to make. In most cases when water is not present the need for a seal course is clearly not there. Additional information about seal courses can be found in Bridge Construction Memo 130-4.0.

# Tremie Concrete

Tremie concrete is a name given to the method of placing concrete under water through a pipe or tube, which is called a tremie. The tremie can either be rigid or flexible. Concrete flow can be either by gravity with a hopper located at the top of the pipe, or by direct connection to a concrete pump.

The purpose of this equipment is to enable continuous placement of monolithic concrete underwater without creating turbulence. 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 maintained by continuing to charge the pipe. The operation must continue until completion. The tremie pipe is immersed in concrete during placement. Some factors which 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 7 sacks of cement per cubic yard. |
| 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. |

Of the various plug devices, the inflated rubber ball is about the most practical. However, a tip plug can cause long tremie pipes to float.

## Seal Course Inspection

In addition to the usual pre-pour matters, such as access and suitability or adequacy of equipment, sufficient soundings should be taken to verify elevations. Particular care should be given to the perimeter of the cofferdam and the pile locations. Soundings can be accomplished using a flat plate of suitable size and weight on the end of a rod or rag tape.

This device can be used to not only determine elevations, but, to some extent, can 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 thickness throughout and the absence of excessive low spots. The *Standard Specifications* require a minimum cure period of 5 days before dewatering.

## Thickness of Seal Course

A chart for determination of seal course thickness 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. 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 130-4.0 and Bridge Design Aid “Seal Course” included in Appendix I.

## 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 for a moderately deep cofferdam. At each stage intermediate bracing systems are installed before proceeding deeper. Depending on the particular design, these internal braces restore the stability of the system.

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 occur along the joints between adjacent sheets. Sawdust, cement, or other material can be used to plug these, and this type of leak is ordinarily not a problem. This is usually done by dropping the material into the water adjacent to the leaking sheets. 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 would be helpful in keeping the work area reasonably dry. Obviously, sumps should be located outside the footing limits.

Prior to proceeding with footing work, all high spots in the seal have to be removed. All scum, laitence, 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 when the seal is placed.

## Safety

Cofferdam work presents safety problems unique to this type of construction. Among them are limited access, limited work areas, damp or wet footing, and deep excavations. Provisions must be made for safe access in terms of adequate walkways, rails, ladders, or stairs into and out of the lower levels.

The work may be within a waterway, in which case additional safety regulations may apply. These would include provisions for flotation devices, boats, warning signals, and suitable means for a rapid exit. The Construction Safety Orders should be consulted for specific requirements.

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23) Standard Test Methods. State of California, Department of Transportation, 1978 (with revisions).

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25) Trenching and Shoring Manual. State of California, Department of Transportation, 1990 (with revisions).

# Appendices

# APPENDIX

# A Foundation Investigations

Table of Contents

Please note that these conversion tables are approximate. They can be used by characterizing the soil as being either predominately granular or cohesive. If possible, the conversion of the penetration index (N Value) should be checked by performing labatory or in-situ 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, $\phi$ | | 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.

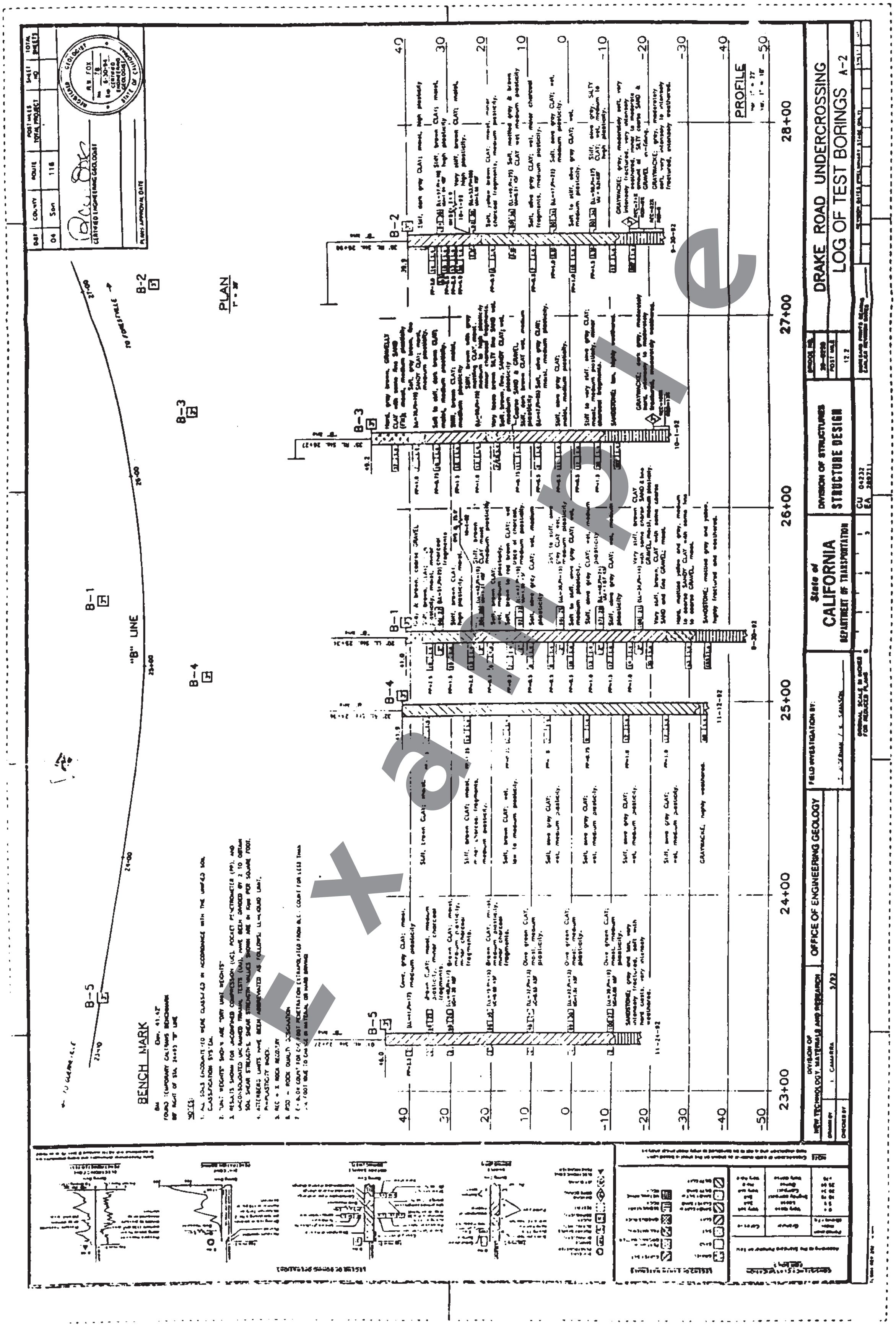
# COHESIVE SOILS

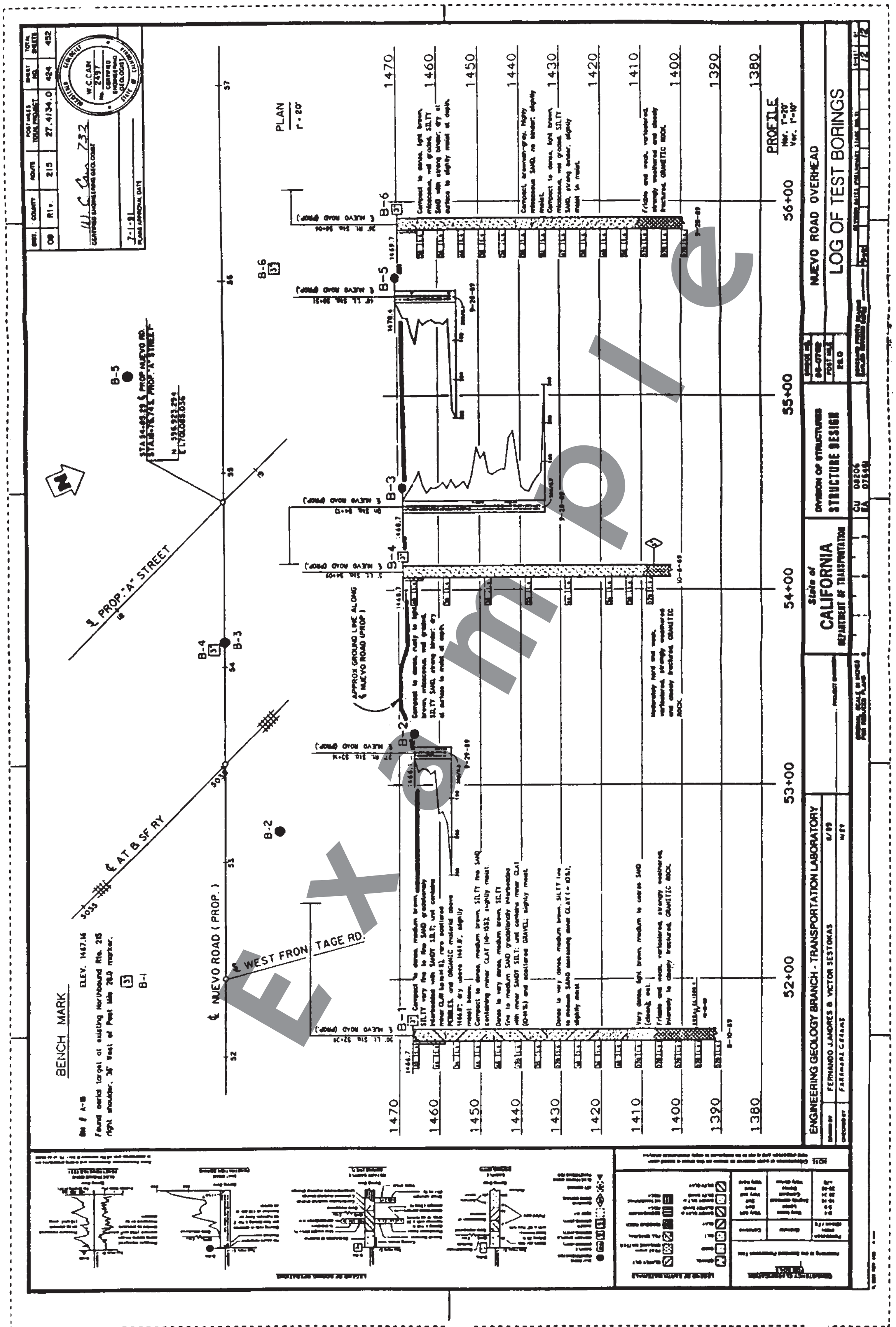
| CONSISTENCY | VERY SOFT | | SOFT | | MEDIUM | | STIFF | | VERY STIFF | | HARD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| $q_u$ = unconfined comp. strength (PSF) | | 500 | | 1000 | | 2000 | | 4000 | | 8000 | |
| Standard Penetration Resistance, N = Blows/Ft * | | 2 | | 4 | | 8 | | 16 | | 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.


DRAKE ROAD UNDERCROSSING
LOG OF TEST BORINGS A-2
PLAN
PROFILE
BENCH MARK
"B" LINE
B-1
B-2
B-3
B-4
B-5
State of
CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES
STRUCTURE DESIGN
OFFICE OF ENGINEERING GEOLOGY
23+00
24+00
25+00
26+00
27+00
28+00
Example


Example
NUEVO ROAD OVERHEAD
LOG OF TEST BORINGS
ENGINEERING GEOLOGY BRANCH - TRANSPORTATION LABORATORY
State of CALIFORNIA DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES STRUCTURE DESIGN
BENCH MARK
PLAN 1" = 20'
PROFILE
℄ NUEVO ROAD ( PROP. )
℄ WEST FRONTAGE RD.
℄ AT & SF RY
℄ PROP. "A" STREET
B-1
B-2
B-3
B-4
B-5
B-6
52+00
53+00
54+00
55+00
56+00
1470
1460
1450
1440
1430
1420
1410
1400
1390
1380

**CONSISTENCY CLASSIFICATION FOR SOILS**

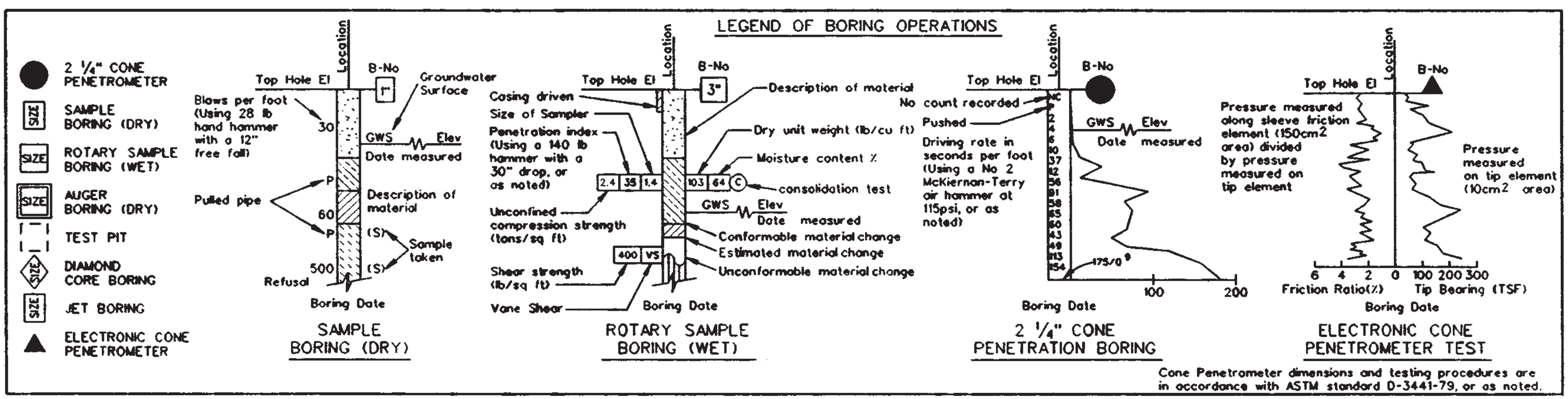
According to the Standard Penetration test

| Penetration index (Blows/Ft) | Granular | Cohesive |
|---|---|---|
| 0-4 | Very loose | Very Soft |
| 5-9 | Loose | Soft |
| 10-19 | Slightly compact | Stiff |
| 20-34 | Compact | Very stiff |
| 35-69 | Dense | Hard |
| >70 | Very dense | Very hard |

**LEGEND OF EARTH MATERIALS**

- CLAY
- SILT
- SAND
- GRAVEL
- SANDY CLAY / CLAYEY SAND
- SANDY SILT / SILTY SAND
- SILTY CLAY
- CLAYEY SILT
- ORGANIC MATTER AND/OR PEAT
- SEDIMENTARY ROCK
- METAMORPHIC ROCK
- IGNEOUS ROCK

NOTE:
Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis



## CONSISTENCY CLASSIFICATION FOR SOILS

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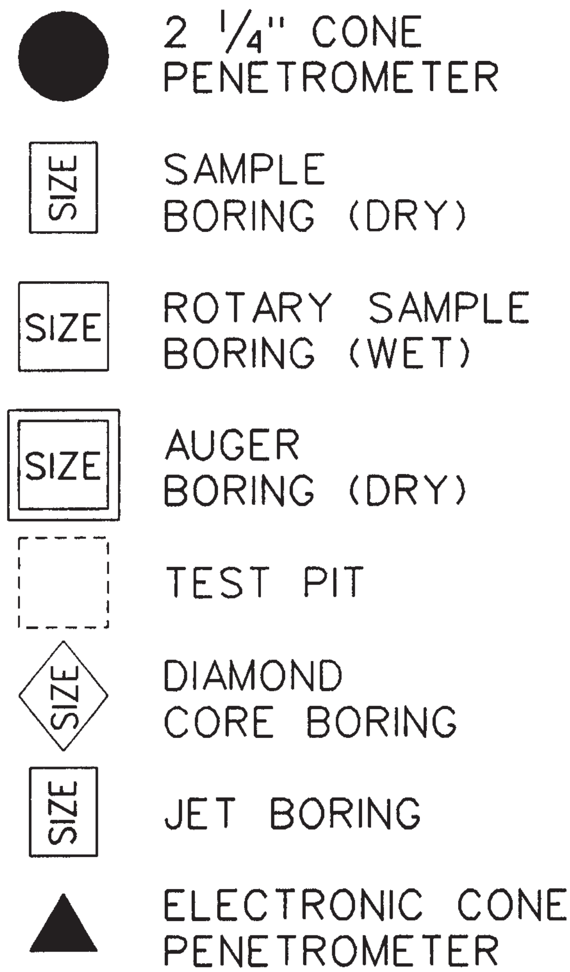
| 10-19 | Slightly compact | Stiff |
| 20-34 | Compact | Very stiff |
| 35-69 | Dense | Hard |
| >70 | Very dense | Very hard |

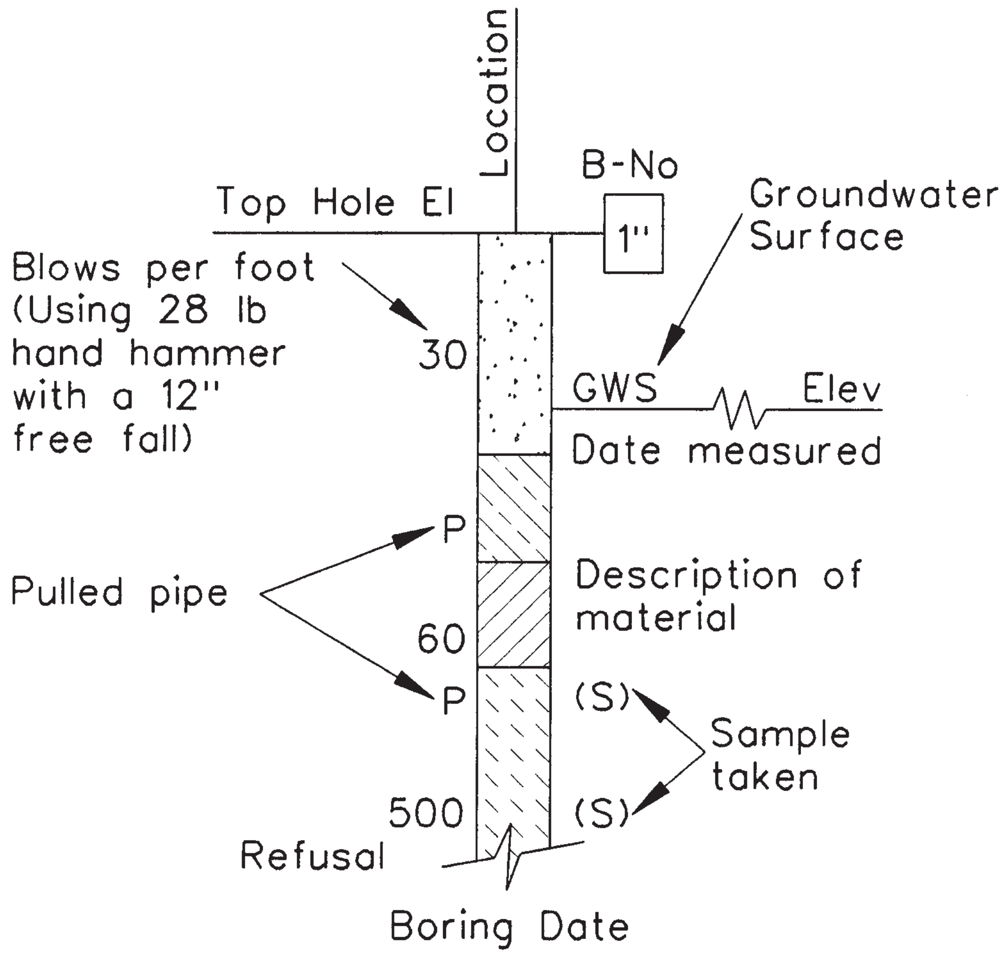
## LEGEND OF EARTH MATERIALS

| | |
|---|---|
| CLAY | CLAYEY SILT |
| SILT | ORGANIC MATTER AND/OR PEAT |
| SAND | SEDIMENTARY ROCK |
| GRAVEL | METAMORPHIC ROCK |
| SANDY CLAY<br>CLAYEY SAND | IGNEOUS ROCK |
| SANDY SILT<br>SILTY SAND | |
| SILTY CLAY | |

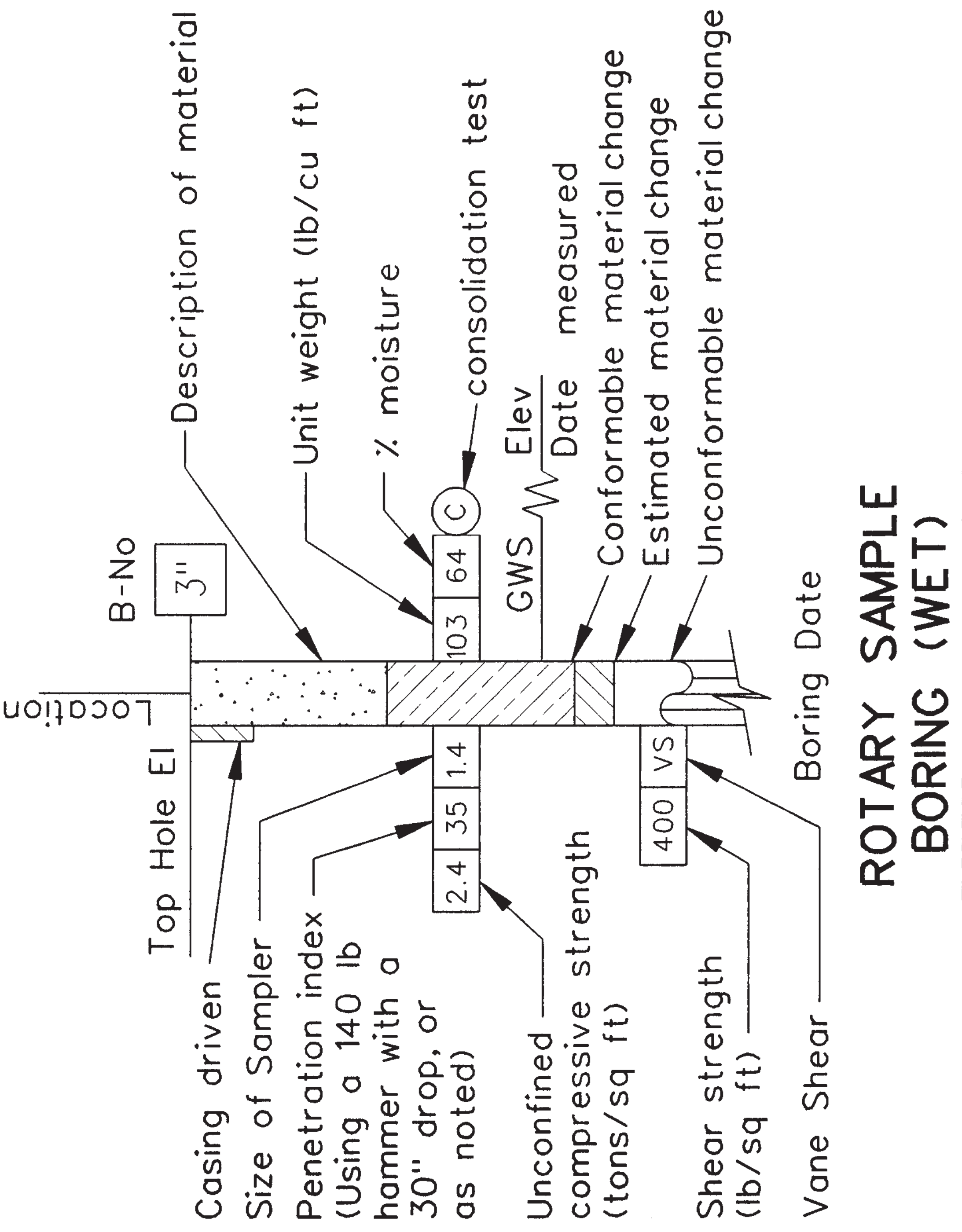
NOTE:

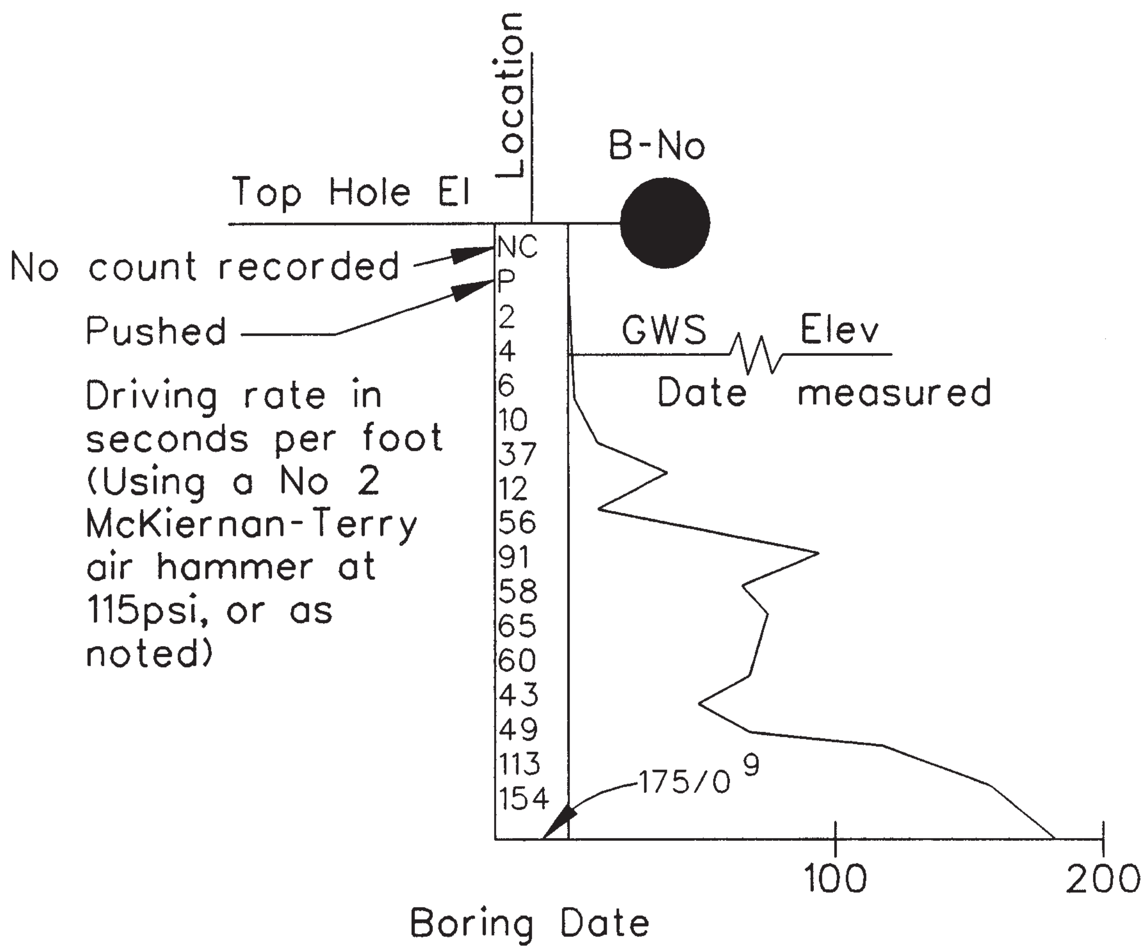
Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis


2 1/4" CONE PENETROMETER
SIZE
SAMPLE BORING (DRY)
SIZE
ROTARY SAMPLE BORING (WET)
SIZE
AUGER BORING (DRY)
TEST PIT
SIZE
DIAMOND CORE BORING
SIZE
JET BORING
ELECTRONIC CONE PENETROMETER

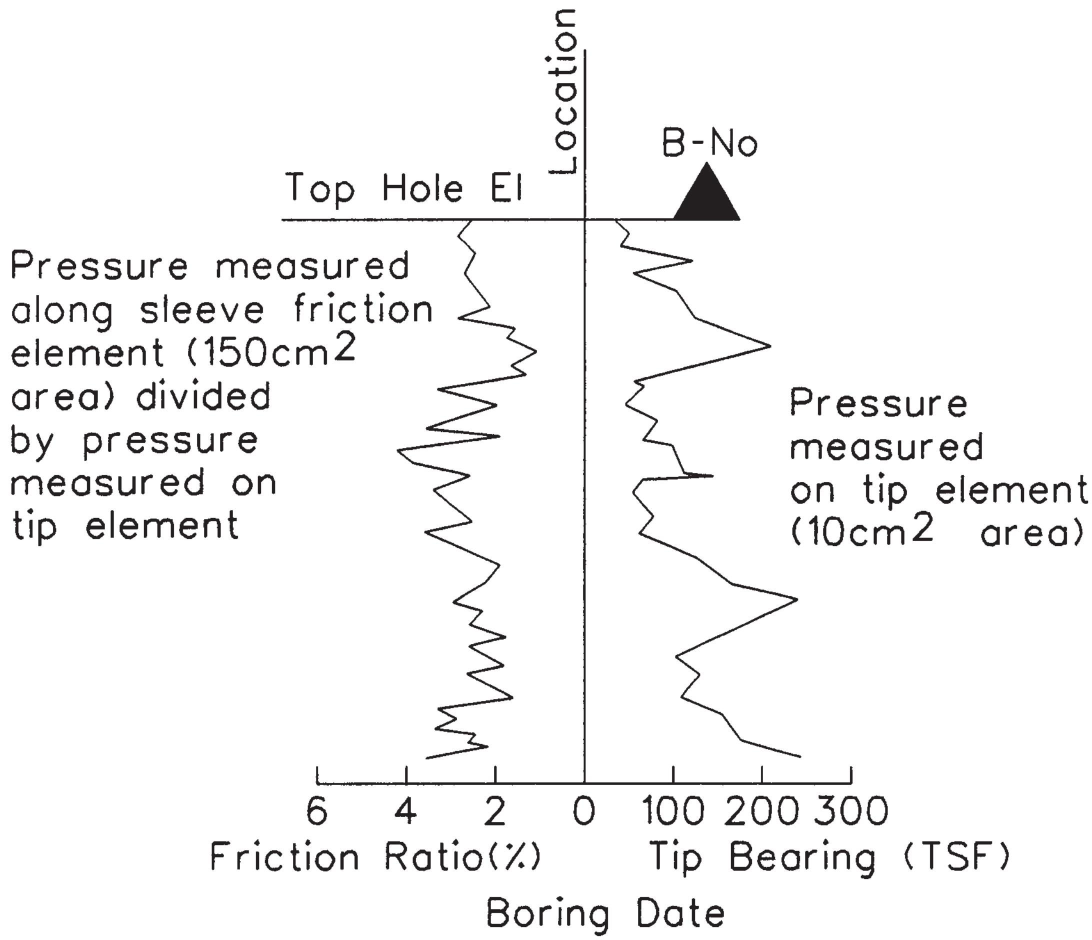



**SAMPLE BORING (DRY)**


Top Hole El
Location
B-No
3"
Casing driven
Size of Sampler
Penetration index (Using a 140 lb hammer with a 30" drop, or as noted)
Unconfined compressive strength (tons/sq ft)
Shear strength (lb/sq ft)
Vane Shear
2.4
35
1.4
400
VS
103
64
C
GWS
Description of material
Unit weight (lb/cu ft)
% moisture
consolidation test
Elev
Date
measured
Conformable material change
Estimated material change
Unconformable material change
Boring Date
ROTARY SAMPLE
BORING (WET)




**2 1/4" CONE PENETRATION BORING**




**ELECTRONIC CONE PENETROMETER TEST**

Cone Penetrometer dimensions and testing procedures are in accordance with ASTM standard D-3441-79, or as noted.

**State of California** **Business, Transportation and Housing Agency**

# Memorandum

**To** : MR. M. W. HORN, Acting Chief
Office of Structure Design

Attention Mr. E. K. Thorkildsen
Design Section 6

**Date** : June 2, 1994

**File No.** : 05-SBt-156-7.3/R15.2
05-027101

San Benito River Bridge
Bridge No. 43-0044

**From** : **DEPARTMENT OF TRANSPORTATION** – 227-7206
**Division of New Technology, Materials and Research**
**Office of Engineering Geology**

**Subject** : Foundation Investigation

A foundation investigation was conducted for the San Benito River Bridge, Bridge No. 43-0044, by the Office of Engineering Geology . The San Benito River Bridge is part of the new alignment of Route 156, Hollister Bypass. The investigation included six 45.7 m (150-foot) deep mud rotary borings and seven 57 mm (2-1/4 inch) cone penetrometer borings drilled in November/December 1993. Also included are three mud rotary borings and one electronic cone penetrometer test performed in September/October 1992 by District 5. Additional borings (beyond the bridge site) are included in District 5 "Materials Information" (MI) package.

**Geology**

The general geology of the site is Cenozoic alluvial deposits of the San Benito River. The subsurface materials are interbedded clays, silts, sands, gravels and scattered small cobbles. The following list gives generalized descriptions and elevations of the soils at the San Benito River Bridge site. Please refer to the Log of Test Borings for better detail.

| Generalized elevations | Generalized Soil Descriptions |
|---|---|
| 71.6 m (235 ft) - 68.6 m (225 ft) | slightly compact to compact sand & gravel |
| 68.6 m (225 ft) -57.9 m (190 ft) | soft to stiff silty clay, slightly compact to compact clayey silt and compact to dense sand |
| 57.9 m (190 ft) - 45.7 m (150 ft) | compact to very dense sand and gravel with some small cobbles (up to 3-6 inch) |
| 45.7 m (150 ft) -30.5 m (100 ft) | interbedded compact to hard clay and dense to very dense silt and sand |
| 30.5 m (100 ft) - 24.4 m (80 ft) | very dense sand, silt and hard to very hard silty clay |

**Ground Water**

Ground water was encountered in the San Benito River channel as shown on the Log of Test Borings. The highest groundwater level measured was elevation 66.8 m (219 ft) in B-2 (near Bent 4). No ground water was encountered at Abutment 1 which is located outside the active river channel. The ground water elevation and surface flow are extremely variable due to the hydrology of the river basin, the seasonal rainfall, and the water withdrawal by the local water district.

Mr. M. W. Horn
June 2, 1994
Page 2

Bridge No. 43-0044

Bent 4 is located in the live channel of the San Benito River. Water was flowing in the channel in November/December 1993, 0.6-0.9 m (2-3 feet ) deep at Bent 4. Surface water may need to be diverted during construction.

**Scour Depth**

The San Benito River is actively degrading, producing scour around existing bridges. Approximately one mile upstream from the proposed San Benito River Bridge site, Bridge No. 43-07 (along Highway 152), has a history of scour and embankment erosion problems. Three years after the bridge was built (1953), tetrahedron slope protection was necessary to protect the abutments. Beginning in 1978, man-made alterations to the river channel caused the river to degrade exposing the bent piles as much as 1.5 m (5 feet) below the pile cap (the City of Hollister made channel improvements upstream for the sewer treatment plant in 1978, and aggregate mining in the riverbed commenced downstream in 1989). Maintenance on the embankments and bents requires continuous addition of rip-rap and backfill.

Man-made alterations to the river will strongly influence the amount of scour and degradation of the channel. Aggregate mining will occur upstream and downstream of the San Benito River Bridge. Therefore, predicted scour depth elevations (due to mining) provided by Preliminary Investigations will be used for pile cut-off elevations. The Division of Structures, Preliminary Investigations Section, has predicted scour depth elevations to 56.7 m (186 ft) at the bent locations and 61.3 m (201 ft) at the abutments.

The historical scour depth elevation in the San Benito River channel, as determined from soil borings, extends to elevation 68.6 m (225 ft) at the channel edges and elevation 67.1 m (220 ft) in the active channel. The current design locates Abutment 1 24.4 m (80 ft) west of the channel edge with the bottom of pile cap elevation at 70.4 m (231 ft). Due to expected river alterations and subsequent channel erosion, the soil beneath Abutment 1 could be eroded, exposing the pile cap. Therefore, the bottom of pile cap at Abutment 1 should be constructed below the historical scour depth elevation of 68.6 m (225 ft). Rock Slope Protection could protect the pile cap, but future maintenance may be required. Abutment 6 should be constructed below historical scour depth elevation 67.1 m (220 ft). Abutment 6 is located in the active river flood plain with the bottom of pile cap placed at elevation 65.5 m (215 ft).

Rock Slope Protection (RSP) shall be placed at both Abutments 1 and 6. At Abutment 6, RSP shall extend to elevation 64.0 m (210 ft), 1.5 m (5 ft) below the bottom of pile cap elevation. At Abutment 1, RSP shall extend to elevation 67.1 m (220 ft), 3.4 m (11 ft) below the proposed bottom of pile cap elevation, and 1.5 m (5 ft) below the historical scour depth elevation at channel edge.

**Fault and Seismic Data**

The site is located near two major active faults. The nearest known potentially active fault is the Sargent fault (M=6.75). The Sargent fault is located 1.9 km (1.2 miles) northwest of the site. The Calaveras fault (M=7.5) is located 3.1 km (1.9 miles) to the east. The predicted maximum credible horizontal bedrock acceleration is 0.7g from the Calaveras fault. Use Design Force Coefficient Curve " >47.7" (>150).

Mr. M. W. Horn
June 2, 1994
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Bridge No. 43-0044

**Settlement Periods**

District 5 Materials Laboratory has analyzed the foundation soil settlements at the approach embankments for the San Benito River Bridge. Abutment 1 is anticipated to have 17.8 mm (0.7 inch) of long-term settlement requiring a 30-day fill delay prior to pile installation. Abutment 6 is not anticipated to have long-term settlement and no fill delay is required. Please contact Ron Richman, District 5 DME for further details.

**Corrosion**

Preliminary test results show that the San Benito River site is a non-corrosive environment.

**Foundation Recommendations**

Cast-In-Drilled-Hole (CIDH) piles using slurry displacement are recommended for support of the structure shown on the "General Plan" dated 12/14/93. Specified pile tip elevations for 3.05 m (120-inch) and 0.91 m (36-inch) diameter CIDH piles are shown in Tables 1 and 2. The design loads were supplied by Structure Design and are noted in the tables.

Due to future aggregate mining in the river, the pile cut-off elevations correspond to the predicted scour elevations as determined by Preliminary Investigations (report dated 11/23/93). The piles are considered to be unsupported by soil above the predicted scour elevations of 56.7 m (186 ft) at the bent supports and 61.3 m (201 ft) at the abutments.

The bent support design loads, given in Table 1, do not include the pile weight [1673 kN (188 tons)] from the column bottom to the cut-off elevation [elevation 66.4 m (218 ft) to 56.7 m (186 ft)]. Therefore, the specified pile tip elevations for Bents 2, 3, 4, and 5 were determined by adding 1673 kN (188 tons) to the design load.

Table 1

3.05 m (120-inch) CIDH Pile Data (2)
Bent Supports

| Location | Diameter | Design Loading (1) (Service Load) | Pile Cut-off Elevation (3)(4) | Specified Tip Elevation |
|---|---|---|---|---|
| Bent 2 | 3.05 m (120-inch) | 15040 kN (1690 tons) | 56.7 m (186 ft) | 25.3 m (83 ft) |
| Bent 3 | 3.05 m (120-inch) | 15580 kN (1750 tons) | 56.7 m (186 ft) | 26.8 m (88 ft) |
| Bent 4 | 3.05 m (120-inch) | 15580 kN (1750 tons) | 56.7 m (186 ft) | 25.6 m (84 ft) |
| Bent 5 | 3.05 m (120-inch) | 15040 kN (1690 tons) | 56.7 m (186 ft) | 26.5 m (87 ft) |

(1) The ultimate compressive capacity is 2x design load.

(2) Method of support is friction and end bearing.

(3) CIDH piles are considered unsupported by the soil above the cut-off elevation.

(4) No casing is to remain below the cut-off elevation

Mr. M. W. Horn
June 2, 1994
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Bridge No. 43-0044

Table 2

0.91 m (36-inch) CIDH Pile Data (2)
Abutment Supports

| Location | Diameter | Design Loading (Service Load) (1) | Pile Cut-off Elevation (4) | Specified Tip Elevation (3) |
|---|---|---|---|---|
| Abutment 1 (5) | 0.91 m (36-inch) | 1340 kN (150 tons) | 61.3 m (201 ft) | 51.8 m (170 ft) |
| Abutment 6 (6) | 0.91 m (36-inch) | 1340 kN (150 tons) | 61.3 m (201 ft) | 51.8 m (170 ft) |

(1) The ultimate compressive capacity is 2x design load.

(2) Method of support is skin friction.

(3) Battered CIDH piles shall not be used.

(4) CIDH piles are considered unsupported by the soil above the cut-off/scour depth elevation.

(5) 30-day fill delay required for Abutment 1. See District 5 Materials report.

(6) No fill delay required for Abutment 6.

**Constructability**

Slurry displacement is recommended for constructing the CIDH piles at all support locations. Caving and loss of slurry mud may be a problem, especially in the sand and gravel layer between elevation 57.9 m and 45.7 m (190-150 ft – generalized elevations – refer to Log of Test Borings for detailed elevations). Jetting may create large voids especially in the sand layers; therefore, jetting should be avoided. Due to shallow tip elevations and low ground water elevations at Abutments 1 and 6, slurry displacement may not be necessary. The tip elevation at Abutment 6 extends 1.5 m (5 feet) into ground water as measured on November 30, 1993 (ground water could be higher during construction).

If temporary casing is used during construction, **all casing below the scour depth/cut-off elevation shall be removed**. The piles will not achieve design load if casing remains below the scour depth.

LAUREL S. JENSEN
Associate Engineering Geologist

R. W. FOX, C.E.G. No. 78
Senior Engineering Geologist

cc: Preliminary Report
R.E. Pending File
DBarlow - Specs & Estimates
District 5 (2)
ELeivas - OEG
Engr. Geology (4)

# APPENDIX

# B Contract Administration

## Table of Contents

## ENGLISH TO METRIC (SI) CONVERSION FACTORS

| Quantity | from English | to Metric (SI) | Multiply by | Quick Conversions |
|---|---|---|---|---|
| Mass | lb | kg | 0.453592 | 1 lb (mass) = 0.5 kg |
| Force | lb | N | 4.44822 | 1 lb (force) = 4.5 N |
| | kip | kN | 4.44822 | 1 kip (force) = 4.5 kN |
| Force/unit length | plf | N/m | 14.5939 | 1 plf = 14.5 N/m |
| | klf | kN/m | 14.5939 | 1 klf = 14.5 kN/m |
| Pressure, stress, modulus of elasticity | psf | Pa | 47.8803 | 1 psf = 48 Pa |
| | ksf | kPa | 47.8803 | 1 ksf = 48 kPa |
| | psi | kPa | 6.89476 | 1 psi = 6.9 kPa |
| | ksi | MPa | 6.89476 | 1 ksi = 6.9 MPa |
| Length | inch | mm | 25.1 | 1 in = 25 mm |
| | foot | m | 0.3048 | 1 ft = 0.3 m |
| | | mm | 304.8 | 1 ft = 300 mm |
| Area | sq inch | sq mm | 645.16 | 1 sq in = 650 sq mm |
| | sq foot | sq m | 0.09290304 | 1 sq ft = 0.09 sq mm |
| | sq yard | sq m | 0.83612736 | 1 sq yd = 0.84 sq mm |
| Volume | cubic inch | cu mm | 16386.064 | 1 cu in = 16,400 cu mm |
| | cubic foot | cu m | 0.0283168 | 1 cu ft = 0.03 cu m |
| | cubic yard | cu m | 0.764555 | 1 cu yd = 0.76 cu m |
| Moment | lb-ft | N-m | 1.3558 | 1 lb-ft = 1.4 N-m |
| | lb-in | N-m | 0.11298 | 1 lb-in = 0.11 N-m |
| Energy | ft-lb | Joule | 1.3558 | 1 ft-lb = 1.4 J |
| Unit Weight | lb/cu ft | kN/m3 | 0.1572 | 1lb/cu ft = 0.16 kN/ m3 |
| | lb/cu in | kN/m3 | 271.43 | 1lb/cu in = 271 kN/m3 |
| Moment of Inertia | in4 | mm4 | 416,620 | 1 in4 = 420,000 mm4 |
| Coefficient of permeability | ft/min | m/min | 0.3048 | 1ft/min = 0.3 m/min |
| | ft/sec | m/sec | 0.3048 | 1 ft/sec = 0.3 m/sec |
| | in/sec | mm/sec | 25.4 | 1 in/sec = 25 mm/sec |
| Coefficient of consolidation | sq in/sec | sq m/yr | 20,346 | 1 in2/sec=20,300 m2/yr |
| | sq in/sec | sq cm/sec | 6.452 | 1 in2/sec = 6 cm2/sec |
| | sq ft/sec | sq cm/sec | 929.03 | 1 ft2/sec= 929 cm2/sec |

Notes:

1. In a "soft" conversion, an English measurement is mathematically converted to its exact metric conversion.
2. In a "hard" conversion, a new rounded, metric number is created that is convenient to work with and remember.
3. Avoid using centimeter for length.
4. The pascal (Pa) is the unit for pressure and stress (Pa=N/sq.m)
5. Structural calculations should be shown in MPa or kPa.

**DEFORMED REINFORCING BARS**
**Metric (SI) – AASHTO M31M/ASTM A615M (Grade 420)**
**English – AASHTO M31/ASTM A615 (Grade 60)**

| Bar Size Designation No. | | Nominal Dimensions | | | | Diameter | |
|---|---|---|---|---|---|---|---|
| | | Area | | Weight | | | |
| SI | English | sq mm | sq in | kg/m | lb/ft | mm | in |
| 10 | 3 | 71 | 0.11 | 0.560 | 0.736 | 9.5 | 0.375 |
| 13 | 4 | 129 | 0.20 | 0.944 | 0.668 | 12.7 | 0.500 |
| 16 | 5 | 199 | 0.31 | 1.552 | 1.043 | 15.9 | 0.625 |
| 19 | 6 | 284 | 0.44 | 2.235 | 1.502 | 19.1 | 0.750 |
| 22 | 7 | 387 | 0.60 | 3.042 | 2.044 | 22.2 | 0.875 |
| 25 | 8 | 510 | 0.79 | 3.973 | 2.670 | 25.4 | 1.000 |
| 29 | 9 | 645 | 1.00 | 5.060 | 3.400 | 28.7 | 1.128 |
| 32 | 10 | 819 | 1.27 | 6.404 | 4.303 | 32.3 | 1.270 |
| 36 | 11 | 1006 | 1.56 | 7.907 | 5.313 | 25.8 | 1.410 |
| 43 | 14 | 1452 | 2.25 | 11.380 | 7.650 | 43.0 | 1.693 |
| 57 | 18 | 2581 | 4.00 | 20.240 | 13.600 | 57.3 | 2.257 |

## Comparison of Bar Reinforcement Currently Used with Metric Bars as Contained in the Current ASTM Specifications and Metric Bars as CRSI has Proposed to ASTM

| A615M or A706M Bars in Current ASTM | | | | | A615 or A706 Bars Currently Used | | | | | Metric Bars as Proposed by CRSI | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| No. | Diam in | Diam mm | Area in² | Area mm² | No. | Diam in | Diam mm | Area in² | Area mm² | No. | Diam in | Diam mm | Area in² | Area mm² |
| 10 | 0.444 | 11.3 | 0.16 | 100 | 3 | 0.375 | 9.5 | 0.11 | 71.3 | 10 | 0.394 | 10.0 | 0.12 | 78.5 |
| 15 | 0.628 | 16.0 | 0.31 | 200 | 4 | 0.500 | 12.7 | 0.20 | 126.7 | 13 | 0.512 | 13.0 | 0.21 | 132.7 |
| 20 | 0.770 | 19.5 | 0.47 | 300 | 5 | 0.625 | 15.9 | 0.31 | 197.9 | 16 | 0.630 | 16.0 | 0.31 | 201.1 |
| 25 | 0.993 | 25.2 | 0.78 | 500 | 6 | 0.750 | 19.1 | 0.44 | 285.0 | 20 | 0.787 | 20.0 | 0.49 | 314.2 |
| 30 | 1.175 | 29.9 | 1.09 | 700 | 7 | 0.875 | 22.2 | 0.60 | 387.9 | 25 | 0.984 | 25.0 | 0.76 | 490.0 |
| 35 | 1.405 | 35.7 | 1.55 | 1000 | 8 | 1.000 | 25.4 | 0.79 | 506.7 | 30 | 1.181 | 30.0 | 1.10 | 706.9 |
| 45 | 1.777 | 45.1 | 2.48 | 1600 | 9 | 1.128 | 28.6 | 1.00 | 644.7 | 35 | 1.378 | 35.0 | 1.49 | 962.1 |
| 55 | 2.176 | 55.3 | 3.72 | 2400 | 10 | 1.270 | 32.3 | 1.27 | 817.3 | 43 | 1.693 | 43.0 | 2.25 | 1452.2 |
| | | | | | 11 | 1.410 | 25.8 | 1.56 | 1007.4 | 57 | 2.244 | 57.0 | 3.96 | 2551.8 |
| | | | | | 14 | 1.693 | 43.0 | 2.25 | 1452.3 | | | | | |
| | | | | | 18 | 2.257 | 57.3 | 4.00 | 2581.2 | | | | | |

# APPENDIX

# C Footing Foundation

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

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 AND TRANSPORTATION AGENCY

**DEPARTMENT OF TRANSPORTATION**
**DIVISION OF CONSTRUCTION**
**Office of Transportation Laboratory**
**P. O. Box 19128**
**Sacramento, California 95819**
**(916) 444-4800**

**California Test 112**
**1978**

# METHOD FOR INSTALLATION AND USE OF EMBANKMENT SETTLEMENT DEVICES

## A. SCOPE

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

Settlement devices are used to monitor the rate and magnitude of settlement occurring at a point within or beneath an embankment during and subsequent to construction. The data obtained from these devices aid the engineer in determining the allowable rate of loading 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 three parts:

I. Fluid-Level Settlement Devices
II. Pipe Riser Settlement Device
III. Settlement Data Analysis

The fluid level vented standpipe unit may be used at most locations. A sealed standpipe unit must be installed at locations where ground water may interefere 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 300 linear feet (92m). 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 Materials and Research personnel will assist in these cases.

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 during the full duration of fill construction and surcharge removal. Because of the vulnerability of this unit to damage by the contractor's operations, the pipe riser settlement device should be used only on those projects where the fluid-level type of settlement device would be impractical.

## PART I. FLUID LEVEL SETTLEMENT DEVICES—VENTED STANDPIPE UNIT

### A. APPARATUS

1. Vented Standpipe Unit (Figure 1)
2. Indicating Unit (Figure 2)
3. ⅜ in. (.95 cm) Polyethylene Tubing
4. Hand Tools—shovel, bar, posthold auger, hammer, adjustable wrenches, etc.
5. Water Container (approximately 1 gallon (3.78 L) capacity)

### B. INSTALLATION

1. Select a location for the standpipe unit on the ground after approximately one foot (.3 m) of fill has been placed above original ground and generally within the area where the maximum height of embankment will be placed (Figure 4).
2. Select a point outside of the toe of the proposed embankment for the indicating unit (Figure 4). Select this location so that sufficent vertical distance will be available for lowering the indicating unit as the standpipe unit settles. A hand level may be used to estimate the desired elevations for the indicating 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 3 to 5 feet (.9 to 1.5 m) above the desired elevation for the

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standpipe unit, prepare a pit and trench in the embankment for the standpipe unit and tubing (Figure 4). The bottom of the pit should normally be about one foot above original ground. The trench should be cut to the same depth at the pit and should have a slight downward slope to the indicating unit location. Make sure that the trench is clear of any future contractor's operations 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 indicating unit post at the previously selected point for the indicating unit. This post can be either a metal sign post or a 4 x 4-inch (10 x 10 cm) timber.

8. Using a hand level, attach the indicating unit to the post so that the 2.0 foot (.61 m) graduation on the indicating unit scale is approximately level with the top of the spill tube on the standpipe unit.

9. Push the ⅜-inch (.95 cm) water line through the metal tube conduit in the center of the vented standpipe unit until the end is approximately ⅛-inch (.32 cm) above the top. Push the ⅜-inch (.95 cm) air vent line through the other conduit until approximately ⅝-inch (1.59 cm) extends out the top (Figure 1).

10. Unroll the water and air vent lines loosely in the trench from the standpipe to the indicating unit. It might be desirable to encase both lines in ¾ in. (1.9 cm) flexible metal conduit for additional protection under rocky material.

11. Cut and attach the water and air lines to the indicating unit as shown in Figures 2 and 3. Then fill the system by pouring water in the sight tube of the indicating 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. One gallon of water is more than adequate for 300 foot (92 m) of tubing. When filling, attempt to keep the water level in the sight tube near the 2.0 foot (.61 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. 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 indicating unit on the post to provide an initial reading of approximately 2.0 feet (.61 m).

Adjacent to the bottom of the indicating unit place a reference nail in the post at the elevation of the 0.0 graduation. This provides a reference point for surveys and relocation of the indicating 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 2.0-foot (.61 m) graduation of the indicator unit. Then loop the air line inside of the indicating unit over the lock hasp (Figure 2). The end of the air line should be pointing down 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 indicating unit.

During cold weather when the air line is too stiff to be looped, cut the air line at the 1.0-foot (.3 m) graduation mark. Then insert the end of a one-foot length of ¼- inch (.64 cm) plastic tubing in the end of the air line and loop the smaller tubing inside the indicating 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 12 inches (30.5 cm). 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 12 inches (30.5 cm) 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 cases should compaction equipment be allowed directly over an installation until a minimum of one-foot (30.5 m) of compacted material has been placed over the standpipe unit.

### SEALED STANDPIPE UNIT

#### A. APPARATUS

1. Sealed Standpipe Unit (Figure 5)
2. ½ in. (1.27 cm) plastic drain tubing
3. Remainder of apparatus is the same as for the vented standpipe unit.

#### 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 procedure in B-1 thru B-5 (section on vented standpipe unit).
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 procedure in B-7 and B-8 (section on vented standpipe unit).
5. Attach the ⅜-inch (.95 cm) water tube to the base plate as shown in Figure 5.
6. Unroll the water, air vent and drain tubes loosely in the trench from the standpipe to the indicating unit.
7. Follow procedure in B-10 and B-11 (section on vented standpipe unit).
8. Complete the assembly of the sealed standpipe unit by attaching the outer galvanized pipe, other fittings, air vent and drain tubes.*
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 procedure in B-13 and B-14 (section on vented standpipe units).

### 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 ± 0.01 ft (± .02 cm) by survey. This elevation should be checked periodically to correct settlement readings for settlement of indicating 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 0.2 feet (.51 cm).
   c. Read again after one hour. The water level after adding the water should drop to the first reading or slightly above it.
   d. If little or no drop is observed, or if the water level is below the 0.05 foot (.13 cm) graduation, refer to Section B, Maintenance, of this Part I.
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 date of reading.
   Column (2)—Record the water level reading from graduated scale on indicating unit (after adding water as indicated in paragraph 2-b above).
   Column (3)—Record the latest elevation of either the "0" graduation on the indicating unit or the nail reference as determined by survey.
   Column (4) & (5) not used.
   Column (6)—Enter the changes in water level elevation and reference elevation since the last reading. This is the sum of differences between the current and immediately previous entries in columns 2 and 3.
   Column (7)—Enter total settlement as *minus* 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 height of fill at surface as determined by survey (optional).
   Column (9)—Enter height of fill above original ground in total feet.
   Column (10)—Enter number of calendar days elapsed since the settlement device was installed.
   REMARKS
   Column (11)—Enter any information that would be helpful in the analysis of data as shown. If it is necessary to lower the indicating unit on the post, enter date and verticle distance lowered; be sure to include the corrected values in Column 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 0.2 foot (.51 cm). 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 excessive additional water; just observe 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 one foot (.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 indicating unit until water is observed in the

* 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.

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sight tube, then recharge 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 fails, 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, forcing air through the water line will 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 in recharging the system with water not to use too much water or in not introducing large voids in the system. For this reason, recharging should be performed only by personnel experienced in this type of activity.

3. If the water level in the sight tube is below the 0.05-inch (.13 cm) 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 inch graduation on the indicating unit and the reference point. Remove the unit from the post and lower it approximately 1.5 feet (.46 m) or until water is observed in the sight tube. If possible, and without adding water, adjust the height of the indicating unit on the post so that the water level in the sight tube is approximately at the 2.0-foot (.61 m) graduation. Add a small quantity of water and check the water level before attaching the indicating unit to the post. After adjusting the height of the indicating unit, again measure the vertical distance between the 0 inch graduation on the indicating unit and the reference point, and record the correction on the settlement data form (Figure 7, Col. 3).

4. It may be necessary to add antifreeze to the water in cold climates to prevent the water from freezing in the sight tube. In these cases, it is best to add the antifreeze solution during the installation and use the antifreeze solution in lieu of water before each reading. This method will help prevent error caused by the higher density solution which would be present at only one end of the system if you added water.

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

6. 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 indicating 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 in paragraph 3 above.

## PART II. 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, if any, 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 4 to 6 feet (1.2 to 1.8 m) of embankment material has been placed, excavate a pit to a depth of approximately one foot 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 ¾-inch (1.9 cm) pipe floor flange to the center of the wood platform with bolts or lag screws. Then screw a 6-foot (1.83 m) length of ¾-inch (1.9 cm) pipe into the floor flange. Place a pipe coupling on the top of the ¾-inch (1.9 cm) 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 1½-inch x 5 ft. (3.83 cm x 1.52 m) protective sleeve, which may be either rigid polyvinyl chloride (PVC) or iron pipe, over the control pipe until it is about 2 inches (5 cm) above the floor flange. Put duck seal or other material to hold the protective sleeve in place (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 independent 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 fine embankment material free of large rocks and clods around the settlement device to a depth of one foot (.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. It has been found that a 6-foot (1.83 m) long 1 x 4 inch (2.5 x 10 cm) post painted with alternate one-foot wide stripes of red and white are 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 ¾-inch (1.9 cm) 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, follow the following procedure:

a. By survey, 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. While tightening the joint, do not allow the coupling and control pipe to turn. Turn only the coupling.

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 by survey.

e. Record the length of additional control pipe added under Col. 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 5-foot (1.52 m) length of proptective sleeve to the existing sleeve and secure the post to the top.

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

Column 1—Record date read.
Column 2—Not used.
Column 3—Not used.
Column 4—Record elevation to top of control pipe as determined by survey.
Column 5—Record length of control pipe.
Column 6—Record the height of the riser above the original ground (Col. 4 minus Col. 5).
Column 7—Record the total settlement in feet. This figure is obtained by subtracting the figure in Col. 6 for the day being read from the figure at the top of Col. 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 height of fill above original ground in total feet.
Column 10—Record number of calendar days elapsed since the settlement device was installed.
REMARKS—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 III. SETTLEMENT DATA ANALYSIS

**A. SCOPE**

The procedure is given in this Part III for plotting and analyzing settlement data obtained from all types of settlement devices described in this method. A comprehensive settlement analysis is very complex and requires 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 are 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 settle-

ment 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. This may include the removing of 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 District Materials 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 not 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 outlined in paragraph 2b above.
b. When the plotted data indicates 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 at least throughout the life of the contract; longer periods are necessary if significant rates of settlement are discernible.

a. The time interval between readings may be increased as the indicated rate of movement decreases.
b. Collection of data may extend over a period of several years on selected projects. This long-term settlement data is frequently useful in the design of embankments where similar conditions are encountered.

**End of Text (16 pgs.) on Calif. 112**

California Test 112
1978

## VENTED STANDPIPE UNIT

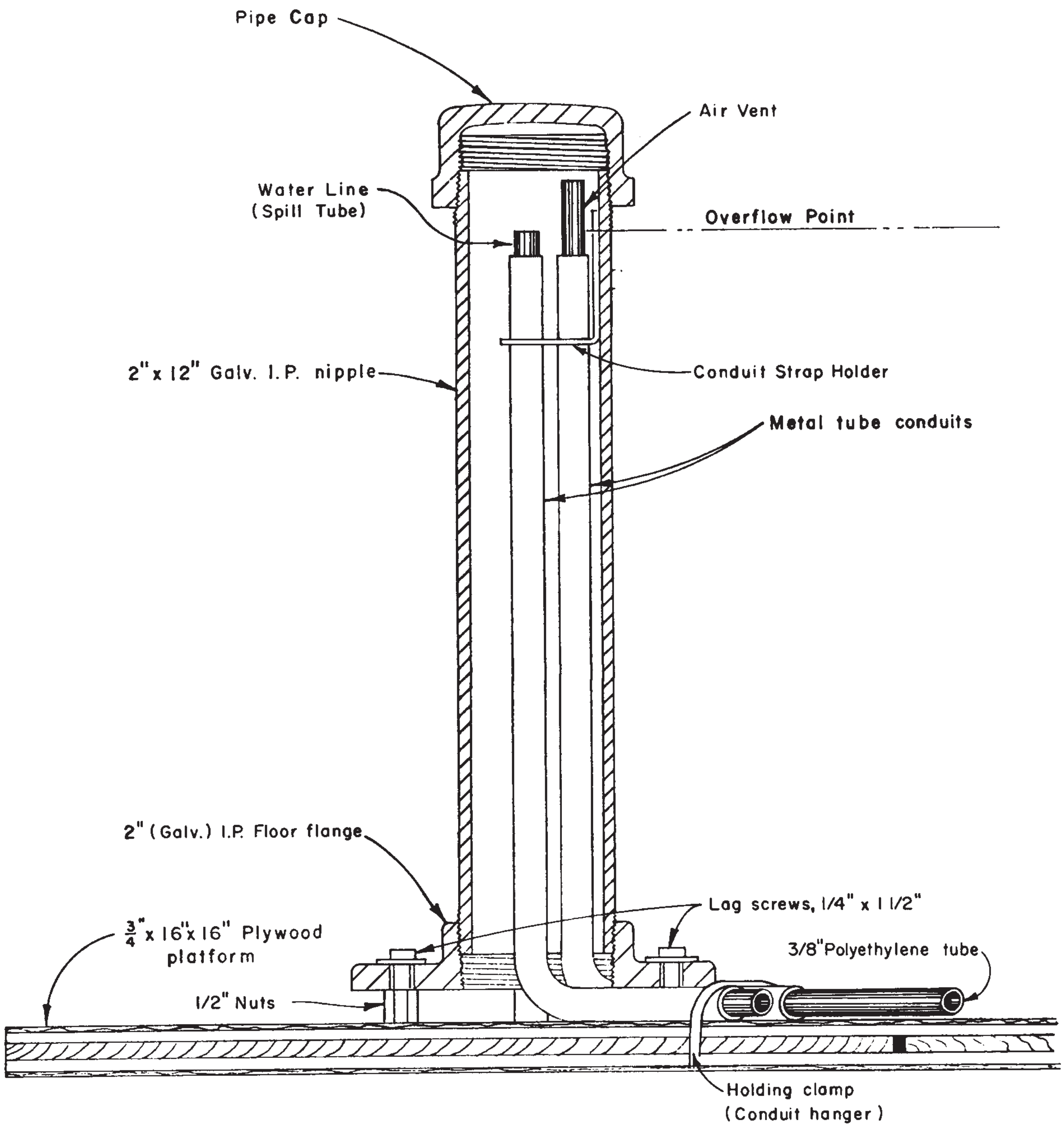


FIGURE 1

6—77624

7

**California Test 112**
**1978**

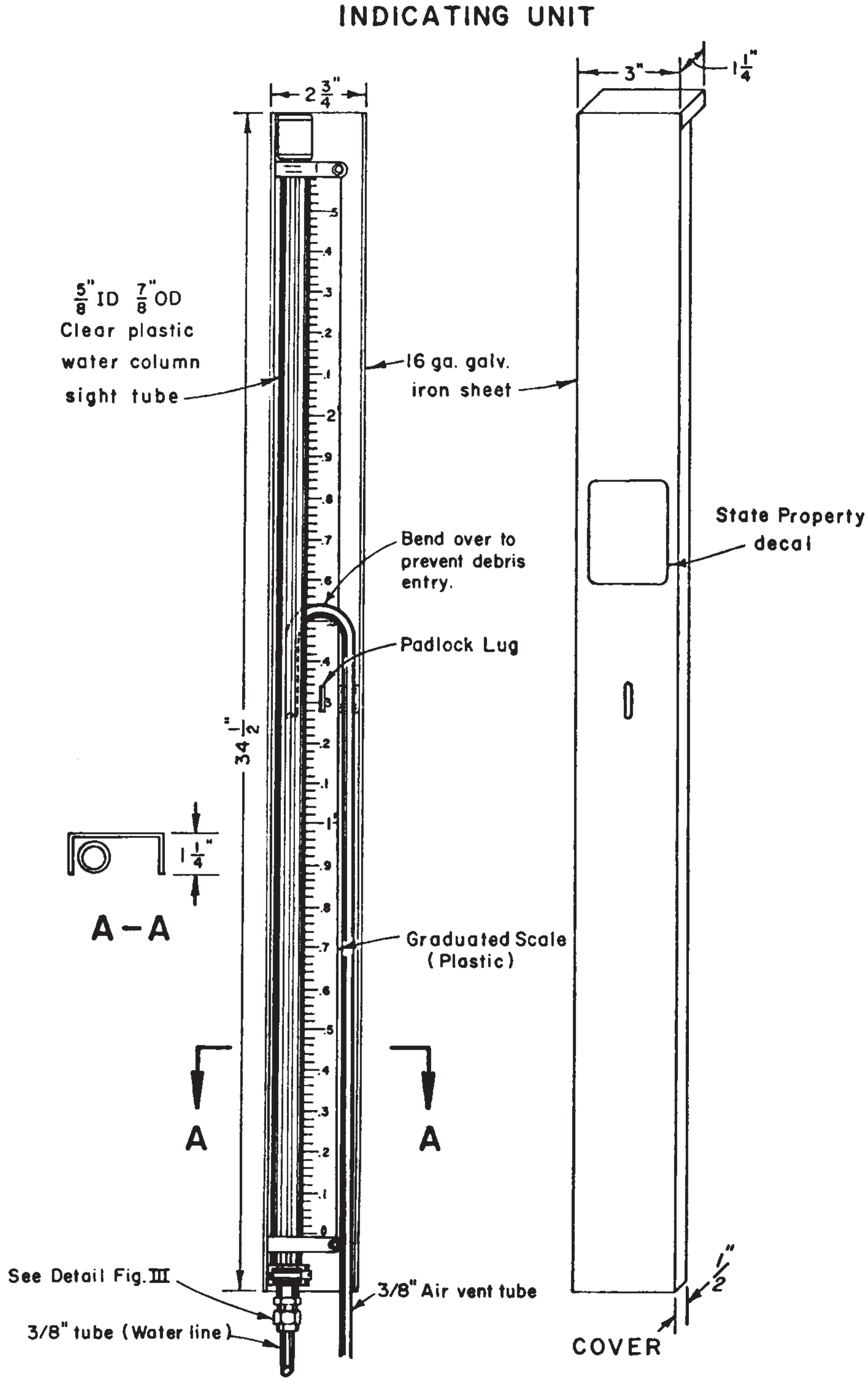


FIGURE 2

8

California Test 112
1978

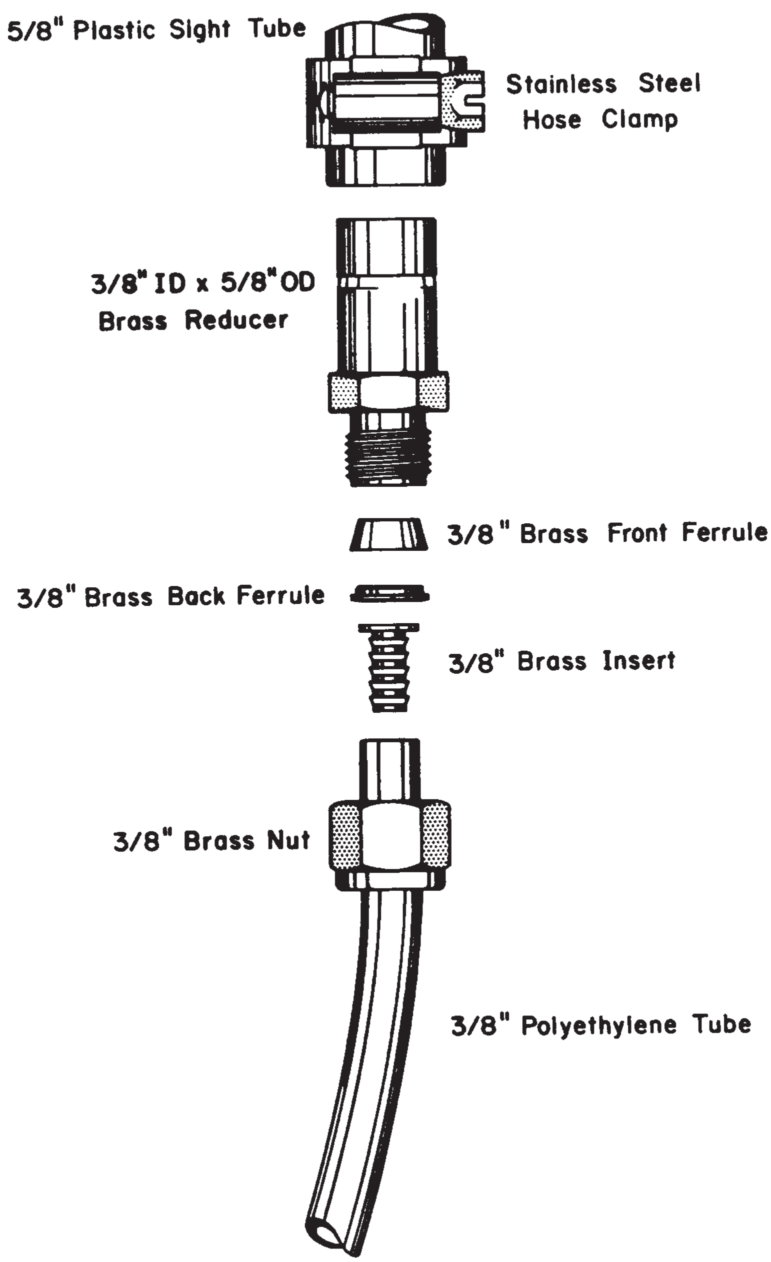


FIGURE 3

9

California Test 112
1978

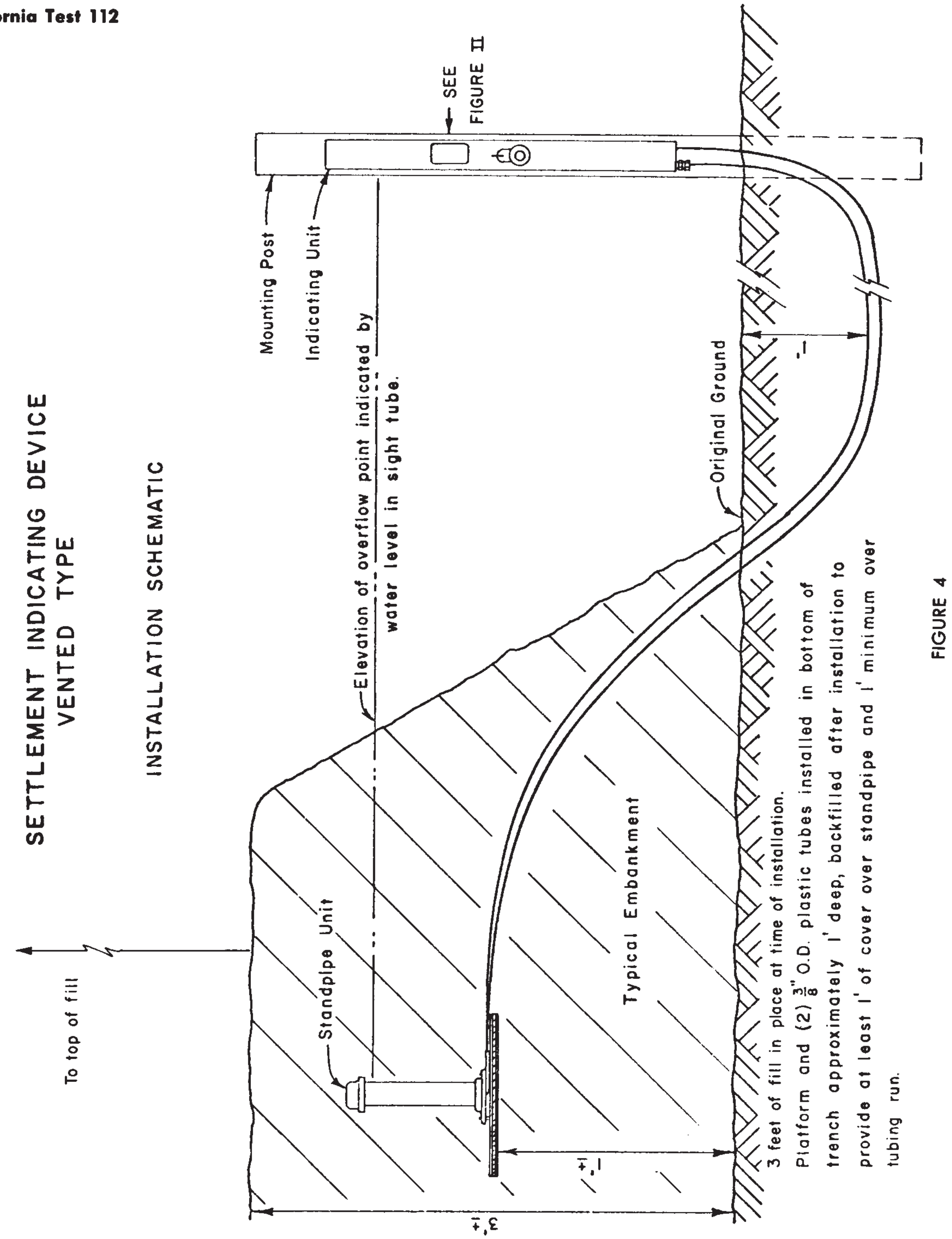


FIGURE 4

10

**California Test 112**
**1978**

SEALED STANDPIPE UNIT

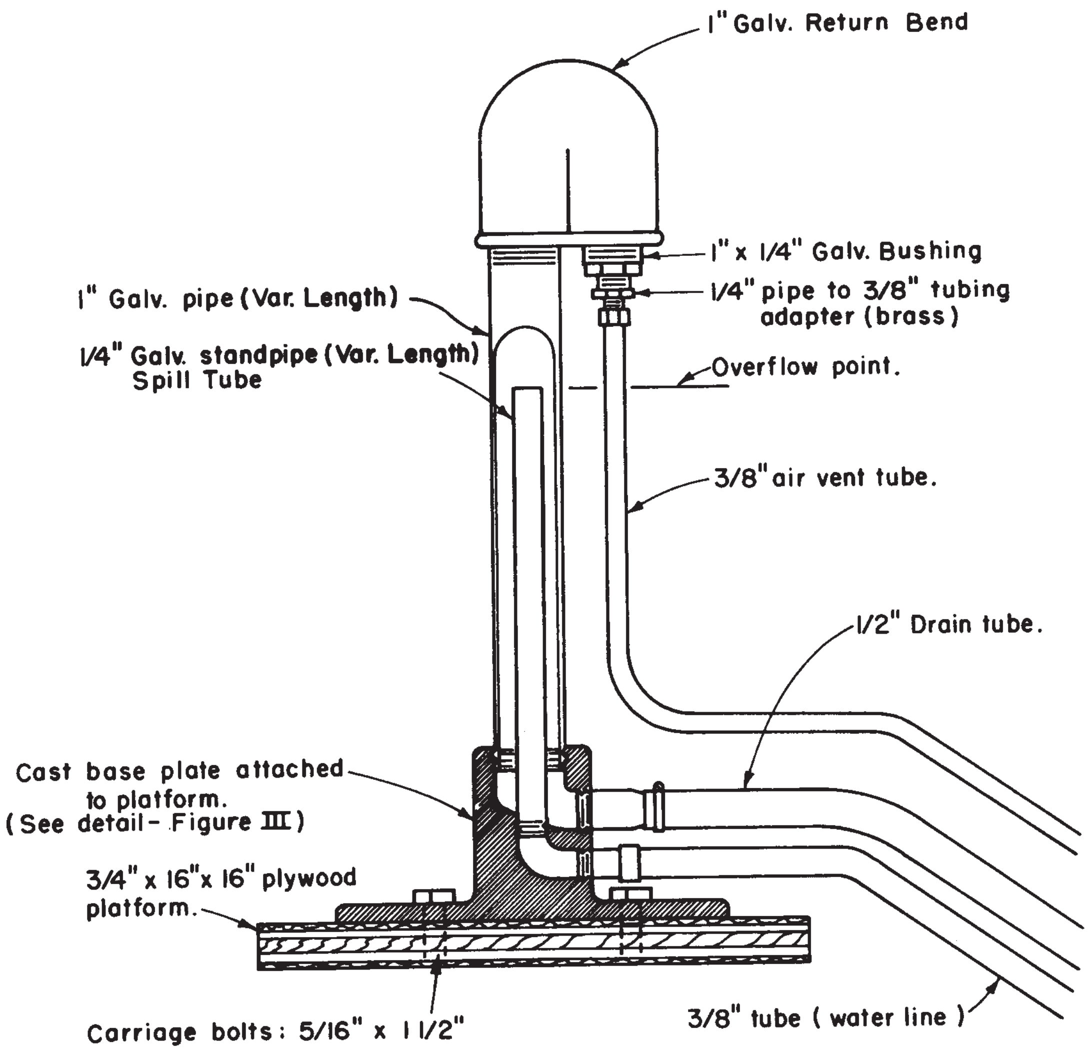


FIGURE 5

11

**California Test 112**
**1978**

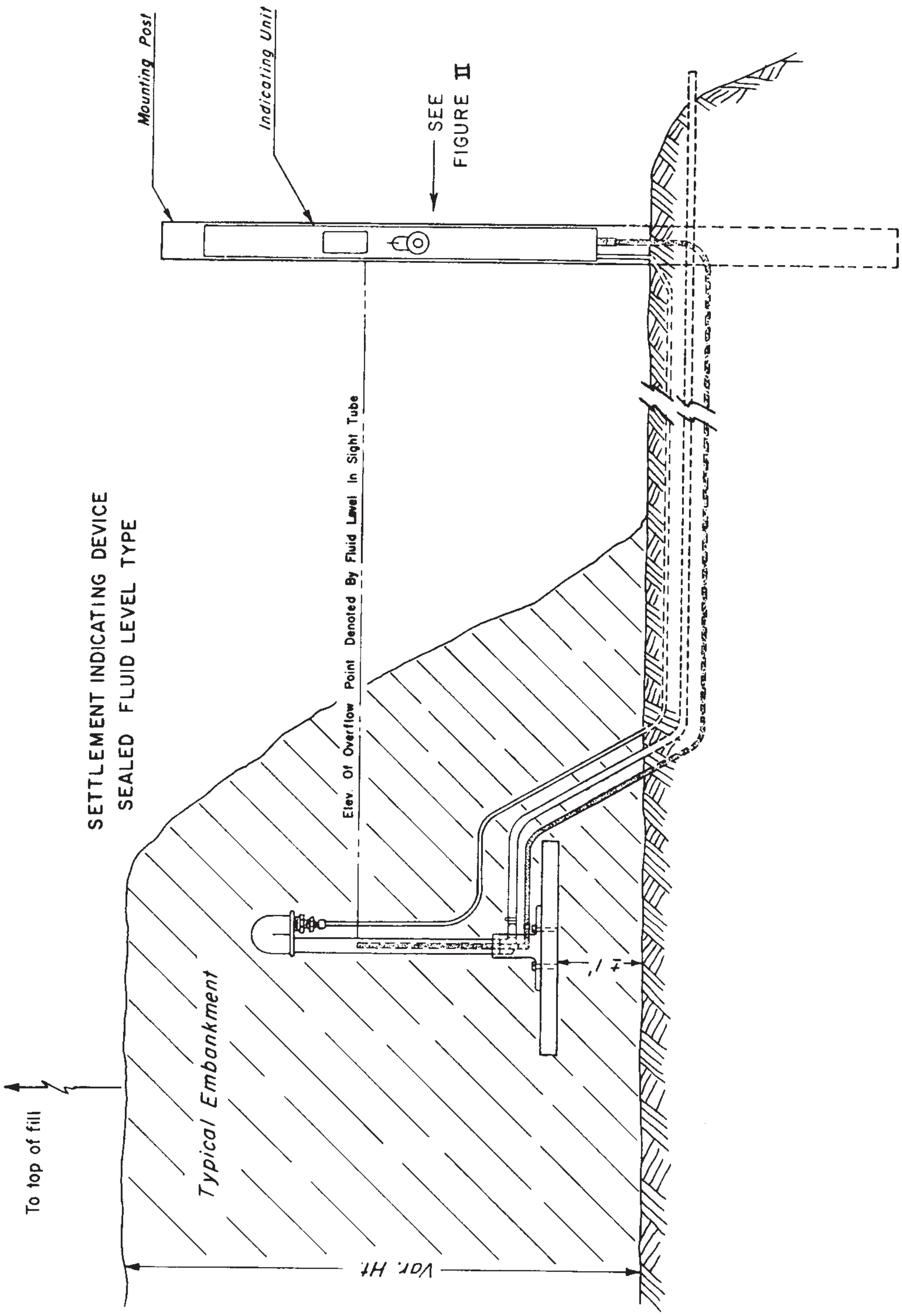


FIGURE 6

California Test 112
1978

STATE OF CALIFORNIA

# SETTLEMENT DATA

DEPARTMENT OF TRANSPORTATION

| JOB STAMP | | |
|---|---|---|
| | INSTALL. LOC. | |
| | BR. NO. | UNIT NO. |
| | STA. & OFFSET | |
| | DESIGN HT. (ft) | SURCHARGE (ft) |
| | WAITING PERIOD (Days) 270 | |
| | CONTROLLED LOADING 3 FT PER WEEK | |

COMMENTS:

| DATE | WATER TUBE READING | REFERENCE MAN. ELEVATION | RISER PIPE | | ELEVATION CHANGE | SETTLEMENT IN FEET | FILL HEIGHT | | ELAPSED TIME DAYS | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | ELEVATION TOP OF PIPE | LENGTH OF PIPE | | | SURFACE ELEVATION | FILL ABOVE ORIGINAL GROUND | | |
| 3-3-66 | 2.49 | 780.86 | | | 0 | 0 | 781 | 3 | 0 | PLATFORM SET ON O.G. |
| 3-4 | 2.44 | | | | 0.05 | 0.05 | 784 | 6 | 1 | |
| 3-7 | 2.42 | | | | 0.02 | 0.07 | 787 | 6 | 4 | TO CONVERT FEET TO METERS MULTIPLY FEET BY 0.3048 |
| 3-11 | 2.35 | | | | 0.07 | 0.14 | 790 | 9 | 8 | |
| 3-18 | 2.16 | | | | 0.19 | 0.33 | 793 | 12 | 15 | |
| 3-25 | 1.97 | | | | 0.19 | 0.52 | 796 | 15 | 22 | |
| 4-1 | 1.80 | 780.86 | | | 0.17 | 0.69 | 799 | 18 | 29 | |
| 4-8 | 1.65 | | | | 0.15 | 0.84 | 802 | 21 | 36 | |
| 4-15 | 1.54 | | | | 0.11 | 0.95 | 805 | 24 | 43 | |
| 4-22 | 1.47 | | | | 0.07 | 1.02 | ✓ | 27 | 50 | SURCHARGE COMPLETED |
| 4-26 | 1.41 | 780.84 | | | 0.08 | 1.10 | ✓ | ✓ | 54 | |
| 5-12 | 1.34 | 780.84 | | | 0.07 | 1.17 | ✓ | ✓ | 70 | |
| 5-28 | 1.30 | | | | 0.04 | 1.21 | ✓ | ✓ | 83 | |
| 6-8 | 1.26 | | | | 0.04 | 1.25 | ✓ | ✓ | 97 | |
| 7-6 | 1.27 | | | | + 0.01 | 1.24 | ✓ | ✓ | 125 | |
| 8-1 | 1.24 | 780.81 | | | 0.06 | 1.30 | ✓ | ✓ | 151 | |
| 9-2 | 1.22 | | | | 0.02 | 1.32 | ✓ | ✓ | 183 | |
| 10-26 | 1.20 | 780.79 | | | 0.05 | 1.37 | ✓ | ✓ | 217 | |
| 11-7 | 1.18 | | | | 0.02 | 1.39 | ✓ | ✓ | 249 | |
| 13-1 | 1.17 | 780.78 | | | 0.01 | 1.40 | ✓ | ✓ | 273 | 3.7" MAIN SINCE LAST READ |
| 1-5-67 | 1.16 | 780.78 | | | 0.01 | 1.41 | ✓ | ✓ | 308 | SETTLEMENT TERMINATED |
| 9-28-67 | 1.14 | 780.78 | | | 0.02 | 1.43 | 799 | 31 | 574 | 1-17-67 SURCHARGE |
| | | | | | | | | | | REMOVED BY 2-1-67 |

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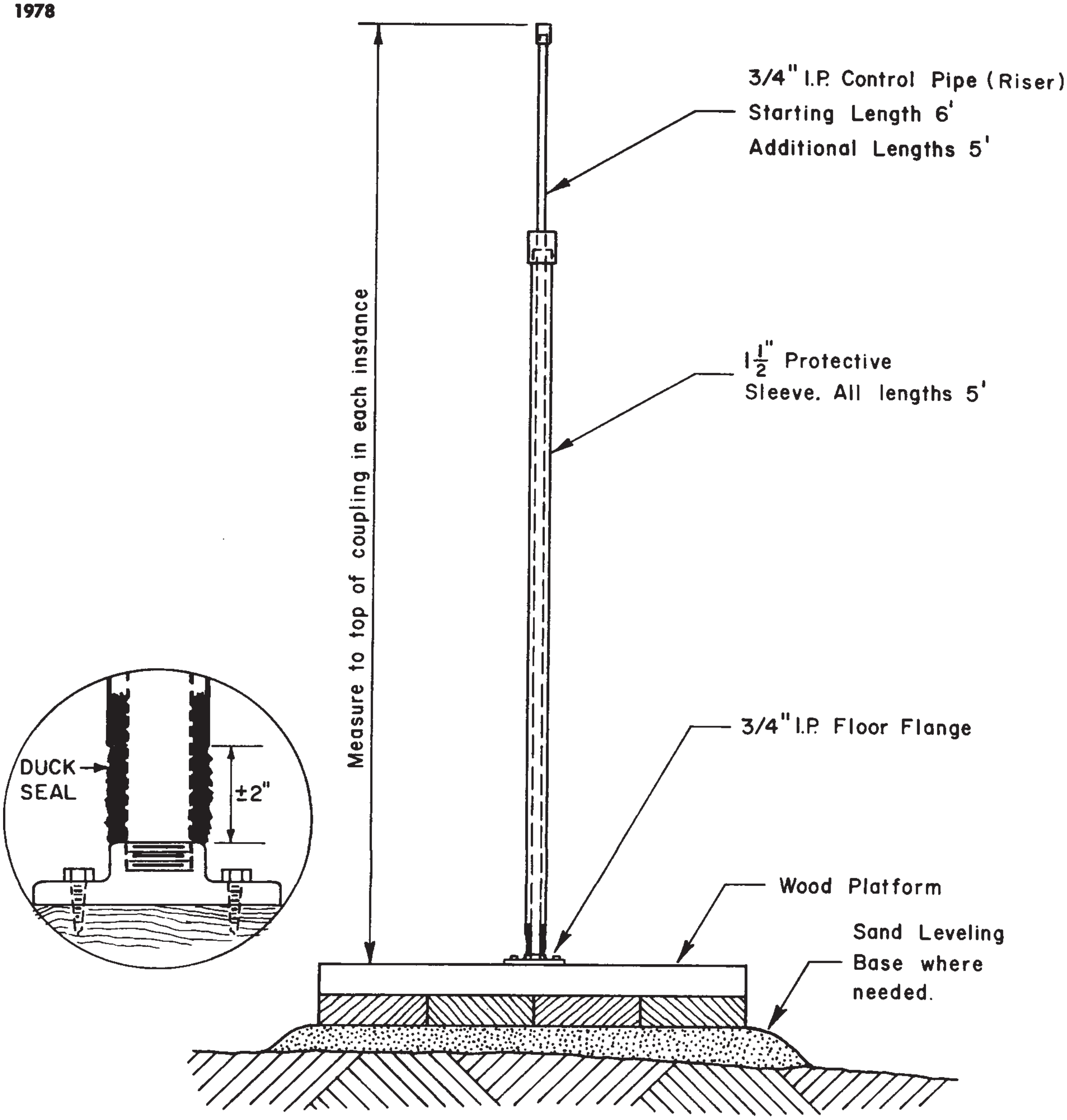
FIGURE 7

13

**California Test 112**
**1978**




SETTLEMENT INDICATING DEVICE
RISER PIPE TYPE

FIGURE 8

14

California Test 112
1978

STATE OF CALIFORNIA

# SETTLEMENT DATA

DEPARTMENT OF TRANSPORTATION

| JOB STAMP | INSTALL. LOC. | | COMMENTS: |
|---|---|---|---|
| | BR. NO. | UNIT NO. | |
| | STA. & OFFSET | | |
| | DESIGN HT. (ft.) | SURCHARGE (ft.) | |
| | WAITING PERIOD (Days) | | |
| | CONTROLLED LOADING | | |

| DATE | WATER TUBE READING | REFERENCE NAIL ELEVATION | RISER PIPE | | ELEVATION CHANGE | SETTLEMENT IN FEET | FILL HEIGHT | | ELAPSED TIME DAYS | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | ELEVATION TOP OF PIPE | LENGTH OF PIPE | | | SURFACE ELEVATION | FILL ABOVE ORIGINAL GROUND | | |
| 2-25-70 | | | 15.42 | 10.71 | 4.71 | | | 5 | 0 | |
| 2-27 | | | 20.68 | 16.01 | 4.67 | 0.04 | | 8 | 2 | To convert feet to meters multiply feet by .3048 |
| 3-3 | | | 25.96 | 21.30 | 4.66 | 0.05 | | 10 | 6 | |
| 3-6 | | | 25.97 | 21.30 | 4.67 | 0.04 | | 15 | 9 | |
| 3-10 | | | 25.94 | 21.30 | 4.64 | 0.07 | | 20 | 13 | |
| 3-20 | | | 31.16 | 26.58 | 4.58 | 0.43 | | 25.5 | 23 | |
| 3-27 | | | 41.50 | 37.20 | 4.30 | 0.41 | | 33.5 | 30 | |
| 4-6 | | | 56.76 | 53.07 | 3.69 | 1.02 | | 49.0 | 40 | |
| 4-15 | | | 56.58 | " | 3.51 | 1.20 | | 49.5 | 49 | FILL COMPLETED |
| 4-29 | | | 56.46 | " | 3.39 | 1.32 | | " | 63 | |
| 5-7 | | | 56.38 | " | 3.31 | 1.40 | | " | 71 | |
| 5-13 | | | 56.36 | " | 3.29 | 1.42 | | " | 77 | |
| 5-23 | | | 56.30 | " | 3.23 | 1.48 | | " | 87 | |
| 5-29 | | | 56.31 | " | 3.24 | 1.47 | | " | 93 | |
| 6-6 | | | 56.27 | " | 3.20 | 1.51 | | " | 101 | |
| 6-12 | | | 56.36 | " | 3.29 | 1.42 | | " | 107 | |
| 6-20 | | | 56.23 | " | 3.16 | 1.55 | | " | 115 | |
| 6-26 | | | 56.18 | " | 3.11 | 1.60 | | " | 121 | |
| 7-7 | | | 56.18 | " | 3.11 | 1.60 | | " | 142 | |
| 7-24 | | | 56.16 | " | 3.09 | 1.62 | | " | 149 | |
| 8-4 | | | 56.13 | " | 3.06 | 1.65 | | " | 160 | |
| | | | | | | | | | | |
| | | | | | | | | | | |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) |

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FIGURE 9

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California Test 112
1978

STATE OF CALIFORNIA

SETTLEMENT DATA

DEPARTMENT OF TRANSPORTATION

| JOB STAMP | INSTALL. LOC. | | COMMENTS: |
|---|---|---|---|
| | BR. NO. | UNIT NO. | |
| | STA. & OFFSET | | |
| | DESIGN HT. (ft.) | SURCHARGE (ft.) | |
| | WAITING PERIOD (Days) | | |
| | CONTROLLED LOADING | | |

| DATE | WATER TUBE READING | REFERENCE NAIL ELEVATION | RISER PIPE | | ELEVATION CHANGE | SETTLEMENT IN FEET | FILL HEIGHT | | ELAPSED TIME DAYS | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | ELEVATION TOP OF PIPE | LENGTH OF PIPE | | | SURFACE ELEVATION | FILL ABOVE ORIGINAL GROUND | | |
| 2-25-70 | | | 15.42 | 10.71 | 4.71 | | | 5 | 0 | |
| 2-27 | | | 20.68 | 16.01 | 4.67 | 0.04 | | 8 | 2 | To convert feet to meters multiply feet by .3048 |
| 3-3 | | | 25.96 | 21.30 | 4.66 | 0.05 | | 10 | 6 | |
| 3-6 | | | 25.97 | 21.30 | 4.67 | 0.04 | | 15 | 9 | |
| 3-10 | | | 25.94 | 21.30 | 4.64 | 0.07 | | 20 | 13 | |
| 3-20 | | | 31.16 | 26.58 | 4.58 | 0.43 | | 25.5 | 23 | |
| 3-27 | | | 41.50 | 37.20 | 4.30 | 0.41 | | 33.5 | 30 | |
| 4-6 | | | 56.76 | 53.07 | 3.69 | 1.02 | | 49.0 | 40 | |
| 4-15 | | | 56.58 | – | 3.51 | 1.20 | | 49.5 | 49 | FILL COMPLETED |
| 4-29 | | | 56.46 | 〃 | 3.39 | 1.32 | | 〃 | 63 | |
| 5-7 | | | 56.38 | 〃 | 3.31 | 1.40 | | 〃 | 71 | |
| 5-13 | | | 56.36 | 〃 | 3.29 | 1.42 | | 〃 | 77 | |
| 5-23 | | | 56.30 | 〃 | 3.23 | 1.48 | | 〃 | 87 | |
| 5-29 | | | 56.31 | 〃 | 3.24 | 1.47 | | 〃 | 93 | |
| 6-6 | | | 56.27 | 〃 | 3.20 | 1.51 | | 〃 | 101 | |
| 6-12 | | | 56.36 | 〃 | 3.29 | 1.42 | | 〃 | 107 | |
| 6-20 | | | 56.23 | 〃 | 3.16 | 1.55 | | 〃 | 115 | |
| 6-26 | | | 56.18 | 〃 | 3.11 | 1.60 | | 〃 | 121 | |
| 7-7 | | | 56.18 | 〃 | 3.11 | 1.60 | | 〃 | 142 | |
| 7-24 | | | 56.16 | 〃 | 3.09 | 1.62 | | 〃 | 149 | |
| 8-4 | | | 56.13 | 〃 | 3.06 | 1.65 | | 〃 | 160 | |
| | | | | | | | | | | |
| | | | | | | | | | | |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) |

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FIGURE 9

15

**California Test 112**
**1978**

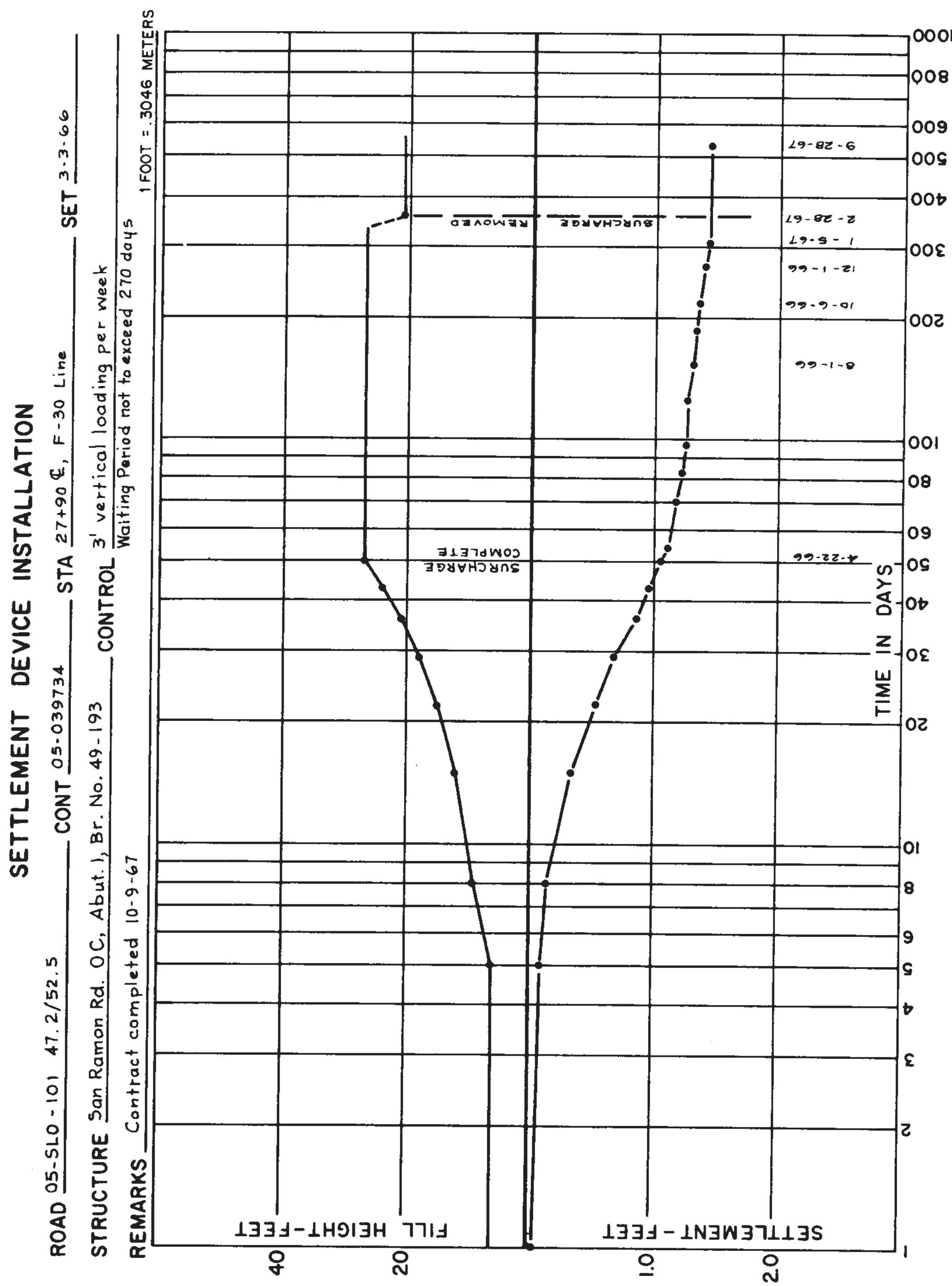



FIGURE 10

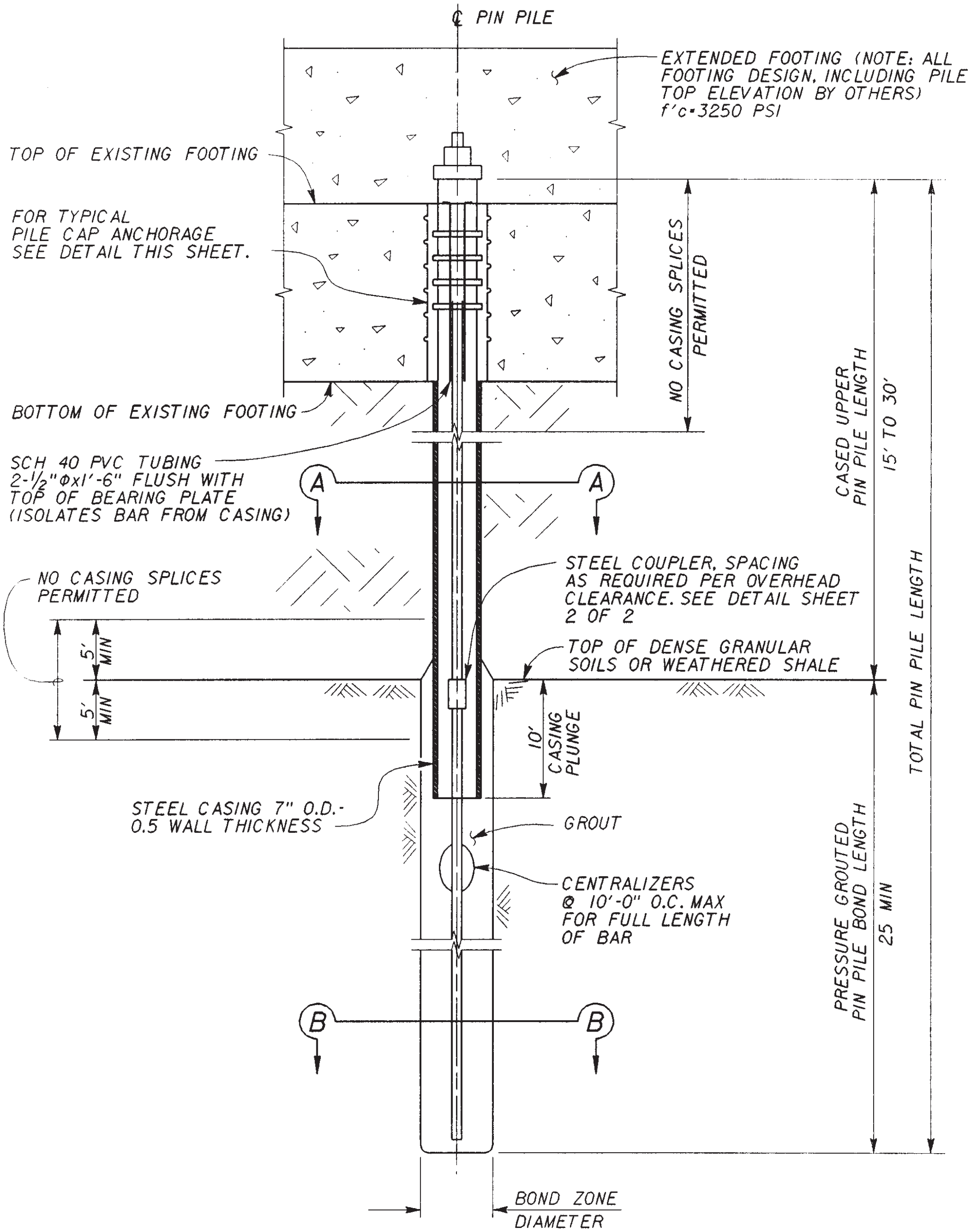
16

# APPENDIX

# D Alternative Pile Types

## Table of Contents

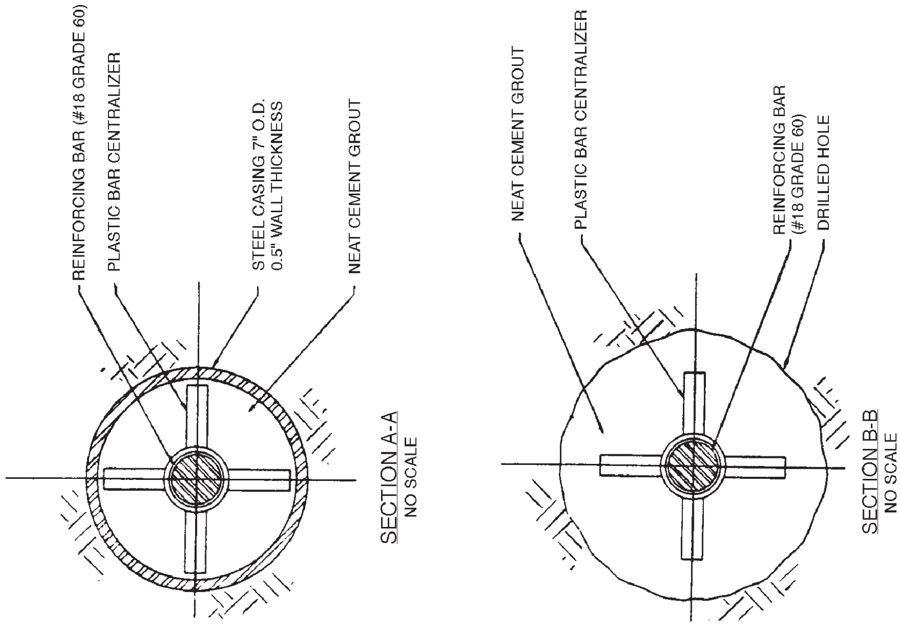
## Nicholson Pin Pile

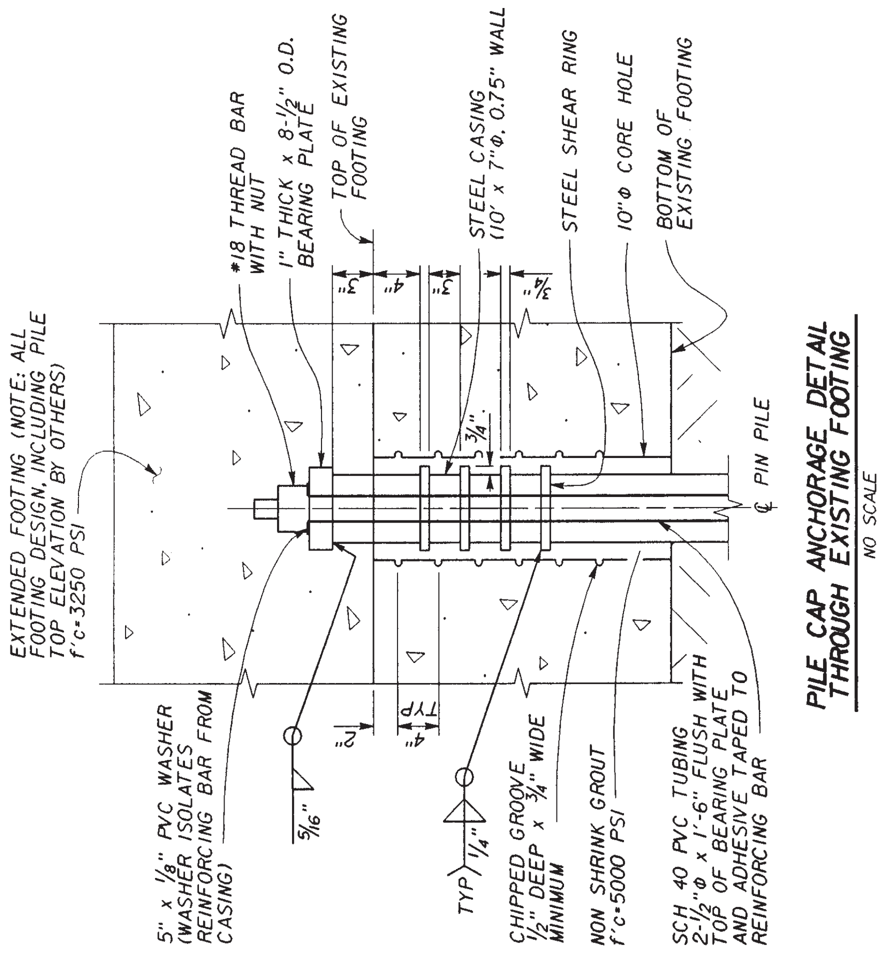



PIN PILE DETAIL

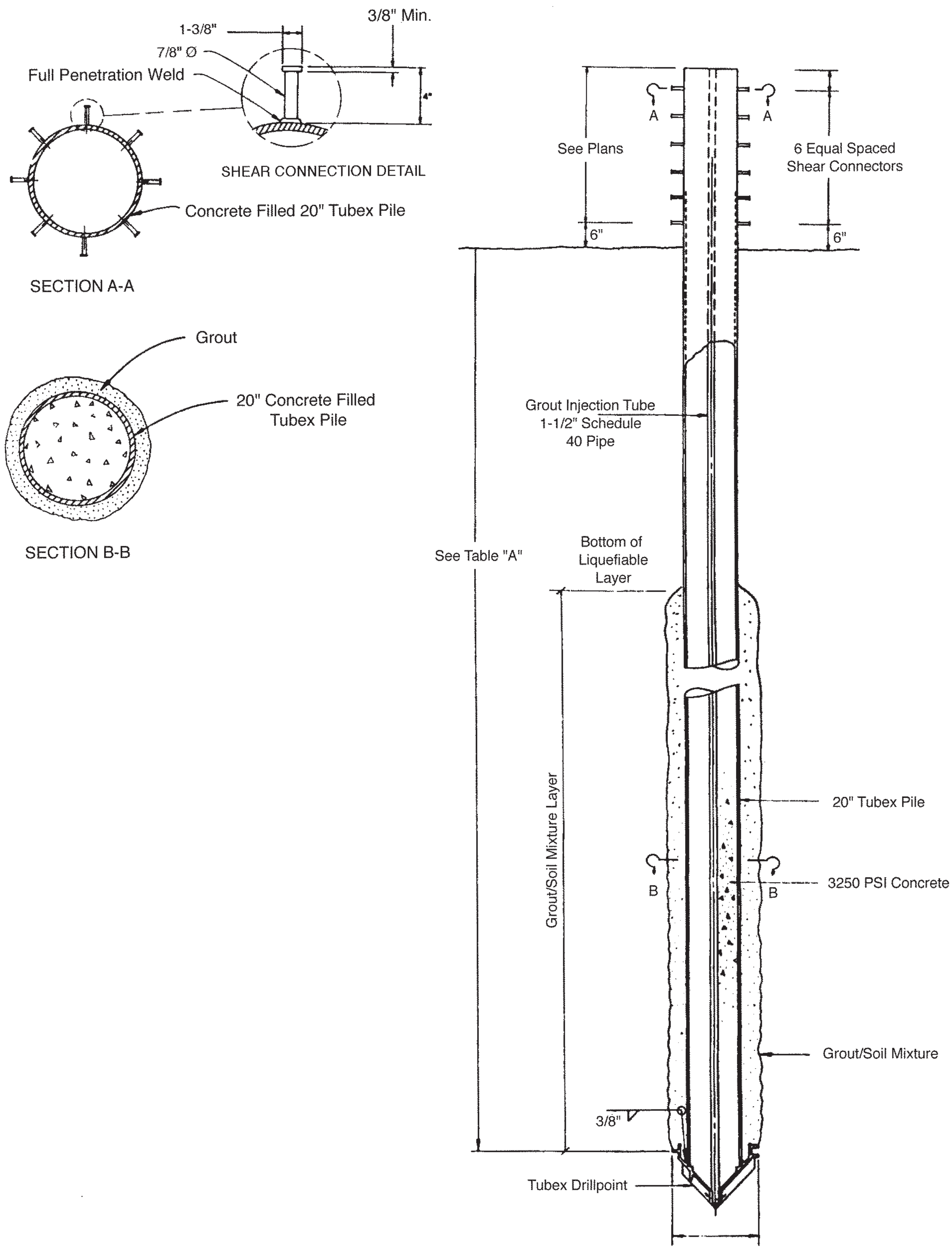
NO SCALE

## Nicholson Pin Pile

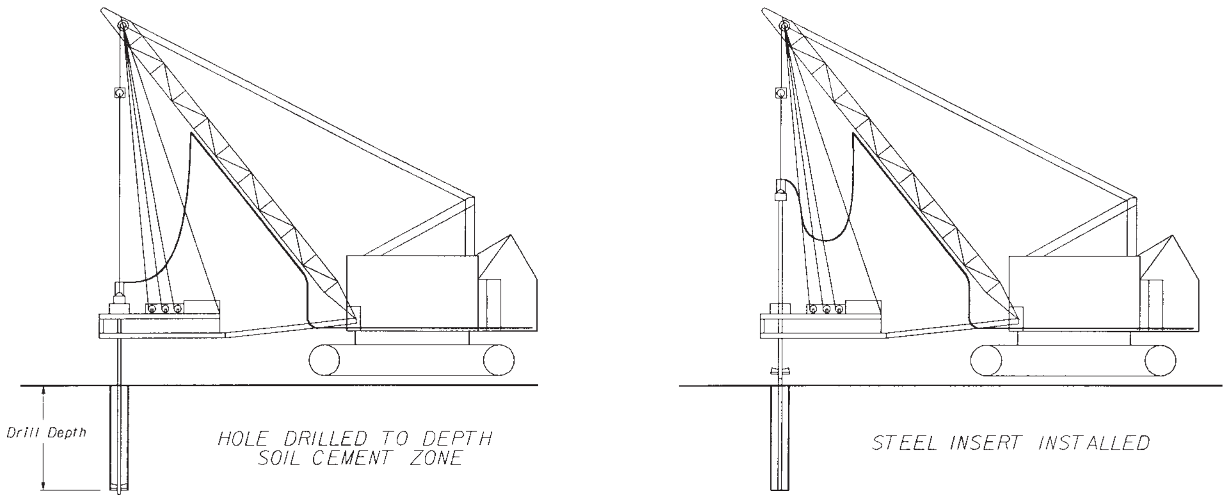


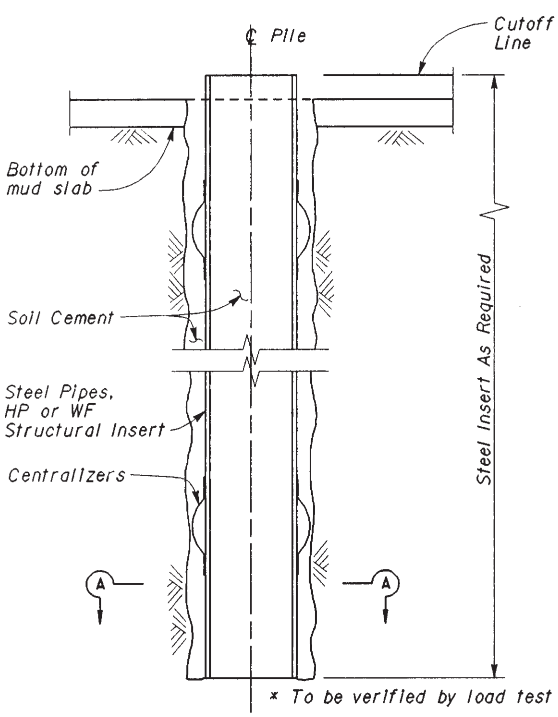



## Tubex Grout Unit

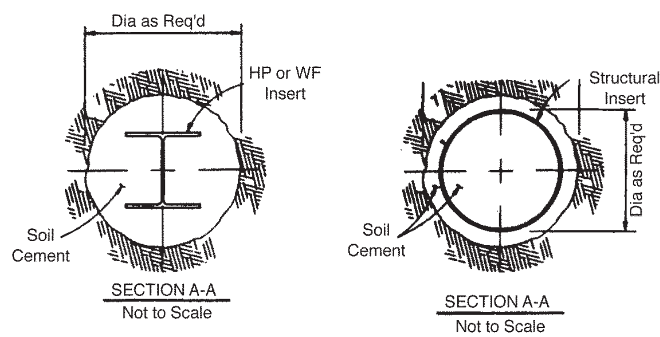


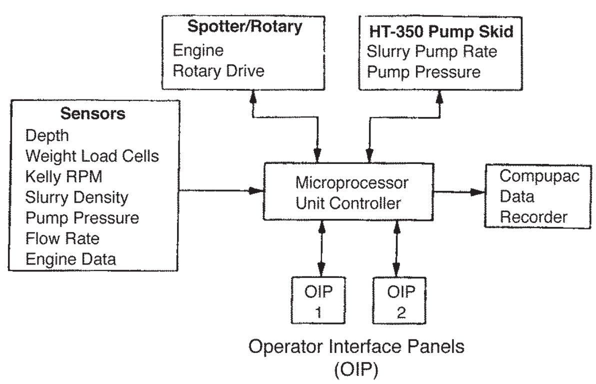
## GeoJet Foundation Unit




**GeoJet FOUNDATION MEMBER**
NO SCALE




GeoJet Process Control Schematic

Caltrans

# APPENDIX

# E Driven Piles

## Table of Contents

Four commonly used pile driving formulas are the ENR, the JANBU, the HILEY, and the PACIFIC COAST. They are described on the following pages. These are offered to show the differences in complexity and approach used by their authors. The formula description is followed by examples used to illustrate how results will differ depending on the formula used.

## PILE DRIVING FORMULAS

### ENGINEERING NEWS (ENR)

$P = 2E/(s + 0.1)$

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 $P = W\,H / (s + c)$

$W$ = 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

$$P = \frac{W\,H}{k_u s} \times Z$$

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:

$$k_u = C_d\left[1 + \sqrt{1 + \frac{\lambda}{C_d}}\right]$$

$$C_d = 0.75 + 0.15\,\frac{W_p}{W}$$

$$\lambda = \frac{W\,H\,L}{A\,E\,s^2}$$

when: $W_p$ = weight of pile in pounds

$L$ = length of pile in inches

$A$ = area of pile in sq. in.

$E$ = modulus of elasticity of pile in psi

$Z$ = conversion factor for units and safety with this formula.

HILEY FORMULA

$$P = \frac{e_f \, W \, H}{s + \frac{1}{2}(c_1 + c_2 + c_3)} \times \frac{W + n^2 W_p}{W + W_p} \times Z$$

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

$$P = \frac{E_n \dfrac{W + k W_p}{W + W_p}}{s + \dfrac{PL}{AE}} \times Z$$

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 sq. in.

$E$ = modulus of elasticity of pile in psi

$k$ = 0.25 for steel piles

0.10 for other piles

$Z$ = conversion factor for units and safety with this formula

## COMPARISON OF FORMULAS

**Problem Conditions:**

Hammer: Delmag 30 $E_m$ = 66,100 ft-lbs

Ram Weight = 6,600 lbs

Max. Stroke = 10 ft

Set or Penetration = 0.844 inches

Length of pile = 80 feet

Assume hard driving

Case 1 : 12" PC/PS Concrete Pile

Case 2 : 12 BP 53 Steel Pile

---

**Engineering News Formula**

For both piles: $P = 2\,E/(s + 0.1)$

$= 2 \times 66100/(0.844 + 0.1)$

$= 70$ tons

---

**Janbu Formula**

Case 1: $P = \dfrac{WH}{k_u\,s} \times \dfrac{1}{3 \times 2000}$

$= \dfrac{6600\,(10\times12)}{3.034\,(.844)} \times \dfrac{1}{3 \times 2000}$

$= \dfrac{309290}{3 \times 2000}$

$= 51.5$ tons

$C_d = 0.75 + 0.15\,W_p/W$

$= 0.75 + 0.15(11850)/6600$

$= 1.019$

$\lambda = \dfrac{W\,H\,L}{A\,E\,s^2}$

$= \dfrac{6600\ (10 \times 12)\ (80\times12)}{144\ (2.5 \times 10^6)(.844)^2}$

$= 2.965$

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

$= 1.019\left[1 + \sqrt{1 + \dfrac{2.965}{1.019}}\right]$

$= 3.034$

Case 2: $P = \dfrac{WH}{k_u\,s} \times \dfrac{1}{3 \times 2000}$

$= \dfrac{6600\ (10\times12)}{2.473\ (.844)} \times \dfrac{1}{3 \times 2000}$

$= \dfrac{379450}{3 \times 2000}$

$= 63.2$ tons

$C_d = 0.75 + 0.15\ (4240)/6600$

$= 0.846$

$= \dfrac{6600\ (10\times12)\ (80\times12)}{15.58\,(30 \times 10^6)\ (.844)^2}$

$= 2.284$

$k_u = 0.846\left[1 + \sqrt{1 + \dfrac{2.284}{0.846}}\right]$

$= 2.473$

**HILEY FORMULA:**

Case 1:

$$P = \frac{e_f\ W\ H}{s + (c_1 + c_2 + c_3)} \times \frac{W + n^2 W_p}{W + W_p} \times \frac{1}{2.75 \times 2000}$$

$$= \frac{1.00(6600)(10 \times 12)}{0.844 + \tfrac{1}{2}(.37+.32+.10)} \times \frac{6600 + 0.25^2(11850)}{6600 + 11850} \times \frac{1}{2.75 \times 2000}$$

$$= \frac{254300}{2.75 \times 2000} = 46.2 \text{ tons}$$

Case 2:

$$P = \frac{1.00(6600)(10\times12)}{0.844 + \tfrac{1}{2}(0.0+.48+.10)} \times \frac{6600+4240(.55)^2}{6600 + 4240} \times \frac{1}{2.75 \times 2000}$$

$$= \frac{507900}{2.75 \times 2000} = 92.3 \text{ tons}$$

---

**PACIFIC COAST FORMULA:**

Case 1:

$$P = \frac{E_n \dfrac{W + k\ W_p}{W + W_p}}{s + \dfrac{P\ L}{A\ E}} \times \frac{1}{4 \times 2000}$$

$$= \frac{66100(12) \times \dfrac{6600 + 0.1(11850)}{6600 + 11850}}{0.844 + \dfrac{P\ (80 \times 12)}{144\ (2.5\text{x}10^6)}} \times \frac{1}{4 \times 2000}$$

$$= \frac{334690}{0.844 + .0000027P} \times \frac{1}{4 \times 2000}$$

$$= \frac{228940}{4 \times 2000} = 28.6 \text{ tons}$$

Case 2:

$$P = \frac{66100(12) \times \dfrac{6600 + 0.25(4240)}{6600 + 4240}}{0.844 + \dfrac{P\ (80 \times 12)}{15.58\ (30 \times 10^6)}} \times \frac{1}{4 \times 2000}$$

$$= \frac{560508}{0.844 + .0000021P} \times \frac{1}{4 \times 2000}$$

$$= \frac{353420}{4 \times 2000} = 44.2 \text{ tons}$$

## TABULATION OF COMPARISON RESULTS

| FORMULA | CASE 1 | CASE 2 |
|---|---|---|
| | 12" PC/PS | 12 BP 53 |
| Pile Length = 80 feet | | |
| ENR | 70 tons | 70 tons |
| JANBU | 51 | 63 |
| HILEY | 46 | 92 |
| PACIFIC COAST | 28 | 44 |
| Pile Length = 40 feet | | |
| ENR | 70 tons | 70 tons |
| JANBU | 67 | 77 |
| HILEY | 65 | 106 |
| PACIFIC COAST | 44 | 61 |

## Example Calculation Minimum Hammer Energy

GIVEN:

Bearing Capacity = 70 tons

Proposed Hammer is Delmag 30–23

Manufacturer's Maximum Energy Rating 66,100 ft–lbs

CHECK HAMMER ENERGY, PER (SECTION 49–1.05)

FROM THE ENR EQUATION, $$P = \frac{2 * E}{(S + 0.1)}$$

REARRANGING $$S = \frac{2 * E}{P} - 0.1$$

PLUG IN GIVEN VALUES $$S = \frac{2 * 66{,}100}{140{,}000} - 0.1 = 0.84 \text{ in.} > .125 \text{ in.}$$

THE HAMMER MEETS THE ENERGY REQUIREMENTS OF SECTION 49–1.05.

## Example Calculation Blow Count Chart

GIVEN:

BEARING CAPACITY = 70 TONS

PROPOSED HAMMER IS DELMAG 30–23

HAMMER WEIGHT = 6,600 lbs.

---

ASSUME E = HAMMER WEIGHT X STROKE

USING ENR EQUATION

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

REARRANGE

$$S = \frac{2 * E}{P} - 0.1$$

UNITS FOR S ARE INCHES PER BLOW

CONVERT S TO FEET PER BLOW

$$\frac{S * 1\ \text{ft}}{12\ \text{in}} = \frac{\text{FT}}{\text{BLOW}}$$

$$\frac{S}{1} = \frac{2 * E}{P} - 0.1$$

INVERTING THE EQUATION TO GET BLOWS PER FOOT

$$S = \frac{12}{(2 * E/P - 0.1)}$$

EXAMPLE: ASSUME A 9 FOOT STROKE FOR THE GIVEN HAMMER

E = 9 * 6,600 lbs = 59,400 ft_lbs

S = 12 / ((2 * 59,400 / 140000) – 0.1) = 16 BLOWS PER FOOT

CONTINUE CALCULATIONS FOR VARYING STROKE HEIGHT

**Example Blow Count Chart**

PILE BLOW DATA

job stamp

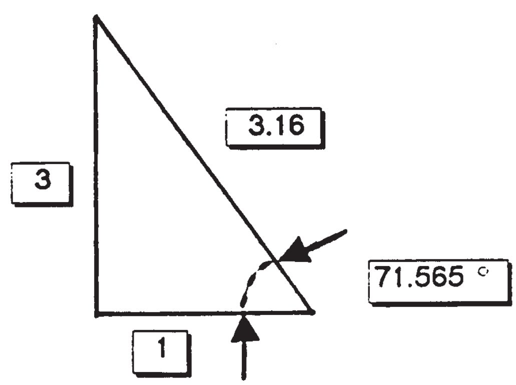
| | | |
|---|---|---|
| PILE CAPACITY | 140000 | POUNDS |
| HAMMER | D 30-23 | |
| PISTON WEIGHT | 6,600 | POUNDS |

| stroke (feet) | reqd. blows | stroke (feet) | reqd. blows |
|---|---|---|---|
| 10 | 14 | 7.0 | 21 |
| 9.5 | 15 | 6.5 | 23 |
| 9 | 16 | 6 | 26 |
| 8.5 | 17 | 5.5 | 29 |
| 8 | 18 | 5 | 32 |
| 7.5 | 20 | 4.5 | 37 |

**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 Calculation Timing of Single Acting Hammer

ACCELERATION = 32.2 ft / sec^2

VELOCITY = 32.2 ft / sec^2 * T sec

DISTANCT = 32.2 ft / sec^2 * $\frac{T\ \hat{}2}{2}$

$$= \frac{g * T\hat{}2}{2}$$

WE ASSUME THAT $T_{fall} = T_{whole} / 2$

THE EQUATION BECOMES DISTANCE = $\frac{g}{2} * \frac{(Twhole)\hat{}2}{(2\hat{}2)}$

WHICH BECOMES DISTANCE = $\frac{g * T\ \hat{}2}{8}$

PLUG IN 32.2 FOR g DISTANCE = 4.02 * T ^2

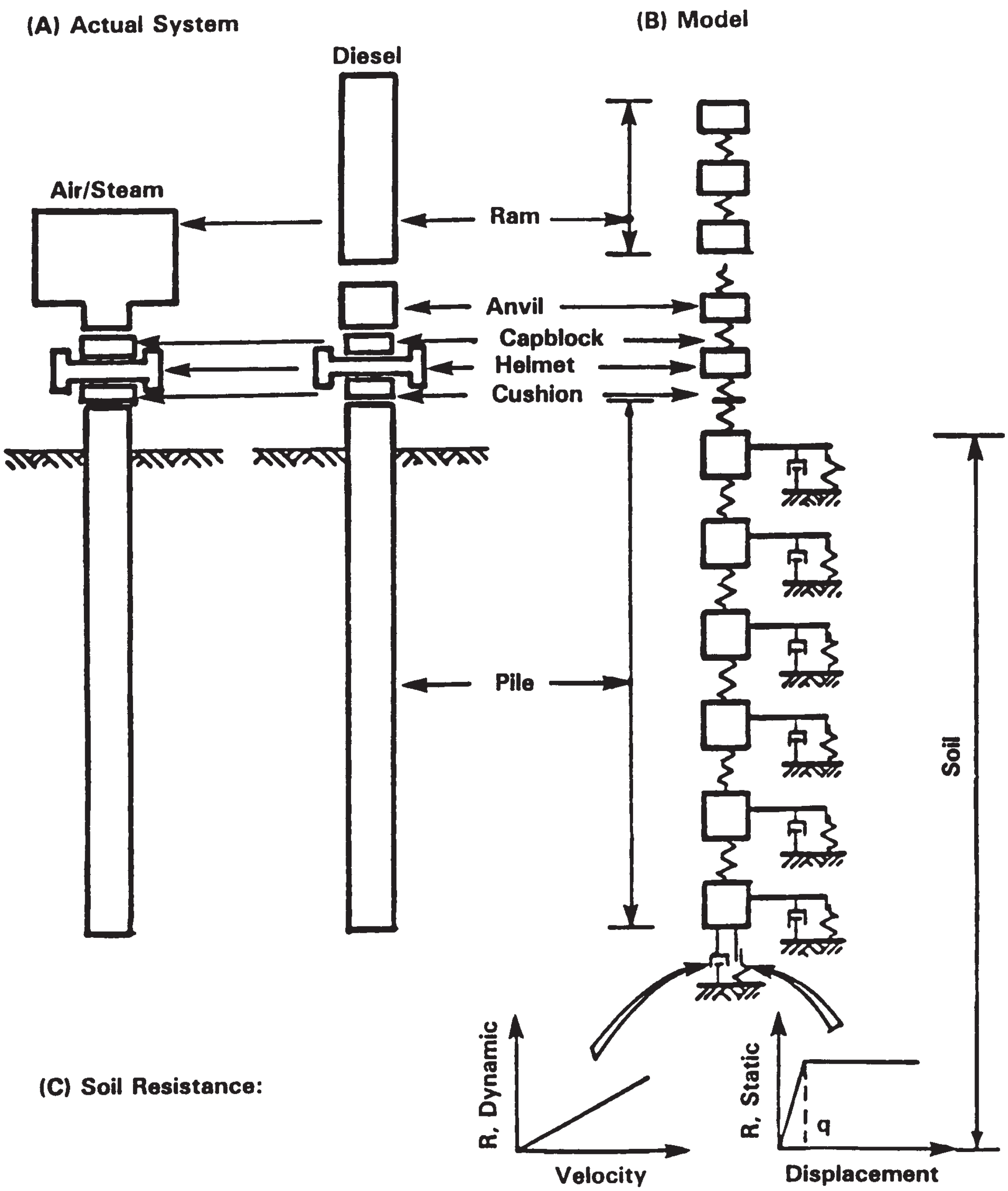
A CORRECTION OF MINUS 0.3 WAS ADDED FOR CORRELATION TO FIELD MEASURED VALUES.

DISTANCE = 4.02 * T ^2 – 0.3

## Example Pile Pick Calculation

This section was not available at the time of publication.

## Example Wave Equation Model




***WAVE EQUATION MODEL***
***(A) THE SYSTEM TO BE ANALYZED; (B) THE WAVE EQUATION MODEL; AND (C) THE COMPONENTS OF THE SOIL RESISTANCE MODEL***

## Example Wave Model and Output

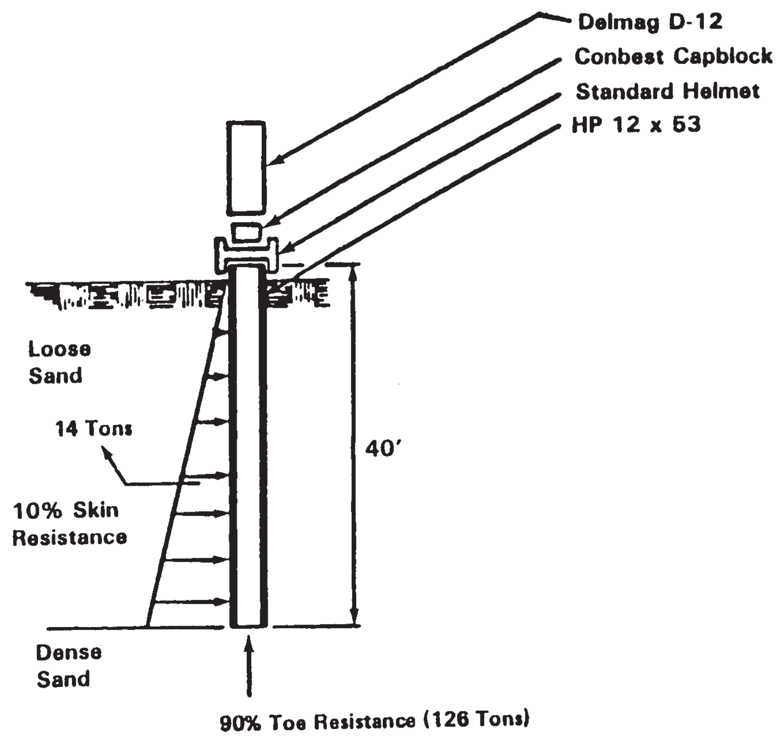
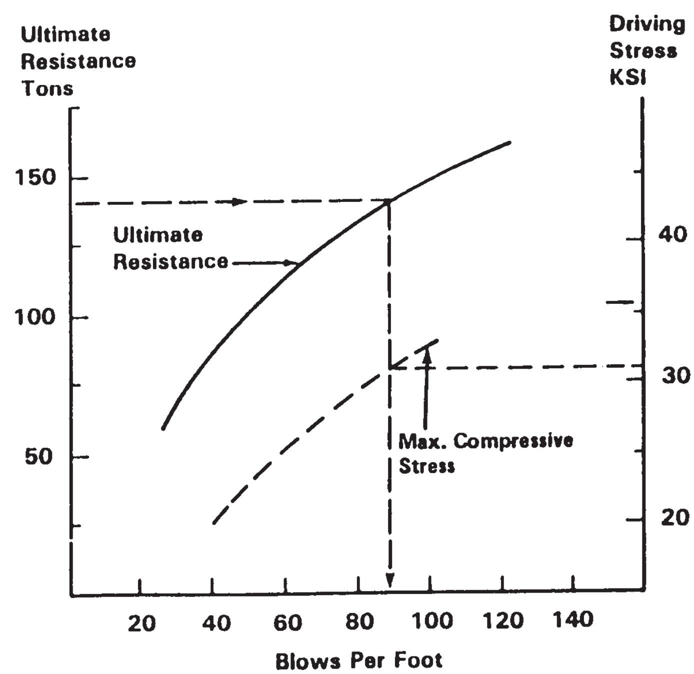
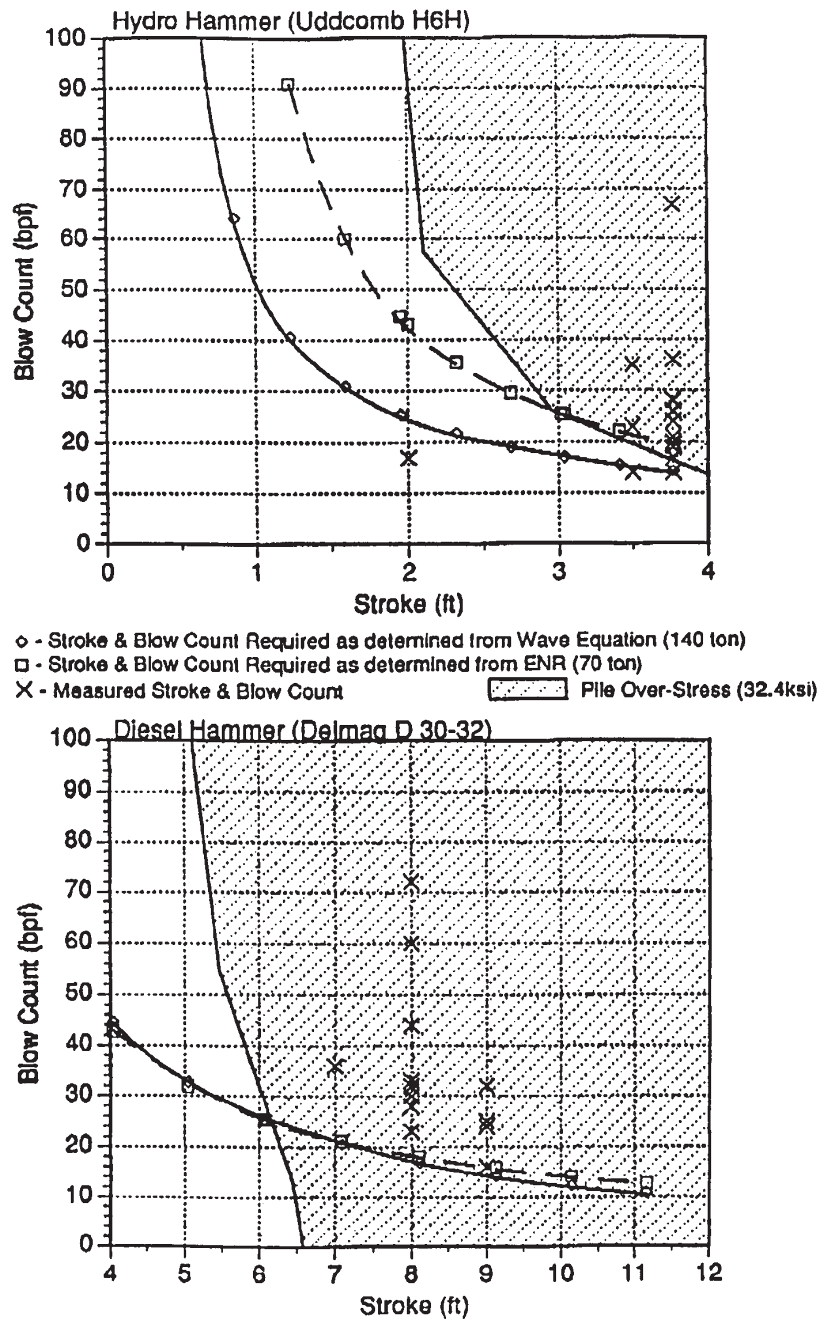



**FIGURE 16-7 DESIGN DATA — EXAMPLE NO. 1**

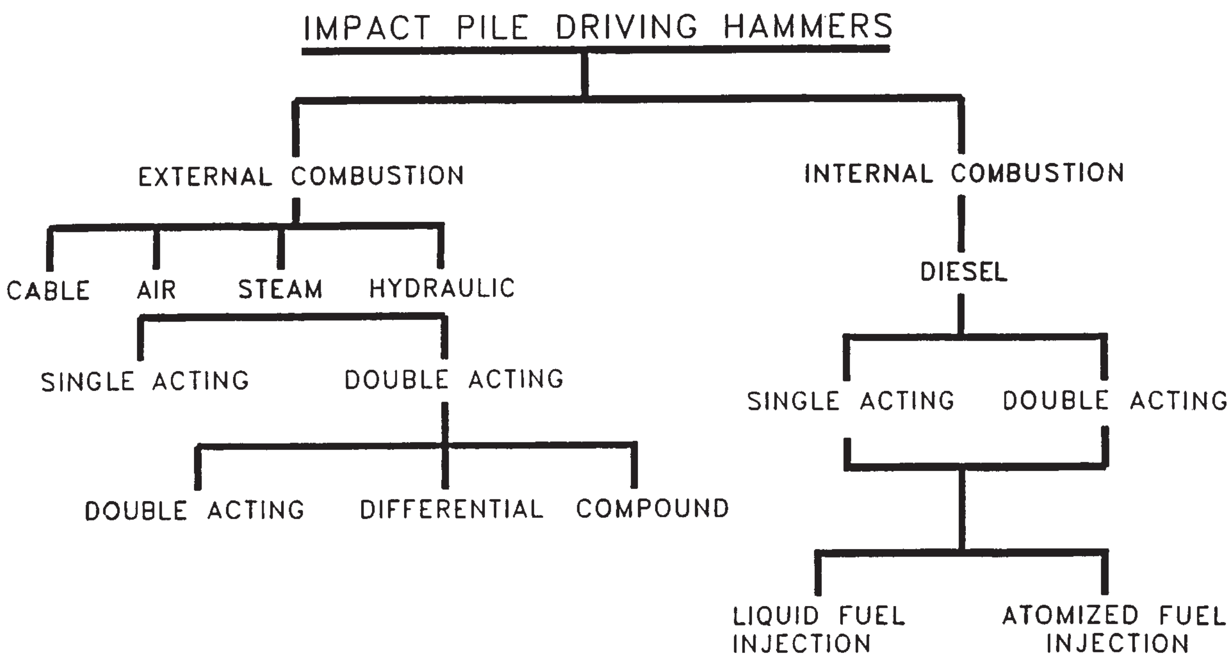


## Example Wave Equation Field Acceptance Charts




**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)**

**Flow chart of Impact Pile Driving Hammers**

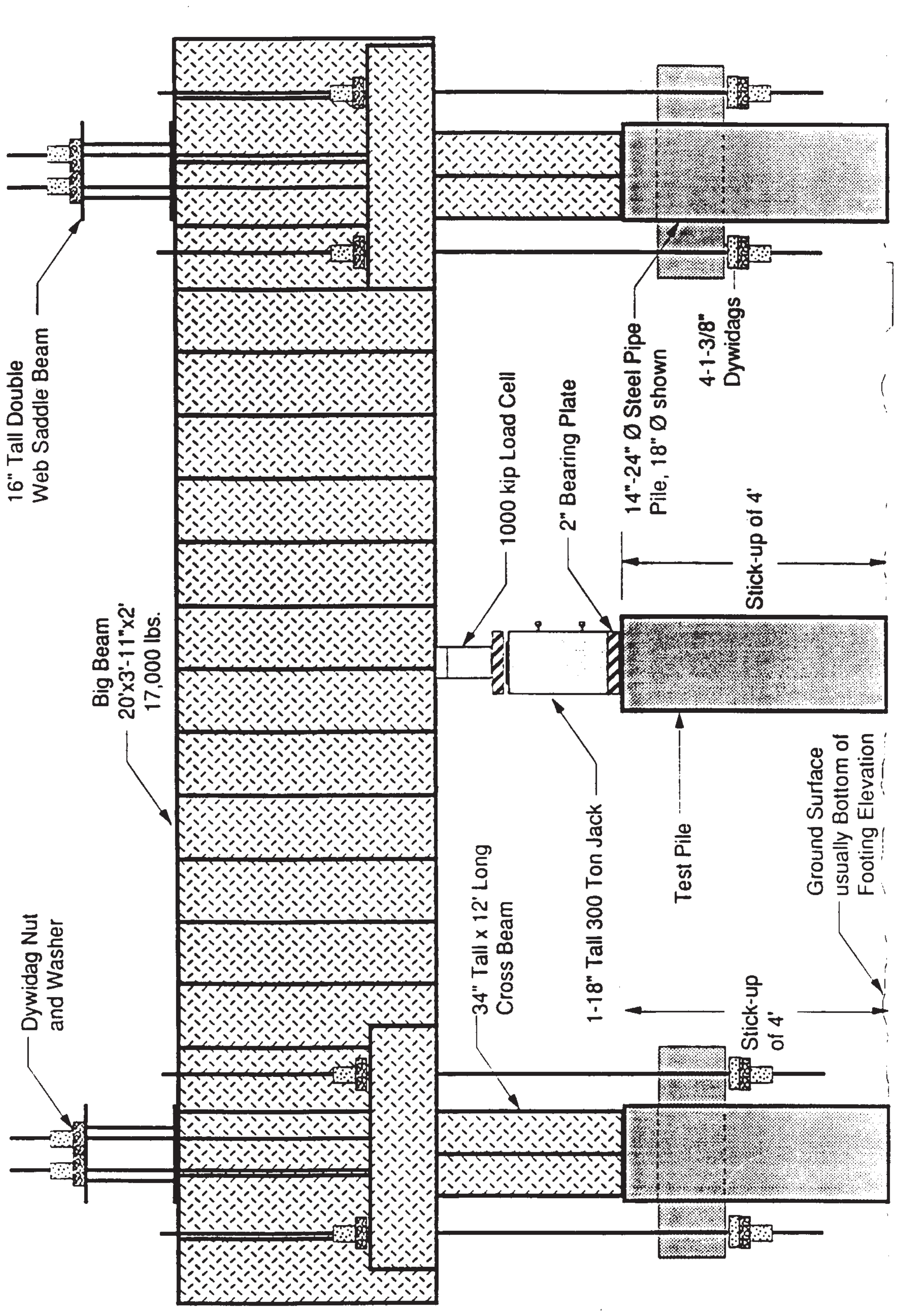


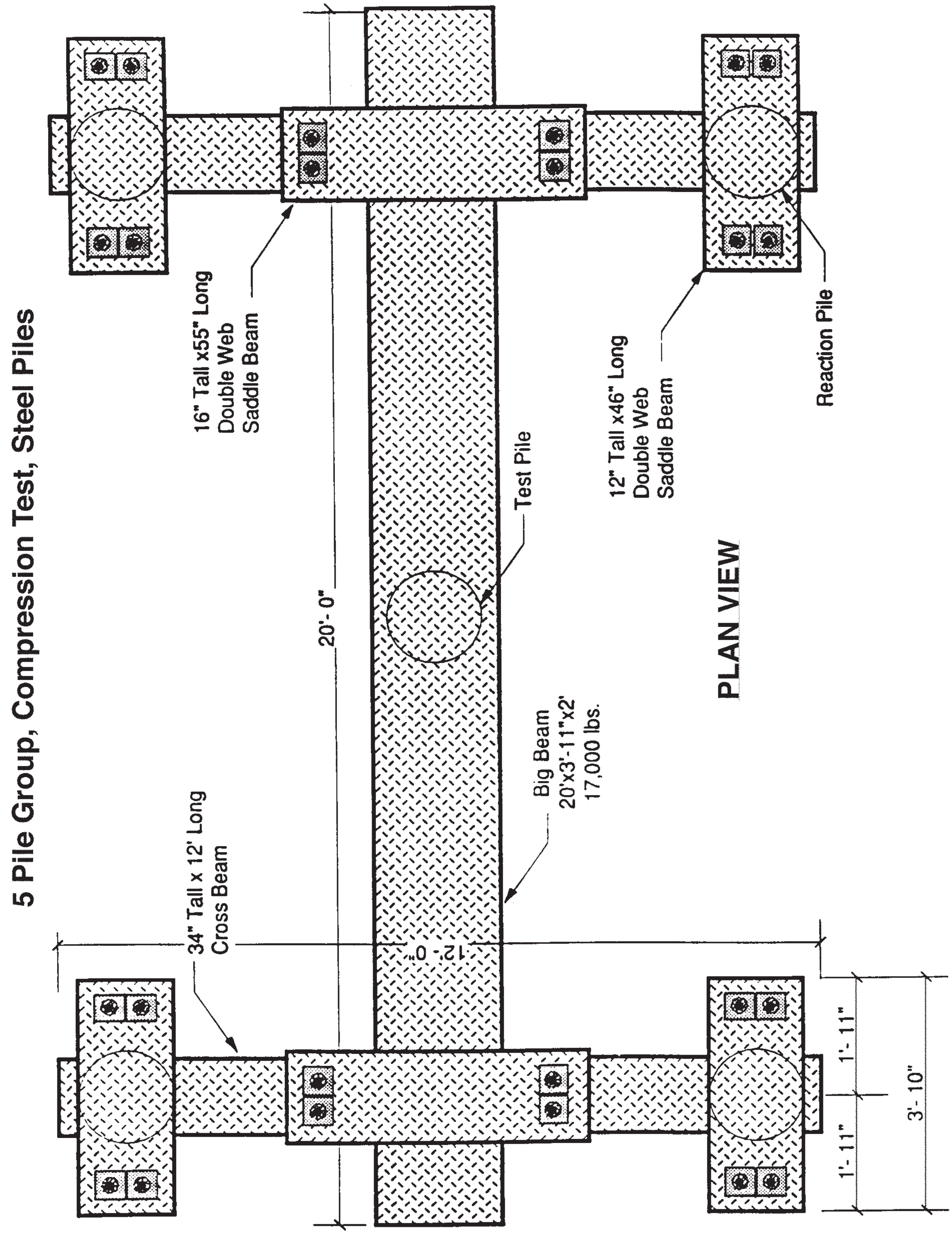
APPENDIX

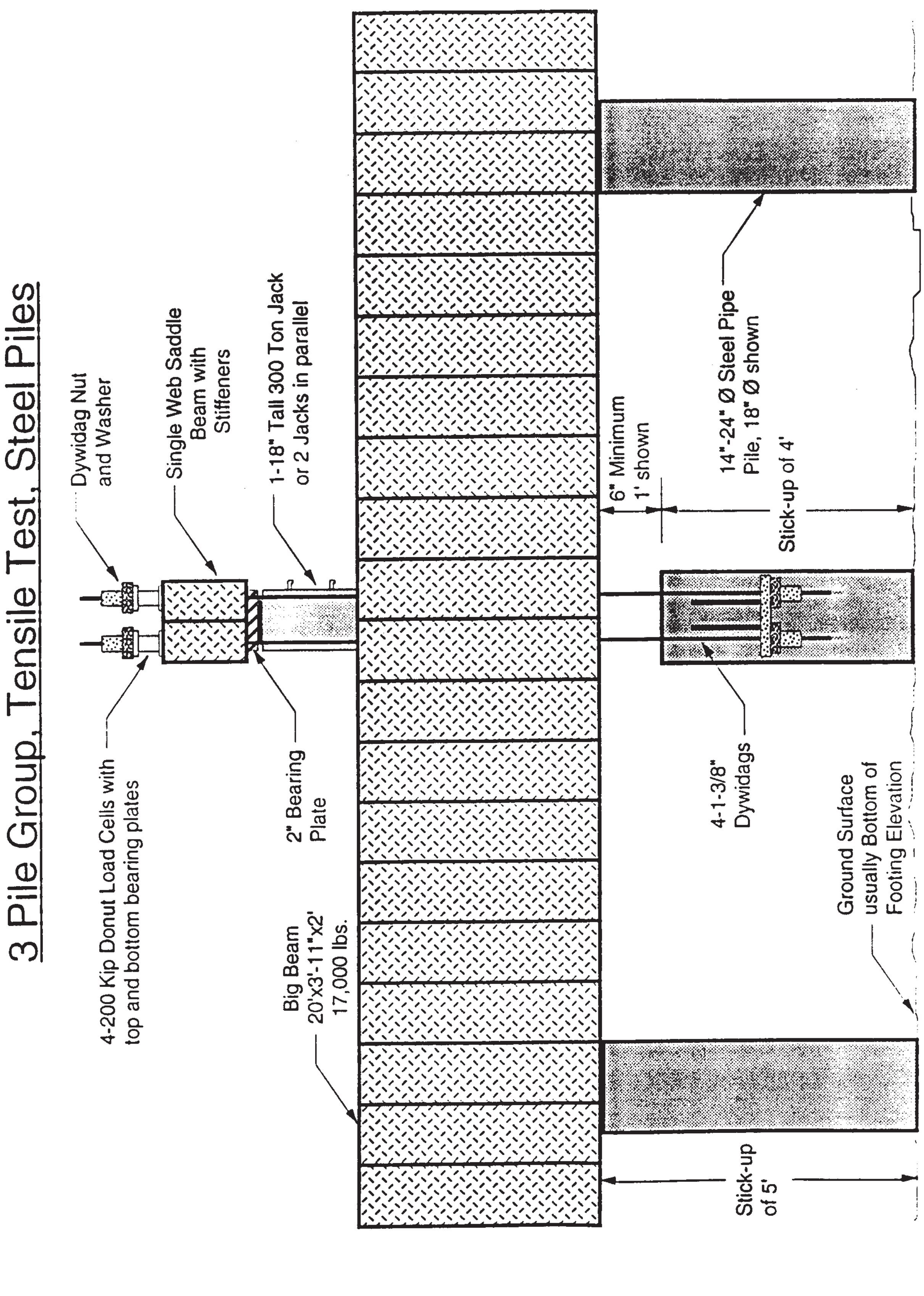
# F Static Pile Load Testing and Dynamic Monitoring

## Table of Contents

**5 Pile Group, Compression Test, Steel Piles**




5 Pile Group, Compression Test, Steel Piles
16" Tall x55" Long Double Web Saddle Beam
12" Tall x46" Long Double Web Saddle Beam
Reaction Pile
Test Pile
20'- 0"
PLAN VIEW
34" Tall x 12' Long Cross Beam
Big Beam 20'x3'-11"x2' 17,000 lbs.
12'- 0"
1'- 11"
1'- 11"
3'- 10"


3 Pile Group, Tensile Test, Steel Piles
4-200 Kip Donut Load Cells with
top and bottom bearing plates
Dywidag Nut
and Washer
Single Web Saddle
Beam with
Stiffeners
1-18" Tall 300 Ton Jack
or 2 Jacks in parallel
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'

Joe

State of California Business, Transportation and Housing Agency

# Memorandum

To : MR. A. P. BEZZONE, Chief
Office of Structure Construction

Attention MR. AMER BATA

Date : September 13, 1993

File No. : 07-Ven-23-PM 23.3/24.2
07-067043
Santa Clara River Bridge
Br. No. 52-118

From : **DEPARTMENT OF TRANSPORTATION**
Division of New Technology, Materials and Research
Office of Geotechnical Engineering

Subject Static Tensile and Compressive Pile Load Test Results; Pier 11.

A static tensile and static compressive pile load test has been completed at Pier 11 of the Santa Clara River Bridge, Bridge Number 52-118. Test results were transmitted verbally to Amer Bata, the Structures Representative, on August 25, 1993. This memo will serve to transmit data from the load tests and commentary on the performance of the tested piles. Attached you will find a copy of the vicinity map, general plan, foundation plan, footing details, load test layouts, plots of test pile displacement versus applied load, and relevant Log of Test Borings.

## Pile Installation

Pier 11 is located at Station 152+22 on the center line of the Santa Clara River Bridge, Route 23, see Attachments A, B, and C. Five HP 12x74, Class 70, Steel H-Piles were driven on August 9, 1993 with a Delmag D30-32 diesel hammer (maximum energy is 73.7 kip-feet with a ram weight of 6.6 kips). The blow count for the final foot of driving was 31 blows per foot, with a stroke of 8.0 feet. All four reaction piles (No. 396, 399, 413, and 416) were driven to a specified tip elevation of 355.8 feet. The test pile 406 was driven to a tip elevation of 356.1. Test pile 406 had a stick-up above ground of 4.0 feet for a total length of 39 feet, and a length of penetration of 35 feet, measured from the bottom of footing elevation, see Attachments D, and F. The bottom of footing elevation is 391.5 feet. There was no predrilling below bottom of footing elevation. The "Driving Records" are shown in Attachment E. Based on the ENR formula and the final blow count of 31 BPF with a stroke of 8.0 feet, the allowable compressive load for test pile 406 is 220 kips.

## Subsurface Conditions

Rotary boring B-2 is 95 feet south of Pier 11 and is the boring nearest Pier 11. From the attached Log-of-Test-Borings dated February 1992, it can be seen that the piles were driven through very dense sand with cobbles with SPT blow counts greater than 70 BPF from the bottom footing at elevation 391.5 feet to elevation

**MR. AMER BATA**
**September 14, 1993**
**Page 2**

383 feet. From elevation 383 feet to 373 feet the soil is very hard clay and sandy clay. From elevation 373 feet to elevation 356 feet, the piles were driven into very dense gravelly sands. The groundwater table was measured in December, 1991, at elevation 398 feet at this boring.

Static Compressive Load Test

The compression test was performed on pile 406 on August 24, 1993, fifteen days after installation. The pile load test group consisted of four anchor or reaction piles and one test pile. See Attachment F for the "Pile Load Test Layout." Twenty-kip load increments were applied to the test pile in five minute load durations to a load of 140 kips, the design load. This load was then cycled down to zero, and then back up to 140 kips in one minute load durations. The pile was then loaded to 280 kips, double the design load, in twenty-kip load increments and five minute load durations. Again the pile was cycled to zero and back up to 280 kips in one-minute load durations. The pile was then loaded to 520 kips, triple the design load, in twenty-kip load increments and five minute load durations. Again the pile was cycled to zero and back up to 520 kips in one-minute load durations. Twenty-kip load increments and five minute load durations were used to increase the compressive load on the pile until pile plunging occurred. A maximum of 600 kips was applied to the pile at which no further load could be applied due to continuous jacking.

After the pile plunged, the pile was loaded in 150 kip increments and one minute load durations until plunging occurred again. The maximum load achieved on this final cycle was 570 kips. A plot of the load versus displacement for the test pile is shown in Attachment G.

Static Tensile Load Test

The tension test was performed on pile 406 on August 25, 1993, sixteen days after installation. The same pile load test group used in the compression test was used in this tension test, including the same reaction piles and test pile. Twenty-kip load increments were applied to the test pile in five minute load durations to a load of 70 kips. This load was then cycled down to zero, and then back up to 70 kips in one minute load durations. The pile was then loaded to 140 kips in twenty-kip load increments and five minute load durations. Again the pile was cycled to zero and back up to 140 kips in one-minute load durations. Twenty-kip load increments and five minute load durations were again used to increase the tensile load on the pile until pile pull-out occurred. A maximum of 240 kips was applied to the pile in tension at which no further load could be applied due to continuous jacking.

MR. AMER BATA
September 14, 1993
Page 3

After the pile plunged, the pile was loaded in 60 kip increments and one minute load durations until plunging occurred again. The maximum load achieved on this final cycle was 240 kips. A plot of the load versus displacement for the test pile is shown in Attachment H.

Interpretation of Load Test Data

The maximum load applied to pile 406 in compression was 600 kips, and the displacement at this load was 1.244 inches. The compressive capacity of the test pile is 480 kips based on Caltrans half-inch failure criterion, and 520 kips based on Davisson's failure criterion. The permanent displacement of pile 406 was 0.79 inches after the first compression cycle to plunge the pile. The maximum load applied to pile 406 in tension was 240 kips, and the displacement at this load was 1.322 inches. The tensile capacity of the test pile is 190 kips based on Caltrans half-inch failure criterion. The permanent displacement of pile 406 was 1.123 inches after the tension test.

Conclusions

The capacity of compression test pile 406 is 480 kips. It is our understanding that the design ultimate compressive requirement for Pier 11 is 280 kips. As such the tested pile meets the compressive pile capacity requirements for this bent. The capacity of tension test pile 406 is 190 kips. It is our understanding that the design ultimate tensile requirement for Pier 11 is 140 kips. As such the tested pile meets the tensile pile capacity requirements for this bent. These two piles were tested in compression and tension, 15 and 16 days after driving, respectively, no additional pile capacity due to pile set-up is expected over time.

For a DelMag D30-32, a blow count for the final foot of driving of 31 BPF, and an 8.0 foot stroke, the ENR formula predicts an allowable load of 220 kips. Based on the results of the static compression load test, however, the allowable load for this pile can be considered to be 240 kips (measured ultimate load divided by 2). Therefore, the ENR formula appears to provide a reasonable estimate of pile capacity for the tested pile and this geology.

These two tests cover the control area from Pier 11 to Abutment 13. Since the tested pile sustained loads above those required, it is recommended that this control area be released for production piles.

Joe

State of California — Business, Transportation and Housing Agency

**Memorandum**

To : MR. A. P. BEZZONE, Chief
Office of Structure Construction

Attention MR. WES JOHNSON

Date : October 14, 1992

File No. : 07-LA-105/405
07-060283
NW Conn. Tunnel
Br. No. 53-2437

From : **DEPARTMENT OF TRANSPORTATION**
Division of New Technology, Materials and Research

Subject Dynamic Pile Test

A dynamic pile test using the Pile Driving Analyzer (PDA) was performed on October 6 and 7, 1992, at the left footing, Station 42+80, of the North West Connector Tunnel. The 24 foot, 14"x14" Class 100 prestressed concrete pile #44A was driven with a DelMag 30-32 hammer. The test pile was predrilled 10 feet with a 14" diameter auger, as were other piles within 50 feet, due to hard driving. As shown on the attached Log of Test Borings the test pile was driven through stiff sandy and clayey silt, and was founded in dense fine sand at specified tip elevation 15.0 feet. The test pile was monitored during initial driving between a penetration of 5 feet and 19 feet. At 19 feet of penetration a compressive capacity of approximately 280 kips was indicated. The pile was not monitored between 19 feet and 23 feet of penetration. The pile was left to set-up for one day and then monitored during the restrike to final penetration of 24 feet. A DelMag 36-32 was used for the restrike.

The one foot restrike indicated the compressive capacity increased approximately 100 kips to 380 kips. Compressive stresses on the pile determined from the PDA were about 3500 psi which is well below the allowable driving stress of 5250 psi for this pile. The factor of safety for pile compressive capacity is estimated to be 1.9.

Conclusions

Although the factor of safety for pile compressive capacity is estimated at 1.9 rather than 2.0, as would be required for a static compressive load test, this value is, given the accuracy of pile dynamic monitoring equipment and techniques, sufficiently close to 2.0 as to warrant no corrective action. Furthermore, it is reasonable to expect additional pile set-up will occur beyond the one day allowed for in this test, thus providing a factor of safety for compressive pile capacity in excess of 2.0.

Measured compressive driving stresses were well below allowable levels. As such, no pile damage due to over stressing is suspected.

Mr. Wes Johnson
October 14, 1992
Page 2

If you have any questions regarding these load test results, please contact me (916)454-6952 (ATSS 497-6952).

REGISTERED PROFESSIONAL ENGINEER
DOUGLAS E. BRITTSAN
No. C043854
Exp. 10/93
CIVIL
STATE OF CALIFORNIA

Douglas E. Brittsan

DOUGLAS E. BRITTSAN, P.E.
Transportation Civil Engineer
Office of Geotechnical Engineering

Attachments

cc: RHPrysock-OGE (w/o attachments)
Dan Speer-OGE
OEG-2 copies (w/attachments)
Ted Jensen-OSD
Bridge Files-OEG
Pile Test Files -OGE

# APPENDIX

# G Slurry Displacement Piles

## Table of Contents

# SECTION 1
# MUD WEIGHT (DENSITY)

**1.1 Description.** This test procedure is a method for determining the weight of a given volume of liquid. Mud weight may be expressed as pounds per gallon (lb/gal), pounds per cubic foot (lb/ft³), grams per cubic centimeter (g/cm³), or kilograms per cubic meter (kg/m³).

**1.2 Equipment**

**a.** Any instrument of sufficient accuracy to permit measurement within ±0.1 lb/gal (or 0.5 lb/ft³, 0.01 g/cm³, 10 kg/m³) may be used. The mud balance (Fig. 1.1 and 1.2) is the instrument generally used for mud weight determinations. The mud balance is designed such that the mud cup, at one end of the beam, is balanced by a fixed counterweight at the other end, with a sliding-weight rider free to move along a graduated scale. A level-bubble is mounted on the beam to allow for accurate balancing. (Attachments for extending the range of the balance may be used when necessary).

**b.** Thermometer: 32-220°F (0-105°C)

**1.3 Procedure**

**a.** The instrument base should be set on a flat, level surface.

**b.** Measure the temperature of the mud and record on the Drilling Mud Report form.

**c.** Fill the clean, dry cup with mud to be tested; put the cap on the filled mud cup and rotate the cap until it is firmly seated. Insure that some of the mud is expelled through the hole in the cap in order to free any trapped air or gas (see Appendix D for Air Removal).

**d.** Holding cap firmly on mud cup (with cap hole covered), wash or wipe the outside of the cup clean and dry.

**e.** Place the beam on the base support and balance it by moving the rider along the graduated scale. Balance is achieved when the bubble is under the center line.

**f.** Read the mud weight at edge of the rider toward the mud cup. Make appropriate corrections when a range extender is used.

**1.4 Procedure-Calibration.** The instrument should be calibrated frequently with fresh water. Fresh water should give a reading of 8.3 lb/gal or 62.3 lb/ft³ (1000 kg/m³) at 70°F (21°C). If it does not, adjust the balancing screw or the amount of lead shot in the well at the end of the graduated arm as required.

**1.5 Calculation**

**a.** Report the mud weight to the nearest 0.1 lb/gal or 0.5 lb/ft³ (0.01 g/cm³, 10 kg/m³).

**b.** To convert the reading to other units, use the following:

$$\text{Density} = g/cm^3 = \frac{lb/ft^3}{62.3} = \frac{lb/gal}{8.345} \quad \text{(a)}$$

$$kg/m^3 = (lb/ft^3)(16) = (lb/gal)(120) \quad \text{(b)}$$

$$\text{Mud gradient, psi/ft} = \frac{lb/ft^3}{144}, \ \frac{lb/gal}{19.24}, \ \text{or} \ \frac{kg/m^3}{2309} \quad \text{(c)}$$

**TABLE 1.1**
**DENSITY CONVERSION**

| 1 | 2 | 3 | 4 |
|---|---|---|---|
| pounds per gallon (lb/gal) | pounds per cubic foot (lb/ft³) | grams per cubic centimeter (g/cm³)* | kilograms per cubic meter (kg/m³) |
| 6.5 | 48.6 | 0.78 | 780 |
| 7.0 | 52.4 | 0.84 | 840 |
| 7.5 | 56.1 | 0.90 | 900 |
| 8.0 | 59.8 | 0.96 | 960 |
| 8.3 | 62.3 | 1.00 | 1000 |
| 8.5 | 63.6 | 1.02 | 1020 |
| 9.0 | 67.3 | 1.08 | 1080 |
| 9.5 | 71.1 | 1.14 | 1140 |
| 10.0 | 74.8 | 1.20 | 1200 |
| 10.5 | 78.5 | 1.26 | 1260 |
| 11.0 | 82.3 | 1.32 | 1320 |
| 11.5 | 86.0 | 1.38 | 1380 |
| 12.0 | 89.8 | 1.44 | 1440 |
| 12.5 | 93.5 | 1.50 | 1500 |
| 13.0 | 97.2 | 1.56 | 1560 |
| 13.5 | 101.0 | 1.62 | 1620 |
| 14.0 | 104.7 | 1.68 | 1680 |
| 14.5 | 108.5 | 1.74 | 1740 |
| 15.0 | 112.5 | 1.80 | 1800 |
| 15.5 | 115.9 | 1.86 | 1860 |
| 16.0 | 119.7 | 1.92 | 1920 |
| 16.5 | 123.4 | 1.98 | 1980 |
| 17.0 | 127.2 | 2.04 | 2040 |
| 17.5 | 130.9 | 2.10 | 2100 |
| 18.0 | 134.6 | 2.16 | 2160 |
| 18.5 | 138.4 | 2.22 | 2220 |
| 19.0 | 142.1 | 2.28 | 2280 |
| 19.5 | 145.9 | 2.34 | 2340 |
| 20.0 | 149.6 | 2.40 | 2400 |
| 20.5 | 153.3 | 2.46 | 2460 |
| 21.0 | 157.1 | 2.52 | 2520 |
| 21.5 | 160.8 | 2.58 | 2580 |
| 22.0 | 164.6 | 2.64 | 2640 |
| 22.5 | 168.3 | 2.70 | 2700 |
| 23.0 | 172.1 | 2.76 | 2760 |
| 23.5 | 175.8 | 2.82 | 2820 |
| 24.0 | 179.5 | 2.88 | 2880 |

*Same as specific gravity (sg).

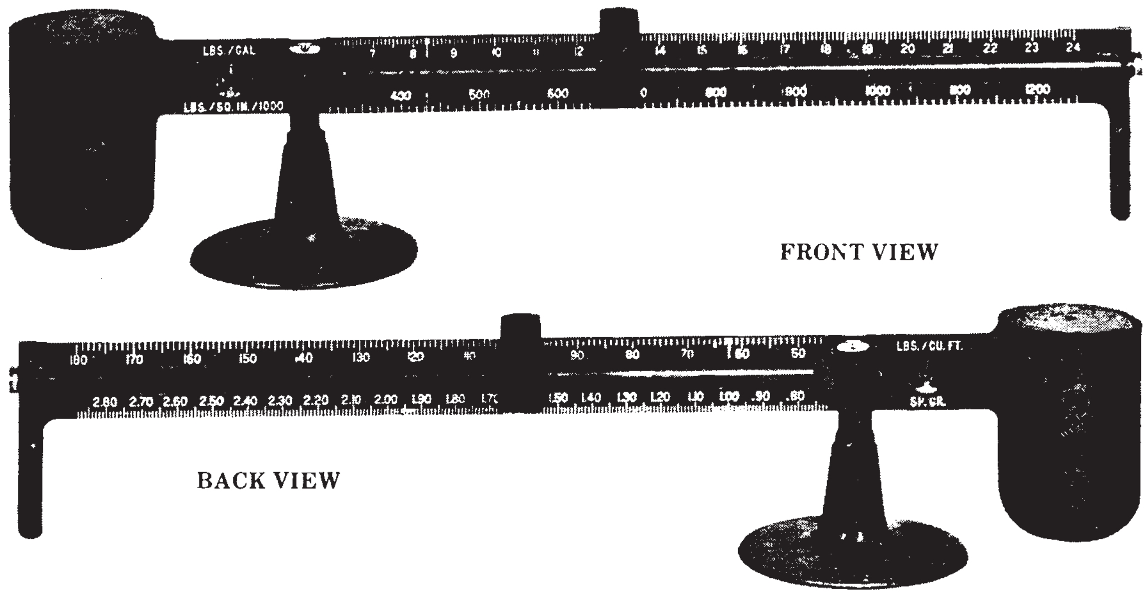



**FIG. 1.1**
**MUD BALANCE**

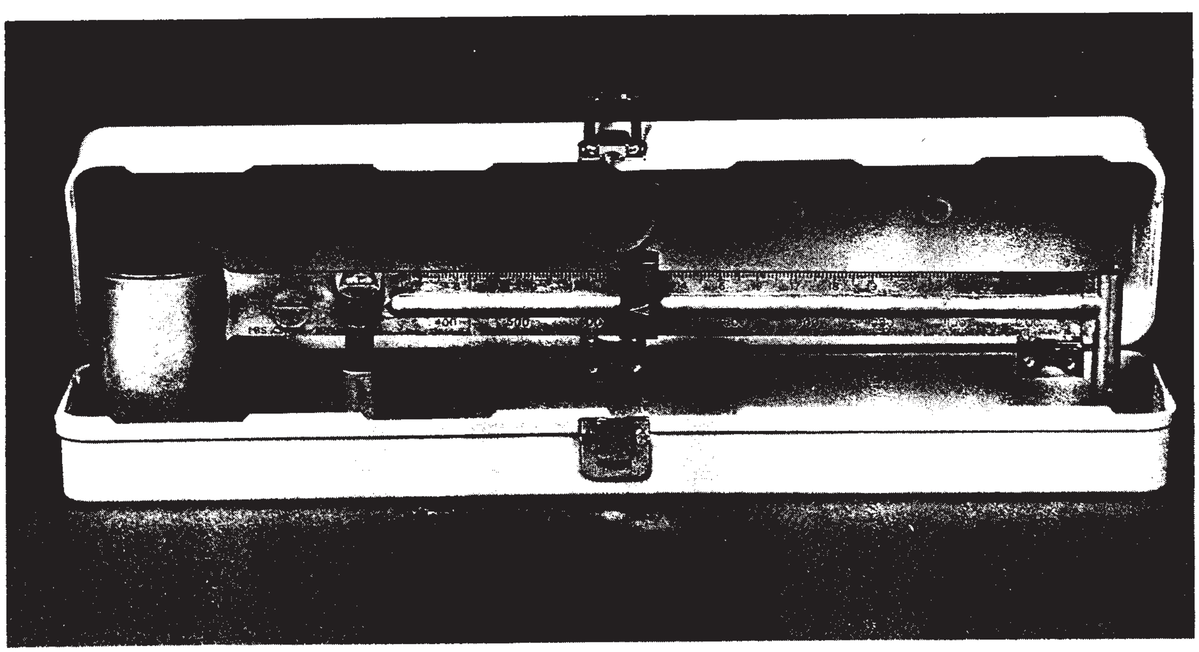


**FIG. 1.2**
**MUD BALANCE AND CASE**

# SECTION 2
# VISCOSITY AND GEL STRENGTH

### 2.1 Description

**a.** The following instruments are used to measure viscosity and/or gel strength of drilling fluids:

1. Marsh Funnel — a simple device for indicating viscosity on a routine basis.
2. Direct-indicating viscometer — a mechanical device for measurement of viscosity at varying shear rates.

**b.** Viscosity and gel strength are measurements that relate to the flow properties of muds. The study of deformation and flow of matter is rheology. An in-depth discussion of rheology is made in API Bulletin 13D: *The Rheology of Oil-Well Drilling Fluids.*

## MARSH FUNNEL

### 2.2 Equipment

**a.** Marsh Funnel

A Marsh Funnel (see Fig. 2.1) is calibrated to outflow one quart (946 $cm^3$) of fresh water at a temperature of 70 ±5°F (21 ±3°C) in 26 ± 0.5 seconds. A graduated cup is used as a receiver.

Specifications
Funnel Cone
Length ........................ 12.0 in. (305 mm)
Diameter....................... 6.0 in. (152 mm)
Capacity to bottom of screen........... 1500 $cm^3$
Orifice
Length .........................2.0 in. (50.8 mm)
Inside Diameter ............... 3/16 in. (4.7 mm)
Screen .................................. 12 mesh
Has 1/16 in. (1.6 mm) openings and is fixed at a level ¾ in. (19.0 mm) below top of funnel.

**b.** Graduated cup: one-quart

**c.** Stopwatch

**d.** Thermometer: 32-220°F (0-105°C)

### 2.3 Procedure

**a.** Cover the funnel orifice with a finger and pour freshly sampled drilling fluid through the screen into the clean, upright funnel. Fill until fluid reaches the bottom of the screen.

**b.** Remove finger and start stopwatch. Measure the time for mud to fill to one-quart (946 $cm^3$) mark of the cup.

**c.** Measure temperature of fluid in degrees F (C).

**d.** Report the time to nearest second as Marsh Funnel viscosity. Report the temperature of fluid to nearest degree F (C).

FIG. 2.1
MARSH FUNNEL AND CUP

# SECTION 5
# SAND

**5.1 Description.** The sand content of mud is the volume percent of particles larger than 74 microns. It is measured by a sand-screen set (see Fig. 5.1).

**5.2 Equipment**

**a.** 200-mesh sieve, 2.5 in. (63.5 mm) in diameter

**b.** Funnel to fit sieve

**c.** Glass measuring tube marked for the volume of mud to be added. The tube is graduated from 0 to 20 percent in order to read directly the percentage of sand.

**5.3 Procedure**

**a.** Fill the glass measuring tube with mud to the "mud" mark. Add water to the next mark. Close the mouth of the tube and shake vigorously.

**b.** Pour the mixture onto the clean, wet screen. Discard the liquid passing through the screen. Add more water to the tube, shake, and again pour onto the screen. Repeat until the tube is clean. Wash the sand retained on the screen to free it of any remaining mud.

**c.** Put the funnel upside down over the top of the sieve. Slowly invert the assembly and insert the tip of the funnel into the mouth of the glass tube. Wash the sand into the tube by playing a fine spray of water through the screen. Allow the sand to settle. From the graduations on the tube, read the volume percent of the sand.

**d.** Report the sand content of the mud in volume percent. Report the source of the mud sample, i.e., above shaker, suction pit, etc. Coarse solids other than sand will be retained on the screen (e.g., lost circulation material) and the presence of such solids should be noted.

FIG. 5.1
SAND-CONTENT SET

State of California Business, Transportation and Housing Agency

# Memorandum

: MR. FRANK YANAMURA, Chief
Office of Structure Construction

Attention: MR. MOHAMED FOUAD
Resident Engineer

Date : September 8, 1994

File No. : 11-SD-805-17.5/17.8
11-058503
Mission Valley Viaduct
Indicator Pile Test Program

From : **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Office of Structural Foundations

Subject : Gamma Test Results

Gamma test results are attached for Shafts 2, 3, and 4 at Site 3 of the Mission Valley Viaduct Indicator Pile Test Program. Testing conducted by State personnel used a CPN Model 502 DR Depth Probe equipped with a 10 millicurie source of Cs137. Results presented herein were initially discussed during the Foundation Review meeting held on Wednesday, August 31, 1994 in Sacramento. A summary of relevant information is as follows:

Cast-in-place concrete Pile No. 2 consisted of an 24 m (80 ft) long by 1200 mm (48 in) diameter drilled shaft. Installation, which occurred between August 23 and August 29, utilized full length casing and polymer slurry drill fluid to hold the excavation open. Gamma testing occurred on August 29, 1994. No problems were reported related to tube blockage or water infiltration. In general pile integrity looked good. The only significant anomaly noted was at the top 1 m (3 ft) to 2 m (6 ft) of the shaft. On production piles, such an anomaly would warrant further evaluation. Due to the proximity of the anomaly to the ground surface, excavating surface material to allow direct evaluation would be the preferable course of action.

Cast-in-place concrete Pile No. 3 consisted of an 24 m (80 ft) long by 1200 mm (48 in) diameter drilled shaft. Installation, which occurred on August 25 and 26, utilized bentonite slurry to hold the excavation open. Gamma testing occurred on August 29, 1994. The only problem noted during testing occurred when the gamma probe would not pass beyond a depth of 21 m (70 ft) in tube No. 4. The other three tubes were read without difficulty. In general, the only significant anomaly noted was at the top 0.1 m (1 ft) to 0.6 m ( 2 ft) of the shaft. On production piles, such anomalies would warrant further evaluation. As with Pile No. 2, excavating surface material to allow direct evaluation would be the preferable course of action.

Cast-in-place concrete Pile No. 4 consisted of an 24 m (80 ft) long by 1200 mm (48 in) diameter drilled shaft. Installation, which occurred on August 22 through 24, utilized a polymer slurry to hold the excavation open. Gamma testing occurred on August 25, 1994. Tube No. 4 was read for the entire length of the shaft, while tubes 1 through 3 were blocked at depths of 20.2 m (66.4 ft), 4.5 m (14.8 ft), and 11.5 m (37.8 ft) respectively. Although these problems with probe access could be related to shaft defects, a more likely conclusion is that they occurred due to improper handling of the reinforcement cage during shaft construction. Accounts from field personnel support this assumption. Of the access tubes tested, the only significant anomaly noted was in Tube No. 4 at the bottom of the shaft. In an effort to further investigate the anomaly encountered

MR. MOHAMED FOUAD
September 8, 1994
Page 2

and to compensate for blocked access tubes, four additional 50 mm (2 in) diameter access holes were drilled. However, after water was encountered within each hole, efforts to conduct additional gamma testing was suspended by the State. At the consent of the Resident Engineer, the Contractor hired a consultant to test the original blocked shafts with a 25 mm (1 in) diameter waterproof probe having a 100 millicurie source of Cs137. Test results developed with this probe, faxed to this office on September 2, 1994, confirm the existence of an anomaly at the bottom of this shaft. If this was production work, we would recommend that this shaft be rejected.

If you have any questions or comments, please call me at (916) 227-7163 or (CalNet) 498-7163.

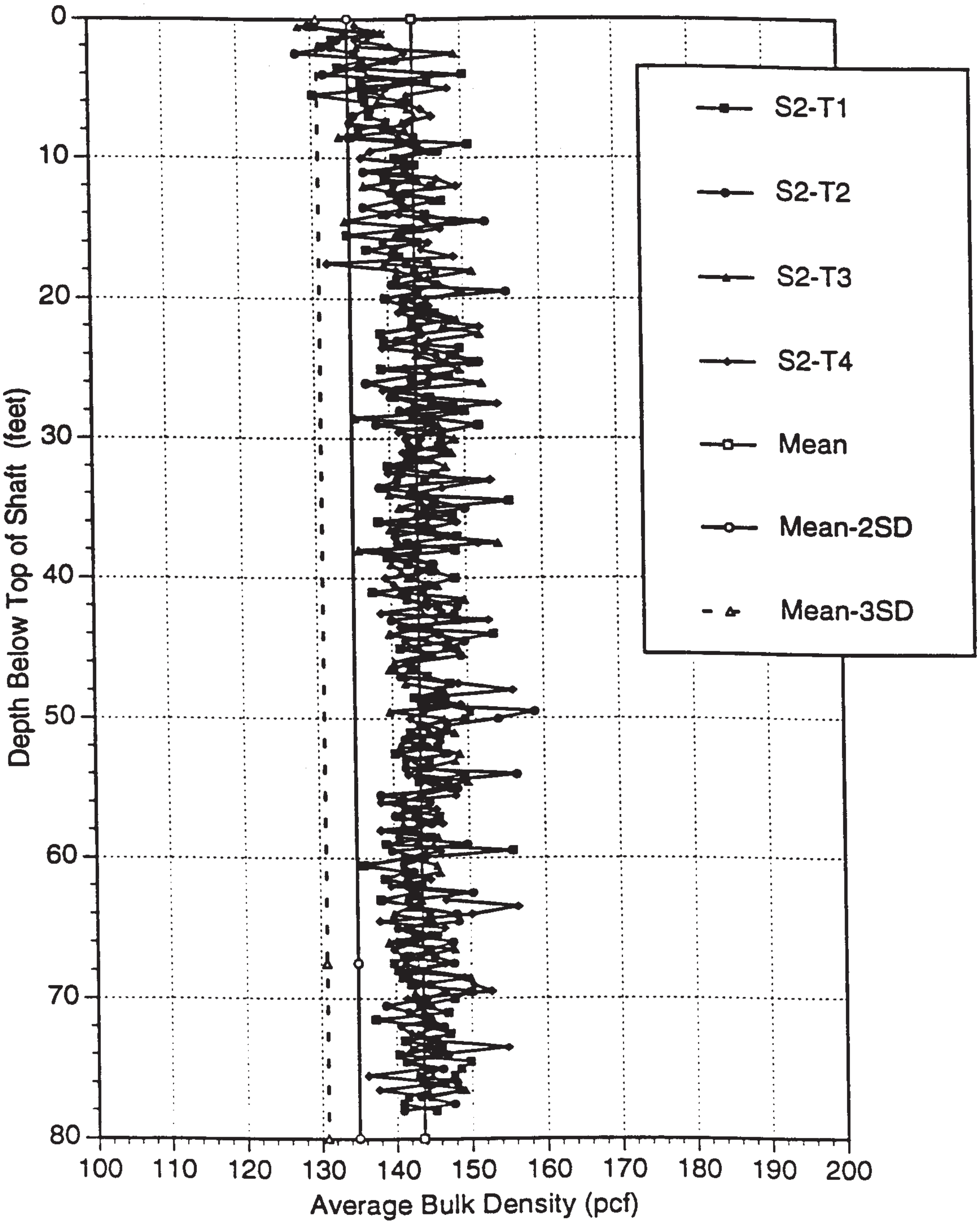
KEITH E. SWANSON
Associate Materials & Research Engineer
Foundation Testing and Instrumentation Section

Attachments

KES/jlm

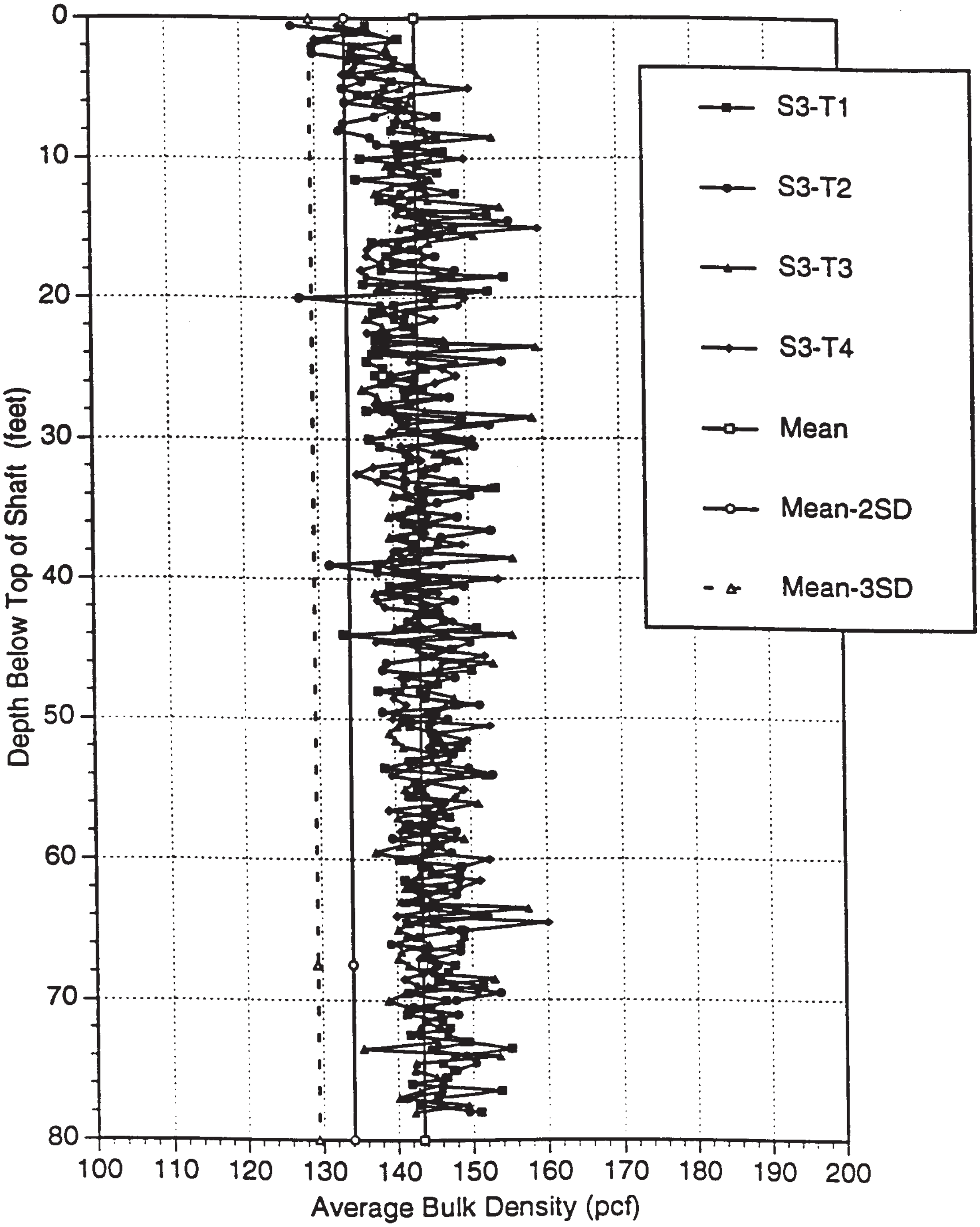
cc: ELeivas
DSpeer
AAbghari
TJensen-OSD
RStott-OSC
AAsnaashari-OSD
Pile Test File-OGE

REGISTERED PROFESSIONAL ENGINEER
KEITH E. SWANSON
No. GE 2267
Exp. 3/31/95
GEOTECHNICAL
STATE OF CALIFORNIA



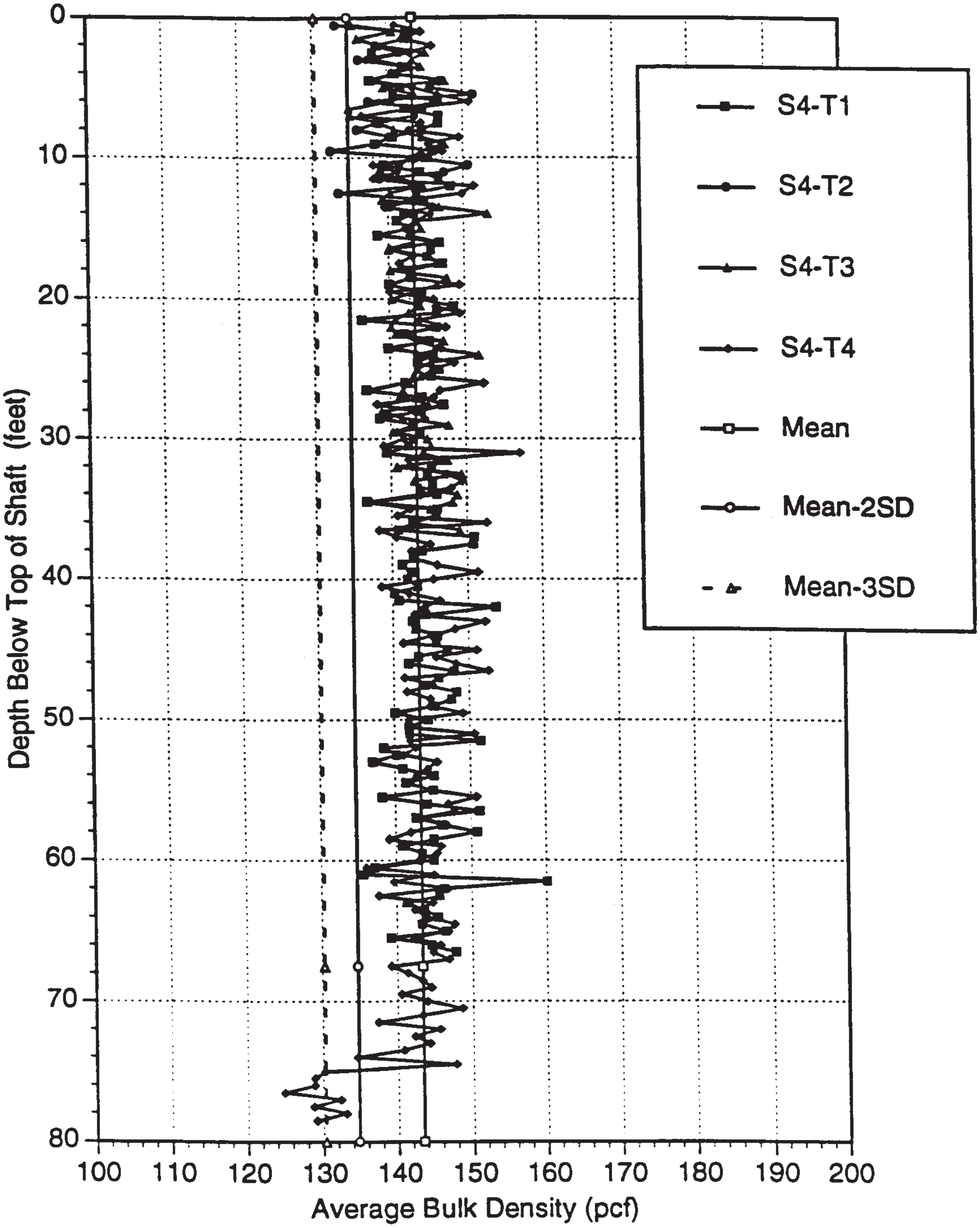

**Gamma Test Data**
**Mission Valley Viaduct IPTP**

| | | |
|---|---|---|
| 11-SD-805 | Site 3 | Bridge No.57-0718F |
| 11-058503 | Shaft 2 | Tested 8/94 |




**Gamma Test Data**
**Mission Valley Viaduct IPTP**

| | | |
|---|---|---|
| 11-SD-805 | Site 3 | Bridge No.57-0718F |
| 11-058503 | Shaft 3 | Tested 8/94 |




**Gamma Test Data**
**Mission Valley Viaduct IPTP**

| | | |
|---|---|---|
| 11-SD-805 | Site 3 | Bridge No.57-0718F |
| 11-058503 | Shaft 4 | Tested 8/94 |

# APPENDIX H

# Tiebacks, Tiedowns, and Soil Nails

## Table of Contents

# Tiebacks – Specific Project

A retaining wall, incorporating tiebacks and Cast-In-Drilled Hole (CIDH) concrete piles, was constructed near the Route 805/163 interchange in San Diego in order to protect the foundation of a multistory apartment house. One wall of the apartment house was located about 4 to 6 feet from the retaining wall layout line and about 20 feet above the new street grade in front of the retaining wall.

The Contractor first excavated approximately 5 feet below original ground and then drilled the holes for the tiebacks. The tiebacks were approximately 30 feet in length and were drilled at an angle of 30 from the horizontal. Next came the drilling of the 60 inch diameter CIDH piles on 10 foot centers. The tieback anchors were then placed and the bottom 15 feet were grouted. The CIDH piles were cast in full section to the elevation of the bottom of the retaining wall. The Contractor then dropped a form down in the hole and cast the top portion (20 feet) in approximately a half section.

After the grout and concrete had reached the required strength, the rods were stressed and left ungrouted for 7 days so they could be tested for residual stress. They were then grouted and the remaining street excavation completed (15 feet). A retaining wall was then constructed with the half section of the CIDH pile cast in with the wall section.

The pile spacing and the pile half section were designed to provide temporary support for the vertical face of the excavation during construction of the retaining wall.

# APPENDIX I

# Cofferdams and Seal Courses

Table of Contents

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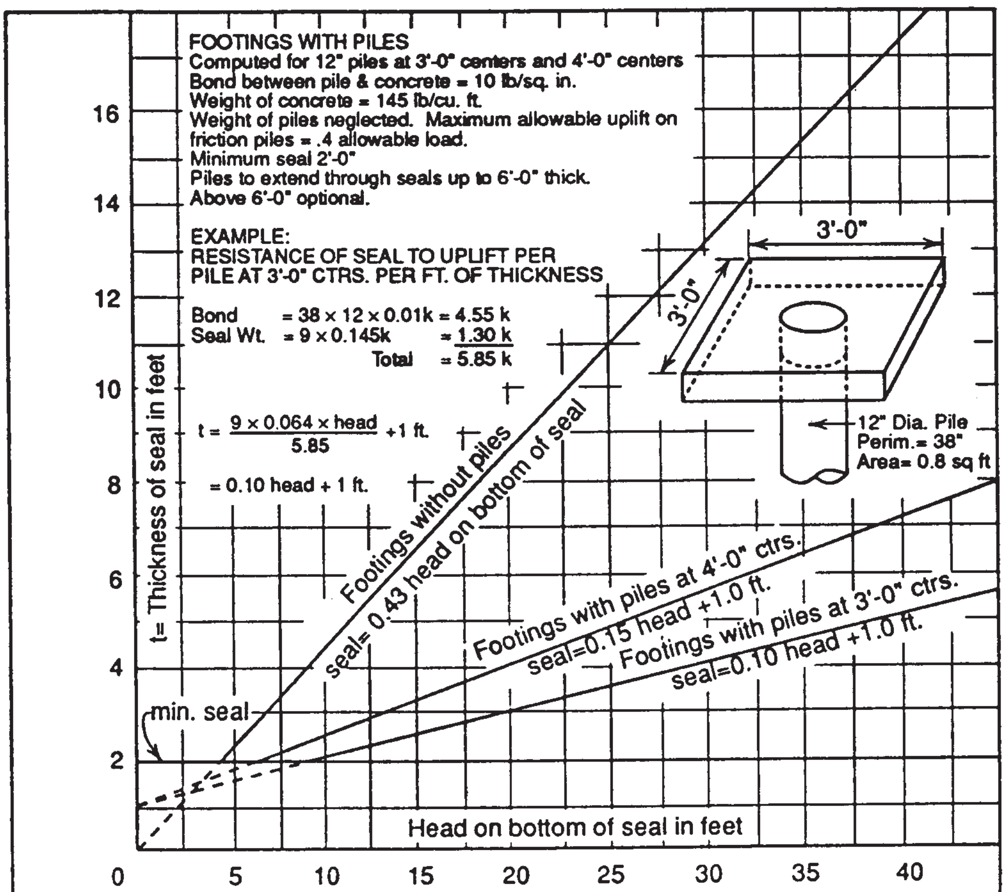
## Seal Courses

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.

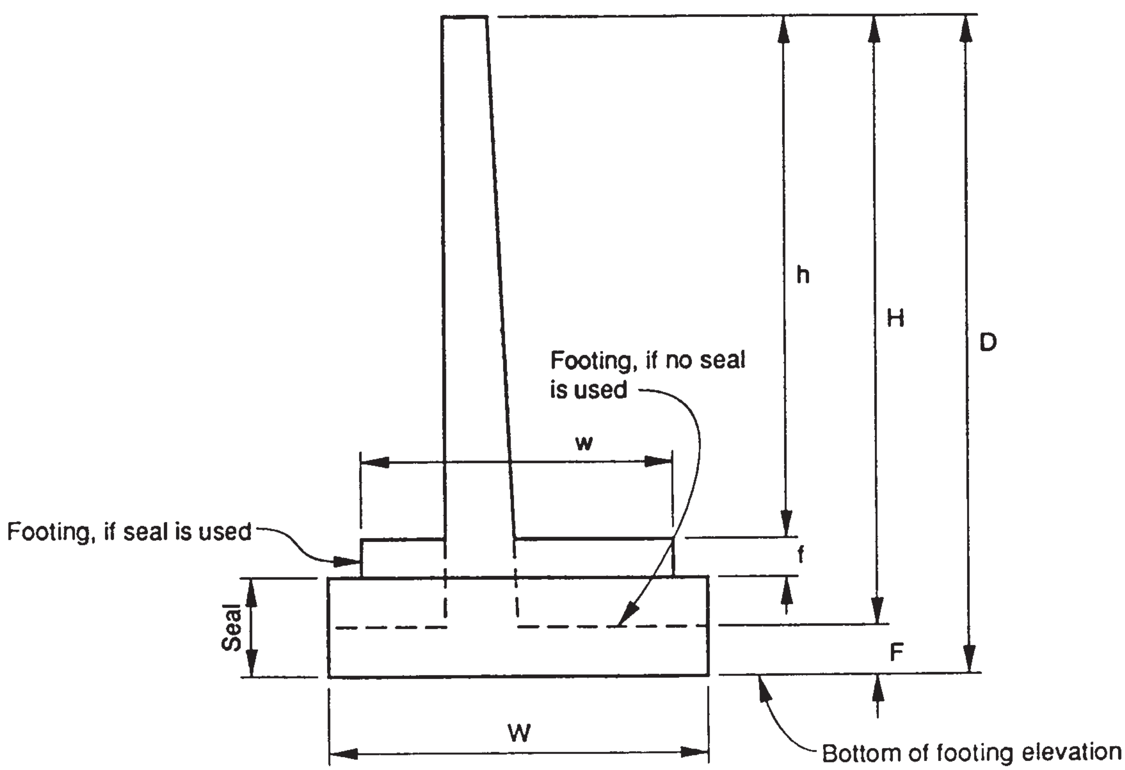
An additional 1 ft. of seal is provided for pile footings to allow for irregularity to top and bottom surfaces.

"Head" may be greater than the depth of standing water if footing is located in heavy viscous soil such as "Bay Mud"

Caltrans BRIDGE DESIGN AIDS JANUARY 1982

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**

# 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**