BORED PILES

(English Translation of French Publication, LES PIEUX FORES)

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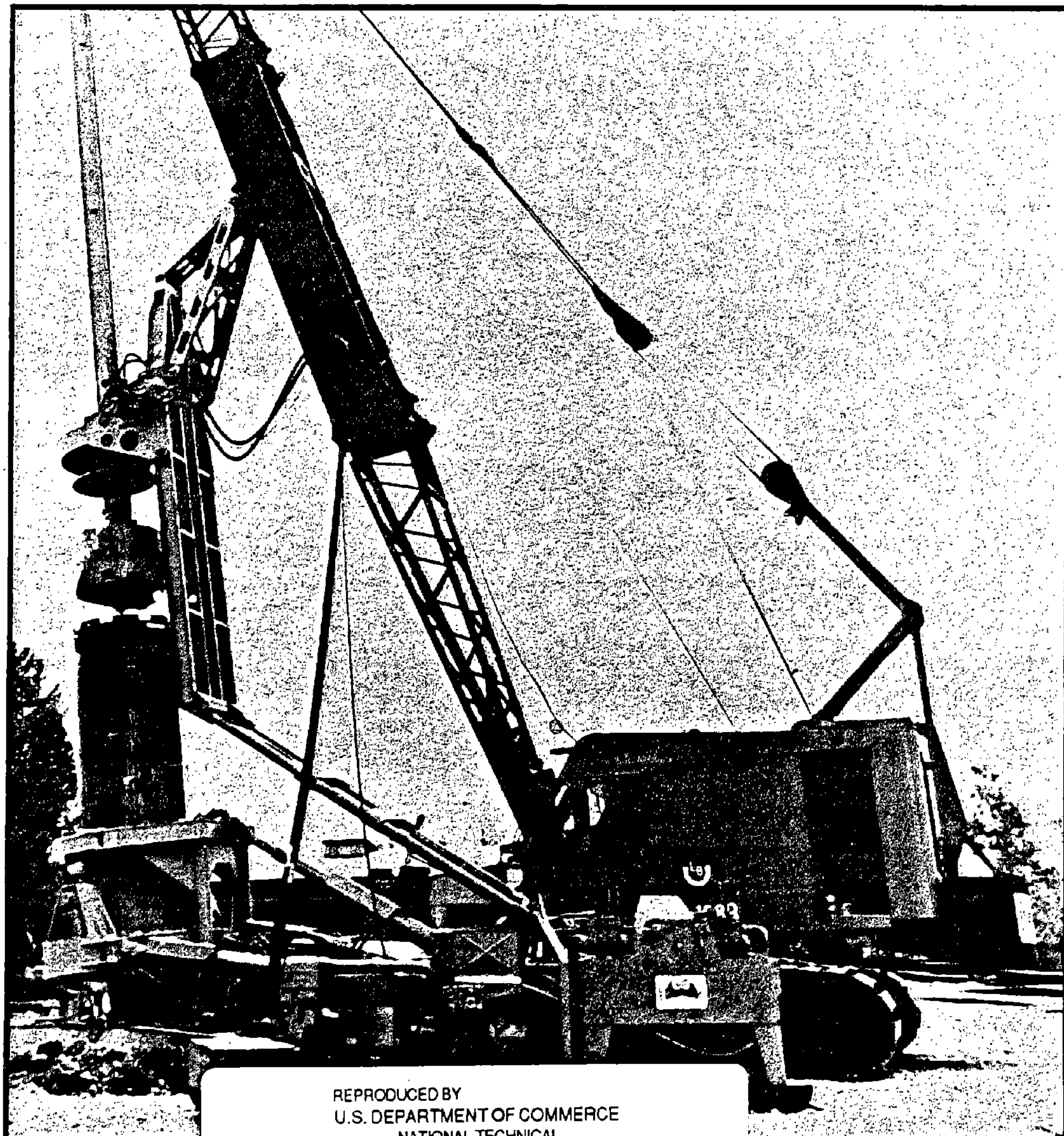

FOREWORD

This technology transfer report is an English translation of a French publication, Les Pieux Fores, in which the construction, inspection, and testing of bored piles (drilled shafts, drilled piers, caissons) as practiced by government agencies in France are discussed in detail. This report will be of interest to geotechnical, structural and construction engineers involved in design and construction of drilled shaft foundations.

Sufficient copies of the report are being distributed to provide a minimum of one copy to each FHWA Region office and Division office, and two copies to each State highway agency. Limited copies of the report will be available to State highway agencies from the FHWA Office of Implementation (HRT-10).

R. J. Betsold
R. J. Betsold
Director, Office of Implementation

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16. Abstract

The construction, inspection, and testing of bored piles (drilled shafts, drilled piers, caissons) as practiced by government agencies in France are discussed in detail. Some of the topics that are treated in the document are contracts, methods of making the excavation, casings and liners, rebar cages, tests for completed piles, and methods of repair of defective piles. A number of technical notes are included at the end of the work that deal with various types of tools, machines, and construction techniques.

Of special interest in the publication is the attention to detail with regard to various construction techniques. The engineers, owners, and contractors in the United States who are involved in the design and construction of drilled shafts will find the book to be of great interest. Construction details turn out to be critical with regard to the success of this type of deep foundation.

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PRINCIPAL WRITERS:

J.-P. BRU	Laboratoire régional des Ponts et Chaussées de Bordeaux
M. CARISSAN	Laboratoire régional des Ponts et Chaussées de Toulouse
G. LACOSTE	SETRA - Division des ouvrages d'art

CONTRIBUTORS TO THE DOCUMENT:

P. BRABANT	Laboratoire régional des Ponts et Chaussées de l'Est parisien
M. BUSTAMANTE	Laboratoire central des Ponts et Chaussées
J.-A. CALGARO	SETRA - Division des ouvrages d'art
O. COMBARIEU	Laboratoire régional des Ponts et Chaussées de Rouen
D. CORNET	CETE de Nantes - Division des ouvrages d'art
P. DELUDE	Laboratoire régional des Ponts et Chaussées de Bordeaux
C. HAHUSSEAU	Direction départementale de l'Equipment de la Gironde
G. HAIUN	SETRA - Division des ouvrages d'art
A. MILLAN	SETRA - Division des ouvrages d'art
A. ROBERT	Laboratoire régional des Ponts et Chaussées de Bordeaux

NOTE: Taking into account the rapid evolution of the methods described herein on a national scale, as well as internationally, some references to materials, procedures, or societies may turn out to be inaccurate at the time of publication of this document. The authors respectfully request that the reader accept their apologies.

BORED PILES

English Translation

of

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LIST OF ABBREVIATIONS

CCAG	:Notebook of General Administrative Clauses
CCTG	:Notebook of General Technical Clauses
CCTP	:Notebook of Special Technical Clauses
CMP	:Code for Public Contracts
CPC	:Notebook of Common Specifications including Article 68, Clause 36
CPS	:Notebook of Special Specifications
CPST or CPS-Type	:Standard Notebook of Special Specifications
DCE	:Report of Consultation with Companies
DIG 70	:Guide to Aid Contractors and Construction Supervisors
DJ 75	:Guidelines for Evaluation of Bids
DTU	:Standardized Technical Guidelines
FT	:Technical Notes
GGOA	:General Guide for Structures
GMO 70	:Guide for Construction Supervisor (from GGOA)
RPAO	:Special Regulation for Bidding

PREFACE

With the invention of mechanical excavating equipment at the turn of the century, techniques of boring piles using temporary casings or supports were developed. The use of bentonitic slurry to ensure stability of a large-diameter borehole is more recent and did not appear until 1948.

The rapid development of this technique for these "formed" or cast-in-place piles is linked to the development of boring tools and machines that, through their power and size, routinely allow construction of shafts under any soil conditions of 1.00 m to 1.50 m in diameter and, in exceptional cases, of 2.50 m to 3.00 m in diameter.

This considerable progress has brought about a decrease in costs for deep foundations which are often preferable to shallow or mat foundations due to increased reliability.

However, numerous observations made on job sites by the Regional Laboratories of Bridges and Roads have shown that construction operations for bored piles do not always imply such reliability. These operations may be the source of difficulties and improper construction that can be very serious due to difficulty of detection and insufficient post-construction testing.

From this experience and methodical analysis of the main construction problems, studies have been made by the Laboratories of Bridges and Roads to propose proper recommendations for increasing the quality of piles and to propose specific methods and tests. The results of this work have allowed a group of engineers composed of specialists from SETRA* and the Laboratories of Bridges and Roads to write this document. In addition to general recommendations for use by the Department of the Ministry of Transportation and the Ministry of the Environment and Standard of Living, this document offers various public contracting agencies and private and professional research groups the principles of construction and testing adapted to site conditions and structural characteristics for each operation that are part of the construction of a bored pile: boring, casing or lining, reinforcing, and concreting.

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*See page xiii for explanation of abbreviations.

For construction supervisors, this document presents the regulatory texts in current use and gives useful information about drafting of contracts, judgement of bids, and technical proposals by the construction companies. This document defines the organization and composition of tests to be performed at various construction stages and is a helpful guide for inspectors.

For contractors, this document is a complementary aid for training of personnel with regard to the Ministry's requirements as well as the quality of work that is expected.

This "review of the state-of-the-art" cannot be considered a final result of studies on the subject due to the fact that it opens the way to essential technical research and technology in several areas.

This latter aspect attests to the concern of the authors to present, as carefully and completely as possible, the knowledge and actual methods involved in construction and testing of bored piles, and also adds to the quality and relevance of the following text which will undoubtedly have a large audience both within and outside of our Administration.

Michel Fève
Chief Engineer of Bridges and Roads
Director of Highways and Traffic

PREFACE TO ENGLISH TRANSLATION

The Laboratoire Central des Ponts et Chaussees (LCPC) and the Service d'Etudes Techniques des Routes et Autoroutes (SETRA) have performed a useful service for owners, contractors, and engineers by the preparation of this comprehensive publication on the construction and inspection of bored piles, and appreciation is expressed to that agency for permission to make this translation. The material herein is especially valuable because there is a scarcity of similar information in the technical literature and because most of the problems that have been encountered with bored piles in the past have been construction related rather than design related.

The support of the the Office of Implementation of the Federal Highway Administration of the English translation will make available many important concepts, techniques, and details to a wide audience in the United States and abroad. The information should be particularly useful to those who are engaged in the design and construction of transportation-related facilities.

The reader will find many references to agencies in France and to French publications. The List of Abbreviations should be useful and it is believed that the reader will have a reasonably good understanding of the intent of the material from the context in which it is presented. Nevertheless, some of the procedures are uniquely related to French practice and could be only marginally related to the way construction is accomplished in the U. S. However, the writer has found the entire publication to be of interest and expects that most readers will respond in much the same way.

Some comment is in order about the procedures employed in making the translation. The first translation of the document was made by Ms. Linda Iverson. Mr. Mohammed Kayyal and Mrs. Mercedes Bernal assisted in the early stages. The writer amended the translation to employ terms and phrases that are consistent with usage in the U. S. Following completion of the translation of the manuscript, it was sent to Mr. John Walkinshaw, a bilingual employee of the Federal Highway Administration, who contributed his time to read the publication and to make appropriate comments. The resulting draft was submitted to French-educated engineers for review and suggestions. The writer and Ms. Iverson then prepared a semifinal draft that was reviewed by Mr. Steven Glaser who is experienced in construction of bored piles. The final copy was prepared by Ms. Susan Brady and Ms. Linda Iverson; it was given a final reading by the writer to find minor corrections and omissions. Mrs. Lola Williams supervised the preparation of the copy. The final, camera-ready copy was submitted to FHWA and simultaneously for review by LCPC. Appreciation is expressed to all those who made a contribution. While many people worked on the translation, the writer accepts full responsibility for the quality of the document.

The following portion of this Preface presents a brief discussion of each chapter with respect to the construction practice in the U. S. The comments that follow are meant in no way to diminish the importance of the methods and procedures that are given by the French writers. Rather, the comments are meant to be helpful to the reader who is generally unfamiliar with the practice of construction and inspection in the U. S.

Chapter 1. The Different Types of Cast-in-Place Piles. Most of the types of cast-in-place piles and the methods of construction are in some use in the U. S. Barrettes have been used to date only to a minor extent, perhaps because the rotary method of advancing an excavation is used almost exclusively rather than percussion drilling with rock breakers and hammergrabs.

The distinction that is made in the types of casing is unfamiliar to most practioners in the U. S. "Lost-pipe" casing, or casing that cannot be recovered, is strongly avoided in the U. S. as certainly it must be in France.

While barrettes and percussion drilling, for example, have had little use in the U. S. to date, the translation should be valuable in that those methods and others that are discussed may suggest more efficient construction methods in certain situations.

Chapter 2. General Aspects of Bored-Pile Contracts. Some sections in this chapter, a number of the references, and several quotations are related particularly to French practice. The organization of a project in France is distinctive; therefore, some terms used in the translation may not be in common use in the U. S. For example, the term "construction supervisor" is used throughout the document. In the United States the construction supervisor could be the engineer-in-charge, a chief inspector, or a special representative of the owner. Even though some of the sections in this chapter have no direct relevance to U. S. practice, the material should be useful to the reader.

The reader may wish to give some attention to the following points that are discussed in Chapter 2:

organization of the project,
staking of locations for foundations,
storage of materials and equipment on site,
evaluation of capabilities of equipment to be used,
environment at site with respect to safety of personnel and efficiency of construction,
construction control,
disposal of spoil, and
procedures that can be employed to deal with special problems that arise.

As may be seen, the document addresses a number of situations that are common to any job where bored piles are to be installed.

Chapter 3. Boring. There is a wealth of information in this chapter and nearly all of it is relevant to practice in the U. S. Some of the tools and techniques that are presented are seldom used but the information, nevertheless, could be quite useful in a general sense. Of particular importance is the suggestions for dealing with difficult subsurface conditions, such as karstic soils, and the techniques for obtaining an excellent concrete-rock contact at the base of a pile.

The recommendation against the use of battered piles is consistent with the views of most of the designers and contractors in the U. S. The emphasis on the careful cleaning of drilling mud prior to the placement of concrete is well placed; however, the detailed suggestions that are made for treating the slurry with additives in difficult situations fall outside of the expertise of most engineers and contractors. A specialist would need to be called in for those cases.

The emphasis on the measurement of the geometry of a borehole is of interest; however, modern analytical techniques can show that the axial capacity is affected only slightly, if at all, by accidental batter and that the additional bending stresses may be very small.

The document puts emphasis on many details that can influence the quality of a completed pile, and properly so; the information on construction details is of considerable importance. However, the impression may be left that there are only a limited number of solutions to difficult problems with the consequence that the danger of a defective foundation is more severe than actually is the case. Of the thousands of bored piles that are installed annually in the U. S., only an extremely small number show any signs of excessive settlement.

A statement is made that sites where there are large particles, such as boulders, should be avoided with bored piles and that some other type of foundation should be designed. In many instances, no other type of foundation is feasible and many techniques have been developed for installing bored piles at such sites.

Chapter 4. Casing and Liners. A considerable amount of attention is given in this chapter to the use of flexible or semi-rigid liners. Such liners are rarely used in the U. S.; however, the suggested uses as described herein may lead to some desirable applications.

There are some instances in this chapter where costs of materials are given in francs. No attempt was made to convert such costs into dollars because the result could be misleading.

Chapter 5. Rebar Cages. There are many practices in France with respect to rebar cages that are similar in the U. S. The stockpiling of the cages, attention to details concerning the lifting and righting the cages, the providing of centering devices, and the protection of the reinforcement during the placement of concrete are all common factors concerning practice in the two countries. However, as shown is the following paragraphs there are some significant differences.

French practice apparently requires that some reinforcing steel be put in any pile. While some agencies follow that practice, the trend is to use rebar only when needed to withstand bending stresses or to provide strength for columns. Research has shown that the soil surrounding a pile, even if weak, will provide support such that the pile will not behave as a long column. The theory on the behavior of piles under lateral loading is effective in providing values of bending moment due to accidental batter and accidental eccentricity, as well as bending moments due to lateral loading, and the rebar can be tailored to fit the actual requirements.

Most of the reinforcing steel that is used in the U. S. is not weldable, because it has been manufactured from reclaimed steel such as railroad rails and bodies of old automobiles. French practice, on the other hand, is to employ weldable steel. While the difference in cost between nonweldable and weldable steel may be significant, the ability to weld certainly gives the contractor an advantage. The cages are structurally much stronger and consequently are easier to handle and to place.

The manual gives a fixed dimension for the width of the annular space between the cage and the inside of the borehole; the practice in the U. S. is to make that dimension a function of the size of the largest of the coarse aggregate.

The manual indicates that French practice is to employ a "basket" at the bottom of the rebar cage or at least to bend in the longitudinal bars at the bottom of the cage to keep the cage from rising due to the force from the fluid concrete. While this practice could have merit in some instances, U. S. practice is to restrain the cage at its top.

Chapter 6. Concreting. The attention given in this chapter to the design of a concrete mix, to the manufacture and transportation of the concrete, to its placement, and to the various phases of the control of the entire operation is well placed. Construction engineers in the U. S. are well aware that the operations involving the concrete for drilled shafts are critical with regard to the quality of the foundation.

The statement in the chapter that the slump cone is ineffective in obtaining a measure of the fluidity of concrete is of interest. The recommendation for the use of the fluidometer for the determination of the optimum water content for the concrete of a pile and other such suggestions are worthy of study by engineers and contractors.

That tests have been made at construction sites of the movement of the concrete as it is placed in a borehole, indicates that the research of the LCPC has been extensive and detailed. Therefore, the credibility of the information presented in the manual is enhanced.

The warnings in the chapter about the numerous mistakes in design and construction that can lead to defective piles suggest two comments. Firstly, the material in this chapter and in other chapters is aimed at those who have had experience in the design and construction of bored piles. Thus, each of the suggested procedures can be evaluated against those that have been in common use in the U. S. Secondly, as noted earlier, some readers may conclude that many bored piles that are constructed are defective but the extremely small percentage of drilled shafts that have performed poorly belie such a conclusion.

Chapter 7. Tests for Completed Piles. The chapter gives much information on the testing of piles that have been built with a particular emphasis on piles for which defects are suspected. Methods identical or similar to those that are described have been in use in the U. S. but the tests are not routine as implied in the chapter. However, the routine use of access tubes for geophysical testing seems to be an excellent idea.

The point is made in the chapter that none of the methods that are described, or a combination of the methods, can give conclusive information on defects or the lack of them. This point is important and should be considered when a test method is being considered.

Chapter 8. Poor Construction and Repairs for Bored Piles. Useful information is presented in the chapter. Of particular interest are the presentations concerning the repair of piles where defects were found. The methods that were employed could perhaps find some general acceptance.

If a defect is found in a pile, many owners would want to have proof that the defect was repaired to the point where the pile has gained its design strength. The only positive way to give such proof is to perform a full-scale loading test and such tests are impractical in most instances.

Technical Notes. Much of the information contained in this section of the document pertains to equipment and procedures that are uniquely related to practice in France. Therefore, the material may have little or no direct relevance to work in the U. S.

However, the material should be valuable from the standpoint of general interest. Many of the methods may suggest some innovations that could be quite valuable.

Lymon C. Reese
University of Texas at Austin
January 1986

CHAPTER 1

THE DIFFERENT TYPES OF CAST-IN-PLACE PILES

Piles have always been used as a foundation alternative. As construction techniques have evolved, numerous types of piles have appeared on construction sites. Even the concept of these piles has considerably evolved (what does a wooden pile, 20 cm (8 in.) in diameter, have in common with a reinforced concrete shaft that could in exceptional cases reach 2 to 3 m (6.5 to 10 ft) in diameter? Chapter 3 and the Technical Notes may be consulted for the kinds of equipment used.)

The gradual tendency over the last ten years has been to abandon prefabricated piles that are driven in favor of cast-in-place piles. However, the problems associated with all aspects of cast-in-place piles are as numerous, if not more numerous, as those found in prefabricated piles that are driven.

The present document deals only with cast-in-place piles and shafts constructed by excavating the ground (bored piles). The classification of a particular type of cast-in-place pile is necessary and important and will be discussed in the following paragraphs.

Piles can be constructed by methods that cause no displacement or densification of the surrounding soil or by methods that do cause displacement. The method of installation is important because the resulting state of stress in the soil will influence side friction, base resistance, and resistance to lateral loading. Drilled shafts are built without densification of the soil.

Without considering the types of equipment that are used (see Technical Notes), the following construction procedures are taken into account in classifying cast-in-place piles (drilled shafts):

-- piles constructed with or without casing, taking into account the methods for placing this casing

-- piles constructed in dry soils, below water, or with bentonitic slurry.

1.1. TERMINOLOGY

(See also [3], § 8.3)

a) *Barrettes*

Barrettes are load-carrying elements cast in the ground that are 0.60 to 1 m by 2 to 6 m in plan dimensions (2 to 3.3 ft by 6.6 to 19.7 ft) used as bearing elements. They are used in secant or parallel configurations depending on the geometry of the structure they are designed to support (Fig. 1).

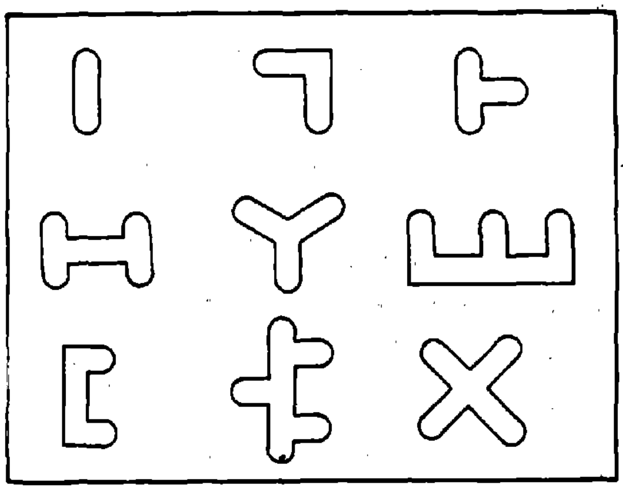

Fig. 1. -- Some types of barrettes.

b) *Mud (Slurry)*

The drilling mud is a colloidial mixture, not a solution, and its basic component is bentonite. Mud is frequently used in the construction of bored piles (Technical Note No. 9 and § 3.4.5) to stabilize excavation walls.

c) *Reinforcing cage*

A group of reinforcing bars are used to make up the longitudinal and transversal reinforcement of a pile. The cage is transported and placed in the borehole before concreting operations.

d) *Bailer with valve*

A cylindrical tool with a valve in its base can be used to permit the withdrawal of excavated soil after the use of the chopping bit.

e) *Driving (Sinking)*

This operation consists of installing a rigid element (pile, casing, sheeting, tube, etc.) by application of pressure, percussion, or jetting. Densification of granular soil occurs during the driving operation.

f) *Borehole*

This operation consists of opening a cylindrical "hole" in the ground by extraction of the excavated material (by common use, borehole has become the name of the excavation itself.)

g) *Kelly drive*

A telescopic or non-telescopic metallic bar of polygonal cross section which transmits to the boring tool the required forces for penetration and/or rotation.

h) *Jetting*

A method for installing a pile or casing which consists of injecting water or mud under pressure at the base of the driven element. It is also used as a boring method in soils of low consistency (see § 3.2.1.1.c).

i) *Piles or shafts*

Article 68 of the CPC, in its commentaries on article 34, makes the following distinction:

-- piles have a diameter less or equal to 80 cm (31.5 in.)

-- shafts have a diameter larger than 80 cm (31.5 in.).

This distinction might sound artificial, which is the reason why the two terms are used interchangeably in the rest of the document when the distinction between them becomes useless.

j) *Trimming*

This operation consists of the leveling of the head of the piles by chipping them to the level of the connecting point of the footings or pile cap, (see § 6.4.3).

k) *Overpouring of concrete*

There is an overpouring of concrete when the volume used is in excess of that calculated, based on the theoretical diameter of the hole (see § 6.4.2).

Overpouring in the order of 10 to 20% is not considered excessive (for example, the diameter of the boring tool is often slightly larger than that indicated in the construction plans). However, any overpouring larger than 20% must be considered abnormal, and the causes of the overpour must be ascertained and remedies found before any other piles are poured.

l) *Auger*

A helicoidal drilling tool is used to advance an excavation employing rotation and a downward force. The auger acts as a screw conveyer to bring soil to the surface.

m) *Auger Bucket*

An auger bucket or drilling bucket is a boring tool that is rotated as it excavates. The tool has two cutting vanes or blades at its base that excavate the soil. The excavated materials are accumulated in the bucket and the bucket is raised to be emptied at the ground surface.

n) *Drop-hammer or chopping hammer*

A heavy drilling tool is lifted and dropped in free fall and is used to penetrate hard layers by disintegrating the soil or rock.

o) *Hammergrab*

The hammergrab is a heavy drilling tool which works in percussion, is guided as it falls (Benoto Method), and has at its base a grab for withdrawing the excavated material.

p) *Rotary drill*

The rotary drill is a drilling tool equipped with chopping bits with teeth or vanes that loosen the soil or rock as the tool is rotated. Generally, the rotary drill is associated with a process known as boring by reverse circulation. The tool is similar to the drilling tools used in drilling oil wells (see Technical Note No. 8).

q) *Tremie tube*

It consists of a concreting pipe (tremie pipe) which is connected either to a funnel-shaped bucket or to a pump.

r) *Temporary casing, thin-walled casing, medium-walled casing, lost-pipe casing**

The pile or drilled shaft can be constructed with the aid of a retrievable pipe which is called a temporary casing. The casing is normally reusable, and is made of steel with a minimum thickness of 1 to 2 centimeters (0.4 to 0.8 inches).

Occasionally, it is necessary to place a protective shell or pipe between the concrete and the soil. The pipe or shell is placed after the

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* Lost-pipe casing is similar to the temporary casing except that it is unretrievable due to unfavorable subsurface conditions. (see § 4.3.6).

excavation has been made (see Chapter 4). If thin steel (a few millimeters thick - a fraction of an inch thick) or other materials (plastic sheets, synthetic materials, etc.) are used, it is called a thin-walled casing. If a rigid tube (7 to 15 mm thick - 0.25 to 0.6 in. thick) is used, it is called a medium-walled casing.

s) *Surface casing*

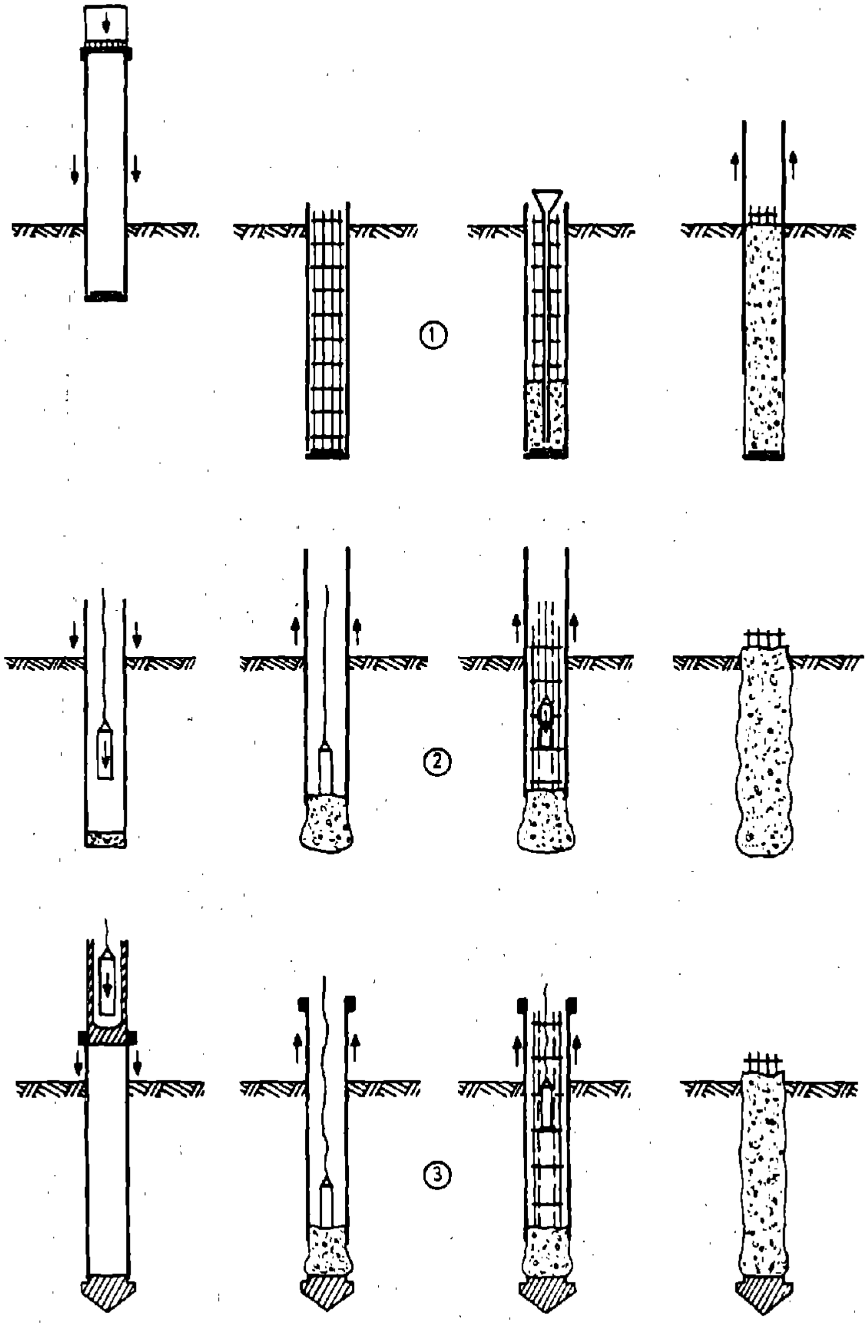
A metallic or concrete-pipe element, often with a cutting edge, is sometimes placed at the top of the excavation to prevent caving and to provide guidance for the drilling tools for the first few meters (several feet). In barrettes, small guide walls are used, similar to those for slurry-trench construction.

1.2. CAST-IN-PLACE PILES WITH DENSIFICATION OF THE SOIL

These piles are of a diameter smaller than 70 cm and are commonly 50 cm (28 in. and commonly 20 inches). They are constructed by pouring concrete in the interior of a closed-ended, metal pipe that is usually driven and that is often withdrawn after concreting.

There are numerous methods (Franki, Express, Paumelle, Vibro, Alpha, Trindel, et al.) which differ from each other in the way the base of the driven pipe is sealed (dry concrete plug, metallic or concrete shoe, special reusable point, lost-steel plate). The methods also differ in the way the concrete is placed and in the consistency of the concrete (dry concrete, plastic concrete poured by means of a tremie, etc.). Several methods that do not permit the installing of reinforcing cages of a significant length cannot be used in some structures. However, these methods can frequently be used in building construction where the loads are such that short reinforcing cages at the top of the piles are adequate and sufficient.

The details of different methods are described elsewhere ([1],[7],[9],[10], [12],[71]). However, Fig. 2 presents the principles of three of these techniques, and it can be noted that methods 2 and 3 allow for the enlargement of the base of the pile due to the compaction of the first portion of concrete by ramming. These two methods require that an additional temporary casing be used inside the reinforcing cage in order to protect it.



Procedures for driving by:

- impacting at the top of the casing 1 and 3
- impacting at the base of a quantity of dry concrete 2

Procedures for concreting by:

- the tremie method 1
- compaction of a dry concrete dropped from the top of the casing 2 and 3

Fig. 2. -- Some construction procedures for piles with retrievable casings.

1.3. BORED PILES (DRILLED SHAFTS, DRILLED PIERS)

1.3.1. Principle

These piles are constructed by excavating the soil by any method, placing a reinforcing cage and finally concreting the excavation (Fig. 3). Bored piles differ from the piles previously described (see 1.2) in that the soil is not necessarily densified.

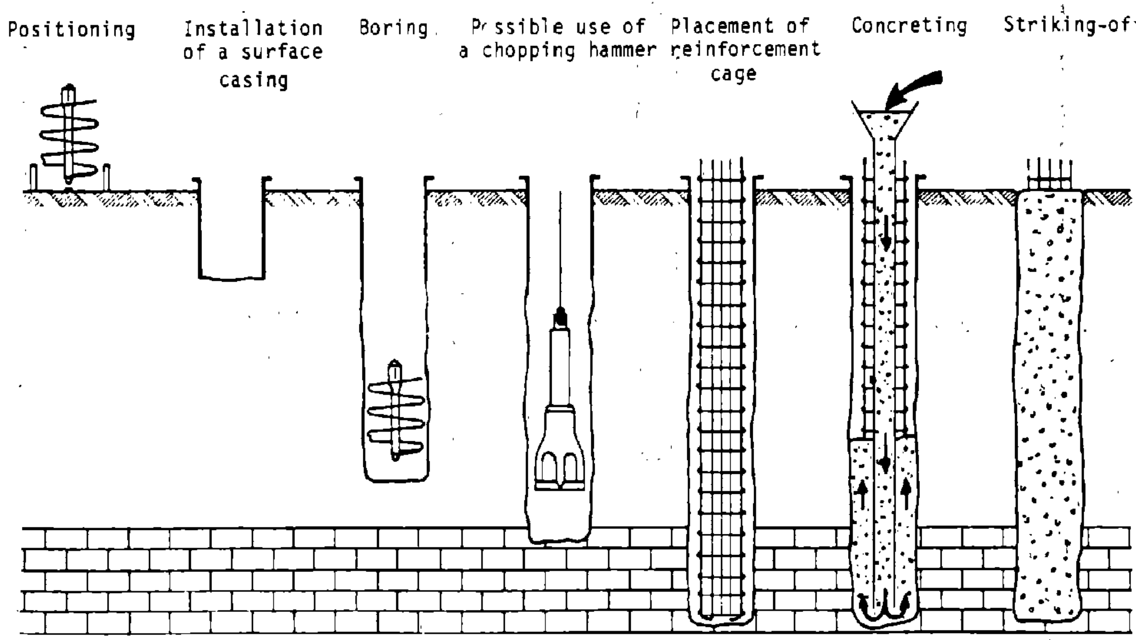


Fig. 3. -- Method of construction of a bored pile.

1.3.2. The different types of piles and their use

Usually a distinction is made between two general methods characterized by *using or not using a temporary casing* (See Chapter 3). However, the diversity of the problems encountered, due to the different geotechnical characteristics of sites, often leads to the simultaneous use of both methods. This justifies the identification of a third method, which is as important as the previous two (§ 1.3.2.3).

1.3.2.1. Drilling with the use of a retrievable or temporary casing.

Within this group of piles, all of which are of a cylindrical shape, are numerous methods presented in detail in Chapter 3.

The use of a retrievable casing is dictated by the presence of unstable strata of soil (recent fills, granular noncohesive soils, talus deposits), by important underground water flow, by mud flow, by karstic or gypsum zones, etc. Under these conditions, the excavated walls of a drilled shaft cannot be stabilized except with the use of a retrievable casing.

In certain cases, soils with high friction and some cohesive soils (muds) could cause difficulties in placing and withdrawing a casing if the pulling power of the equipment is insufficient. Therefore, the use of a casing in such soils is usually limited to piles of average length (maximum of 15 to 20 meters - 50 to 65 feet).

Casing is also used in more stable and hard soil when the drilling tools, unguided in this case, will not ensure a uniform section and a given plumbness of the excavation (as in the drilling equipment shown in Technical Note No. 1). Tubular casing, open at its base, is composed of metallic elements of variable length, screwed, welded or bolted one to the other when advanced into the soil. The casing must be thick enough (minimum of 1 cm - 0.4 in.) in order to resist, without deforming, the stresses to which it is subjected during installation and during the excavation. Deformations as well as damage to the lower end of the casing should be prevented. In certain cases, the bottom of the casing is equipped with a cutting edge which is selected according to the soil to be penetrated (as in the Benoto Method, see Technical Note No. 4).

The casing is either advanced simultaneously with a drilling tool, slightly ahead or behind according to ground conditions, or directly lowered to the bottom of the borehole (or slightly higher if the pile will be socketed into a hard strata).

a) *Casing advanced as excavation progresses*

The casing is driven ahead of the boring tool when soil characteristics are appropriate (uncompacted soils); this method is generally used to drill in loose soils which are susceptible to sloughing.

The drilling tools are advanced ahead of the casing in the case of compact, usually cohesive soils. In the case of hard soils or soils containing boulders, the use of a drop-hammer or chopping hammer is frequently required. The casing then can be lowered by sinking, aided at times by a slight hammering with the drilling tools or by driving of the casing by percussion (Benoto method).

b) *Casing driven directly to the bottom of the borehole*

This method allows the separation of the phases of placing the casing and extraction of the excavated material, a process which saves time and avoids the expensive immobilization of the system for driving the casing.

This technique is used in loose soils, and the extraction of the excavated material is done in one continuous operation. However, when compacted soil or boulders are encountered, the driving of the casing must be stopped and the whole operation must be performed in several steps.

The casing is generally driven with a vibratory hammer (see Technical Note No. 6), a method which is particularly suited for granular soils below the water table. A diesel or steam hammer could also be used in driving a casing (see Technical Note No. 5) in all kinds of loose soils. There is a wide variety in the equipment used to perform this job. Other methods are sometimes employed (jacking, jetting) but their use is limited to particular cases.

1.3.2.2. Boring without casing.

Within this second group, it is convenient to distinguish bored piles using mud or slurry stabilization from those that do not require the use of any particular method to stabilize the borehole walls, namely those constructed in the absence of water (dry soils) or sometimes under clear water.

a) *Bored piles under slurry, barrettes*

Piles in which the excavated boreholes are stabilized by bentonitic slurry during drilling are called bored piles under slurry. The slurry is later displaced by concrete that is placed by use of the tremie method.

The use of this technique implies two conditions:

-- the drilling mud stabilizes the sides of the excavated borehole (filter-cake formation, adequate hydrostatic pressure...)

-- there is no significant mud loss, *particularly not sudden loss*.

The first condition is usually satisfied, except when involving newly constructed embankments that are poorly stabilized, or very soft soils that tend to creep and may cause narrowing of the bore of the pile, or when unstable, screed-covered slopes are involved. Difficulties also arise when construction is in close proximity to other structures constructed on shallow foundations or on friction piles which prohibits the loosening of the soil, especially when the soil is cohesionless. Of course, this latter restriction is even more serious if penetration of resistant layers requires the use of a drill bit.

The second condition deals primarily with calcareous or gypsum terrains, in which the presence of karst or soluble pockets may cause heavy losses of drilling mud and the sloughing of upper layers during drilling operations.

Other than these particular cases, the permeability of soils commonly encountered is not an obstacle for the use of this procedure, with the exception of soils of high permeability with an average over 10^{-2} to 10^{-1} m/s (3.28 ft/s to 0.33 ft/s). Excessive consumption of drilling mud will occur in cases where the permeability is high.

The procedure is well suited in terrains that contain an aquifer, free or under pressure. Good execution involves the use of a sufficient fluid level; a minimum of 1 m (3 ft) should be maintained between the mud level in the borehole and that of the free surface (hydraulic head) of the aquifer. When there is underground water flow, and the stability of the walls of the borehole might be uncertain, it is wise to provide a thin-walled casing (see § 4.2.1.2) which would help prevent the leaching of the fresh concrete.

Almost all types of tools can be adapted for use when drilling with mud (diamond or tungsten chopping bits, augers, hammergrabs, buckets, etc.), but generally these tools must be guided at the top of the borehole in order to assure that the excavation is drilled to comply with the tolerances on position.

The drilling may be executed:

-- *with mud circulation*

direct, when the drilling mud is forced by a very strong pump down through the drill pipe, and up through the annular space between the pipe and the walls of the borehole (see § 3.2.1.1.c)

inverse, when the excavated materials are extracted with the drilling mud by means of a suction pump through the drill pipe (case of rotary drills).

-- *without or with very little mud circulation*

the mud is simply discharged at the top of the borehole then pumped to the recycling station.

Whichever method is adopted, it is necessary to provide a facility for preparing and regenerating the mud (see Technical Note No. 9). The slurry should be recycled regularly during the drilling operation.

b) *Bored piles in dry soils*

This technique is principally used in cohesive soils above the water table, where the drilling can proceed in the dry with the use of simple equipment (augers, hammergrabs). There is a wide range of diameters and dimensions for these piles, circular section, square, rectangular or irregular shape, but it is rare that their depth exceeds 12 meters (40 feet).

1.3.2.3. Combined method.

The combined method may be an alternative when the upper layers of soil that are to be bored are unstable and/or require the use of a temporary casing. This method is also possible when construction is to be performed at an aquatic site from a barge or from a newly constructed embankment. The boring may continue, without a casing, using bentonitic mud, if the underlying layers so permit, and can attain depths that are impossible with use of a temporary casing alone (considerable friction on the casing; see § 1.3.2.1).

Below the unstable soil layers the boring may be continued, without casing, using bentonitic slurry. This mixed method allows attainment of depths that would not be practical to be cased (excessive friction, extraction problems; see § 1.3.2.1).

1.4. ADVANTAGES AND DISADVANTAGES

This kind of comparison is usually subject to uncertainty because a number of parameters are totally or partially overlooked in the analysis (price, region, type of work, nature of job, etc.). The advantages and disadvantages presented in the next paragraphs are for information only.

1.4.1. Cast-in-place piles with densification of the soil

ADVANTAGES

-- fast execution

-- good contact between the soil and the tip of the pile

-- concrete is placed in a dry state

-- development of the lateral friction by densification and due to the lateral stresses of the soil layers

-- neatness of the site

-- estimate of bearing capacity using pile-driving formulas.

DISADVANTAGES

-- risk of false refusal and insufficient socketing, difficult to penetrate hard layers

-- cannot be adapted to cavernous sites

-- risk of destroying neighboring piles, in which the concrete is still fresh, due to vibration and impacting the concrete

-- limited maximum diameter of 0.70 m (2.3 ft)

-- possible deviations during driving

-- heavy and cumbersome drilling and driving equipment

-- noisy (nuisance in urban areas).

1.4.2. Bored piles

ADVANTAGES

-- large diameter, up to 2.5 m (8 ft) and maybe more, and possibility of constructing elements with different shapes to resist bending

-- possibility of penetrating hard layers

-- qualitative control over the penetrated layers

-- adapts to different lengths.

DISADVANTAGES

-- construction requires specialized personnel and equipment adapted to the drilling and concreting operations

-- control of plumbness and diameter of the borehole is difficult except for piles constructed in dry soils

-- risk of disturbed soil around the pile due to the excavation

-- risk of poor contact at the base due to a poor cleaning of the bottom of the borehole

-- frequently difficult to keep site clean.

CHAPTER 2

GENERAL ASPECTS OF BORED-PILE CONTRACTS

Specific Construction Problems

Along with some background information, this chapter presents the principal administrative problems encountered in contracts for bored piles. Also, the different phases of construction are indicated that are worthy of inspection or testing.

The technical problems connected with bored-pile contracts are analyzed in detail in the chapters which follow. This chapter covers only one of the many roles of the construction supervisor.

In order to facilitate the use of the present document, the text of this chapter has been expanded in the light of superceding that part of Chapter 8 of the report guide GMO 70[1] *which concerns bored piles*.

Paragraphs 8.4 to 8.7 and paragraph 8.9 in GMO 70 have been reviewed and revised, but this presentation as a whole has been modified and follows the chronological order of the different phases of construction.

2.1. THE CALL FOR BIDS

In the following section, the portion of the provisions on the construction of deep foundations, applicable to cast-in-place piles, is summarized and brief comments are presented.

2.1.1. The regulatory documents

For the drafting of construction contracts, the basic regulatory texts are:

--the Code of Public Contracts (which is denoted by the initials CMP, the last edition dated September 1, 1976);

-- the Specifications for General Administrative Clauses (CCAG) of January 21, 1976, which replaces the former Article 1 of the Specification of Local Regulations (CPC);

-- Article 68 of the CPS [4];

-- Circular No. 75-147 of September 25, 1975, pertaining to methods of making a call for bids and the evaluation of the bids concerning con-

[1]GMO: Guide for Construction Supervisor, Level II of the GGOA

struction in the national road-system network, in which it is stated that the provision of foundations at a fixed price is forbidden.

In order to facilitate the work of the construction supervisor, some recommendations have been cited, of which the most important are:

-- the Guide to Aid Contractors and Construction Supervisors, November 1, 1976, which replaced the Directive of the Prime Minister of October 2, 1970, known under the initials DIG 70.

-- the CPS Standard (December, 1969 edition) updated in April, 1974, Article 3.09: "Cast-in-place piles") [5].

Finally the Report Guide FOND 72 [1] contains a certain amount of information that is useful in the drafting of contracts.

2.1.2. Choice of the procedure for a call for bids

The first task for the construction supervisor is to choose a procedure for a call for bids considering the relative portion of the Construction of Piles with the assistance of the Guide.

When it pertains to bored piles, the work related to the material in this section is generally subcontracted. If the work on bored piles has a secondary importance in relation to that of the overall project, the normal procedure should be that of a contract with the general construction company (§e5 of the Guide). It is, then, very desirable to include in the RPAO the stipulation that each general contractor indicate in the contract the specialized company that will be charged with construction of the piles.

If the amount of work on foundations is important in relation to the overall project, it can be advantageous to employ the procedure involving a contract with a joint contractors group (§e6 of the Guide).

In any case, the construction supervisor may resort to the procedure of a contract with a general contractor with specialized sections, described in §e8 of the Guide. To employ this procedure, the document on the call for bids should distinguish the main section (to be constructed by the prime contractor) from the auxiliary sections (to be constructed by specialized contractors who will be named in the contract as co-contractors or as accepted sub-contractors).

Theoretically, this allows the construction supervisor to replace a co-contractor or a sub-contractor to the prime construction company under the provisions of the contract, if there is an obvious technical or financial advantage. However, the replacement of a subcontractor is rarely done in practice because the general construction companies often consider themselves definitely linked with certain sub-contractors.

2.1.3. Special regulations concerning the call for bids (RPAO)

Two cases arise involving foundations. In the first case, the design documents require a basic "solution"; the RPAO can then foresee the possibility of "alternates", and should indicate in a precise way the range of admissible alternates. This model may be the most desirable for foundations of large scope; these jobs often pose some specific problems that may be resolved in different ways depending on the techniques which govern the best interests of the owner and the contractors. This does not pertain only to those variables associated with construction but pertains as well to variables associated with design.

In the second case involving routine jobs, freedom of initiative should be left to the contractors who are competing for the project. In this case, the RPAO should define the nature of the technical *proposals* that are made. For example, these proposals can have a bearing on the choice of the type of pile (cast-in-place or precast, etc.), on the choice of method of construction in the case of cast-in-place piles, etc. The technical proposals include specifics that the competing contractors are *compelled* to furnish in their offers so as to complete the contractual definition of the work within the technical plan.

As a rule, foundations for routine jobs are open to technical proposals. In the case of foundations composed of driven piles (metal or reinforced concrete) of relatively short length, and in consideration of the experience of the supervisor, the call for bids may possibly be undertaken without alternates.

Finally, we point out that a special regulation concerning the call for bids of the RPAO model is forthcoming from SETRA.

2.1.4. Document concerning special technical clauses (CCTP)

To prepare the Document Concerning Special Technical Clauses (CCTP), the construction supervisor refers to Article 3.09 of the CPS model, a reference previously cited. It is at present incomplete concerning a number of points pertaining to cast-in-place piles. Nevertheless, it is a good point to start from and, while waiting for the next publication, some recommendations can be found in the directives of the Report-Guide FOND 72.

2.1.5. Contents of the geotechnical report

The contents of the geotechnical report are quite variable according to the nature of the work to be performed. Nevertheless, to aid in the evaluation of the bids, this report must be quite complete, especially in the case of foundations of piles. A good report is based, in the first place, on a good preliminary survey and on tests of the geotechnical properties of the soils. It is not worthwhile to produce results of 50 soil identification tests if no measure of shearing strength is used. In the second place, a good report must deal with the problem of the interpretation of the tests that were made and with the selection of the method for testing the foundation. It is possible, when practical, to make reference in the CCTP to the basic concepts in the Report-Guide FOND 72.

2.1.6. Evaluation of bids

To evaluate the bids, the supervisor may refer to two documents:

-- the Report-Guide FOND 72

-- the Directive for the Evaluation of Bids, edited by SETRA under the initials DJ 75.

2.2. EXPECTED OPERATIONS BEFORE WORK BEGINS

2.2.1. Organization of the construction supervision

Generally, and independent of the level of supervision, the work is performed in *three different stages*.

The first stage occurs, properly speaking, before the work begins. The materials to be used must be checked for conformity with the CCTP: casings, linings, steel for reinforced concrete, etc. Studies may possibly be made to determine the design of the concrete mix and, if drilled piles in slurry are involved, to determine the characteristics of this slurry (obviously made with water available on the site).

The second stage involves the inspection of all phases of the construction. This inspection is continued throughout construction (drilling, concreting, finishing, etc.). A test of the quality of the slurry used in drilling should certainly be planned.

Finally, the third stage relates to the testing of the quality of the completed piles.

Generally, the first and the third phases of the inspection or testing are performed using a specialized laboratory, and the second phase of inspection is done by staff working directly under the construction supervisor.

The arrangements of the details of the inspection or testing must be planned in advance, on the one hand, so as not to slow down the job at the time the work begins and, on the other hand, in order to avoid long and expensive delays. For example, if provisions for access tubes to be placed within the piles to allow a geophysical investigation are not handled before concreting, the access tubes must be omitted because they cannot be placed in hardened concrete.

The final quality of the piles results from good construction and thus there must be good inspection. For example, it is useless to place a high-strength-concrete mix if the tremie is not being used properly or if delays occur between vibration. The piles will present some anomalies in both cases.

Good inspection naturally assumes that inspection personnel are sufficiently qualified or at least sensitive to the problems of drilled piles. This is why we have included in § 2.4 a "check-list" of the principal points to be observed in the inspection process. If the list is adhered to, it will then create a significant incentive for the construction companies to perform well. It is desirable for the representatives of the construction supervisor to operate in strict collaboration with the specialized laboratory team for which the responsibilities should be clearly defined.

2.2.2. Staking

Sections 6.3 to 6.5 of the GMO 70 define general staking, special staking, and supplementary staking.

Each pile is often marked on the ground with a single "stake" that marks the position of its future axis. Naturally, the marking of the piles is a contractual obligation of the construction company but just staking the axis is insufficient for piles of large diameter. The risk of an error in positioning the top of the pile is significant. The construction supervisor can then ask that a more reliable and efficient marking be employed; for example, with three or four stakes arranged around the future perimeter of the hole. It is useless to try to obtain a positioning of a pile within 5 cm (2 in.) using only one axial mark that will disappear at the time of the lowering of the drilling tools.

2.2.3. Pile tests

When the importance of the job and the results of the geotechnical study so justify, one or several pile tests may be conducted.

The most common test is the static test employing vertical loading. The conditions for selection and performance of this test are explained in section 3.5.5 of the Pilot-Guide FOND 72. The means of performing the test are described in the document of the LCPC: Method of Performing a Static Test of Deep Foundations (May, 1970) and is alluded to in Article 68 of the CPC in Clauses 29 and 39.

It should be noted that a vertical-load test *is not a quality-control test for a completed pile*: it is a test that, so far as is possible, should be of use in the designing of the foundations. Thus, it is preferable to perform this test, when it is stipulated by the CCTP, before the completion of the construction plans. However, the cost of the test is such that its performance cannot be recommended during the study stage except under independent funding.

Another test is sometimes performed: a static test employing lateral loading. The test is is designed to measure the reactive capacity of the soil when the pile is subjected to lateral loads. The lateral-load test also should be performed before completion of the construction plans. However, recent progress made in the use of curves and graphs in the prediction of soil reaction to lateral loading often allows elimination of the field test.

2.2.4. Plan for installation of piles

The construction of drilled piles is often subject to some hazards; jobs without problems are uncommon. Although the term "pile-driving plan" is not generally used in the case of cast-in-place piles, a document of this type should be required of the contractor in a clause included in the CCTP. The plan for the installation of piles should be developed in detail so that as many hazards as possible are dealt with.

To cite an example, consider a foundation on piles in a soil for which the preliminary survey has not indicated special problems. However, at the time of the construction of a pile, the drilling tool encounters a hard, local obstacle before reaching the depth of the foundation. The obstacle must be broken up using impact tools. Also, the presence of recently constructed piles close to this hole can lead to postponement of the operation of drilling so as not to damage the nearby piles where the concrete has not yet set. In order not to disturb job progress, it is preferable to plan a construction sequence of the piles such that the pile under construction is relatively distant from recently constructed piles.

2.3. INSTALLATION OF CONSTRUCTION MATERIALS ON THE JOB SITE

2.3.1. Right of inspection of the construction supervisor

In the case of prefabricated piles (Article 68, Clause 26 of the CPC) as well as in the case of cast-in-place piles (Article 68, Clause 37 of the CPC), the contractor makes the selection of the equipment and the necessary materials to build the structure unless specifically stated in the CCTP. However, and this is important, the details of the selection must be submitted to the construction supervisor for approval.

In practice, the construction supervisor should concern himself with this matter at various stages of the project:

-- at the time of the discussion of the "technical proposals", at the time of evaluation of the bids and at the drafting of the contract. This stage takes on a special importance in the case of cast-in-place piles because, once the design selection is made, the possibilities of selection of equipment become very limited. However, technical conditions imposed by the site prevail over equipment arrangements by the contractor;

-- at the time of the drafting of the Summary Order of Duties that references the requirements of Article 2.4 of the CCAG of 1976 and the CPC, Article 68, Clauses 26 and 37 (installation and equipment).

2.3.2. Installations on the job site (Figs. 4 through 7)

The importance of the job determines the precautions and arrangements to be taken at the time of the installation of the equipment.

Clause 28.2 of the CCAG of 1976 requires that the construction program, to which the plan of the installation on the job site and of temporary structures are linked, be submitted for the endorsement of the construction supervisor. The documents must be submitted at least ten days before expiration of the period of preparation, or, if such a period is not stipulated in the CCAG, one month at the latest after the notice of the contract.

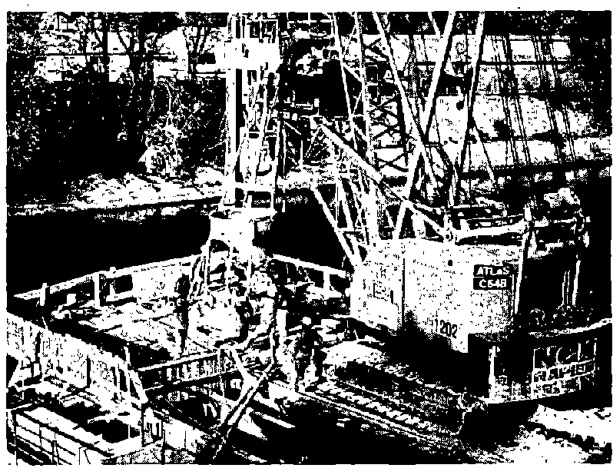


Fig. 4. -- Arrangement on a cofferdam.

Fig. 5. -- Equipment at job site.

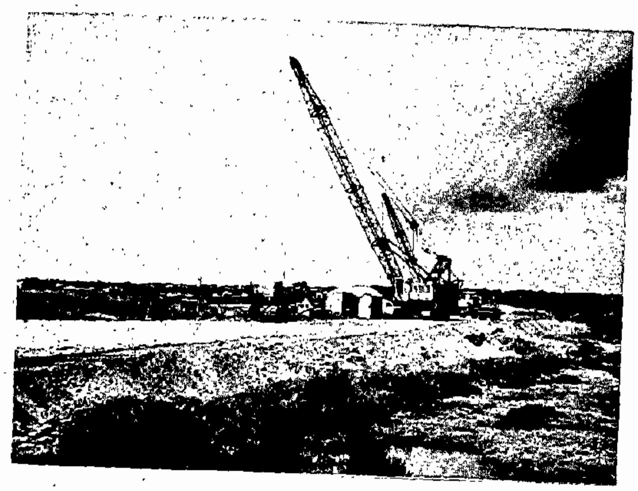


Fig. 6. -- Job site access road.

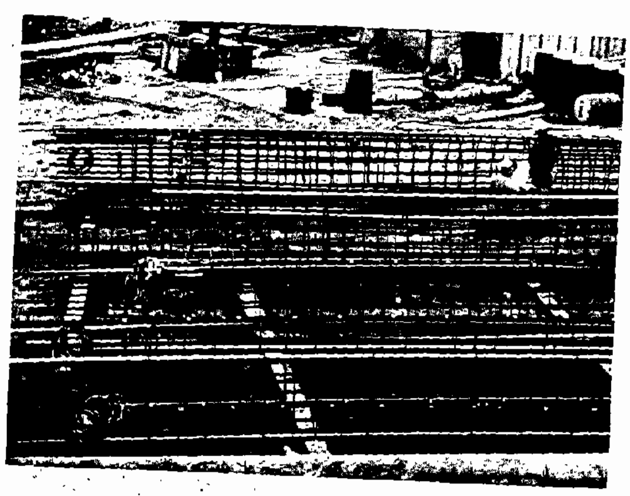


Fig. 7. -- Method of stockpiling reinforcing steel.

In practice, experience has shown that good construction procedures are linked to the preparation of an efficient work atmosphere, planned in such a way as to permit:

-- movement of machines and men over soil of satisfactory condition, possibly stabilized;

-- practical disposal of the drilling spoil;

-- proper handling and storing of prefabricated rebar cages (room for the expected crane, lifting cables and arrangement, and storage area.)

Each of these conditions can result in unfortunate incidents when precautions are not taken. For example, concrete trucks have been observed to be bogged down one hundred meters or so from the job site due to poor access roads, which made the concreting operations rather acrobatic. Finished piles have literally disappeared under the debris emanating from other piles during the course of construction; when the earthwork machines arrive to evacuate the spoil, the pile heads have been damaged. Finally, prefabricated rebar cages are often poorly stockpiled, deformed, and may have broken welds. What can their efficiency be for reinforcing the concrete?

When piles are constructed with bentonitic slurry, the work atmosphere should be specially studied if the job site is to remain accessible. It should be foreseen, for example, that some portion of the recovered slurry will inevitably be lost around the pile or at the pump station. The spilling of slurry is particularly important when site constraints are severe and when the work is in an urban environment.

It is important to note that the finished head of the pile rarely coincides with the level of the natural terrain; if the ground surface has been modified by cutting or filling or by activities at the job site, the completed piles may have excessive height that must be trimmed or, on the contrary, an insufficient height that requires the lengthening of the reinforcing steel and the placing of additional concrete.

2.3.3. Technical capability of the equipment

Before any work is begun, the construction supervisor must take care to be sufficiently informed about the characteristics and the constraints of the functioning of the construction equipment.

This especially involves (see Technical Notes):

a) *the limits (in qualitative terms) of the capability of the equipment* in connection with the intentions of the project:

-- if the foundations should be deeper than anticipated, to what point will the equipment be able to go, or will it be possible reasonably to adapt the equipment to the new situation?

-- if the piles are to be constructed with a batter, what batter angle is possible? Can the batter angle vary in a continuous fashion or does it correspond to no more than two or three intermediate values?

-- if there are hard soils to be penetrated, including the founding layer at the base of the piles, will it be possible to do the hard drilling in an acceptable and productive manner? To narrow down the question and provide a reasonable answer, the construction supervisor should consult the geotechnical report and look at the results of the in-place tests that were performed.

b) *The eventual production to be expected from the drilling equipment* in the different layers of soil to be drilled; this does not generally concern the construction supervisor. However, when special difficulties are foreseen that require changes during the course of work, and that affect the specifications, it is necessary that the construction supervisor be aware of the facts concerning the problem he will have to resolve.

c) *the overall dimensions of various pieces of equipment* whether terrestrial or marine. Consideration is given to access, presence of electrical lines, cables, water lines, etc. The performance of some current equipment is dealt with in Chapter 3 under drilling, and there is some information on equipment in the Technical Notes with respect to their dimensions.

d) *the constraints of the use of the equipment*: in particular, will it be possible to construct the piles in any order or according to the installation plan, without additional cost?

e) *the possible construction precision* required to comply with the tolerances specified in the contract (CPST, Clause 3.09.41).

f) *the capacity and number of machines* for removal of the excavated soil materials and the capacity and number of cranes.

g) *the adequacy of the equipment for treatment* of the bentonitic slurry and the methods for testing the slurry (see Chapter 3).

h) *the methods of placement of the concrete* and the handling of the rebar cages.

i) *nuisances (noise and vibrations)* which can prohibit the use of certain machines on urban sites or in the proximity of certain other structures of facilities that may be damaged (railways for example); on this subject see § 7.4, second line of the GMO 70.

2.3.4. Work and the environment

As with all civil engineering works, the construction of drilled-pile foundations can pose a certain number of problems with respect to the environment.

These problems can be anticipated, as should be the case with all difficulties brought on by the job, or can arise unexpectedly during construction.

The main nuisances are the following:

a) *Interruption or detours of traffic on roads and streets*

b) *Interruption of or re-scheduling of railway traffic.*

This latter problem is especially sensitive and at times the costs to be paid to the SNCF are such that they can influence the choice of the type of piles. The criteria of rapid construction or of guarantees of safety are often overriding. Thus, as a result of their importance, these criteria may be set forth in the RPAO.

c): *Noise, congestion, pollution*

These nuisances have an importance especially for work in urban areas. For example, pollution of the access roads by trucks that travel through mud must be avoided.

d) *Risks for projects or buildings constructed nearby*

The construction of deep foundations is always critical near projects or buildings previously constructed. Naturally, from the outset of the project planning, the nature and geometry of the foundations of these nearby facilities must be understood as well as possible. Such an understanding is not always easy when old structures are involved, especially in an urban area. The new construction should always be carried out with prudence. To cite an example, the assumption can be made that the construction of piles will require that a casing be driven into the soil to a great depth using a vibratory pile hammer. If the upper layers of the soil are loose sand, for example, it will densify due to the vibration and the ground surface could possibly experience settlement over a wide area. Therefore, if the construction is in the vicinity of the foundations of an old building, some distress in that building could result.

In the same vein, during construction previously unidentified, buried objects may be encountered (possibly even ammunition). Then it will be necessary, after proper identification of the obstacle, to choose the equipment judiciously which allows the obstacle to be by-passed or to choose the technique that causes the least amount of disruption.

For sensitive cases, the GMO advises that the obstacle be staked and that the nature of the problem be made known to a building official.

2.4. PROBLEMS DURING CONSTRUCTION

2.4.1. State of the equipment

Once the type of equipment is agreed upon there should be time to make sure that the equipment stored at the job site is in satisfactory condition, that the equipment can perform as expected, and that the desired results can be readily achieved. Contact with the previous user of the equipment can be highly instructive.

If the initial performance of the equipment proves disappointing, it is worthwhile for the construction supervisor to investigate the causes. The contractor should be informed at once of the deficiencies and suggestions made in order to avoid possible complaints after the fact.

2.4.2. Qualification of personnel

Good construction of drilled piles depends, in a significant way, on professional attitudes, and on qualifications and experience of the drilling and concreting foreman. To this end, Clause 36 of the CCAG of 1976 deals with the possibility for the construction supervisor to demand that the contractor make a change of employees for insubordination, inability, or lack of integrity.

2.4.3. Pre-construction requirements

2.4.3.1. Receiving materials

For all matters regarding the receiving of materials, it is desirable to refer to Chapter 5 of the GMO which is perfectly explicit on the subject.

In the case of bored piles, it is important to note that liners, temporary casing, and permanent casing specified in the contract must be inspected prior to being needed on the job. The inspection is essentially visual and involves the checking of the length, thickness, and diameter of these items.

2.4.3.2. *Special case of concrete*

There are currently ***regional formulas for concrete for piles*** that are quite satisfactory and are well know, notably by the Regional Laboratories of Bridges and Roads. The use of these regional formulas permits a concrete mix to be selected at the outset of the job without the performance of a design study.

However, a check of the mix is almost always necessary. This check can only be carried out on the concrete at the time of construction of one of the piles of the foundation. Thus, it is necessary to choose a pile which is easy to replace if the results are not satisfactory. This is the opportunity to make a "dress rehersal" as the GMO recommends; that is, to test the adequacy of the equipment and the placement procedure so that, after the test pile is constructed, it is checked according to the methods presented in Chapter 7.

When the construction of the piles involves both a land site and a marine site, a test pile can be considered for each type of site; the testing can be simultaneous or separate, depending on the scheduling of the job.

In a general way, the study and the testing of concrete are the subject of Clause 3.17 of the CPS Standard and of Article 65 of the CPC.

When the concrete is manufactured at a plant, reference should be made to § 9.42 of GMO 70.

2.4.3.3. *Special case of rebar cages*

The rebar cages are generally prefabricated and stockpiled. With regard to reinforcing steel, Chapter 5 of this document presents the general characteristics of rebar cages and gives details on their fabrication.

It is necessary to make a visual inspection of the rebar cages to check for centering devices and for protection of the cages against damage during lifting and placing.

2.4.4. Controls during construction (Figs. 8 through 17)

The principal things to inspect, in chronological order of operation, are the following:

a) ***Location***, centering, and possible inclination of drilling tools or casing.

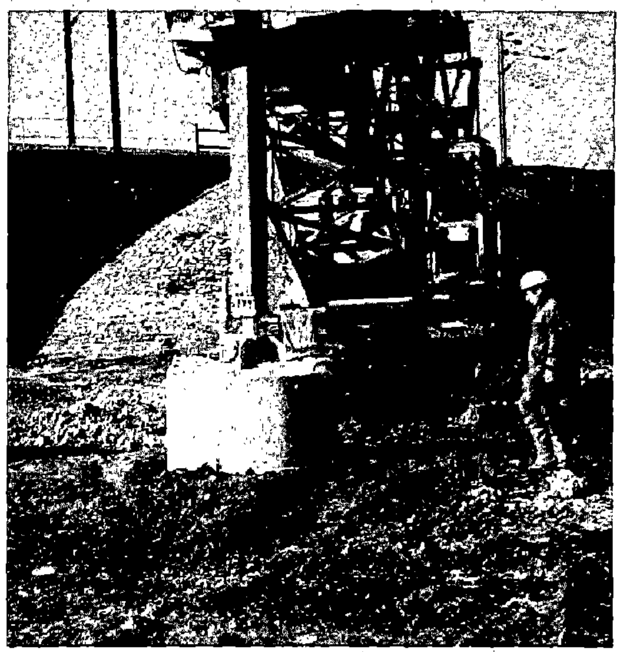


Fig. 8. -- Positioning of tool.

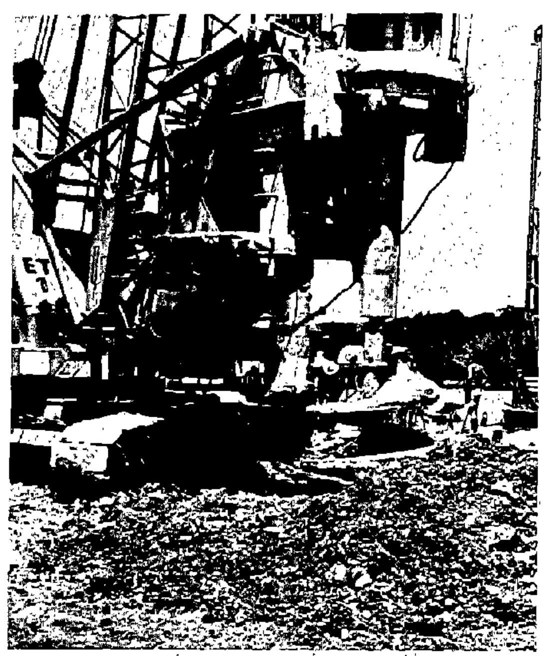


Fig. 9. -- Drilling of first meters.

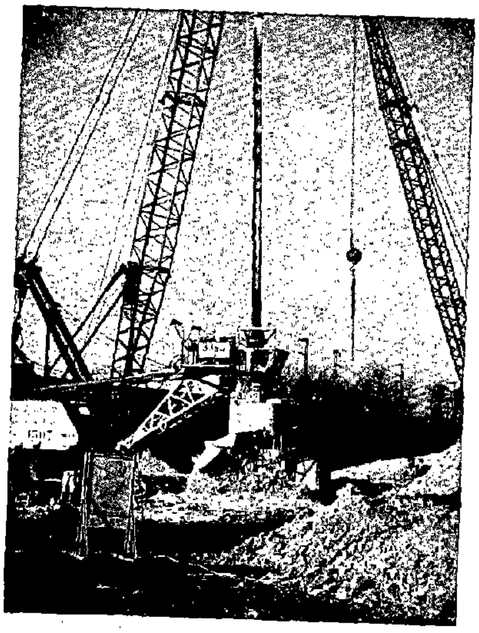


Fig. 10. -- Drilling.

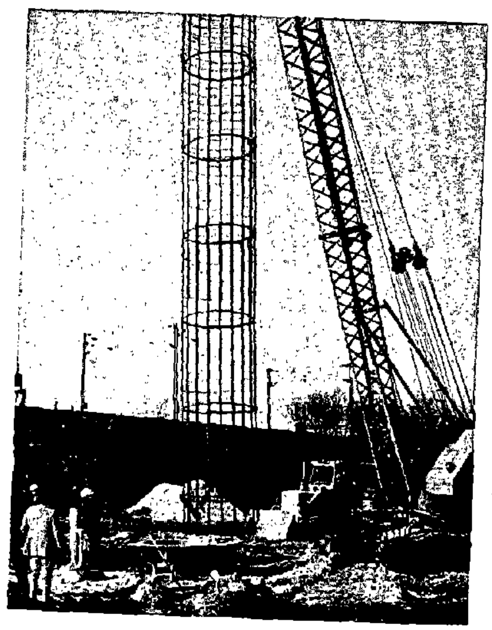


Fig. 11. -- Placement of reinforcing steel.

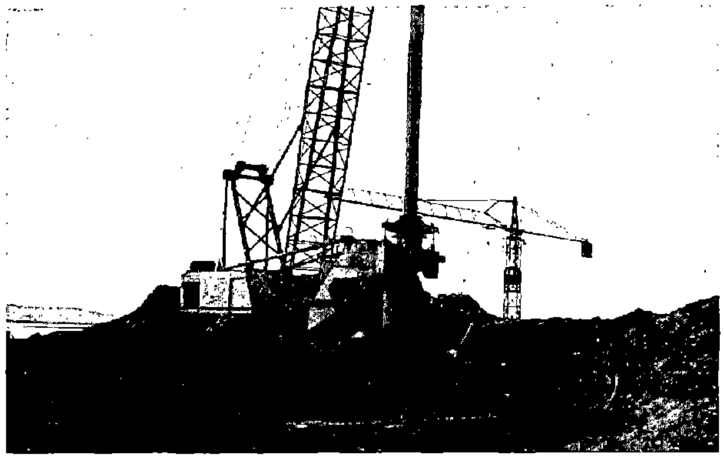


Fig. 12 and 13. -- Evacuation of debris at proper time. Required!

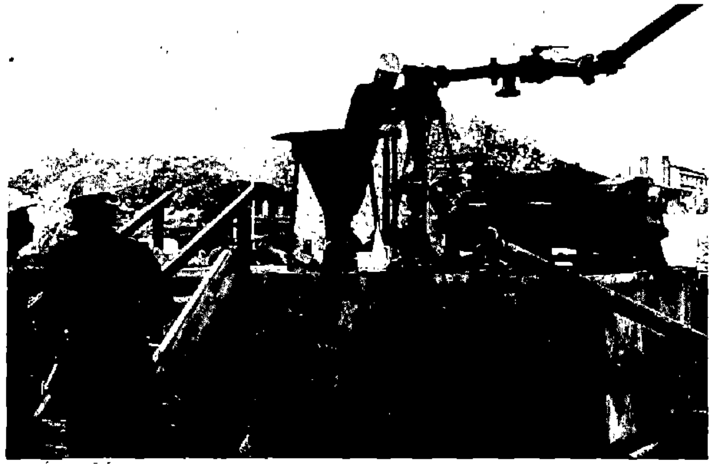


Fig. 14. -- Recycling of slurry.

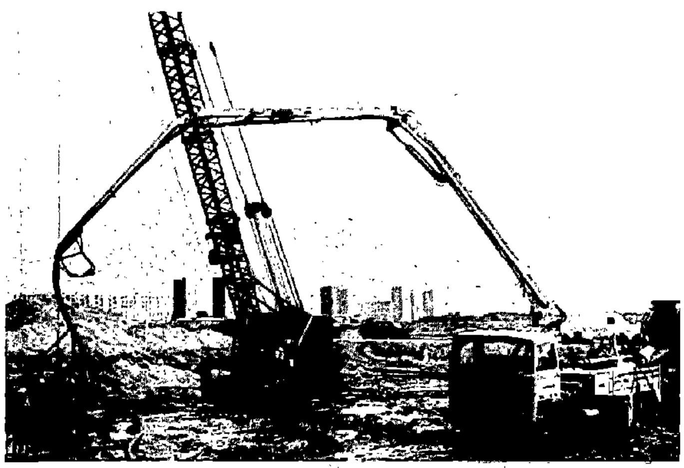


Fig. 15. -- Concreting with pump.

Fig. 16. -- Concreting with tremie.

Fig. 17. -- Trimming.

b) *Protection of the top of the borehole* by surface casing when the pile is not constructed with a temporary casing. The surface casing furthermore permits guidance through the first meter.

c) *Efficient removal of drilling spoil*, which contributes to the general quality of the job.

d) *Accuracy in locating top of pile, its inclination and consideration of tolerances*. The problem of the positioning of the borehole is encountered at various stages of the job, and especially at the time of the approval of the equipment and the certification of the methods to be employed by the contractor.

For piles drilled without temporary casings it is particularly desirable to take note of the straightness of the borehole. The CPS (Article 68, Clause 37.1) requires, to this effect, a verification of the positioning of the boreholes at various levels. However, the CPS Standard (Clause 3.09.41) is not very explicit on this subject.

e) *Control of the construction of the borehole*

The reader is directed to Chapter 3 of this document where the problems of drilling are treated in detail. It is of interest to note that inspection or evaluation applies essentially to the quality of the construction plan or schedule described in Appendix A of this document and established according to the model annexed to Chapter 8 of the GMO [3]. The value of this type of schedule is the following:

First of all, the schedule brings together the actual field conditions encountered by the contractor through the sampling program required of him (CPS Standard 3.09.45) by the geotechnical report. This sampling facilitates the establishment of the geological profile as required by the CPST 3.08,46. The soil and rock samples are to be conveniently organized, classified, and displayed in appropriate boxes or trays.

All of the incidents that may occur during the course of drilling are recorded. Appropriate solutions to the technical problems must be worked out in the case of a discrepancy between the subsurface investigation and the data from the drilling. Such discrepancies may include the determination of the depth of the bottom of the borehole, the need or lack of the use of a casing or a liner, the strata where there may be caving soils, a significant inflow of water or sand boils, the rate of advancing the borehole, the environmental agressiveness of the soil formations, etc. Various problems concerning organization or inspection of the work can arise. A particular problem in inspection is that the measurement of amounts cannot be verified once the foundations are completed, especially in the case of the use of a rock breaker (see § 20.62 of the GMO, pricing, 79 to 84).

The model schedule gives far more information than is included in a simple drilling log. Also, more important than a pile driving log, the model schedule is suggested to be used by the construction supervisor as the basis for discussions with the contractor, ironing out problems between the inspectors and the contractor, and documenting the need for special tools such as rock breakers.

With regard to the use of a rock breaker for the excavation of the borehole, the cost of which can represent a significant part of the total cost for the foundations, the construction supervisor always retains the authority to start or stop the work (CPS Standard 3.09,47). Such authority is indispensable in order to have the ability at the contract level to apply regulation No. 84 on cost overruns - *drilling by the meter and not by the hour* (see § 20.62 of the GMO).

Thus, the company must use tools that perform efficiently in order to avoid being penalized for the excessive use of time.

On the other hand, drilling of soil or rock formations for which the rates were not anticipated in the contract is often a reason for an extra claim.

However, when such a bid item is in the contract but without quantities, *it does not influence the total contract amount*. Thus, *it is important at the time of evaluation of the bids for the construction supervisor to note the proposed cost for this item.* An exhorbitant drilling rate for this item may involve cost overruns that are prohibitive in relation to the initial total of the contract in case the unanticipated drilling is encountered. In some cases it could be argued that the contract is invalid because of unbalanced bid items.

Furthermore, there are currently no specific guidelines for judging the operation of various tools which could be based on rates of advancement and an immediate decision should generally be made for the use of the tool.

The construction supervisor and his representatives must be made aware before agreeing to the use of a tool that it is often effective yet expensive; and is therefore of questionable usefulness.

A suggestion is also made that assignment be given to the sub-contractor to authorize use of a rock-breaking tool:

-- without special reservations if the soils encountered present greater difficulties than anticipated, such as encountering an obstacle or refusal when using standard tools;

-- on the acknowledgement in writing of the cautions given by the construction supervisor concerning the adverse effects of using the percussion tool (the acknowledgement is particularly necessary if the use of the tool is due to a malfunction in the drilling equipment in normal use);

-- and in all cases to take note of the reasons for the use of the rock-breaking tools.

In certain very special cases, another solution may be to include the cost of the use of the rock-breaking tools in the average rate per meter of drilling.

The construction supervisor should then request the details of the price for each meter (foot) of drilling at the time of the submittal of the bids. The use of an average price per meter (foot) of drilling is especially suitable when the results of the geotechnical study predict frequent use of percussion drilling throughout the entire depth of the borehole but fail to indicate the exact position and nature of the layers to be penetrated by the rock-breaking tools.

Within the general guidelines concerning the use of rock-breaking tools, it should be noted that in the case of a rock surface that is very irregular, sloping, or steeply bedded the work of the percussion tools is tedious, especially if the pile is battered. It is difficult to obtain a borehole with a uniform cross-section and the borehole has a tendency to change its slope. It is quite necessary that the top of the casing be shaped properly and kept in position and that the slope of the casing be carefully controlled.

At the time of the drafting of the plan for the installation of the piles, consideration is given to the minimum amount of delay that is necessary in order to avoid damage to the piles previously concreted; if the piles are next to each other, a 24-hour interval should be sufficient to permit the hardening of the concrete.

f) *Control of the drilling slurry*

The testing of the drilling slurry is especially important at the time of constructing the first piles. The slurry should be tested during the course of the drilling and at the completion of the boring. The necessary properties of the slurry are defined in § 3.4.5 and 3.5.9 and the tests that are required are described in Technical Note No. 9. Such tests must be stipulated in the CCTP and should be performed by the construction company. If the case arises, the tests may be performed by an accepted laboratory, and the role of the job-site inspector consists of verifying that the tests are appropriately performed at various levels in the slurry column and that the slurry is regularly recycled, particularly before concreting.

g) *Test of the bottom of the borehole*

This test is essential; it should take place just before the concreting and should consist of verifying that the bottom of the borehole has been properly cleaned out (see Chapter 3). This determines the quality of the contact between the pile and the soil at the base.

h) *Control of the handling of the rebar cages*

This involves a visual inspection that should be carried out by the job-site inspector. Without precautions, these cages are too often abused and deformed before they are lowered into the borehole.

i) *Control of concreting operations*

Chapter 6 of this document is devoted to concreting. In this chapter, it is simply emphasized that the excavation should be concreted *as rapidly as possible*. The concrete should be placed within a few hours after the cleaning-out of the excavation. If the walls of the borehole are not stabilized by a casing, too much of a delay in the concrete placement can possibly allow a collapse of the walls of the borehole. The concreting operations should be monitored from the standpoint of timing, workability of the concrete, position of the tremie during the concrete placement, and recording of the overpour.

j) *Control of the trimming at the top of the pile*

Trimming is prescribed by the CPC (Article 68, Clause 40); that is, the top of the shaft is trimmed to the lower level of the pile cap (CPST, Clause 3.09,54). The necessity of trimming the top of a shaft in the case of concrete cast in a dry hole may be debatable; in any case, the height to be trimmed must be controlled and limited.

The trimming is always paid by the unit but the height to be trimmed can vary in each case depending on the difference of dimensions between the working level (or point to where the concrete is poured) and the bottom of the foundation cap. The length of the pile to be constructed and the length of the trimming can be more significant than anticipated if there has been, for example, an error in the elevation of the working platform.

It is normal:

-- on one hand, to pay for the pile by the meter (foot) between the level of its base and the theoretical mark of the lower face of the foundation cap.

-- on the other hand, to pay for finishing of the top of the pile by the unit, regardless of the amount of concrete to be cut away resulting from errors in measurement that are attributable to the construction company.

2.4.5. Decision to stop drilling: various situations on the job

2.4.5.1. Decision to stop drilling

The decision to stop drilling is up to the construction supervisor (CPST, Clause 3.09,48). The construction supervisor must be prepared to act quickly if the occasion arises because of the obvious need of a speedy decision. *It is generally more worthwhile to make quickly* a somewhat questionable decision than to make a better decision eight hours too late.

However, the above statement will depend principally on a good understanding on the part of the construction supervisor of the various technical factors associated with the boring:

-- the nature and quality of the subsurface investigation that revealed a soil profile and details about the principal strata that were encountered;

-- examination of the soil and rock samples that were obtained from the excavation by the contractor;

-- output of various tools used by the contractor in performing the drilling;

-- tests at the bottom of the borehole.

Concerning the examination of samples, it is necessary to be especially careful in identifying the soil. In regard to soil identification, a factor to be considered is that the terminology may vary according to profession (driller and geologist, for example). Another factor is that the soil recovered from the drilling is disturbed and the drilling operation may have mixed soils from different layers. Therefore, it may not be possibe to tell if soil from a weak layer or a strong layer is recovered from the borehole. It is desirable in all cases that a geotechnical expert be present on the job site to recognize the various soils at the time of construction of the *first pile*.

Concerning the information on the efficiency of drilling, it is necessary to interpret with caution this type of information. The data are strongly influenced by the operator of the drilling machine, by the details of the tools that are employed, and by the nature of the soil that is excavated. In any case, the data on drilling rate that may be obtained are only qualitative.

In regard to another point, it is necessary to remain appropriately cautious with regard to in situ tests (pressuremeters, penetrometers, etc.) that can be performed at the bottom of the borehole. In fact, the measuring devices do not generally give accurate results at shallow depths (the presence of a free soil surface can affect the results); therefore, in situ testing devices cannot indicate the mechanical characteristics of soil or rock near the bottom of the borehole. Assuming that such in situ tests are useful, they only give some information about the quality of the soil at the tip and not about the overall behavior of the pile. Finally, in situ tests are generally very difficult to implement and also interfere with construction activities. For all these reasons, performance of tests at the bottom of the borehole are rare in practice; the techniques are not perfected and the information collected is often not very useful.

In the normal case when the drilling encounters a strong and resistant layer, a decision must be made about the amount of penetration into the stratum. The penetration depth, or socket depth, is a part of the plan (CPST, Clause 3.09,44) and is generally required (see Pilot-Guide FOND 72). However, in construction, it is desirable to take care that the depth of drilling into a strong layer of rock is not excessive; replacement of the compact rock with concrete would not be advantageous and should be avoided ***except in the case of significant horizontal stresses***.

Confirmation of the depth of penetration into the founding stratum is useful at the time of the boring because changes can be made, for example, when the top of the strong formation is not found near where anticipated but is a little higher or a little lower.

2.4.5.2. Changes in the construction plans

Conception, justification, and establishment of the construction drawings for a foundation are based on the information furnished by the preliminary survey of the soil and of the site. Even when this preliminary survey is well conducted and thorough, it cannot present all the geotechnical particulars nor the geological aspects that become evident at the actual time of construction of the foundation. For this reason, some changes in the plans or in the methods of construction may be necessary. To illustrate job situations, two cases are discussed:

a) *The simple case* is where the soils encountered are homogeneous and well identified before the contract is signed, when the plan is thorough and has been completed before beginning the job, and when there has been no postponement due to a variation or to a technical problem at the time of bidding.

When the plans have been reviewed and the site visited prior to the beginning of the job, the project falls into the category of the simple case.

When the job begins, the only items left to check are that the assumptions and conclusions made in design are verified in the field. These assumptions should be clearly found in the design calculations and the technical drawings. Any reconciliation can be made while installing the first pile (or the test pile).

The possible changes that will need to be made should be minor and could involve:

-- the level of trimming the top of the pile,

-- the exact depth attained by the bored piles of a foundation,

-- the use of certain equipment for drilling or for placement of the rebar cage or the concrete,

-- changes due to the rate of drilling which may influence the overall progress of the work, etc.

When the construction supervisor looks at the schedule of work, he will ensure that the rebar cages are not fabricated at a faster rate than the expected rate of placement of these components. In this way, if a change in the construction plan is needed despite the precautions that were taken, it will be less difficult and costly to make the changes than to have many unneeded rebar cages on the job site.

In a similar fashion, the construction supervisor will examine the construction plan (see § 2.2.4 of this chapter) and will see that construction is begun with widely spaced piles, if the mobility of the equipment so permits without any difficulty. Thus, in case of problems, remedies may be found without too much trouble by modifying the spacing of the other piles in the foundation.

This strategy of the construction supervisor should be applied with much precaution and discernment because it does not, contrary to the following case, rely just on the explicit clauses of the contract. The construction supervisor should act toward the contractor more by persuasion than by the voice of authority. He should not go beyond reasonable limits in suggesting the use of the equipment because the contractor may issue a claim for additional charges.

b) ***The other case***, which sometimes arises, is where the construction conditions are different from those initially anticipated. This may occur for a number of reasons: encounter with an obstacle during the course of drilling that was not apparent at the time of the preliminary survey, physical characteristics of the foundation soil that are different from those deduced by interpretation of the geotechnical tests, setbacks on the job site, etc.

The problem is to resolve each setback in a specific fashion. Some examples are cited of such setbacks that require modifications of the plans or of the construction method.

The foundation of a bridge will be constructed within the stream bed of a river and a cofferdam with diaphragm walls will be employed. In the preparation for the construction of the diaphragm walls, the excavation tool encounters a block of concrete prior to reaching the full depth. This kind of setback could not have been foreseen at the onset because it involved one of the units to provide protection against scour of a pier for a bridge located at some distance up-river. After authorization by the construction supervisor, the contractor could use different equipment to continue with construction of the cast-in-place walls. It goes without saying that a clause in the contract to deal with the problem that was encountered could have been easily overlooked because the obstacle could not have been foreseen. In this case, the setback could be truly unforeseeable. On the other hand, in other cases, special consideration should be taken with the clauses in the construction contract, especially when the work is in an urban environment.

Piles for the foundations for a long viaduct are placed in a soil that is expected to provide sufficient capacity for barrettes of appropriate dimensions. At the time of the construction of the first barrettes, the properties of the foundation soil as it appeared when lifted by the drilling tools did not correspond with the geotechnical characteristics estimated in the preliminary tests. A new program of soil investigation, employing in situ tests, was quickly undertaken for each support. The result was that the behavior of the soil at the level of the base of the barrettes were overestimated at the time of the preliminary survey. The overall concept for the foundations of the viaduct had to be completely modified. This type of setback may have been solely due to the use of tests that were improper for the solution of problems posed by such foundations.

When the physical characteristics of a foundation soil are not as anticipated, it is necessary to determine whether the phenomenon is localized or not, and to proceed to some additional tests to define the modifications needed for the subsequent piles. It may be that the resistance of the soil is higher than anticipated. With the results of the additional soil investigation, the construction supervisor can make the decision to raise the level of foundation after having consulted with engineers specialized in soil mechanics.

Certain setbacks on the job may require modifications in the entire concept of the foundation design. An example is where a drilling tool remains at the bottom of the borehole and is not retrievable. If the pile is practically finished, this is not so serious; the excavation may be concreted and the pile will have one strongly reinforced point! On the other hand, if only one part of the pile is bored, it will generally be necessary to plan for the construction of a supplementary pile. If space is limited, the concept for the foundation as a whole may require revision.

There is no general solution for resolving such problems. All the special solutions that may be found cannot be covered in the contract and in the administrative plan. Moreover, in some cases, insufficient information may present an intrinsic difficulty that may not be remedied by an additional program of soil investigation.

2.4.6. TESTS FOR COMPLETED PILES

At present, the contractual documents are relatively poor on the subject of tests following the completion of a pile. This is due to the fact that the methods of testing in current use have recently been developed. On the other hand, the development of bored piles of large diameter has been accompanied by an increased need for caution during construction.

The reader may refer to a discussion of the methodology of the testing of completed piles in Chapter 7 of this document. It may be stated simply that there are actually a number of methods that allow testing of the quality of piles that have been constructed.

As is noted in Chapter 7, such proof tests must be defined with precision before work begins. The amount of testing to be done should vary in accordance with the importance of the job and the general concept used in the design of the foundations. A bored pile in a line of six piles of 1 meter in diameter, for example, will deserve more careful testing than a pile in a group of 15 piles of 0.60 meter in diameter. The result of poor construction of the larger pile is obviously more severe than for the smaller one.

The non-destructive or proof testing is done on:

-- the soil-concrete contact, at the base of a pile,

-- the homogeneity and the integrity of the concrete of the shaft.

In addition to data from non-destructive testing of a few piles, information can be gained that may be of interest for the construction of the other piles on the job. The results of the tests may allow improved construction procedures to cope with a difficult problem. The arrangement of the non-destructive testing should be made in such a way that possible repairs can be made in case defects are found in some of the piles.

CHAPTER 3

BORING

Of all the operations that govern the construction of a pile when the soil must be excavated, it is the boring that should be the object of special attention. It is the first operation and is influenced by a large number of often imprecise parameters.

Boring is the operation that involves the greatest number of unforeseeable difficulties of a geotechnical nature because of the natural heterogeneity of subsurface conditions. A related problem is that predictions cannot always be made about the applicability and reliability of the various methods of machine excavation. Such difficulties occur despite precautions taken during the preliminary surveys (subsurface and site reconnaissance) or following the relevant regulations that appear in guidelines such as the CCTP.

In an attempt to limit these hazards, this chapter includes:

-- a presentation of available procedures as complete as possible; additional detail is given in the Technical Notes (TN) at the end of this manual;

-- some recommendations about these different procedures in relation to the essential criteria of achieving quality boreholes and in relation to the nature and particularities of the soils encountered.

Finally, this chapter includes the principal tests that may be used to investigate the quality of construction and the durability of foundations on bored piles.

3.1. BORING WITH USE OF A RETRIEVABLE CASING

3.1.1. Chopping tools or grab buckets (TN Nos. 1 and 2)

This involves a procedure for excavation of a borehole using a chopping tool, a cylindrical grab bucket on a cable, or a lightweight hammergrab (which permits simultaneous breaking up of the soil by percussion and extraction of the debris). These tools are used inside a casing that penetrates under its own weight or is driven.

The rock materials are preliminarily crushed by percussion with the aid of a chopping tool (Fig. 18).

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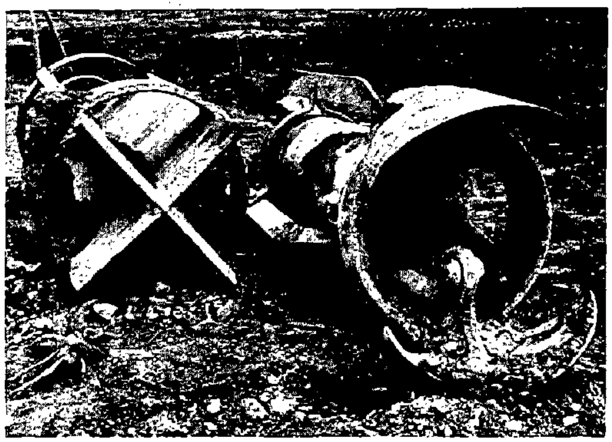


Fig. 18. -- Drill bit and grab bucket.

These tools may be used with a simple winch and hoist (with a timber or metal tripod) or with more elaborate machines using inclinable, metal derricks that can be dismantled or moved without dismantling. A derrick of the Migal type, for example, may be used.

Such an outfit allows construction of piles of diameters varying from 0.40 m to 0.85 m (1.3 ft to 2.8 ft) and in exceptional cases up to 1.2 meters (4.0 feet).

3.1.2. BENOTO method (see TN No. 4)

This method differs from the preceding, from which it has been derived, by use of pushing, twisting or rocking of the temporary casing to achieve penetration. Depending on the type of machine used, the diameters of piles that may be constructed by this method vary from 0.60 m to 1.20 m (2.0 ft to 4.0 ft) and exceptionally up to 2 m (6.6 ft) (Fig. 19).

A variation of this procedure involves the use of a driver-extractor of the Foncex type, for example. Thus, the system of handling the casing is totally independant of the boring system. The power unit in the boring system may be a crane that utilizes tools as varied as drill bits, hammergrabs, and hydroelectric or mechanical grab buckets (Fig. 20).

3.1.3. Open, driven casing

In contrast to the preceding techniques, the placement of the temporary casing in the method described here precedes the actual operation of boring. The casing is placed with the use of hydraulic rams or with a double-acting hammer (see TN No. 5) to the desired depth and the soil

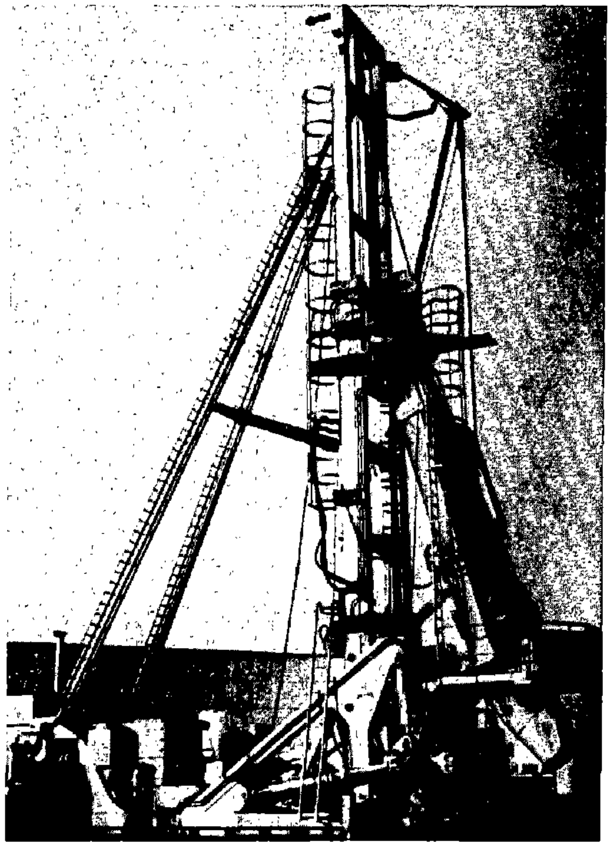


Fig. 19. -- General view of an EDF Benoto machine.

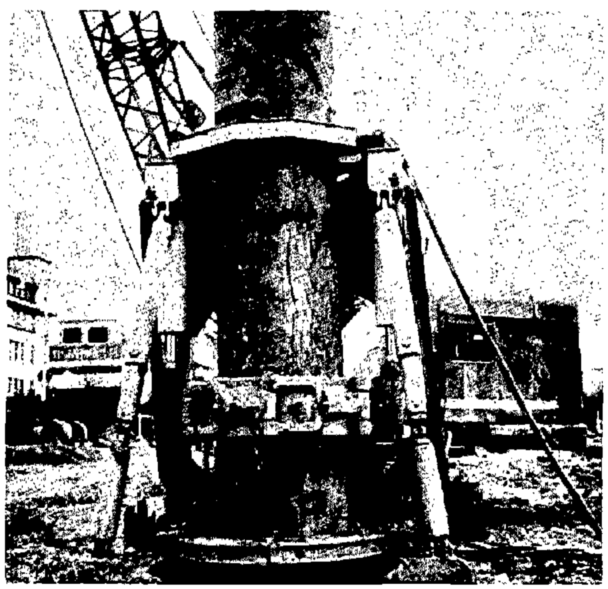


Fig. 20. -- Driver-Extractor, Foncex Type (Document from Benoto).

that is penetrated is then extracted from the casing in one operation with the aid of tools previously described.

When refusal is encountered prematurely, the casing is cleaned out and the excavation is carried through the strong stratum of soil using a chopping tool; then the installation of the casing is continued. These operations are repeated as often as necessary until the casing reaches the desired penetration.

Technical Note No. 3 gives information about the thicknesses of the wall of a casing in relation to its diameter.

3.1.4. Open, vibrated casing

Based on the same principle as the preceding, this method makes use of vibro-percussion to sink a steel casing. Technical Note No. 6 describes the vibratory drivers that are currently in most use and indicates, according to diameter (ranging from 0.40 m to 2 m - 1.3 ft to 6.6 ft), the wall thicknesses of the casings that are generally used (Fig. 21).

3.1.5. Special systems

3.1.5.1. Atlas piles

This is a pile of from 36 to 60 cm (14 to 24 in.) in diameter that is placed without the excavation of the soil. A casing is installed by rotation with a machine of the Atlas type that has at its base an auger and a special tip. This tip is disconnected at the bottom of the borehole and remains in place as the casing is extracted as concreting progresses (Fig. 22).

3.1.5.2. Drilled pile with continuous-flight auger

This procedure is derived from techniques used in preliminary soil surveys and consists of using a tool which is a continuous auger with a hollow stem. A loose plug seals the bottom of the hollow stem during the placing of the tool. Concrete can then be pumped through the hollow stem as the auger is retrieved. The Tecvis Soletanche pile is an example of this kind of pile (Fig. 23).

This technique does not permit the placement of reinforcement and is mainly used for the construction of piles that are subjected only to compression (foundations of some buildings, for example).

Fig. 21. -- PTC vibratory driver, Type 40 A2.

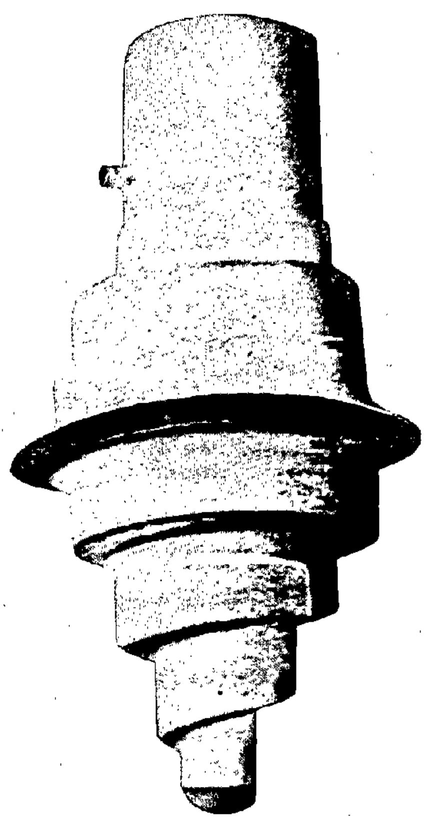


Fig. 22. -- Auger tip of an Atlas Pile.

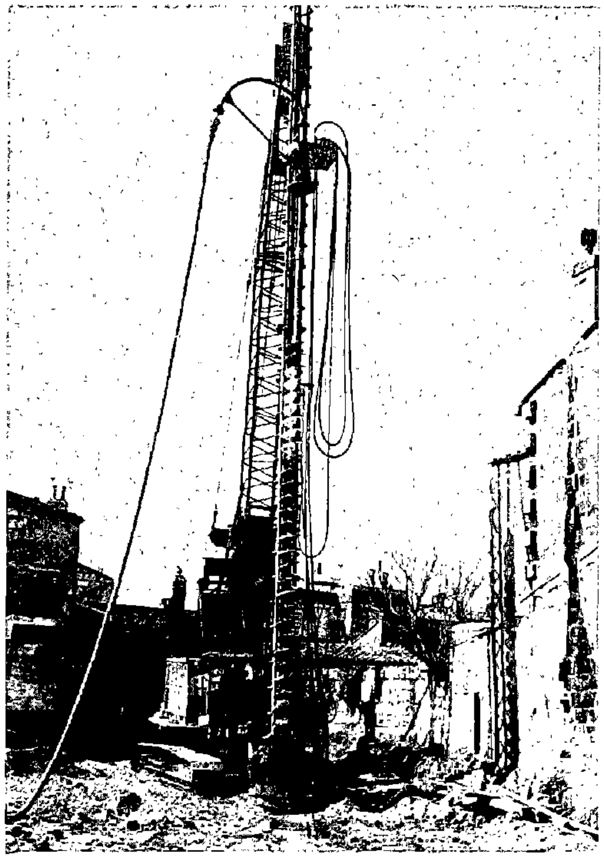


Fig. 23. -- Continuous auger drill, Tecvis Soletanche method.

3.2. BORING WITHOUT TEMPORARY CASING

3.2.1. Boring with drilling fluid

3.2.1.1. Bentonitic slurry

a) *Drilling mud*

Sometimes called "heavy water", drilling mud is a colloidal suspension (not a solution) with a bentonitic base in which the water appears in the form of:

-- free water between the particles of clay. By drying out or by filtration in the soil, this free water is removed and a plastic residue collects on the wall of a borehole which constitutes a filter cake or membrane;

-- absorbed water, meaning water that is fixed in a rigid fashion over the surface of the particles of bentonite;

-- absorbed water or molecules that integrate with the particles to transform them into "gel".

Drilling mud is generally obtained by dispersing at least 50 kg (110 lb) of bentonite for each cubic meter of water (264 gallons). In addition, additives are often used to increase weight; improve sealing capability; enhance viscosity or enhance fluidity; combat against contamination; lower or raise pH; and finally reduce the filtrate or the quantity of free water which filters through the cake (see § 3.4.5).

In saline formations, use is made of either treated muds or special clays of the attapulgite or sepiolite type that, contrary to bentonites, do not flocculate in the saline water. Their elevated colloidal stability in saturated brines allows the preparation of muds of high yield.

It is recalled that the yield of a mud is expressed in cubic meters of mud made up at a Marsh viscosity of 30 to 35 seconds (see §3.5.9.4) for a tonne (2205 lb) of clay dispersed. For information only, a pure bentonite presents a yield from 15 to 18 m^3/t (489 to 577 ft^3/T, where T is 2000 lb). Higher yields involve treated bentonite or special clays.

The essential function of a bentonitic mud is to maintain the wall of the borehole despite the absence of a casing. The formation of a slightly permeable cake which lines the borehole wall and the hydrostatic pressure exerted on this wall limits the risks of caving-in, reduces mud losses, avoids the infiltration of the ground water into the borehole, and limits the swelling of certain soils. It is advantageous to ensure that the pressure from the drilling fluid be maximum; therefore, the level of the mud in the borehole should be maintained at the highest possible level.

When circulating the drilling fluid in the borehole, among other things the mud permits the removal of sediments which are suspended in the mud.

The upper part of the borehole is always protected by a guide casing (or collar) of 3 to 6 m (9.8 to 19.6 ft) in length. The short casing is used to prevent the caving or collapse of the near-surface soils that would result from construction-equipment loads, or the disturbance related to the placing of concrete and reinforcing cage. In the case of barrettes, this protection is ensured by a low guide wall or by a metal collar of the appropriate shape (Fig. 26).

Finally, it is obvious that on an aquatic site the length of the guide casing is increased for the distance from the bottom of the river bed (or of the sea) to the top of the work platform.

The mud is manufactured at a plant (Fig. 24 and Technical Note No. 9) that consists of:

-- *a mixing unit* with bentonite-storage silos, a turbine mixer, a storage tank for freshly mixed mud [with a capacity related to the size of the job, 50 m^3 to 300 m^3 (1765 ft^3 to 10,590 ft^3)] and the tanks for storage of the mud that is returned from the borehole;

-- *a treatment unit* with a vibrating sieve (mesh of 1 mm^2 (1.5 x 10^{-3} in^2) for eliminating the large particles from the return tanks; and one or more hydrocyclones for eliminating the fine sands and the silts.

The drilling mud is thus regenerated or recycled and is checked to see that specifications are met (see § 3.5.9), and is then sent to the special storage tank. The treated mud is then returned to the circulation system after the addition of new mud. Thus, the properties of the mixture returned to a borehole are satisfactory despite the presence of 1 to 2% of sand.

b) *Tools*

Many types of tools may be employed in making an excavation in the presence of drilling mud; these tools include grab buckets, helicoidal augers, drilling buckets, tungsten or diamond core drills, and various rotating drills. Bentonitic mud is also perfectly suitable to the construction of boreholes by simple percussion *drilling*.

GRAB BUCKETS (see TN Nos. 2 and 10)

These tools are suitable, according to their shapes, for the boring of cylindrical shafts and for barrettes. According to the method of boring, distinction is made between:

-- *buckets suspended on a cable* (Fig. 25) in which the guidance is enhanced by the use of a surface casing. The equipment that is used is varied in regard to the method for handling the bucket. The bucket may be handled by cable or by hydraulic or hydro-electric equipment.

-- *buckets attached to a kelly* (Fig. 26). The kellys in current use permit excavations of 40 m (131 ft) in depth and with round or rectangular buckets. Round buckets vary from 120 to 180 cm (47 to 71 in.) in diameter and rectangular buckets range from 30 to 150 cm (11.8 to 59 in.) in width

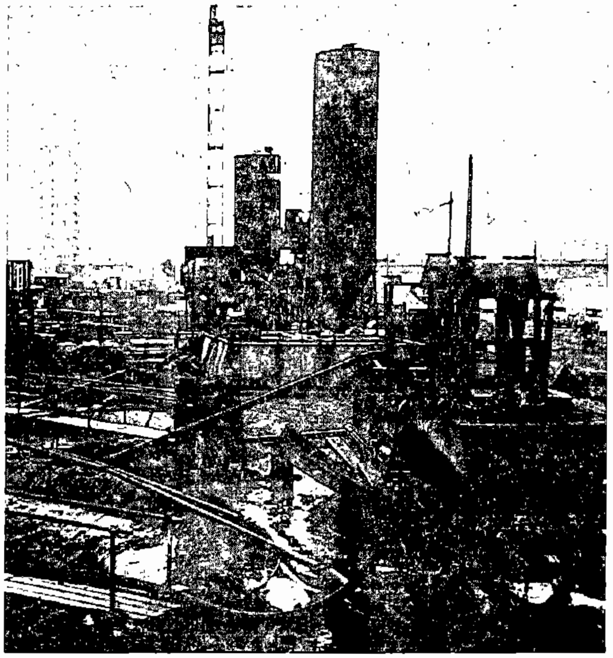


Fig. 24. -- General view of a plant for manufacturing and regenerating mud.

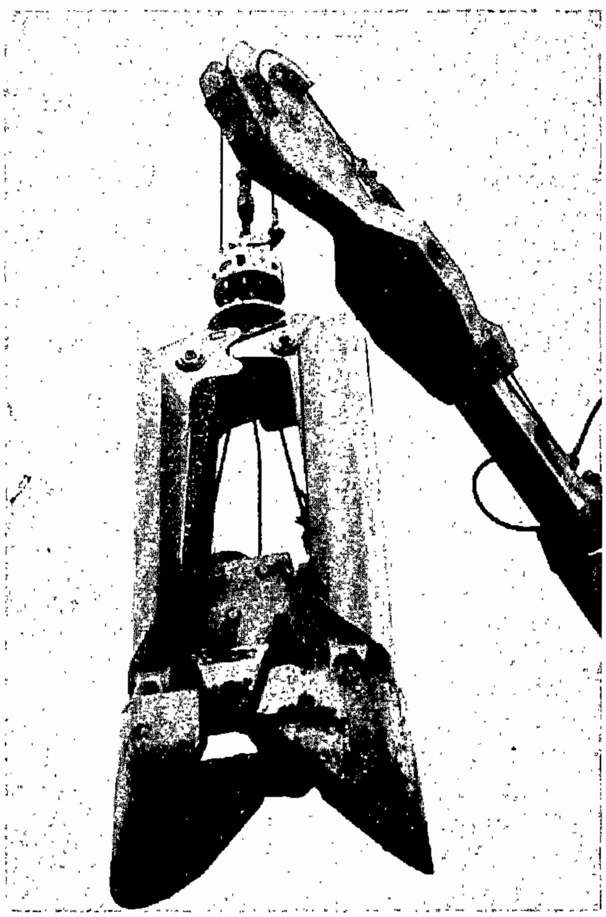


Fig. 25. -- Type SC Benoto bucket suspended and operated by cables.

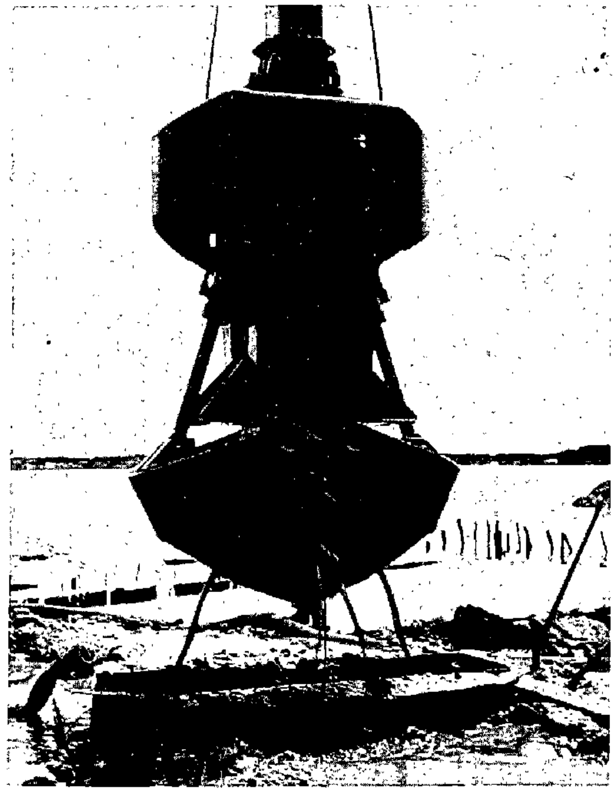


Fig. 26. -- Drilling for barrette pile with a kelly bucket (note the presence of the metal collar at the head of the borehole).

and from 120 to 300 cm (47 to 118 in.) in length. The operation of these buckets is mechanical, hydraulic, or hydro-electric;

-- ***buckets hung on derrick of a hydraulic shovel either directly or with a long extension***. These buckets are used with cranes, such as Artois or Provence or Benoto, or Poclain shovels (Fig. 27) of the G type (GY 120, GC 120, GC 150). According to the shape of the bucket that is selected, the shape of the boreholes will be round, square or rectangular and the depth can reach 15 to 16 meters (49 to 52 ft). This procedure is obviously reserved for construction of shafts and of barrettes in soils which do not require casings.

FLIGHT AUGERS AND DRILLING BUCKETS

These tools are used exclusively for the construction of cylindrical piles. Augers and buckets, in principle, can be used alternatively on the same job, depending on the nature of soils encountered and the pref-

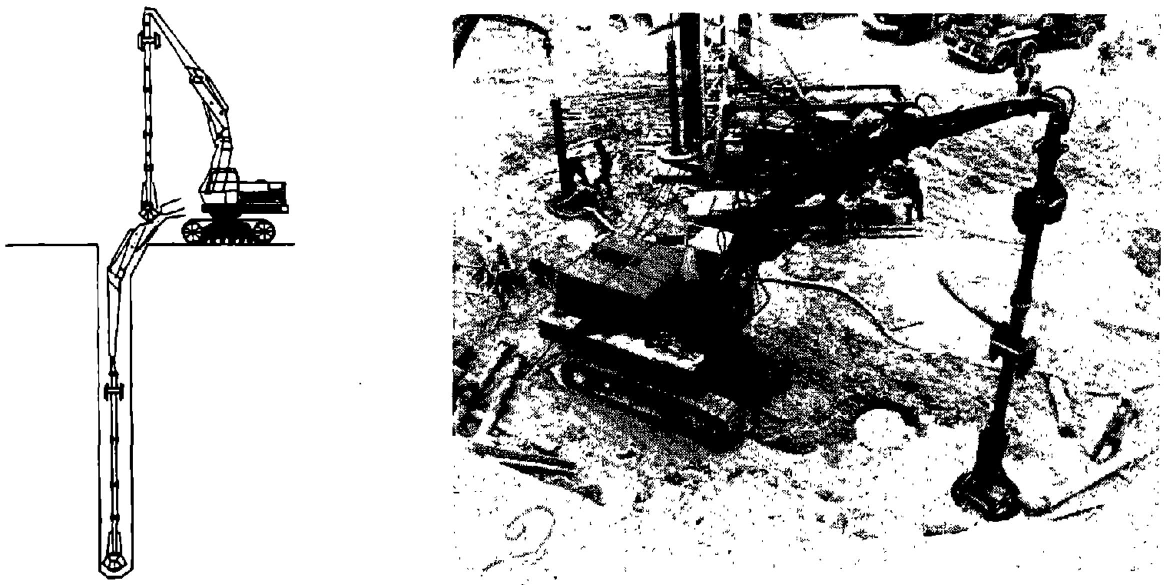


Fig. 27. -- Cylindrical drilling with a Poclain shovel equipped with a long extension.

erence of the workmen. As a general rule, flight augers (Fig. 28) are suitable at the start of the boring to drill through dense layers while drilling buckets (Fig. 29) are more efficient in loose soils and permit a better cleaning of the bottom of the hole.

Augers or buckets are attached to the end of a single kelly or to a telescopic kelly (Fig. 30). The kelly is turned by the rotary table of a machine made by manufacturers such as Calweld, Watson, Trevisani, or Williams. The power unit with rotary table, depending on the model, (see TN No. 7) may be mounted on a truck or on a crane. Each of these types of mountings has specific mobility advantages.

The types of regular or special augers vary according to the manufacturer, and the design of the auger is a function of the holes to be bored (see TN No. 8).

Given the importance of the torque required to drill through soils of medium hardness that lie beyond about 20 m (66 ft) of depth, the dimension of the drilling bucket rarely exceeds 1.50 m (5.0 ft) in diameter.

However, boreholes of more than 3 m (9.8 ft) in diameter may be completed by modifying an auger with cutters that extend its diameter (see TN No. 8).

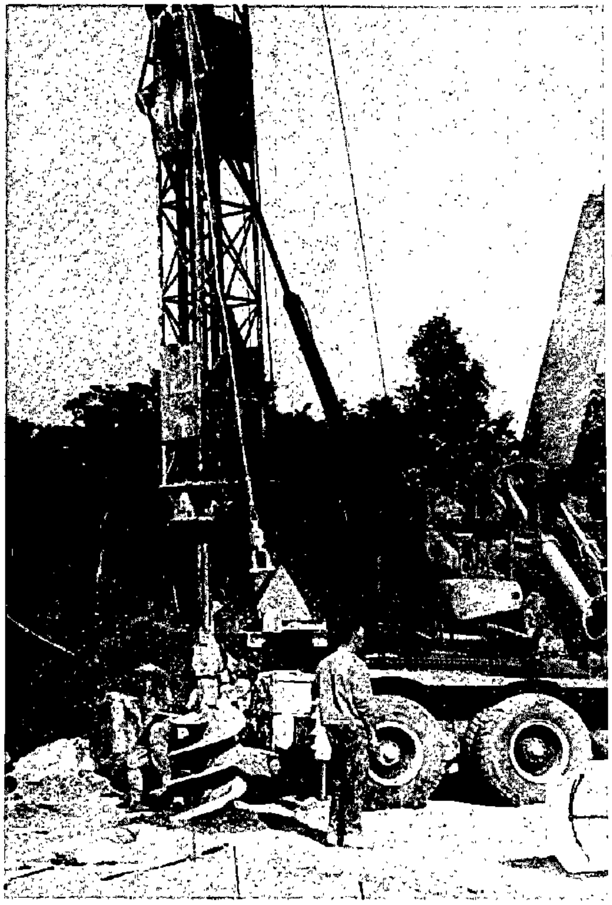


Fig. 28. -- Example of flight auger (Williams).

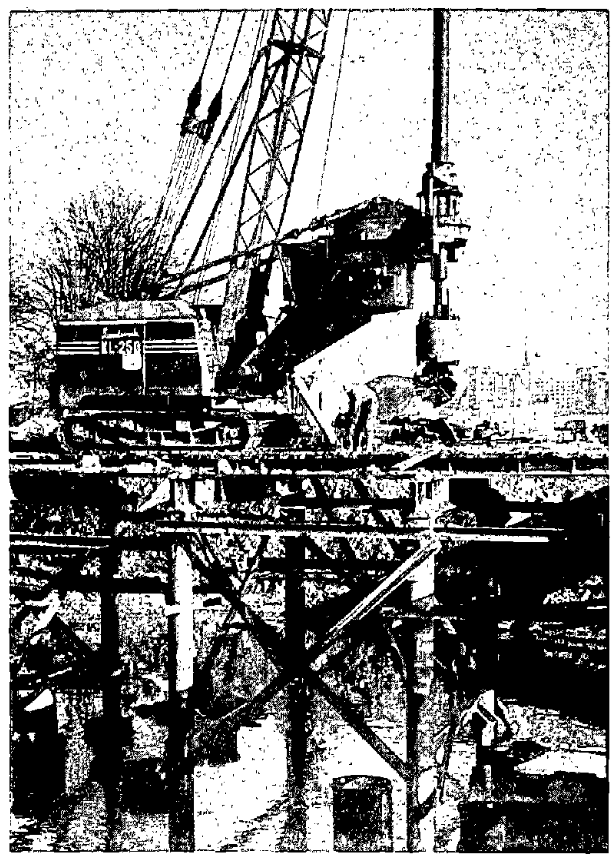


Fig. 29. -- Bucket in open position.

Fig. 30. -- Telescopic kelly.

At the bottom of the borehole, underreaming tools permit diameters two or three times that of the shaft (belled foundations). This technique is little used in civil engineering.

SIMPLE TUNGSTEN-TOOTHED CORERS

Corers of 0.30 m to 3 m (1 ft to 9.8 ft) in diameter are used for the drilling of particularly strong soils (Fig. 31) to obtain a good anchorage in a very resistant substratum, or also for the construction of secant piles. However, when the extraction of the core becomes impossible it may be necessary to use other drilling methods.

Fig. 31. -- Example of limestone cores of 85 cm in diameter. (2.540 cm = 1 in.)

ROTATING DRILLS

These tools can be employed to break up strong soils or rock and the fragments are brought up to the surface by reverse circulation inside the hollow stem.

In loose soils, drills with teeth or flat projections are used (Fig. 32). In hard rock, augers with rotary bits can be used (Fig. 33); such augers are derived from tools used in the petroleum industry (see TN No. 8). The diameters of such tools range from 30 cm to 120 cm (11.8 in. to 47 in.), but manufacturers mention that tools exceeding 2 m (6.6 ft) in diameter can be furnished on demand and that the tools can be designed to operate in all the categories of soils that may be encountered.

c) *Methods of Boring*

WITHOUT CIRCULATION

In this case, drilling mud is added into the borehole in proper amounts as the hole is advanced; the mud is recovered by pump during concreting as the column of concrete lifts the mud. This mud, which cannot be treated and reclaimed in a continuous fashion, progressively takes on sediments and may need to be replenished in the course of a boring. Tests are performed regularly on the samples taken at different depths to indicate the necessity of changing the mud. To this end *sampling bottles* may be used that have sufficient capacity (≈ 2 liters - 0.5

Fig. 32. -- Example of rotating drill with flat projections for use in loose soils.

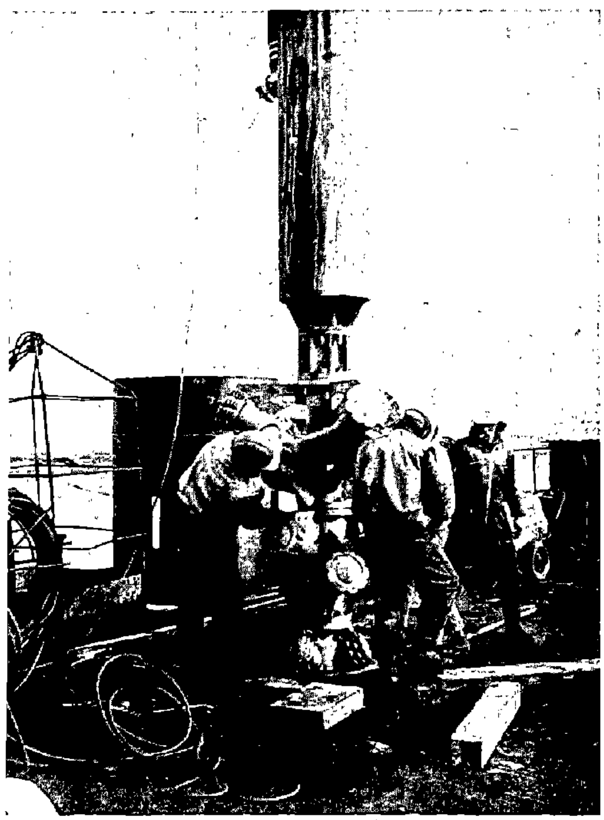


Fig. 33. -- Example of rotating drill with rotary bits for use in hard rock.

gal) for performance of various tests of mud quality (density, sand content, viscosity, pH, filtrate and free-water content; see § 3.5.9 and TN No. 9).

This procedure is mainly used for the construction of boreholes using augers and grab buckets.

BY DIRECT CIRCULATION (Fig. 34)

The water, or the mud, is forced back through the interior of the drill pipe with the aid of a powerful pump and rises in the annular space between the drill pipe and the wall of the hole.

This method allows the continuous recycling of the mud during the course of operation. However, for piles of large diameter the annular space is such that the ascending speed is insufficient for the effective lifting of the sediments or for satisfactory cleaning of the bottom of the borehole.

The drilling techniques corresponding to this method are:

-- ***Drilling under water pressure*** on the order of 0.5 to 1 MPa (72.5 to 145 psi) in less compact soils and for diameters of 30 to 70 centimeters (11.8 to 27.6 in.). The tool (Fig. 35) is fixed to a shaft and is composed of one or more ducts that are mounted on a cutting arrangement with teeth faced with stellite. However, it should be noted that the diameter of the borehole cannot be controlled well if the strength of the soil is low. Localized enlargements of the borehole can occur that are difficult to detect.

-- ***Drilling with an auger with roller bits or with a coring tool*** for all other soil types and with diameters up to 120 cm (47 in.).

BY REVERSE CIRCULATION

This procedure consists of lifting mud, that is supplied to the annular space between the drill pipe and the wall of the borehole, through the interior of the drill pipe. The force to lift the mud loaded with sediments is obtained by suction from the surface through the drill pipe (Fig. 36) or by the injection of compressed air (Fig. 37) near the base of the string of drill pipe. The upward velocity thus obtained allows the complete recovery of the soil and rock broken up by the drilling tool (rotating drill bit in this case).

This method avoids the continual lifting of the drilling tool and permits an increase in the weight on the tool by simple addition of drilling collars, which has the advantage of limiting the vertical forces

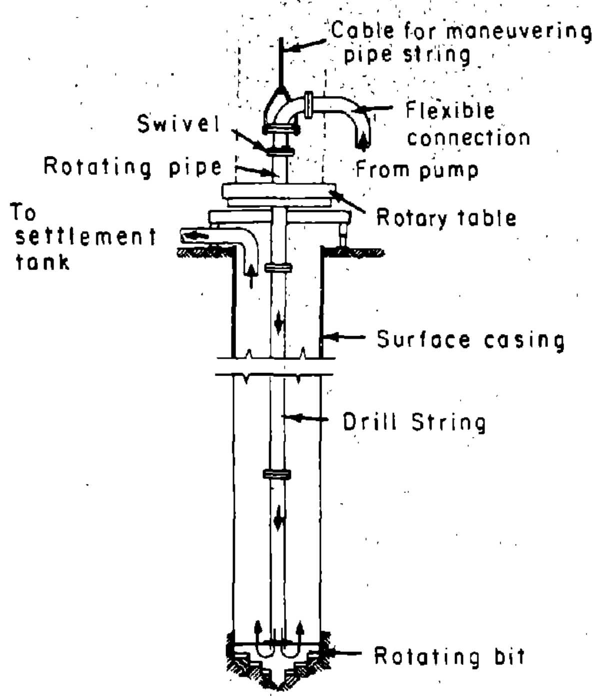


Fig. 34. -- Diagram of principle of direct circulation.

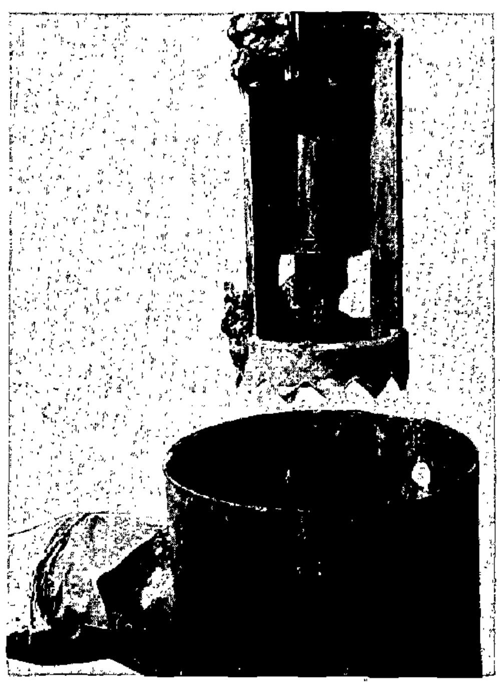


Fig. 35. -- Example of cutting arrangement. To the left of the casing at the head is the pipe for removal of water loaded with sediments.

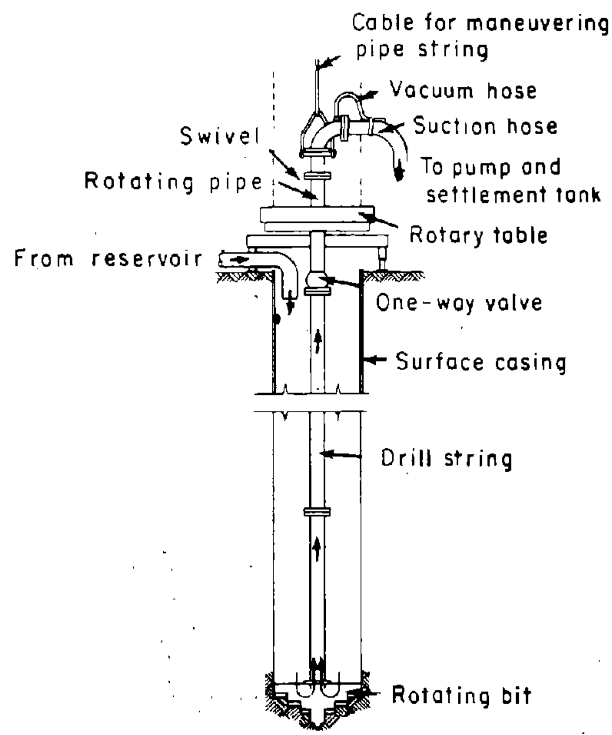


Fig. 36. -- Diagram of principle of reverse circulation by suction.

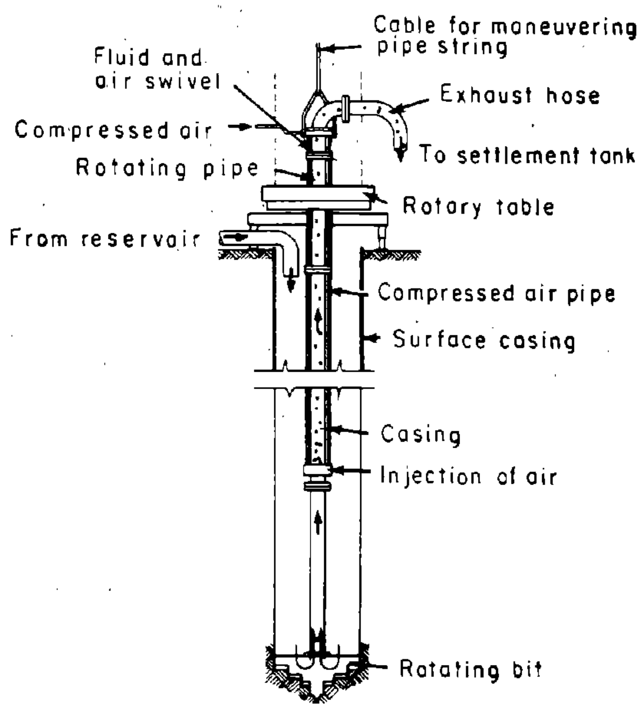


Fig. 37. -- Diagram of principle of reverse circulation by injection of compressed air.

on the rotary table. This is a procedure particularly well adapted for aquatic sites.

Moreover, the continuous, rapid circulation of the purified mud ensures an efficient cooling of the drilling tool.

Equipment for drilling with reverse circulation is supplied by Salzgitter, CIS Soletanche (especially used for the manufacture of diaphragm walls), Hydrofond, and Calweld (see TN No. 11).

Figure 38 shows the dimensions of rock fragments that may be extracted by this method (materials obtained in construction of foundations of piers for the bridge crossing the estuary of the Loire between Saint-Nazaire and Mindin).

Fig. 38. -- Fragments of gneiss raised by reverse circulation during drilling with rotary drill with rotary bits.

The result of the proper use of the reverse-cicrulation method of drilling is an optimal cleaning of the bottom of the borehole, which constitutes one of the essential conditions of the quality of the concrete-soil contact at the base of the foundation, a condition that strongly affects the good behavior of a foundation.

Also cited, based on the exceptional nature of the jobs, are the foundation piles of the Maracaibo bridge and of the overhead monorail in Tokyo. These piles were bored up to 70 m (230 ft) using reverse circulation, a procedure that, at present, seems to be the most efficient from the standpoints of quality of construction and avoidance of construction difficulties as well as the performance of the completed foundations.

3.2.1.2. With clear water

The construction of boreholes with clear water is simply a special case of the general method of boring with bentonitic mud. If geotechnical conditions are sufficiently exceptional so that the wall of the borehole is stable under water (for example, with cohesive soils that are stiff or hard), the question arises as to whether or not piles are actually needed at the site. However, drilling with clear water is valuable at an aquatic site or in compressible clay soils where casing or drilling mud is not required. Clear water should be used as much as possible if caving is not a problem because the use of drilling mud can cause a reduction in the value of the axial capacity of a pile due to loss of resistance in skin friction.

If clear water is used, the level of the water in the borehole is maintained by means of a surface casing at a height well above the phreatic surface in the soil so that the seepage of water is from the hole into the soil. If the water level in the excavation is below the natural water table in the soil, the inward seepage of water can aggravate the risk of caving. Keeping an appropriate head of water at the top of the borehole during all phases of construction of piles also counteracts the possible collapse of the base of the borehole due to inflow of water and can protect fresh concrete from possible deterioration (increase in water-cement ratio) because of the upward flow of water.

3.2.2. Dry boring

Because soils justifying foundations for piles are generally saturated (phreatic surface very close to ground surface), this procedure is unfortunately rare. The possibility of dry boring sometimes arises in very slightly permeable, cohesive soils where boreholes can be constructed using all of the tools and machines previously mentioned, but also, more simply, using mechanical shovels (Poclain for example) equipped with grab buckets. Also, excavations can be made, in a more rudimentary fashion, by hand in very special circumstances (small jobs, small depths, few shoring constraints, or very difficult access to machines).

3.3. BORING BY PERCUSSION

This method, which would be used in exceptional circumstances, consists of breaking up the very dense or rocky soils using heavy tools (chopping bits) in various forms (simple wedge, cruciform, etc.) that are dropped freely in order to penetrate the dense stratum.

Percussion drilling involves a long, difficult and costly operation for which there are no precise guidelines. The contractor can first consider the precise information about the subsurface conditions, taking into account the technical requirements of the project, and secondly consideration is given to the best kinds of equipment to be used for the particular problem (type, dimensions, and weight of the chopping bit in particular) (see TN No. 2).

To obtain adequate penetration of the substratum, use of the chopping bit should allow excavation of the rock for a depth corresponding to two or three diameters if necessary. However, it should be noted that there should be a limit to use of the percussion method. If a considerable amount of rock must be penetrated, either for obtaining an appropriate bearing resistance or anchoring, or because intermediate layers must be penetrated, it may be preferable to make use of rotary drilling equipment using rock bits or core barrels (with bottoms provided with stellite or tungsten teeth).

It should be noted further that the efficiency of the percussion method is undeniable for penetrating natural (boulders) or artificial obstacles. However, percussion drilling becomes much more risky in simple, compacted, soils such as clays and loose marls, for example. In such formations, it is thus preferable to use rotary drilling and to avoid the use of percussion-type drills which are all too often proposed for small jobs, regardless of the geotechnical conditions, under the pretext of limiting the costs of mobilizing and de-mobilizing the equipment. Indeed, the economic advantage that apparently favors the least sophisticated procedures is lost later by cost overruns that result from difficulties and delays that are inherent when techniques are selected that are inappropriate for the job in question.

Once the opportunity for the use of the percussion method is acknowledged, it is necessary to adapt the tools to the specific difficulties for each job site, particularly to improve their guidance (guides on the top of tool, on the bottom, or both on top and bottom, such as in Fig. 39 for example).

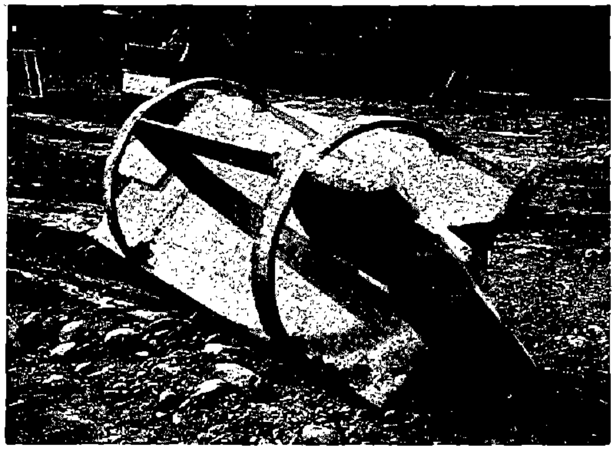


Fig. 39. -- Example of cruciform drill with crown guides.

Finally, whatever the motives might be for justifying the percussion method of boring, it is of interest to note that the cost for such work should be on the basis of the amount of hole drilled and not by the hour. Such an arrangement is as much in the interest of the construction supervisor as in that of the contractor. Thus, there is incentive to make use of the most appropriate tools.

3.4. RECOMMENDATIONS

The qualities of a borehole are dependent, on one hand, on the nature and characteristics of the soils encountered and, on the other hand, on the selection of equipment. The following fundamental characteristics are required:

-- uniformity and regularity of the wall to ensure that pile dimensions are as near as possible to the theoretical dimensions and to avoid, in this way, prohibitive consumption of concrete;

-- consideration of verticality or of inclination, even at the level of the founding stratum that, in the case of a strongly inclined substratum, can pose some problems;

-- consideration of the depth of penetration in the founding stratum that conforms to the requirements of the project, which can certainly influence selection of one type of tool or one procedure over another;

-- proper cleaning of the bottom of the hole, in such a way as to obtain, after concreting, the most perfect concrete-soil contact possible, without accumulation of sediments at the bottom of the boring.

3.4.1. Uniformity and regularity of the wall

Excluding stiff or dry, cohesive soils in which the wall of the borehole is naturally stable and regular, but due to their qualities rarely require deep foundations, five principal types of soils are identified as requiring special precautions. The specific problems of penetration into the founding stratum will also be addressed.

Nevertheless, regardless of the nature of the soils, the uniformity of the wall is better preserved when the duration of the construction of the borehole is short and when the time periods for the subsequent operations (casings, reinforcing steel, concreting) are further reduced.

With regard to the time for the construction of a pile or shaft, a boring without casing should not be undertaken when the concreting may not be performed on the same day.

In particular circumstances, when it is desirable to make full use of the equipment or to regain time lost to downtimes (frequently caused by breakdowns or by delays of supplies), the contractor may wish to begin a borehole or, indeed, several boreholes by the end of the day. But special care must be taken to see that all the drilling and concreting are planned at the latest for the next day.

Obviously, in this constant concern for limiting the risks of cave-in, it is important to coordinate as well as possible the successive operations which are involved in the construction of a pile. In particular, it is very important to obtain concrete and to place it at once and without interruption after the borehole has been completed and is ready for concrete. The installation of a mix plant or the selection of a ready-mix supplier near the job site and the careful organization of the movements of the concrete trucks are very important in regard to quality of the concrete (see § 6.3.2).

One should keep in mind that percussion drilling causes vibrations felt through the upper soil layers and increases the risk of cave-ins in open holes.

3.4.1.1. Cohesive soils of medium or low consistency

Cohesive soils of medium or low consistency can be bored using nearly all procedures. However, for depths beyond 20 m (66 ft), it becomes difficult to push (or drive) and to extract the casing because of the rapid increase in the lateral friction with depth. In such clays, boring with clear water or with bentonitic mud is preferable. The appropriate drilling tools (augers, grab buckets) allow very satisfactory construction progress and the quality of the borehole should be satisfactory. The difficulties arise, in these cases, in getting a borehole with a uniform diameter. The risk of overbreak is due principally to the eccentricity in movement of the drilling tool and the risk increases with depth and with speed of rotation.

Another concern is that the cohesive soils frequently contain layers of sand or gravel which are water bearing and occasionally under hydrostatic head. Localized caving can result and can render ineffective the drilling techniques that were selected initially. Preliminary geotechnical surveys and preliminary hydrogeological studies, particularly allowing an estimation of the nature of the water in the sands and gravels, are required in order to avoid such hazards during the course of the work.

As an example, the case of a job site is cited for which the construction of diaphragm walls with bentonitic mud was interrupted due to the flow of a large quantity of water within a gravel layer interspersed in a cohesive formation that was presumed homogeneous. In this circumstance, it was necessary to use secant piles that were constructed with the use of casing (Fig. 40).

Embankments constructed very recently over compressible soils and without special precautions, can cause, by the lateral stresses that they create, the caving of the boreholes constructed at the toe of their slopes. This is why it is advisable to build such embankments well before the excavation of drilled piles. Such construction arrangements, moreover, permit the limiting of the horizontal forces on the piles and the differential settlement between the embankment and the structure. In such a case, it is evident that piles constructed with a permanent casing are preferable to those constructed using only bentonitic mud.

It should be noted that, when the proportion of sand increases in the cohesive soils, the efficiency of the drilling mud is often reduced. This sometimes leads to last minute, expensive measures in order to ensure the uniformity of the walls of the borehole. In some unusual cases and at the site of an important project, it may be necessary to inject the

Fig. 40. -- Secant piles required because of significant water flow within a gravel layer.

soft, sandy clays with a cement grout before boring begins in order to limit the risk of caving when using bentonitic mud. It is very obvious that, in this case, the use of casing in the drilling operation is often advantageous.

3.4.1.2. Cohesionless soils above water

This involves either natural formations or deposits, or embankments that are more or less heterogeneous.

The procedures that may be used to drill such materials depend on particle-size distribution, density, and depth or magnitude of the layers:

-- *The sands and gravels* can be drilled, for the most part, with the following procedures:

- either a retrievable casing when the depth or strength of the layer and the diameter of the pile are sufficiently moderate so that the frictional resistance against the casing remains compatible with the capability of the available equipment for driving and for extraction of the casing,

- or a bentonitic mud which can be used regardless of the depth or the diameter of the borehole.

In these soils, boring with clear water without casing is obviously not suitable. However, in certain favorable conditions (the upper layers have some cohesion due to a significant proportion of fines or to the presence of a clay matrix), dry boring may be considered with the use of only a surface casing.

-- *In formations composed of elements of a large size* (rock embankments, jetty rocks, scree-covered slopes, fluvial-glacial alluviums), the procedures of boring with bentonitic mud do not permit assurance of a hole with uniform diameter nor the regularity of the wall. It is then necessary to resort to boring methods using thick casings (≥ 1 cm - 0.4 in.), which are placed by pushing, twisting, or rocking (see TN No. 4) rather than by impact driving or by vibration. It is also preferable to make use of drilling tools of a more elaborate sort such as hammergrabs or rotating drills. The boring of large diameters required by these materials moreover calls for a quasi-systematic use of such tools and practically excludes equipment of the grab-bucket type that is used, as a rule, for piles of more modest dimension (Ø ≤ 1 m - 3.3 ft).

Under certain conditions when drilling is possible without instability of the wall, dry boring may be considered using only a surface casing. However, in this type of formation, such favorable conditions rarely occur.

In a general way, it can be said that in these materials boreholes are always difficult and costly even with well-adapted equipment. Thus, where deposits of large-sized elements exist, it is recommended that piles not be employed unless no other solution is feasible.

3.4.1.3. Cohesionless, underwater soils

This involves the same formations as the preceding, but the granular soils are underwater, as found at a fluvial, lakeside, lagoon, or maritime site.

Thus, the preceding considerations are applicable to this category of materials in which the difficulties of drilling, linked with the particle-size distribution, are even more consequential. There are some situations that present especially difficult problems (for example, a water course with torrential flows in high valleys), and special arrangements must be made to maintain the stability of a boring.

Boring procedures with bentonitic mud should be made in such cases with more caution because of the possible ineffectiveness of the slurry in preventing cave-in in loose sands or ineffectiveness in the presence of possible strong flows of water. These flows should have been accurately determined during the preliminary investigations in order to select the appropriate protection for the fresh concrete against watering down, protection such as a permanent casing or membrane (See § 4.2.1). In this regard, at fluvial sites the risks of undermining often add to the risks of water circulation and, as a consequence, it is advisable to anticipate the use of thick casings or membranes that are resistant to erosion. Thus, it may be advantageous to adopt a boring procedure (vibratory hammer for example) that permits the use of non-retrievable casings for these groundwater conditions and particle-size distribution.

Furthermore, the role of water as a lubricator, and the ascending movement of water created by suction at the time of extraction of the casing or drilling tools, create, at the base of the casing, risks of cave-in that are more formidable than in formations which are not submerged. These cave-ins which may be very consequential become especially harmful when the piles are constructed at the foot of an embankment or slope, which may become unstable (Fig. 41).

To limit the severity of such risks, care is then taken in submerged, cohesionless soils, to keep the casing as close to the elevation of the boring tool as possible and maintain a water level in the casing above the natural water table in order to avoid all phenomena of piping at the base of the casing.

These measures, and the exclusion of all violent systems of boring, such as the percussion-drill outfit, turn out to be especially important when the foundation is situated in these soils. Lacking such precautions, the disturbance that results at the base of the piles may effectively eliminate the end-bearing component relied upon during the design process. The end-bearing component may then have to be re-established through pressure injections, which are always very costly and of questionable efficiency.

At present, when the casing suddenly reaches refusal on a rock substratum, the boring is generally continued with a chopping tool that

Fig. 41. -- Crack at the head of an embankment caused by construction of bored piles at the foot of embankment.

produces a hole of irregular shape, particularly in its upper portion which is often very ragged. Between the casing and the rock, cracks may result. Through these cracks, which may be quite significant, cave-ins may occur in the overlying, cohesionless materials (Fig. 42). It is these caved-in soils, raised with the rocky debris, that are evidence of this kind of problem. In the case just decribed it is sometimes said that there is "sealing of the casing" but no seal was made. To seal the opening between a casing and the rock surface, clay may be dumped at the bottom of the borehole and compacted as the casing is raised to just above the contact surface. This cohesive soil forms a plug against the cohesionless materials situated immediately above the rock (Fig. 43).

The casing is then relowered to the rock through this sealed zone and the excess clay is extracted from the interior of the casing. Certainly, the "sealing of the casing" is easier when the rock presents a more or less soft, weathered zone in which even a weak seal is possible (Fig. 44).

Finally, it is necessary to remember that such problems are more frequent when the rock substratum has an irregular surface or when the piles are inclined. When the preliminary surveys predict an irregular

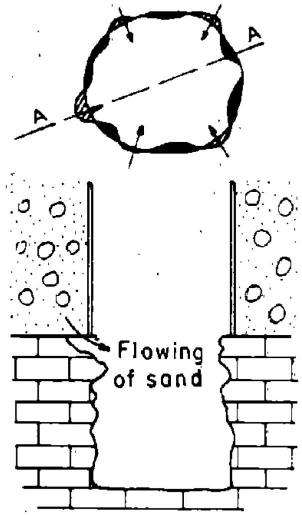


Fig. 42. -- Schematic diagram of the flowing of sand under a temporary casing at the substratum contact.

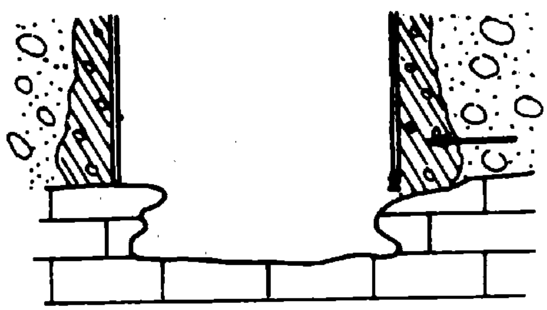


Fig. 43. -- Example of closure of a temporary casing by a clay seal.

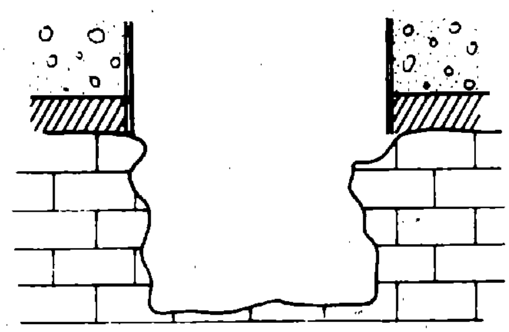


Fig. 44. -- Sealing of a temporary casing facilitated by existence of a soft, weathered zone in the substratum.

surface of the rock at the base of a pile, information on the rock must be communicated to the designers and the contractors so that the proper selection of methods and equipment is made.

3.4.1.4. Soils of archeological interest

These are historical sedimentary deposits (< 2000 years) that are encountered most often in littoral and sublittoral (isolated lagoon-muds) zones or in marshy depressions and valley floors (peat bogs). They may also be reshaped soils or those left by man (embankments, dumps, fill materials from ancient extraction zones, artificial ditches, or natural thalwegs) that are located for the most part in urban and suburban zones. Not only are these soils generally of poor quality and often have heterogeneous mechanical characteristics, they frequently harbor hidden objects (relics, ancient constructions, ancient river networks, ancient foundations, ruins, wooden roadways, etc.) which constitute very serious obstacles to boring.

Because such obstacles often require large construction expenditures and considerable losses of time, their presence should thus be mentioned with the greatest possible precision in the documents that are prepared concerning the site conditions.

In this case, only very complete surveys, including numerous rapid surface-testing methods, are capable of locating these risks on the sites that were initially described to the designer by the preliminary geological studies.

When such obstacles are pinpointed precisely and their significance is well estimated, it is possible to limit construction hazards by stating, in the CCTP, the obligation for the company to use the materials that are the best adapted to the particular conditions of the job. The job may require casings, powerful chopping tools, or rotary drilling methods (drill, coring tools, or rotating drill bit).

3.4.1.5. Karstic soils

Karstic soils pose a double problem in the designing and constructing of boreholes for piles [15].

-- Designing of cost effective and reliable foundations on a karstic site is dependent upon the representativeness of the results of the preliminary survey. It is necessary to recall that there is no precise method capable of giving information about the position, geometry, nor the true extent of the cavities in a formation (except in extreme cases). Sometimes the presence of a karst on a site has been revealed by surface observations (sinkholes, apparent cavities, craters, etc.), by a geological study, or by sounding.

The shortcoming of achieving a good definition of a karstic formation is even more disturbing when structures are modest. On one hand, the designer must minimize the risks and, on other hand, propose foundation solutions that are the most efficient and the least costly. At present, unless the number of borings or soundings is substantial, a sufficient degree of safety cannot be obtained at the planning stage.

Excluding the case of a deliberate choice of a mat foundation, it is during the foundation layout prior to construction that it is desirable to complete the exploration using rapid soundings (wagon-drill for example) at the location of each of the supports of the structure in order to ensure the absence of karst under the anticipated levels of foundation.

It should be noted, however, that in such soils it is preferable, to increase the number of rapid soundings during the preliminary survey in order to obtain the best possible image of the site rather than to research in too much detail the nature and characteristics of the materials that are encountered. Some of the newer methods, such as surface-penetrating radar, may be of value at certain sites.

-- *The problems of constructing* piles on karstic sites essentially consist of losses of slurry and significant overconsumption of concrete.

To offset these difficulties, the solutions, often of variable efficiency, depend on the nature and exact characteristics of the karstic network.

Numerous fissures with moderate dimensions do not require the same arrangements for construction as do significant cavities.

In the presence of a dense network of medium to fine fissures, boring procedures without casings may be used if slurry losses are not excessive. If excessive, it is then necessary to resort to grouting the formation to create a waterproof screen around the piles or the groups of piles. However, costs escalate rapidly and quickly become prohibitive.

In the case of large cavities, it is preferable to use boring methods with casings that have sufficient thickness to be driven (driving is often severe due to the hardness of the rocks generally affected by the karstic phenomena) in the hole created by a chopping tool. Possible overconsumption of concrete may be guarded against either by using a permanent casing, or by prior sealing of the openings in the formation. The sealing operation consists of refilling the encountered caverns with sand-gravel materials compacted with the drilling equipment and continuing with the boring and casing through this reconstituted soil (Fig. 45). One other method consists of injecting the soil with expansive grout in a pilot borehole. This procedure is still considered experimental.

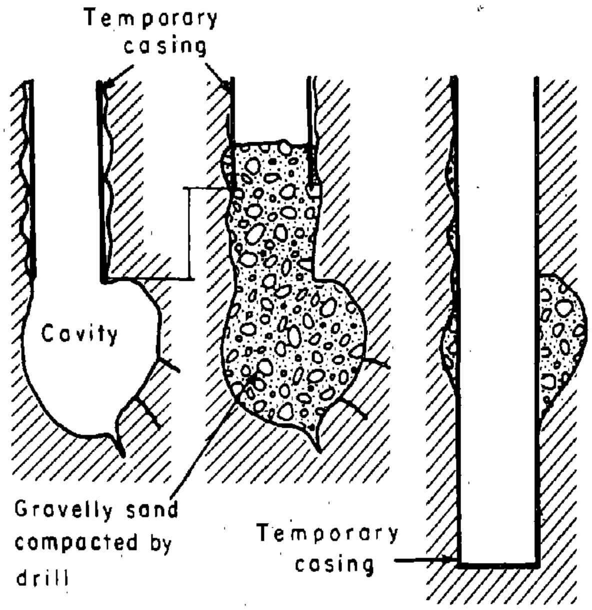


Fig. 45. -- Diagram of principle of refilling of a karst.

3.4.2. Consideration of verticality or inclination

3.4.2.1. Vertical piles

The CPC Specifications (Clause 3.09.41) stipulates that the plumbness of the borehole should not deviate more than 5 mm per meter (0.06 in. per ft). Regular testing of this plumbness should allow for the correction of excessive deviation either by adjusting the casing, or by plumbing the kelly bar. Such corrections cannot be made with precision when the borehole is constructed without casing while using a bucket on a cable, a chopping bucket, or a chopping tool.

3.4.2.2. Battered piles

Generally, the risks of malfunctions and construction difficulties inherent to bored, vertical piles are aggravated when battered piles are involved. Regardless of the procedure used, even with a guide casing, the planned inclination is very difficult to maintain, particularly when long lengths are involved and if drilling by percussion is required. Typically the tolerances for deviation from the planned inclination are the same as vertical piles (5 mm per meter - 0.06 in. per ft).

In particular, the difficulties with socketing the piles into rock as discussed for vertical piles when the substratum is inclined (sealing the casing is often difficult), are complicated by the risk of displacement of the head of the casing which leans against the borehole wall where loose soil can deform and erode.

Finally, it must be noted that the inclination of a borehole may be modified, not only by the presence of obstacles (wreckage, ruins, blocks, etc.), but equally by the dip and strike of the layers of soil and rock. The relative inclination of the boring to the dip of rock influences the breakage of the walls of the borehole (Fig. 46).

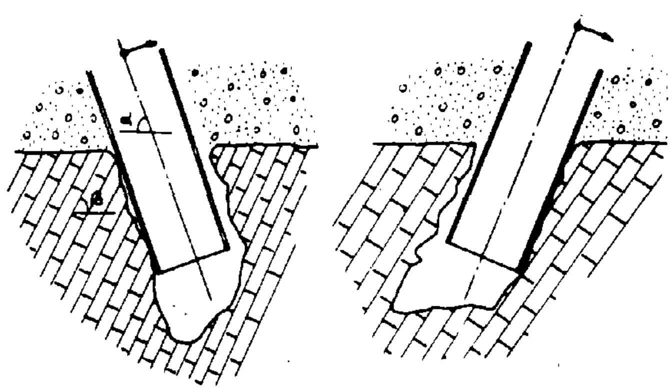


Fig. 46. -- Influence of dip and strike of layers on inclination of piles.

All these difficulties ***demonstrate the risks associated with the use of battered piles*** and support the ***elimination*** of this type of pile from further consideration. The present techniques of vertical boring allow the designer to select a wide variety of diameters or configuration of barrettes to resist the applied loads based on mechanical characteristics of the layers of soil that are penetrated.

3.4.3. Anchorage in the substratum

When the substratum is rock, the difficulty of socketing the piles into the layer to the depth required is dependent on the density and hardness of the rock. To achieve this penetration several procedures may be used: use of tools with stellite, tungsten-carbide teeth, drilling by percussion, or rotary drilling with reverse circulation.

The contractor on a project will have to select the equipment based on the geotechnical studies and the description of the rock quality on one hand, and on the required depth of penetration into the stratum (selected by the designer) on the other hand. The equipment must be able to obtain the specified depth with minimal difficulty and without deliberate modification unless stratigraphic differences between the excavated material and those of the subsurface exploration are found.

In the case of drilling by percussion it is desirable that the energy available for battering be sufficient to penetrate efficiently the rock to a depth of 2 to 3 pile diameters. To this effect, the winch should be capable of ensuring a height of fall sufficient for the chopping tool. Additionally the winch should have sufficient power to lift the carefully selected chopping bit for the diameter of hole to be excavated. This selection is all important because the preferred method of payment is by the excavated depth and not by the equipment hour.

This appropriate selection of equipment for the problem that exists can be detailed down to the shape and dimension of the chopping bits which will avoid jamming of the tool and will avoid a considerable loss of time. Such incidents are frequent when the depth of socketing is larger than the height of the tool. Difficulties are caused when the chopping tool bumps on the wall or when a deviation of the borehole tips the head of the tool, causing the tool to be caught in a crevice or under the casing (Fig. 47). Some guiding devices, such as a casing or a centering guide (Fig. 48 and Fig. 39) will, as a rule, offset these inconveniences.

Finally, it is desirable to guard against hazards inherent to variation of the level of the substratum. When such conditions have been detected by the preliminary geotechnical survey, equipment is selected to handle the worst conditions. In particular, it is important to select kelly bars of sufficient length to attain the greatest depths that are expected to be drilled. This planning effort should avoid changes of

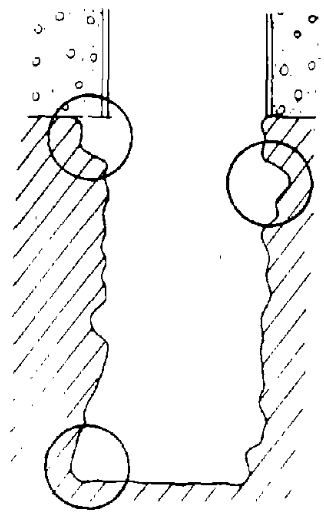


Fig. 47. -- Critical areas related to socketing in a rocky substratum.

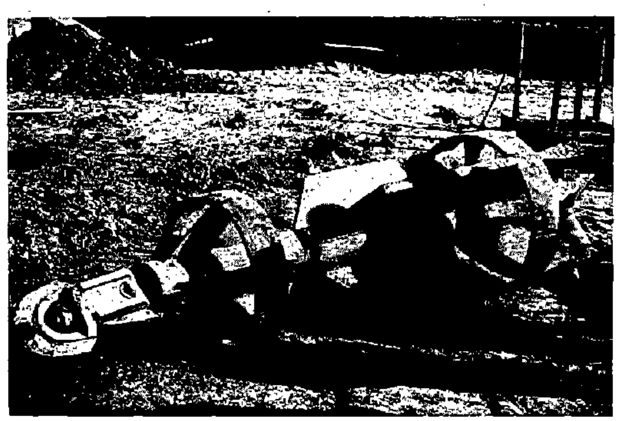


Fig. 48. -- Example of drill with crown guide.

equipment which are always expensive during construction and even more regretable because such changes are often required by minor deficiencies.

3.4.4. Cleaning of the bottom of the hole

Conforming to the special provision of Clause 38 (38.1.1) of Article 68, which is also found in the DTU No. 13-2, the procedure that precedes the start of the concreting is the cleaning-out of the bottom of the borehole.

The cleaning of the bottom of the borehole is an operation performed immediately before concreting; that is, after the placement of the rebar cage and possibly a liner, and should not just be performed in a perfunctory manner at the end of boring. The cleaning of the bottom of the borehole consists of properly eliminating all of the debris located not only at the bottom, but also in suspension in the slurry to ensure concreting under satisfactory conditions, and above all to obtain a good concrete-soil contact, especially at the tip. This cleaning permits not only the shaping of the bottom of the borehole but also the extraction of the remaining soil left after the use of auger-type tools. Finally, the task of the cleaning after installation of the rebar and the liner permits elimination of materials or sediments that have caved in during the delays required for placement of these components. For long piles, these delays can be very significant (several hours). Careful cleaning is very essential to the proper construction of bored piles and it is regretable and dangerous that it is often neglected.

It is necessary to have a clear understanding that improper cleaning always translates to a malfunction at the tip of the pile. It is a mistake to rely on the flushing of the bottom of the hole by the fluid concrete for removing loose material and sediment (see Chapter 6), even in the absence of a basket at the base of the rebar cages, and even if the tremie effectively reaches the bottom of the borehole (Fig. 49). It is easily understood that the cleaning of a dry borehole may be done with no difficulty if the bottom of the borehole is visible; however the cleaning is a completely different matter in the presence of water or of bentonitic mud.

The difficulties of ascertaining whether or not good cleaning has been done and the false assurance that flushing of the fluid concrete provides cleaning are the essential factors which encourage the construction supervisor and the construction company to minimize the sig-

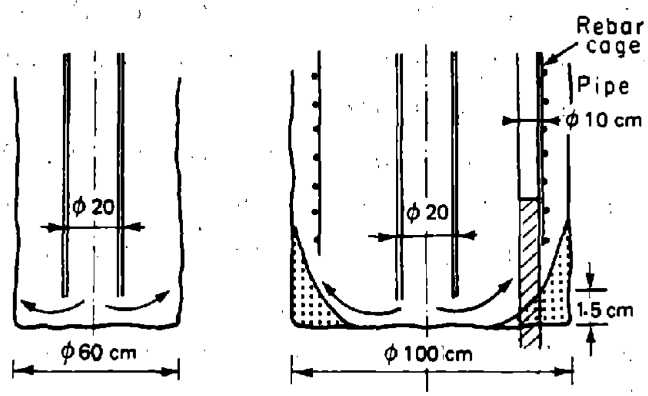


Fig. 49. -- Effect of flushing relative to dimension of piles. (2.540 cm = 1 in.)

nificance of the removal of loose material and to consider it only in the final phase of boring. There is a tendency to use only the equipment which is employed in the construction of the borehole for the cleaning operations and too often suitable methods are not employed. The usual boring procedures do not eliminate the risks of sedimentation of sands during the course of the installation of casings and rebar, nor do such procedures avoid the more or less serious modification of the bottom of the borehole in cohesionless soils. In regard to cohesionless soils, it is particularly evident that the use of any tool that will cause a loosening of the soil at the bottom of the boring is extremely harmful and should be expressly prohibited. A number of examples related to very diverse soils and various techniques of boring (with or without casing, with or without bentonitic-mud circulation) attest to difficulties and numerous defects (Fig. 50) that result from poor cleaning procedures. The problems include the premature halt of the rebar cage by sediments which can attain a depth of several meters when slurry is not properly screened during recirculation or is of poor quality. The presence of sediments in concrete has been detected by geophysical methods or by coring of the pile, and improper conditions at the bottom of the borehole have been found by tests performed under the tip of the pile after its construction.

In all the above cases, the boreholes had been cleaned with the aid of available equipment and procedures on the job site (bailer, grab bucket, pumping and injection of slurry at the top of the borehole, direct

Fig. 50. -- Example of cores from bottom of pile, showing a poor contact between concrete and rock (because of a 40 cm of sand layer).

circulation). Sometimes the cleaning had been stopped, based on insufficient criteria such as the absence of materials raised by the bailer or other tools, or the presence of less than 2% sand ≤ 80 µ in the regenerated mud, a condition that is required for good uniformity of the wall but insufficient in the light of the risks of sedimentation of larger sands, 5 mm (0.20 in.) to 80 µ for example. On the other hand, incidents of improper cleaning are rare (of course, disregarding the risks of cave-in) when the boreholes are constructed under reverse circulation of bentonitic mud. Reverse circulation with slurry is, in effect, the only method that allows the efficient raising of sediments deposited at the bottom of the borehole and the complete elimination of those sediments without further interruption of equipment operations for casings and rebars.

Such a cleaning can continue for as long as necessary (until the fluid in the hole is regenerated if it involves mud or completely cleaned if it involves water) and no interruption is necessary except at the precise instant of concreting.

The possibility of establishing reverse circulation at the end of boring is not, in any way, the sole option of a particular type of boring technique. In effect, regardless of the boring procedure used, it is always possible to establish a reverse circulation by the use of a tube lowered to the bottom or by a series of special pipes. The tube is either connected at the top of the boring to a vacuum pump (Fig. 51 and Fig. 52), or fitted with a device for injection of compressed air near the bottom of the tube (Fig. 53), or is connected to a submerged pump. This relatively simple equipment, which does not constitute a significant constraint, should be required by the CCTP on all job sites with bored piles. At stake is the quality of the contact between the *concrete and the soil at the base of the pile*, which for most piles is an important element in the design capacity of the foundations and certainly justifies the additional expenses which may result.

3.4.5. Required properties of the slurry

3.4.5.1. During boring

In addition to an ability to keep the wall of a boring from caving (and, if the case arises, to raise sediments) during the entire drilling operation, the slurry should retain the qualities required to limit wear on the equipment (especially pumps).

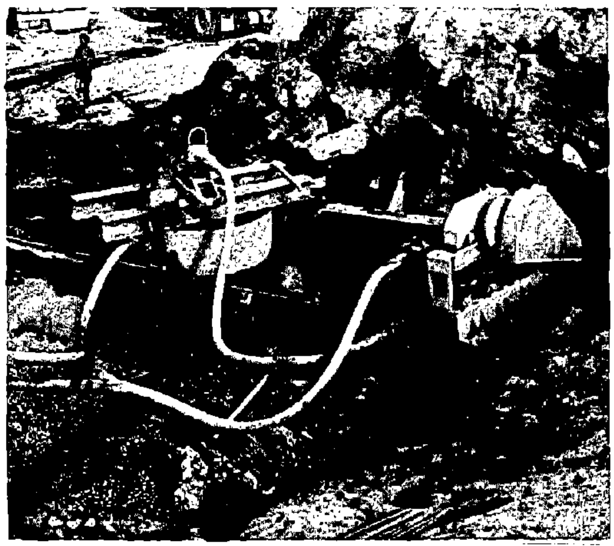


Fig. 51. -- Cleaning of bottom of borehole by circulation of mud using tremie.

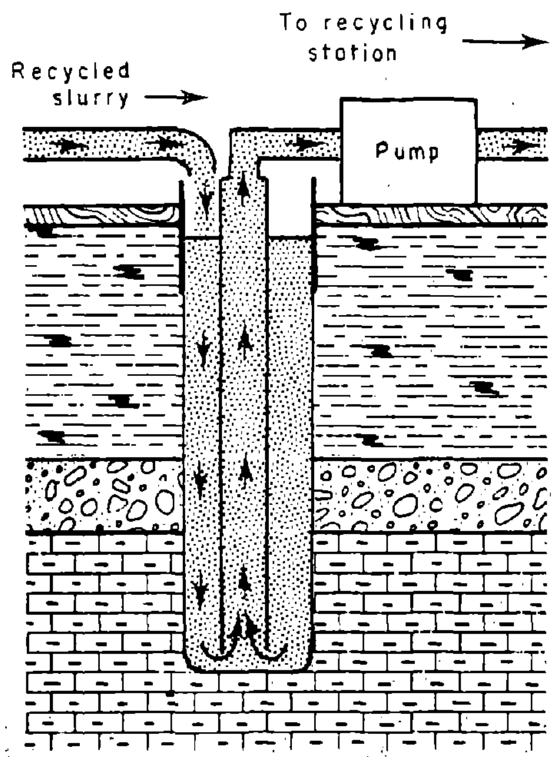


Fig. 52. -- Principle of cleaning of bottom of borehole by circulation of recycled mud.

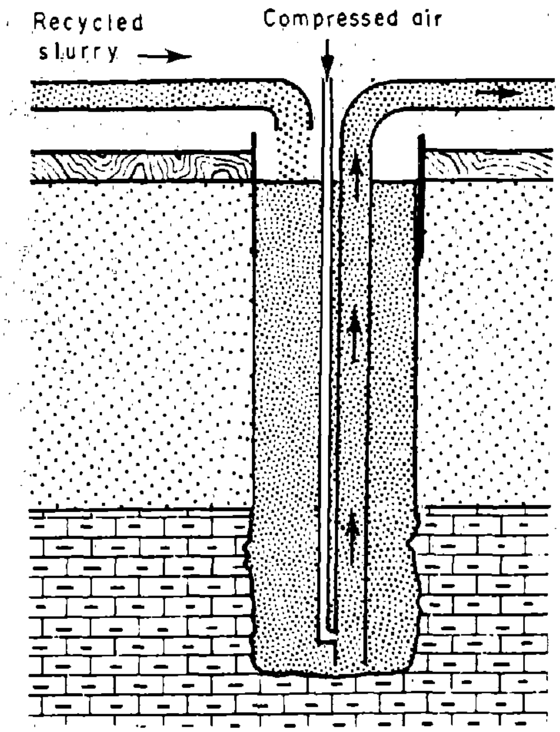


Fig. 53. -- Principle of cleaning of bottom of borehole by use of compressed air.

To fulfill these conditions:

-- the filter cake should be both resistant and thin in order to ensure stability of the wall without being damaged by the drilling tools. For exploratory boreholes (especially for mining and oilwell drilling), the depth of the filter cake measured in the pressed filter test (see TN No. 9) should not exceed 0.5 to 1 mm (0.02 to 0.04 in.) (with a filtrate of 5 to 10 cm^3 - 0.30 to 0.61 in^3) so that the movement of the drilling tools is not impeded. As far as bored piles are concerned, cake depths of 2 to 4 mm (0.8 to 0.16 in.) filtrates of 20 to 30 cm^3 (1.22 to 1.83 in^3) are allowed, these values correspond to a fresh, regenerated mud. It is necessary to recognize that cave-ins may be due to an overly thin filter cake when the mud is fresh (low viscosity) as well as an overly thick cake (contaminated mud, too high free-water content);

-- inevitable contamination of the mud by the materials that are drilled (inert products such as sand, or chemically active clay products) should be combatted by desanding and efficient recycling by a facility specially designed for these purposes. Installation of equipment for treating the slurry should be required by the CCTP (see TN No. 9). The regeneration of the slurry should further be performed in a continuous way and not just when the boring is complete. At the end of the boring the slurry may have a density that is too high, especially if loose soils have been penetrated. The late substitution of fresh mud for the contaminated mud promotes cave-ins. Unless a treatment facility is employed, the slurry is loaded with sand and loses its initial properties; there is an increase of free water (> 40 cm^3 - 2.44 in^3) and a thickening of the filter cake (> 10 mm - 0.39 in.). These inferior qualities of the slurry are detrimental to a uniform condition of the wall of the boring, can cause excess wear of the pump, and will result in a sedimentation that interferes with the proper cleaning of the bottom of the hole;

-- the viscosity of the slurry should be maintained as constant as possible, without abnormal increase due to physical or chemical contamination that may have serious consequences in the construction operations, particularily as related to difficulties in pumping;

-- flocculation should be anticipated, especially in the presence of saline water, gypsum or cement which gives the mud a pH > 10;

-- the density of the slurry, the increase of which is a sign of an elevation of the sand content, should be closely monitored. However, the properties of the slurry should be modified in relation to the hydrogeological conditions of the soils that are drilled. The density of the slurry should be increased by addition of appropriate products (see § 3.5.9.2) to limit risks of intrusion of water under a head (artesian) in the borehole;

-- finally, to reduce eventual mud losses, sealers may be added such as Guar gum, polymer resins, or solid substances. The solid substances can be reused following separation and drying and may consist of wood fibers, sawdust, expansive perlite, powder, or mica flakes, etc.

3.4.5.2. At the end of drilling

Reverse circulation of the regenerated slurry, between the end of boring and the beginning of concreting (especially after placement of rebar and possible linings), constitutes, without doubt, the most efficient method of cleaning the bottom of the hole. Thus, reverse circulation is especially recommended in order to offset the risk of poor concrete-soil contact at the tip (see § 3.4.4).

3.4.6. Treatment of slurry

With Cambefort [8], treatment of physical contamination is distinguished from treatment of chemical contamination.

3.4.6.1. Treatment of physical contamination

This involves reconditioning of the slurry that was contaminated during boring by sands, limes, or clays which increase the density of the slurry as well as its free-water content and the thickness of the filter cake.

The sand eliminates itself by settling or is eliminated even better by circulating the slurry through a system composed of vibrating sieves and centrifugal separators (see TN No. 9).

In certain cases, the slurry may be treated with additives which include:

-- organic colloids (soda alginate, extract of marine algae, carboxymethylcellulose or CMC, starch) which *strongly reduces the free-water content* (thus thinning the cake and enhancing its resistance to contamination) and *increasing more or less the viscosity*;

-- additives, such as tanins (especially quebracho), polyphosphates (pyro, tetra, and hemaxetaphosphates) which *especially diminishes the viscosity*, also lignosulphates which equally act as *filtrate reducers*;

-- *anti-hydrating agents*, such as potassium lignosulphate which is especially effective with plastic and expansive clays;

-- pyrophosphate acid which *lowers the pH* of the slurry. This additive is especially interesting for boring certain expansive marls in which hydration, which occurs when the waters are basic (pH > 11), can be limited by maintaining the pH value between 7.5 and 8. In this respect, experience also shows that in practice marls do not destratify beyond a certain concentration of sodium silicate or in the presence of lime slurry.

3.4.6.2. Treatment of chemical contamination

The treatments described below are applied in case of normal contamination. If the contamination is very significant, it may be preferable to replace completely the polluted mud, or to use inert muds (of oil, of starch or of silicate, for example) even though these techniques are normally employed only for deep boreholes (especially for drilling oilwells) and are rarely justified in civil engineering.

-- *Contamination by sodium chloride.* A dilution of the slurry permits maintenance of the uniformity of NaCl around 1% and the addition of phosphates or tanins may be suitable, but decrease the viscosity. It may then be preferable to use a slurry of starch (or of CMC for concentrations of NaCl lower than 1%).

-- *Contamination by gypsum or anhydrite.* For a weak contamination, the addition of tanins, phosphates, or CMC can suffice. For significant contamination, calcium, in the form of sulfate, may be precipated with the aid of carbonate of soda, dissodic phosphate, or barium carbonate.

-- *Contamination by cement.* As in the preceding, tanins, phosphates, or CMC may first be tried, but to lower significantly the pH it is necessary to use monosodium phosphate, complex phosphates, quebracho, or more simply bicarbonate of soda.

3.5. TESTS OF CONSTRUCTION OF THE BOREHOLE

The construction conditions previously described, on which the essential qualities of a good borehole depend, cannot be met unless regular and careful testing is assured by available personnel who are instructed as to the particularities and difficulties inherent to the construction of piles in place. Such testing is indispensible regardless of the reputation of the company, their references, or their possible experience in such construction.

To avoid all ambiguity with respect to testing during the progress of the work, it is desirable in the CCTP to anticipate and to define with precision the nature and modes of the construction tests. This does not just involve the keeping of an account of the depth drilled and the time spent, as is unfortunately too often the case because of the insufficient knowledge of the inspector or the number of his duties (he may have responsibilities at several job sites). The appropriate kind of inspection requires a regular testing of high quality, achieved through the rigorous establishment of a suitable schedule. In Appendix A, a model is proposed to effectuate a testing progam and has already been referred to in Chapter 2 (§ 2.4.4).

Such testing naturally fits into the group of verifications in which emphasis has been placed on the significance of the level of the organization and the construction methods employed at the job site (see Chapter 2). Furthermore, Chapter 2, which partially restates certain paragraphs of Chapter 8 of the Pilot Guide GMO 70, has already established a chronology of fundamental tests that may be required during various operations that govern the construction of a bored pile.

More specifically, this section involves the recommendations of the principal tests specific to construction of the borehole and the proposal of methods, even though they are very empirical, that may be placed at the disposal of the inspector on the job.

3.5.1. Location of top of pile

The proposed tolerances for the location of the top of the pile generally vary between 5 and 10 centimeters (2 and 4 in.). Such values may easily be followed for vertical piles. But the position of the top of a battered pile is not so easy to control, as as described in 3.4.2, because there is a tendency for the top to displace if drilling is difficult and if the subsurface soils are nonuniform. The desirability of placing several stakes that will assist in locating the top of a pile, such as is recommended in § 2.2.2 may prove to be insufficient when the work space is restricted, is cluttered with equipment, or is not clean and orderly.

When the conditions of the job site so require, there should be no hesitation to locate the pile head more effectively using guides of weak concrete with cylindrical holes of the diameter of the boreholes to be constructed (Fig. 54). Such devices, that also contribute to the initial guiding of the surface casing (which they do not replace) or of the casings, are especially valuable for piles of small diameter which are closely spaced.

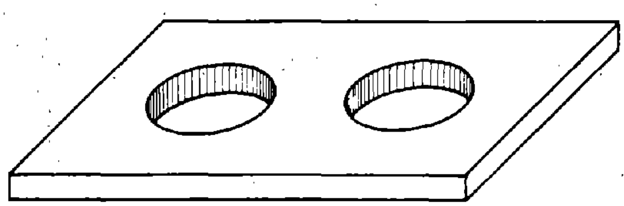


Fig. 54. -- Hole-guide for piles of small diameter.

3.5.2. Verticality or inclination

As guidelines only, the CPA Standard (3.09.41) mentions that the defects in verticality or inclination should not exceed an average of 5 mm per meter (0.06 in. per ft) over the entire length of the pile.

It is very evident that the method of testing the direction of a borehole are variable and must be related to the methods of construction (see § 3.1 to 3.3) and to the geometry of the pile. This tolerance is adopted for every type of pile regardless to its construction method.

3.5.2.1. Verticality

The verticality of bored piles with casings can be verified with satisfactory precision using a plumb-line that is lowered to various levels while being held at the same position at the groundline.

For piles that are constructed dry, it is possible to light the boreholes with electric lamps, mirrors or simply with torches made of paper or other material and proceed with the same checks as if a casing had been used.

On the other hand, the testing of the verticality of piles constructed under bentonitic slurry, especially without casing, is much more difficult. In such a case, however, an estimation of the defects in verticality is possible by observing the verticality of the kelly when the borehole is constructed by rotation (auger, auger-bucket, corer, rotating drill) or using a grab bucket. However, such observations become impractical when tools on cables are used (cable buckets, hammergrabs, percussion drills).

3.5.2.2. Inclination

The difficulties described concerning the testing of the verticality of piles are even more severe when testing the inclination of piles and, as before, only piles constructed in the dry or with casings allow accurate verification. Inclined or battered piles are most often constructed with a temporary casing in order to control the direction of the drilling. Thus, paradoxically, the testing of the location and inclination of these piles is made more frequently than for vertical piles. However, the ability to make accurate tests of the location of inclined boreholes is influenced by the relative dimensions of the piles and their inclination. The depths that may be tested using a plumb line are reduced as the diameter of the pile becomes smaller and as the inclination of the borehole becomes greater. The ability to test the inclination of slender, very inclined piles thus remains very slight.

3.5.3. Stratigraphy and nature of soils

The CPS Standard calls for representative sampling from each of the geological layers that is penetrated or at least a sample every 3 m (9.8 ft) during boring and one every 0.50 m (1.6 ft) in the final 2 meters (6.6 ft). These samples are classified, labeled, and stored in boxes with compartments and allow the establishment of the geologic section that appears on the boring logs. The geologic section is then compared with the results of the preliminary surveys that were made. The correspondences or differences noted in this geologic section should then lead to confirmation of the depths of the piles or, on the contrary, to a modification of initial design.

Such verifications should be followed in a rigorous and systematic way but it is necessary to recognize that the most currently used boring procedures, because they are so efficient, rarely allow for extensive sampling of subsurface soils. It can be assumed that rock formations can be accurately identified when drilling the borehole. However, difficulties in identification are presented when soils are drilled. The drilling modifies the soil properties, especially the consistency, so that the properties of the cuttings have only a distant relationship to the in situ properties as determined by a careful subsurface investigation. This difference is especially important in the case of rotary borings with bentonitic slurry. However, regardless of the procedure used, it would be useless to propose the verification of the mechanical qualities of soils with tests on the cuttings from boreholes. Unfortunately, *in situ* tests that are presently used do not permit the testing of the mechanical characteristics of soils at the bottom of the borehole. There are two problems with in situ tests at the bottom of a borehole. The first problem concerns the placement of the testing device (pressuremeter, penetrometer, sampler) when the borehole is deep. Secondly, the tests are for the most part strongly empirical so that the results are imprecise (tests in a bucket for example, described in 8.95 of the Job Site Guide for the Construction Supervisor [3]).

Conscious of these difficulties, the Laboratories of Bridges and Roads is currently studying the possibility of using specific equipment allowing easy and rapid placement, with capabilities of testing in place the mechanical characteristics of the wall and of the bottom of the borehole. The equipment being considered is a multi-directional penetrometer which will give the operator an indication of the remolding of the soils at the bottom of the hole compared to the sidewalls.

However, since such equipment is not yet in operation, comparison between the soils encountered and those recognized at the time of the geotechnical investigations cannot practically be based on more than an often uncertain visual examination. It is recommended that a specialist be requested to make the visual examination.

Since the permanent presence of a geotechnical engineer is rarely considered, except on very important jobs, the usefulness of the visual examination and of traditional in situ tests turns out to be seriously hampered by the delays that result. If the decision to stop the boring is subordinated to the obtaining of results of such testing, the risk of permanent damage to the pile can then be more serious than the risks avoided. In this case, ***it is then generally more valuable to promptly make a decision that can be critized than to make a better one, eight hours later*** [3].

To aid in the decision about the continuation of a boring it is, however, necessary to have the maximum amount of directly useful information. To this effect, it is desirable to complete the construction report in detail and in a timely manner covering all the parameters concerning the boring (speed of advancing the borehole, speed of rotation, forces on the tool, weight and height of fall of percussion tool, etc.). Also, it is desirable to report, as a function of depth, all observations relative to soil (losses of water or mud, fall of tools, difficulties of penetration, obstacles, color and nature of sediments, etc.) and to report information relative to equipment use (modification or replacement of tools, breakdowns, deteriorations, etc.).

The scrupulous obtaining of all of this information is especially important for the first piles on a job because the construction procedures of the subsequent piles can be adjusted to meet the site conditions if uniformity is indicated by the subsurface investigation. Such information is obviously all the more indispensible on a heterogeneous site where data should be acquired with the same meticulousness for all the piles.

Taking into account the difficulties and the lack of precision which are associated with obtaining the geotechnical characteristics of the subsurface soils as they are being excavated, the decisions to be made during the job will be easier if the initial subsurface investigation has been sufficient and suitably adapted to the site and to the project. In this connection a project that is developed from appropriate geotechnical studies should limit the number of modifications that are needed as the construction progresses, particularly to the base elevation. The soil properties at the base elevation can generally be obtained with better confidence from in situ methods as compared with methods based on results obtained from the testing of cuttings.

3.5.4. Water samples

The possible harmful effects of any subsurface conditions on the concrete in the piles should have been investigated at the time of the initial subsurface studies and the selection of an appropriate concrete mix should have been made as a result.

However, it is recommended that verification be made at the time of the boring of the first pile of whether the ground water is corrosive (pure or saline) or not. To this effect, water samples are taken at different levels, using sampling bottles that are specially designed. Of course, this test is not possible in the case of boring under bentonitic slurry.

3.5.5. Water circulation

-- *Horizontal circulation*, that may invalidate the boring procedure initially selected (see § 3.4.1.1) or that at least may require casing of the piles (see § 3.4.1.3 and § 4.2.1.2), is quite difficult to detect at the time of the geotechnical survey because such circulation generally prevails within cohesionless formations that must be cased to proceed with the boring. If it is thought that horizontal flow of water is encountered, an open and permeable casing can be set in the formation and an effort can be made to measure the speed of the current with the aid of a micro-current meter. The significance of such a test is, however, debatable due to the perturbation of the current created by the casing, even though the casing has been made as permeable as possible.

However, when the probability of such a horizontal flow of water has been revealed by a hydrologic study, it is desirable at the time of the construction of the first pile to confirm or invalidate the presence of the flow by means of a micro-current meter, unless of course, the water circulation becomes apparent because of cave-ins during boring. However, here again, the test with the micro-current meter is difficult or nearly impossible when the piles are constructed using a casing. If the probability of circulation is strong and if a casing is used in construction, it is desirable to reduce the risk and to plan to line the borehole through this zone or to the bottom of the stratum containing the flowing water.

-- *Vertical circulation* is very easy to detect at the time of the geotechnical survey and, except in the case of artesian water layers which are powerful and evident, it is generally easier to neutralize the vertical flow by maintaining during all the phases of construction of the piles a sufficiently high level of water (or of slurry) in the boreholes.

3.5.6. Extent and level of a cave-in

A cave-in during boring, or worse between boring and concreting, is among the most important incidents that may affect a pile constructed without casing. Therefore, it is not sufficient simply to mention cave-ins on the inspector's log but to proceed immediately to measure and examine cave-ins so as to estimate their extent and the nature of the risks that are involved. Such data allow rapid decisions to be made pertaining to measures to be taken, which can go as far as a complete modification of the boring procedure.

A lack of timely decisions for solutions to be adopted, usually due to excessive optimism, often result in taking measures that do not work. Once the pile is finished, it will be necessary to proceed with expensive repairs, possibly to the complete replacement of the pile.

In the case of dry boring, a visual examination may instantly indicate the seriousness of the cave-in, but the information is more difficult to obtain in the case of boreholes underwater and all the more so with bentonitic mud.

Cave-ins are generally detected when the tool is unable to reach the level that it had previously attained. At that point there must be a useful determination of the extent of the cave-in by measures that indicate accurately the depth and diameter of the enlargement. A borehole caliper can be used. Such a device can be used to obtain data in the zones revealed by the preliminary surveys or during boring as more vulnerable. When bentonitic slurry is being used in the borehole, tests of slurry quality may contribute to the detection of cave-ins, but on the other hand, if the cave-ins have been revealed by other signs, it is necessary to require tests of the quality of the slurry (density, viscosity, sand content), before and after treatment. The remedial measure may consist of either thickening the mud by the addition of bentonite or the addition of barite, for example, or by the use of a provisional casing.

3.5.7. Presence of karsts

On a site where there is suspicion of the presence of karsts (cavities) at the time of the geotechnical study, it is obviously desirable to increase the number of soundings or boreholes in order to localize the cavities and to estimate their extent in order to define the most appropriate techniques of construction (see § 3.4.1.5). However, the very random character of karstic phenomena and the absence of effective detection techniques, outside of borings from which the information always remains limited despite the number, create a substantial probability of an unforeseeable encounter of karsts during work.

It is thus desirable, in such conditions, to be especially attentive to all signs which may indicate the presence of cavities or fissures during boring and particularly to watch for freefalls, for increases of the speed of advancement of the tool or the casing, for decreases in the force on the drilling tool, and for losses of boring liquid (water or mud). If karsts escape this vigilance during boring, significant overpour of concrete may result but the foundation is rarely placed in peril if the loss of concrete is in a zone above where the pile gets its support. On the contrary, the danger of a defective foundation is considerably aggravated if the probability of the loss of concrete extends to the base of the piles.

There, the necessity exists on a karstic site to ensure systematically the absence of cavity under the level of foundation of piles by the use of a wagon drill to drill a probe hole at the bottom of each borehole.

3.5.8. Diameter of the borehole

The measurement of the diameter of the borehole allows the detection of possible shrinkage due, for example, to the creeping of soft layers, which may go unnoticed if the creep is not sufficient to prevent the descent of the rebar cages.

Also, such a test permits the measurement of enlargements in the boreholes due to cave-ins or cavities. With such data the cause and position of the possible subsequent overpour of concrete are known and judgment can be used to deal with such an overpour in an appropriate fashion if the case arises.

In order to proceed with the testing, calipers of different types are used. Of these, some are being improved by the LPC so that it will be possible to estimate the amount of remolding of the walls and the bottom of the borehole (see § 3.5.3).

3.5.9. Quality of the slurry

In order that the slurry will possess the required properties for the functions desired, the slurry should be tested using instruments and methods described in Technical Note No. 9. The slurry should have the following characteristics:

3.5.9.1. Free water and thickness of the filter cake

Even under circulation, slurry with a high free-water content always creates a thick filter cake which may be destroyed by the drilling tools. Further, subsurface clays that absorb excess free water may disintegrate and be improperly stabilized by the cake.

In order that the thickness of the cake be of the *order of 2 to 4 mm (0.08 to 0.16 in.), the quantity of free water*(or filtrate) determined by standard tests should be *from 20 to 30 cm^3 (1.22 to 1.83 in^3)*.

3.5.9.2. Density

The normal specific gravity of a fresh-mixed slurry is around 1.03 but may reach 1.05 through the simple concentration of bentonite. However, the increase in viscosity which therein results renders the slurry difficult to pump. In order to make the slurry heavier, when the conditions of the borehole so require (intrusion of water under pressure for example), additives may be used such as barite, hematite, pyrite, siderite, or galenite, which permit attainment of desirable specific gravities up to or even above 2.

3.5.9.3. Sand content

Generally speaking, sands are the group of elements above 80 μ which are contained in the slurry. In order that the slurry may retain its initial properties and so that it does not cause abnormal wear of the pumps, on one hand, and to limit sedimentation at the bottom of the borehole, on the other hand, *the sand content should always be lower than 2% at the end of the treatment of the slurry.*

3.5.9.4. Viscosity

The viscosity should be sufficient to retard or prevent the sedimentation of inert particles and to ensure the proper thickness of the filter cake adjacent to impermeable layers (a property which is linked to the pH). However, the slurry should be weak enough to allow good separation of sand raised by the slurry and weak enough so as not to cause problems in pumping the slurry.

One hour after its mixing, a good slurry should have a *Marsh viscosity around 40 seconds*, or 30 centipoises for a nozzle diameter of 6/32 inch. For information only, the Marsh viscosity of clear water is around 26 seconds.

3.5.9.5. pH

The pH of fresh mud is from 7 to 9.5. Above this, flocculation may result due to changes in the free water and in the viscosity. The measure of pH permits detection of contamination of the slurry by subsurface soils, or by the water which they contain (gypsum formations, saline water, etc.).

3.5.9.6. Thixotropy

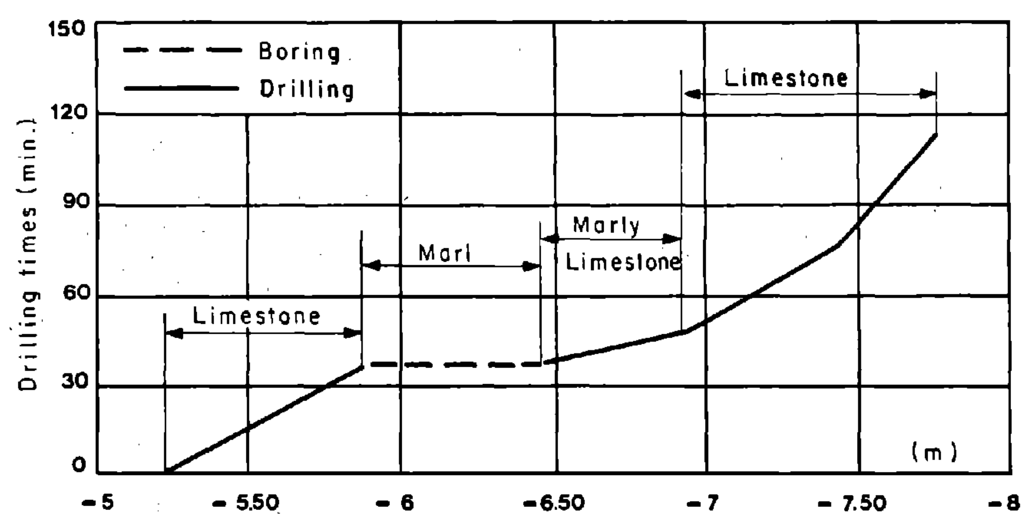
This property of the slurry to turn into a gel when not in movement limits sedimentation at the bottom of the borehole. However, the gelling should be sufficiently limited to permit, without difficulty, the proper functioning of the pumps. Thixotropy is a complex phenomenon which depends on the constituents in the slurry and is difficult to dissociate from the viscosity. (Measures of thixotropy are made most often by measures of viscosity after agitation.) Generally, it may be stated that a slurry with an appropriate viscosity satisfactorily counteracts sedimentation during shutdowns in drilling.

3.5.10. Closure of the casing

When operating with a casing, it is known (§ 3.4.1.3) that difficulties in sealing the bottom of the casing in a rocky substratum may cause a cave-in in the borehole of cohesionless soils that may occur just above the rock. Such an incident should be detected by a detailed test of cuttings at the time of the cleaning out of the bottom of the borehole. If the examination of the cuttings demonstrates that the cohesionless soil is mixed with the rock cuttings, it is certain that the "sealing" of the casing is not assured. If successive cleanings verify that the cave-in of the cohesionless material continues after the end of boring, it is then necessary to use measures proposed in § 3.4.1.3 in order to avoid pollution of the base of the pile and to avoid inclusions in the shaft at the time of concreting.

3.5.11. Making a socket in the substratum

It was noted (§ 2.4.4) that, conforming to the terms of the CPS Standard (3.09.47), the use of a rock-boring tool must be authorized by the construction supervisor. Recommendation has been made that their payment be made by the depth rather than by the hour. Given the necessity, at times debatable, of using the rock-breaking tool [3] and its very high cost (in which the unit price should be shown in the contract but quantities obviously canot be given), it is necessary to collect data on the drilling into rock in an extremely rigorous way. In this regard, the information represented by the schedule of construction proposed in Appendix A can practically be completed using diagrams such as that shown in Figure 55 [2].



Depth (m)	0	0.65	1.25	1.70	2.20	2.55
Time increments (hours)	0.65		0.45	0.85		
Time accumulated (hours)	0.65		1.10	1.95		

Fig. 55. -- Example of drilling record corresponding to socketing of a pile in a rocky substratum.

3.5.12. Cleanliness of the bottom of the borehole

Insistence has been made, in Section 3.4.4, on the major importance of the cleaning of the bottom of the borehole because the quality of the cleaning directly affects the quality of the concrete-soil contact at the tip of the pile. The techniques best adapted to the obtaining of satisfactory results (reverse circulation) have been indicated when operating with water or with bentonitic slurry.

It is highly recommended that meticulous care be taken while checking the cleanliness of the bottom of the borehole. This test generally involves the measurement of the depth of the borehole with a weight at the end of a calibrated line. There is hope that, in the near future, the measurement of the depth of a borehole to ascertain the condition of the bottom may be done by static-penetration tests with the aid of devices that are currently under evaluation (§ 3.5.3 and 3.5.8).

In the case of dry boring,the checking of the condition of the base of the borehole is of course facilitated by the opportunity to examine the bottom of the excavation either by descent (cased boreholes of sufficient diameter), or by lighting from the surface by some means (lamp, mirrors, torches, etc.).

Operations involved with possible caving, of placing of rebar and of concreting, cannot be done properly unless the level of the bottom of the excavation corresponds very precisely to the maximum level reached by the tool. Cleaning is resumed as long as this condition is not met.

Measurement is repeated immediately after each cleaning, and after a pause in order to estimate the significance of sedimentation or possible cave-ins. In this way pulling up of the casing and rebar cages is avoided if they have been placed in the borehole prematurely and if the tools and methods of cleaning are not compatible with the presence of this equipment in the borehole. However, despite these precautions, cave-ins can occur during or after the preparations for the concrete pour. The cave-ins can be a direct result of the placement of the temporary casing or the rebar cages. Thus, it is imperative to also proceed to test for the cleanliness of the bottom of the borehole after installation of the rebar cage and above all immediately before concreting.

CHAPTER 4

CASING AND LINERS

The installation of a casing or a liner provides a protective barrier between the concrete and the soil. The borehole-support member is:

-- rigid for a casing (about 10 mm - 0.40 in. thick),

-- flexible or semi-rigid for a liner (about of 1 to 2 mm - 0.04 to 0.08 in. thick).

An envelope called a sheath or liner is usually placed in the borehole before or along with the reinforcement cage and prior to concreting of the pile.

In certain cases, the temporary casing is voluntarily left in place as a permanent liner. This non-retrievable permanent casing is usually of a standard thickness of 7 to 15 mm (0.28 to 0.60 in.) which is between that of a retrievable temporary casing and that of traditional liners. A permanent casing is also known as a "lost-pipe" casing.

Generally, the installation of a casing or a liner is an operation that is costly and requires care and one that should relate to specific objectives. The need for borehole support should thus be estimated in the planning stage and the characteristics of the protective barrier should be carefully specified in the special provisions (details should be given about the nature, thickness, diameter, and length of the system or systems to be used for borehole support). When information about the casing or liners is omitted, the construction company could object to the placing of such a soil-restraining device and this will probably lead to a claim. Objections relate to the possible slowing of the construction progress and also to the fact that the placement of a protective barrier always requires some improvisations.

For these reasons, casing and liners should be used advisedly and to satisfy the particular requirements examined in this chapter.

4.1. DIFFERENT TYPES OF CASINGS, SHEATHS, AND LINERS

These are classified in three categories, according to thickness, composition, and purpose.

4.1.1. Temporary casing and permanent (lost-pipe) casing

These are composed of large, metal, welded cylinders (Fig. 56) with their thickness depending upon:

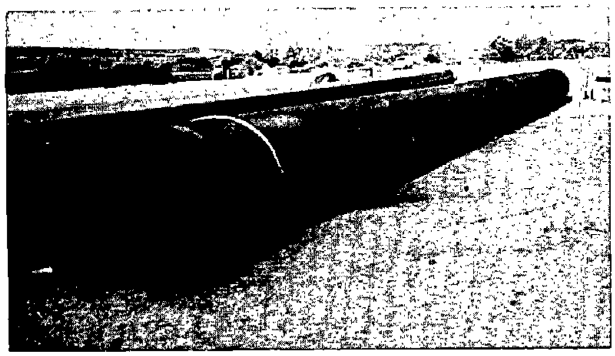


Fig. 56. -- Rigid, metal casing: diameter 1 m, thickness 1 cm. (0.0254 m = 1 in.; 2.540 cm = 1 in.)

-- their diameter and the soils to be penetrated, in the case of lost-pipe casings,

-- function to be fulfilled.

From an economic point of view, it is necessary to understand that recourse to casing can lead to the actual doubling of the cost of a pile. The cost for a linear meter of a pile with a diameter of 1.2 m (3.9 ft) may be estimated as 1000 F, while the cost of a casing with a thickness of 10 mm (0.40 in.) for the same pile is 900 F (cost before tax 1977).

In regard to another important point, it is necessary to consider that the use of permanent casing results in a reduction of skin friction for the areas in which the casing is situated (see [1], Chapter 5.2, Dossier FOND 72).

4.1.2. Semi-rigid liners

These are generally made of corrugated sheet metal (Fig. 57) of thicknesses equal to or less than 1.5 mm (0.06 in.), reinforced or unreinforced by corrugations which double or triple their resistance to crushing (such as Cofratol Casing, for example). In special circumstances they are made of smooth, welded sheets. Placed with or without the protection of a temporary casing, their exterior diameter is *about 10 cm (4.0 in.) less* than the diameter of the temporary casing or the diameter of the borehole. The space thus created between the lining and the soil,

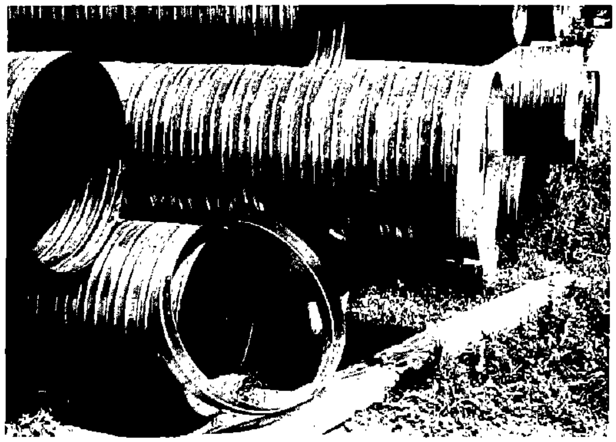


Fig. 57. -- Semi-rigid, Cofratal type.

if not filled in, creates a reduction, indeed a cancellation of the capacity of the pile in skin friction and in lateral resistance.

When the designed function of the liner is not being negated (avoidance of pile-soil interaction for negative skin friction for example), a desirable procedure may be either to inject the annular space with grout or to fill it with sand. This operation should be detailed in the special provisions. The purchasing of these liners may be estimated on the basis of 3.50 F per kilo of steel, before tax (1977 price).

Practically all appropriate sizes of semi-rigid liners are available for piles of circular bore. Certain special shapes are also available such as for oblong barrettes (Spiroval from Davum, for example, Fig. 58).

In the case of possible negative skin friction along one part of the shaft, the liners can be coated at the factory or on the site with bituminous products (see § 4.3.3). Although coatings applied at the factory may definitely prove to be more homogeneous and more uniform, it may undergo deterioration during transportation and handling and often requires repairs on the job site.

4.1.3. Flexible liners (Figs. 59 and 60)

Recently available flexible liners may be made of PVC plastic film or sheets (OTT patent...), of synthetic, fine-squared, wire mesh (Texac...), of polyester felt (Bidim...), of a rubber-coated membrane (Staff patent...), or of a more rigid, plastic membrane (Salvay...).

Fig. 58. -- Bituminous-coated liner for barrette piles.

The cost of these flexible linings is fairly low (3 to 10 F per square meter), and they may be quickly manufactured on the job site. They limit overconsumption of concrete and when flexible ensure a better soil-pile contact by following the form of the borehole (the relative deformation can attain 50%).

Finally, certain permeable materials favor the drainage of soils distributed around the pile, thus permitting a faster re-consolidation of the soil. However, since the value of skin friction that can be mobilized is not well understood, it is rarely taken into account in the calculations.

Furthermore, the fragility of this type of lining requires special precautions during construction, especially upon withdrawal of the temporary casing if one is present. In this case, the rocking back and forth to extract the casing is not used as it may lead to rupture of the lining.

4.2. APPROPRIATENESS AND CHOICE OF LINERS (OR CASING)

The decision to use a liner (or to case) should be made at the planning stage. In this way, the suitable type of liner may be adopted and recommendation may be made for the use of a suitable method of excavation or drilling. Despite all efforts, liners or casings may be needed unexpectedly because of construction problems that the preliminary survey did not indicate (horizontal flow of water, for example, or unforeseen caving). At that time it is essential to examine the effect of the device used:

Fig. 59. -- Flexible liner, OTT type.

Fig. 60. -- Flexible liner, Bidim type.

-- on the design of the foundation and particularly on the possible loss of skin friction (decrease of supporting force) and on the possible loss of lateral resistance (reduction of resistance to horizontal forces);

-- on the compatibility of the respective diameters of reinforcing cages, of liners, and, if the case arises, of temporary casing (see § 5.2.4).

4.2.1. Approaches associated with solution of specific problems

This section involves problems which should be perceived at the time of the soil study. Interference forces, flow of water, risks of caving, and presence of karsts sometimes lead to the possibility of the use of casing or liners.

4.2.1.1. Interference forces (negative skin friction, lateral pressure)

Such phenomena occur when piles that support abutments or retaining walls must penetrate compressible soils and when the compressible soil is loaded with backfill or stocks of heavy materials (see [1], Chapter 5.3). In the case of major structures, preloading of the compressible layers by preliminary construction of an embankment is recommended in order to limit the lateral pressures. Preloading, however, will not eliminate all of the negative friction because some soil settlements will occur over a long period of time and only small relative displacements are required to generate these forces. Therefore, when the negative skin friction is judged to be too large, each pile of the foundation should be equipped with a ***semi-rigid liner which is itself coated with a bituminous layer*** (see § 4.3.3). This liner, generally of spiralled, sheet metal, should protect the shaft over the entire depth of the compressible layer and the overlying soils. This liner should not in any case extend into the underlying, incompressible soils nor into the supporting layers. Liners of corrugated sheet metal are sometimes proposed but are obviously unsuitable for such an application.

4.2.1.2. Flow of water

The obligation to employ a casing or liner is not due to the presence of water, but to its potential to flow freely at certain elevations.

Problems can arise with very permeable, flooded strata and consideration must be given to possible water problems when the subsurface exploration is being done and when planning for the job is underway. Subsurface conditions where the flow could be a problem usually involve granular deposits composed of large sized particles, such as rock embankments, jetties, scree-covered slopes, fluvio-glacial alluvium, or recent natural deposits located in fluvial, lake, lagoon or marine zones. Significant flows of water can also occur in natural rocks that are very fractured or karstic formations (containing solution channels).

The rapid flow of water past fresh concrete can cause damage so that it is necessary to take precautions not only ***during the placement of the concrete but also until it has set***. Therefore, despite the method of boring (with or without temporary casing), it is necessary to anticipate the lining of the shaft at the various levels concerned (partial lining) or for the group of risky zones (continuous lining). According to the nature of the formations where the risks exist and considering the boring procedure to be used, a selection is made of a type of flexible or semi-rigid liner. The respective advantages and disadvantages of these liners have been previously described (see § 4.1 and [2]).

In this way, in cohesionless formations and fissured rock, flexible liners can be an affordable, ***efficient protection***, and with an acceptable amount of lateral resistance and friction. The liners will also limit the amount of overpour of the concrete.

On the other hand, semi-rigid liners are more costly and less efficient with respect to lateral skin friction, but are preferable for rocky and karstic soils because they are less fragile and more easily placed.

4.2.1.3. Scour of concrete

When foundation piles are located at an aquatic site, where there is sediment transport, and where the foundation is not protected by a specific structure such as a sheet-pile wall, it is necessary to protect the piles against scour. Scour is most pronounced during floods. The fresh concrete is protected by the casing used during construction of appropriate thickness (about or equal to 1 cm - 0.4 in.) and left in place.

In some cases, stones or large particles may be transported and it may be necessary to use thicker casing than is desirable to leave in place. In those instances it could be desirable to perform the borings using a temporary casing and to then install a liner of required thickness. This method also avoids the use of excessively long, permanent casing when the thickness of cohesionless soils is much greater than the depth of scour.

4.2.1.4. Karstic formations with solution channels

It was noted earlier that on a karstic site (see § 3.4.1.5), there is no precise method which permits a determination of the size (a few cubic decimeters to several cubic meters) or the distribution of cavities.

The construction supervisor is then confronted with an arbitrary choice which always implies an economic risk which is difficult to estimate beforehand.

When anchorage is achieved in a sound substratum, the least hazardous option, but certainly a costly one, consists of employing a continuous semi-rigid liner over the entire height which is known to be karstic. This guarantees a minimal overpour of concrete, limited only by the risk of the possible flowing of the concrete between the borehole and the liner.

On the other hand, a deliberate choice not to line may turn out to be advantageous, but may also lead to significant overpouring. In this case, several situations may arise on the job site:

-- *Cavities are detected during boring.*

One method of dealing with the problem is described earlier in section 3.4.1.5 and presents an economical solution.

-- *No cavity is detected during the boring:*

- in the absence of a cavity in the vicinity of the borehole, no overpour will be observed,
- on the contrary, the pressure of the concrete may cause the filling of the neighboring cavities due to opening-up of fissures. A significant overpour of concrete may then result.

4.2.2. Approaches associated with solution of construction problems

Discussed in this section is the use of casing or liners to obtain a solution to certain construction difficulties. It is necessary to foresee such difficulties before work is begun. The solutions described in the following paragraphs generally deal with a reduction in the overpour of concrete.

4.2.2.1. Deformation or lateral squeeze of soft soils

Regardless of the boring procedure adopted (with or without temporary casing), an overpour of concrete may be due to the lateral deformation of a compressible layer of soil under the pressure of the fresh concrete. Usually, this expansion as a whole remains very limited and recourse to lining is rarely justified.

However, if there is concern about negative skin friction, the use of a casing or liner to protect against downdrag (see 4.2.1) becomes even more necessary because the lack of such precaution leads to an increase in the downward forces on the pile. When the depth of soft soils is of particular significance, a non-lined pile tends to acquire a flattened-cone form (Fig. 61) which of course amplifies considerably the effect of the negative skin friction.

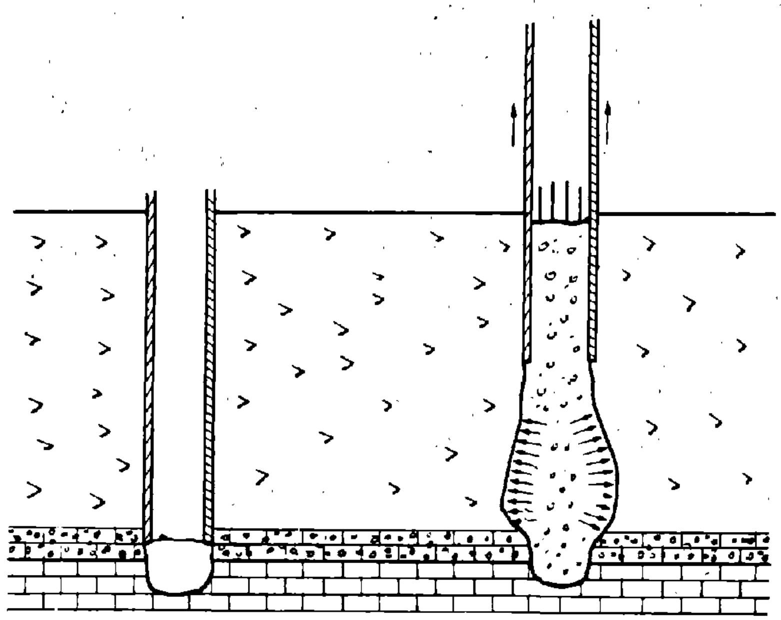


Fig. 61. -- Deformation of a soft layer even though a temporary casing was used.

In the special case of piles constructed alongside or through recently constructed embankments, there may be, on the other hand, concern during concreting of local narrowing of the borehole, brought about by horizontal soil pressures. At the time of extraction of the temporary casing (the use of such an expedient is recommended in this case, see 3.4.1.1), the height of the column of fresh concrete may turn out to be insufficient to counteract the lateral squeeze of the soil. Therefore, the use of a semi-rigid liner may be considered, but it was noted earlier that the use of such a liner would not be necessary if the embankments had been constructed early enough.

4.2.2.2. *Caving during boring*

These cave-ins are associated with the nature of the soil, and with the presence of water, and are related to construction procedures (see § 3.5.6).

If the use of a temporary casing has been selected as the method of curtailing such caving, placement of a liner is not required, because the possibility of caving was specifically eliminated by the selection of the temporary casing.

If boring with drilling mud has been chosen as the method to prevent such caving, again, this procedure maintains the stability of soils without the necessity subsequently to add a liner. It can also be said that, even if some caving occurs during boring (see § 3.5.6), it is advisable to allow the concrete to fill the gaps formed in spite of the overpour which will result. Filling of such gaps will ensure a good contact between concrete and soil that is essential to the mobilization of the skin friction and will also give the foundation an improved capacity to resist lateral loads.

Liners of the semi-rigid type may, however, be considered during construction when the caving noted in the construction of the first few piles is attributable to a poorly selected construction technique. The caving is liable to mean that the extra concrete used for the overpour has a greater cost than that for the liners. It is necessary, however, when estimating the cost of the liners to include such requirements as injection and filling of the annular space (see § 4.1.2). Grouting of the annular space may be required in order to ensure the designed conditions of stability for the pile, but may not have been anticipated as a part of the operation of lining.

4.3. PROBLEMS OF PLACEMENT

The various problems linked with placement of liners and casing are a result of their fragility, their deformability, or their bulkiness.

4.3.1. Transportation

For prefabricated liners (semi-rigid) and casing, the problems are about the same as those posed by transportation of the reinforcement cages (see § 5.5.2).

For casings, however, supply delays are often quite long due to the use of small quantities, delivery delays, or defective material. Also, the desirable sizes of casings are not always found in stock at the manufacturers.

4.3.2. Bracing and lifting

The difficulties of bracing and reinforcing of liners obviously increase with the importance of the components. From their very nature, casings do not pose problems, apart from those of very flexible metal, and are generally placed in the borehole with cranes. However, rough handling can result in impacts that cause deformations which are incompatible with their intended use. The bracing and handling of casing and liners is thus a critical operation which requires provisions to avoid both bending and distortion of the long and less rigid elements.

The most critical operation, as is the case with reinforcing cages, is the raising of the casing or liner into a vertical position (erection, Figs. 62 and 63).

The number of lifting points should be determined from the length and the flexibility of the liner and, at the lower suspension point, a strap is selected in preference to a more usual connection. It is also recommended to employ braces at the ends of the liner (crosspieces for example, as shown in Fig. 64) in order to prevent the out-of-roundness of the liner.

4.3.3. Preparation of bitumized linings on site

When negative skin friction is anticipated, the magnitude of the downdrag may be limited by coating the proposed liners with asphalt or with a product with equivalent specifications (Shell Indaspile type for example) (Fig. 65).

Due to its ability to creep under stresses that are applied slowly, an asphaltic layer placed between the soil and the shaft permits the reduction of the vertical load which is brought about by the settling of compressible layers which the pile penetrates (see [1], Chapter 5.3, Dossier FOND 72).

This operation of asphalting of the casings consists of three phases:

a) CLEANING AND PREPARATION OF SURFACE

Liners that have mud and debris clinging to them are spray-washed. The greasy sections are further brushed and washed with a de-greasing product (detergent) and later are carefully rinsed.

The rust which often collects on liners stockpiled on the job site does not always present a drawback concerning the use of coating, but the

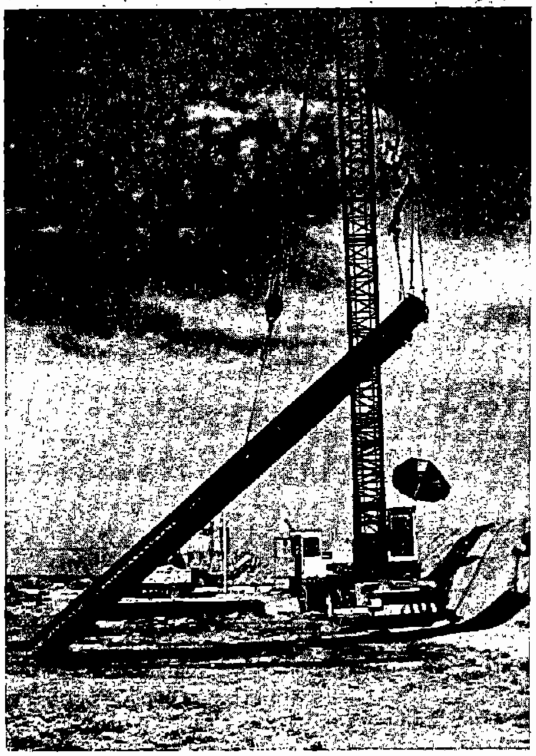


Fig. 62. -- Raising into vertical position of a semi-rigid liner.

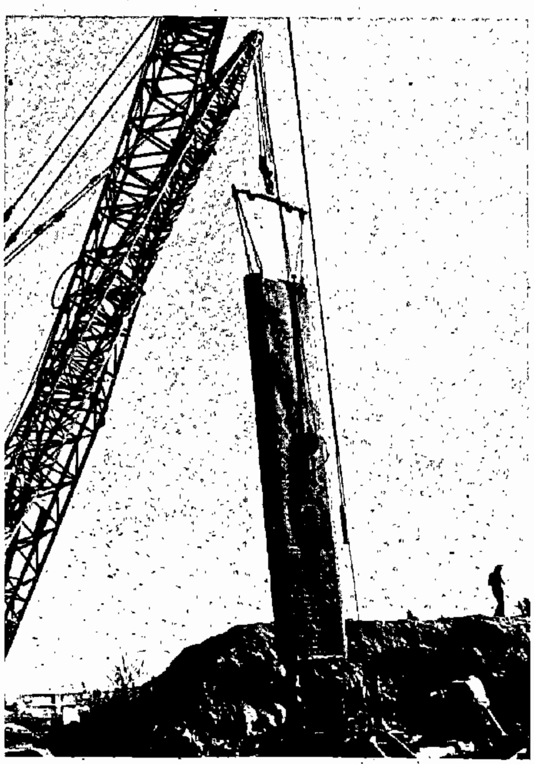


Fig. 63. -- View of a suspension system.

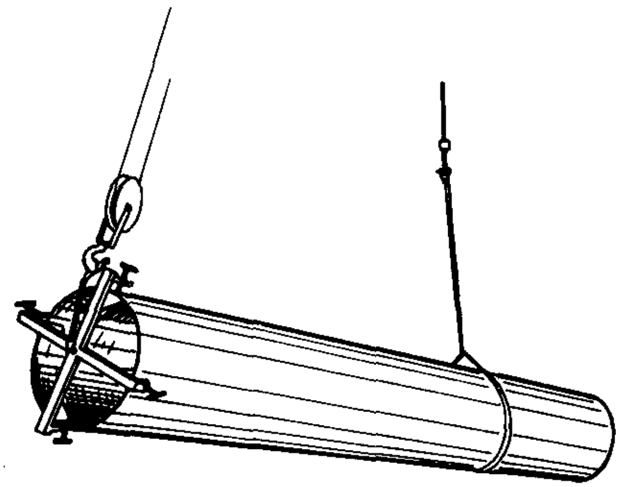


Fig. 64. -- Procedure recommended for bracing of semi-rigid liners (crossbars above, and strap in lower section).

Fig. 65. -- Semi-rigid, bituminous-coated liner.

corrosion may later become more rapid. Thus, it is advisable to use a metal thickness of at least 1 mm (0.04 in.) and to stockpile and coat the sheaths in an appropriate area and in favorable surroundings (low humidity, ventilated space protected from extreme temperatures).

b) SPRAYING THE COATING WITH A HOSE (Fig. 66)

This involves coating the exterior with a cationic emulsion with rapid break and a pure asphalt base (60 to 65%) and, for example, without solvent (formulation of "summer" roadway emulsion, for example).

A breaking time of approximately 15 minutes can easily be obtained using an emulsion with a pH of around 3.5 which will not cause corrosion of the metal.

It is wise to note that emulsions of the anionic type, although not chemically harsh, are not recommended because their break is too slow and too dependant on climatic conditions.

As a rule, spraying (or possibly application with a brush) of approximately 400 to 600 grams (0.9 to 1.3 lb) of emulsion per square meter (10.8 ft^2) permits formation of a continuous coating of sufficient thickness.

c) COATING WITH A BRUSH, IN SEVERAL LAYERS, OF APROXIMATELY ONE CENTIMETER OF BITUMEN (Fig. 67)

The hardness of the asphalt to be used depends on an ambient temperature which permits, on one hand, an easy application and, on the other hand, avoids subsequent risks of creeping which may cause de-coating of the sheath before installation in the borehole.

Selection of a 40/50 penetration asphalt in hot seasons and a 60/70 asphalt in cold seasons is generally satisfactory.

When those soils which are likely to induce negative frictions on the piles which penetrate them are granular (embankments or gravel layers on top of compressible soils), it may be desirable to increase the thickness of the coating and asphalt then becomes less suitable. In this case it may be replaced by pure bitumen which can be poured in thicker layers, but for which the conditions of placement are more complex.

4.3.4. Connection of components

Casings, both temporary and permanent, that are transported along a highway or street, can have maximum lengths of 15 meters (50 feet). To obtain desired lengths, these casings are cut and joined in a clean and level area. The welds must be continuous.

Fig. 66. -- Spraying coating with hose.

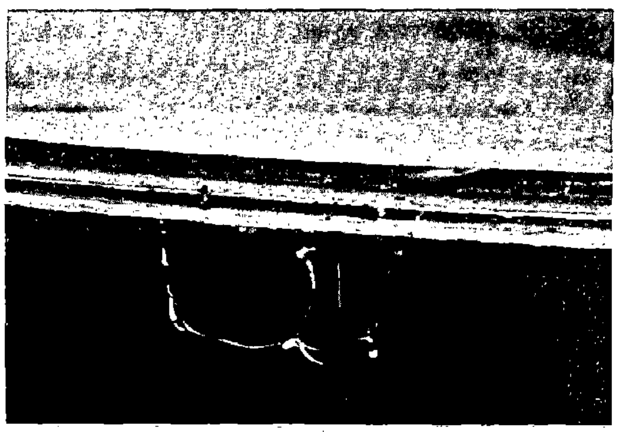


Fig. 67. -- View of tip of a liner showing thickness of a bituminous layer.

Spiral liners are delivered in component lengths of 9 to 15 m (30 to 50 ft), according to their diameters. It is also possible to prefabricate them at the construction site (Fig. 68). The joining of two sections is achieved by riveting or by some other form of coupling such as bands (Fig. 69).

Flexible liners or sheaths of synthetic material are preferably sewn and joined at the job site, and are generally composed of a single component (Figs. 59 and 60).

4.3.5. Centering of casing or liners

In general, assuming the presence of rebar cages, the best possible centering of the borehole liner and rebar cage should be achieved.

Centering of a rebar cage in a casing or liner is obtained by the procedures recommended in Chapter 5.

Centering of a semi-rigid liner in the borehole should be achieved by the use of spacers at the top of the liner. Wooden wedges can be used, for example, or metal pieces can be temporarily joined to the top of the liner or to the temporary casing (Fig. 70). When the temporary-casing procedure is used, improved centering can be achieved by the use of spacers along the length of the casing, 3 or 4 per level (Fig. 71). This modification limits the risk of raising the liner during extraction of the temporary casing.

4.3.6. Placement of casing and liners (Figs. 72 and 73)

Permanent casings (Fig. 74) are placed by gravity, by driving, or by vibration; thus, they play the role of the temporary casing in the construction phase (foundations in rivers, for example). The use of permanent casing may turn out to be inefficient or too costly in difficult soil in view of the thickness required for the casing. The substantial thickness of the casing is needed because of the possible presence of rock, masonry, hard strata, and the like. It is thus preferable to lower a liner or a casing constructed in advance into the borehole if the construction conditions so permit.

In all cases, and especially for bored piles without temporary casing, the risks of hanging up a liner or a casing along the wall of the borehole suggests that the diameter of a borehole should be larger than the liner or casing. The annular space thus created will be partially filled during concreting, over a height which is a function of relative diameters of the liner and borehole and a function of the height of the

Fig. 68. -- Fabrication on job site of a metal, semi-rigid liner.

Fig. 69. -- Example of riveted joint.

Fig. 70. -- Support at top of a casing with welded brackets.

Fig. 71. -- Four spacers on a liner to allow the centering of the liner within temporary casing.

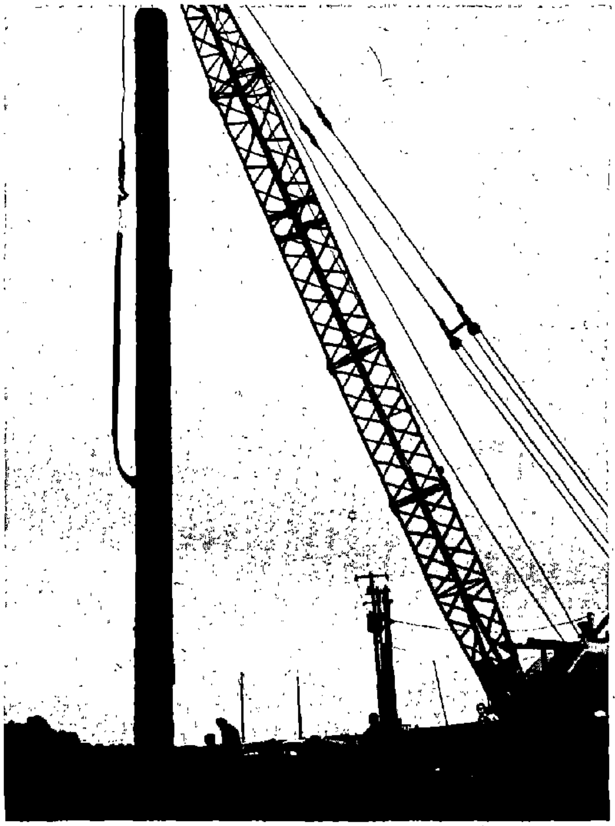
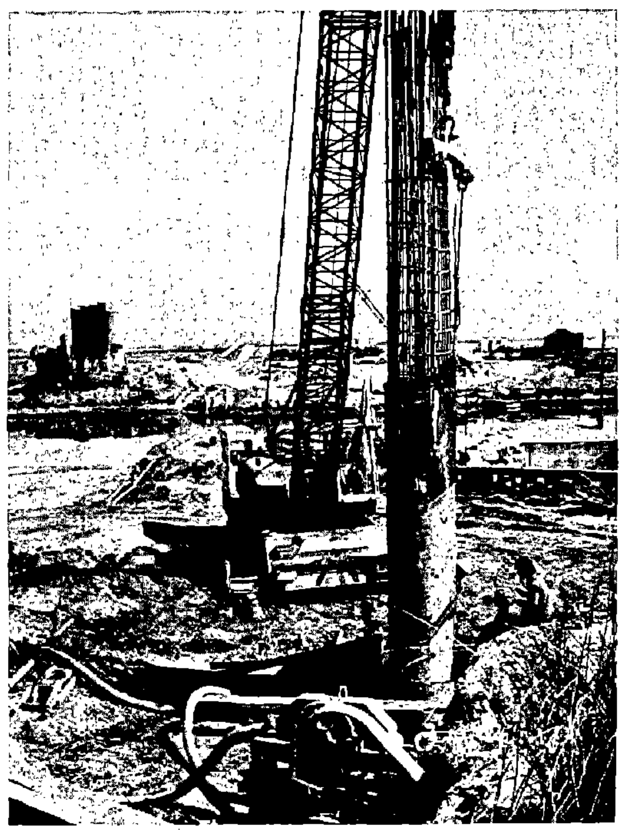


Fig. 72. -- Lowering of a liner before placing the rebar cage.

Fig. 73. -- Simultaneous lowering of liner and rebar cage.



concrete column (Fig. 75). The efficiency of the lining in the light of possible negative skin friction is then compromised, perhaps even destroyed. It is easy to envision that the height of the concrete which fills the annular space between the liner and the borehole is somewhat reduced when a concrete of lower slump is pumped into the shaft (liner).

On the other hand, if the annular space is too small, there may be concern that the extraction of the temporary casing, particularly if it is rocked back and forth, will cause the lifting of the liner because of the presence of aggregates (or of concrete that has set prematurely) between the liner and the casing. The clearance of 5 cm (2.0 in.), recommended in section 4.1.2, seems satisfactory in this regard. However, in any case, special attention must be given to the recovery of the temporary casing.

Use of flexible liners (Figs. 59 and 60) requires precautions that are associated with risks of tearing. A flexible liner should obviously be resistant to concrete pressures (risk of bursting), with the lower portion of the liner receiving the greatest pressure. Thus, the characteristics of the liner, including seam, will be specified in consideration of the stresses that will be imposed by the soil and concrete. At the time of the welding of the components of the rebar cage, potential damage to a textile liner may be reduced by spraying the liner with water in the vicinity of the welding.

Placement of liners (as with rebar cages) in the borehole may lead to caving and thus to additional deposits at the bottom of the borehole because the placing of liners takes longer and is more critical. A careful cleaning of the bottom of the borehole is strongly recommended just before concrete is placed (see § 3.4.4).

4.4. INSPECTIONS

The initial inspection should be carried out at the time of receipt of the construction materials and to verify that they meet the appropriate specifications. The quality of the materials and their dimensions (especially length) should satisfy the guidelines and the materials should be satisfactory for the functions for which they will be used.

Because employing casing and liners is expected to offset significant risks, the use of these items should be rigorously inspected by personnel who are competent and on hand throughout the operation. This

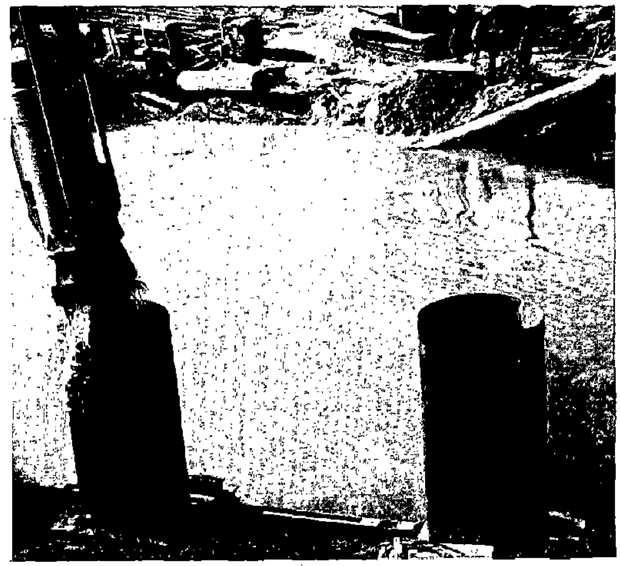


Fig. 74. -- Piles at aquatic site constructed with lost casings.

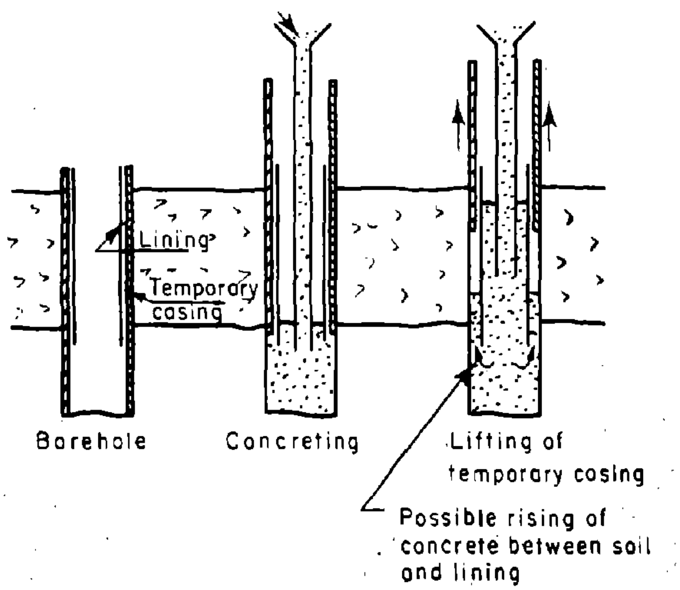


Fig. 75. -- Rising of concrete in annular space between liner and wall of borehole.

phase should not be treated any differently from an inspection point of view than other phases of the construction of piles.

In the case where there is a special concern about the overpouring of concrete, the height of the concrete column is measured and the actual volume of concrete that is placed may be compared with the theoretical volume. The amount of overpour can be estimated as the concreting of a pile is accomplished.

CHAPTER 5

REBAR CAGES

Bored piles for structural foundations are generally reinforced over their entire length. Except for some very special cases (short piles of large diameter in strong and impermeable soil, for example), it is obviously impossible to place steel after the concrete has been poured. The steel reinforcement should thus be prefabricated at a plant or on the job site and then be lowered into the borehole prior to concreting. The assembly of steel bars so prefabricated is called the ***rebar cage*** (Figs. 76 and 77).

In the axial direction, a rebar cage for a pile is composed of longitudinal bars, distributed around the outside of a cylinder. Around the longitudinal bars are transverse reinforcing that is wound around and rigidly attached to the longitudinal bars. The transverse reinforcing is in the form of hoops or helices, also called spirals.

The longitudinal and transverse reinforcing steel is held together with ties or by welding (Fig. 78). Even if the logitudinal connections are made with ties, certain special reinforcing bars (hoops for sizing the assembly, loops for use in lifting, for example) should be welded to the main bars. This is why steel to be used for reinforcing a pile should be made of weldable rebar. Found in Table I is a list of the main characteristics of rebar required for piles.

Of course, periodical updating of the information shown in the table will be necessary. For rebar that is highly stressed, the welding procedure should be compatible with the type of assembly (transverse, end to end, lapped) and with the conditions of manufacture of the cage (on the job site or at the fabrication plant).

5.1. LONGITUDINAL REINFORCING STEEL

5.1.1. Role. Strength of rebar used

Longitudinal reinforcing has the function of resisting, in each section of the pile, the bending moments from the calculated loads or any unexpected loads. Use of bars of high strength σ_{en} > 3,300 bars) is recommended for either smooth or deformed bars. Deformed bars are usually selected since the bond resistance of the smooth bars may be lowered

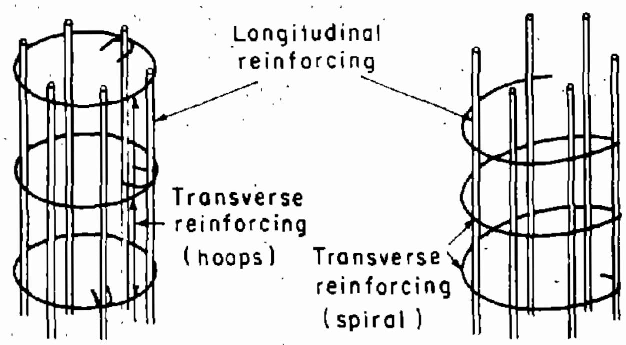


Fig. 76. -- Elements of a reinforcing cage for piles.

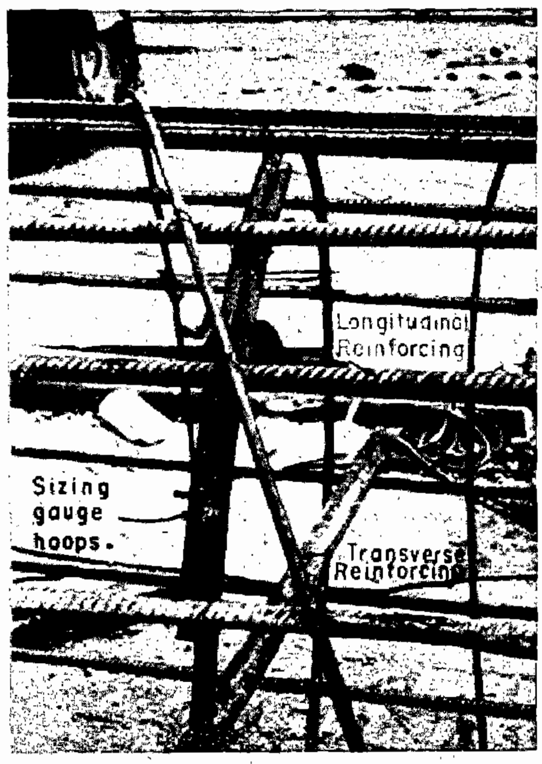
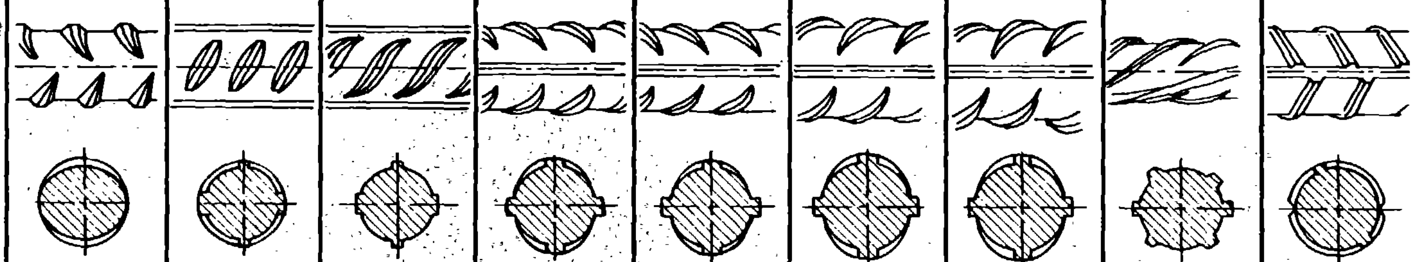


Fig. 77. -- Group of main components in the fabrication of a rebar cage.

TABLE I

CHARACTERISTICS OF WELDABLE REBAR WITH LARGE DEFORMATION

SYMBOL	BR	RHS	HBA	HS	SH	TG	TS	T	W
NAME OF PRODUCT	steel **BIRI**	steel **RODURAC HS**	crenated steel **HI BOND A**	crenated steel **HLE HADES**	crenated steel **HLE SUPER-HADES**	steel **THYGRIP**	steel **THYGRIP S40**	steel **TOR**	steel **WELBOND**
DEVELOPMENT PROCEDURE	cooled, Thomas steel, hardened by traction	naturally hard carbon base oxygen blown	Thomas steel blown with enriched air	cooled, Thomas steel blown with enriched air	cooled, Thomas steel blown with enriched air	Thomas steel blown with enriched air, cooled	Thomas steel blown with enriched air cooled	soft steel heat laminated hardened with cold torsion	naturally hard steel made in electric ovens
IDENTIFICATION NUMBER	n° 21 bis 30-12-71	n° 36 19-02-75	n° 15 ter 30-12-71	n° 5 quater 28-4-71	n° 17 bis 28-4-71	n° 6 bis 20-7-73	n° 26 30-12-71	n° 11 ter 20-7-73	n° 22 ter 20-7-73
APPEARANCE									
MINIMUM ELASTICITY LIMIT: σ'_{eg} (bars)	4120	4120	$\phi \leqslant 20$: 4120; $\phi > 20$: 3920	$\phi \leqslant 20$: 4120; $\phi > 20$: 3920	4900	$\phi \leqslant 20$: 4120; $\phi > 20$: 3920	4120	$\phi \leqslant 20$: 4120; $\phi > 20$: 3920	$\phi \leqslant 20$: 4120; $\phi > 20$: 3920
STANDARD COMMERCIAL LENGTHS (m)	up to 15	6 to 8	up to 15	6 to 15	6 to 15	3 to 15	6 to 15	10/15	12
NOMINAL DIAMETERS (mm)	6 to 14	6 to 20	up to 40	6 to 32	6 to 16	6 to 40	6 to 20	5 to 40	8 to 32

(0.0254 m = 1 in.; 25.40 mm = 1 in.)

Publication Date: January 1978.

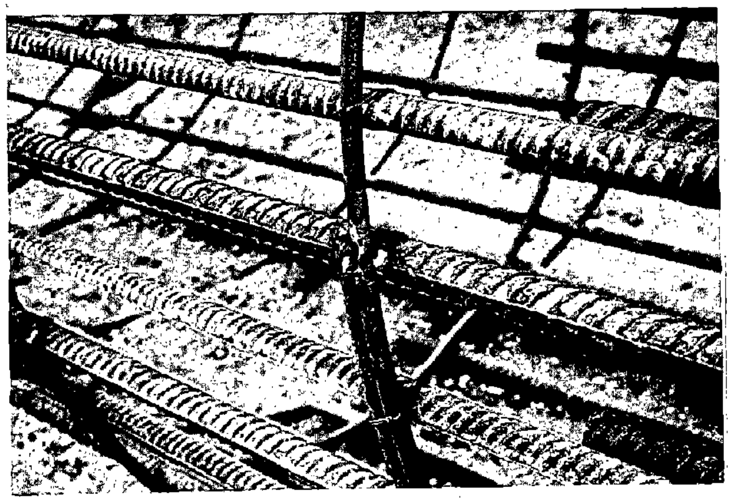


Fig. 78. -- Assembly of reinforcing cage with ties and welds.

considerably due to the presence of a film of mud between the concrete and the rebar.

5.1.2. Diameter of reinforcing

The diameter of the longitudinal bars should be at least 12 mm (0.47 in.) (Article 68, Clause 36.1) and may be as large as 32 mm (1.26 in.) and very exceptionally 40 mm (1.57 in.). It is not desirable to use a diameter over 25 mm (0.98 in.); diameters over 25 mm present problems with welding (assembly with transverse bars) and flexibility of the cage. It is also necessary to consider that above a certain diameter, generally 20 mm (0.79 in.), the allowable stress of rebar drops.

5.1.3. Length of reinforcing steel

The current lengths of rebar that is delivered commercially is 12 to 14 meters (39.4 to 45.9 ft). The maximum commercial length is generally 15 meters (49.2 ft). The hoisting equipment commonly on the job, excluding large cranes, rarely can lift higher than 15 meters. Finally, in compliance with the code for highways (see Section 5.5.2: Transport) the transport of elements of lengths under 14 m (45.9 ft) with a maximum of approximately 15.50 meters (50.8 ft) is permitted.

Thus, rebar cages for long piles (above 15 m) should be broken down in component sections which are assembled at the job site at the time of the lowering of the reinforcing steel into the borehole. The longest section should be the lowest section because this will facilitate the placing of the cage in the borehole.

5.1.4. Construction arrangement

Longitudinal bars are most often distributed uniformly over the circumference or perimeter of the pile. There should be at least five and preferably six bars (CPC, Article 68, Clause 36.1).

In order that the concreting of the pile can be carried out under proper conditions, spacing of the reinforcing should be at least 10 cm (4.0 in.) for small diameter bars (12 to 16 mm - 0.47 to 0.63 in.) and 15 cm (6.0 in.) for large diameter bars (20 to 32 mm - 0.79 to 1.26 in.).

The cross-sectional area of the reinforcing steel should be at least 0.5% of the cross-sectional area of the concrete of the pile (Clause 36.1 of Article 68, 1st title). In Table II, the minimum area of steel for a pile is shown as a function of diameter, taking into account the specifications mentioned previously. The minimum amount of steel for the longitudinal reinforcing ranges between 39 and 46 kg per cubic meter (2.4 and 2.9 lb/ft^3) of concrete.

The distribution of the rebars may be varied over the perimeter of the pile in the case where the main forces causing bending have a preferential direction. However, it is usually preferable to avoid unequal spacing of the rebars because of construction difficulties associated with the need for precise positioning of the rebar cage in the borehole.

5.2. TRANSVERSE REINFORCING STEEL

5.2.1. Role

From the point of view of reinforced concrete, the transverse reinforcing plays three roles.

The main role is to maintain the longitudinal reinforcing by preventing buckling.

TABLE II

Pile diameter (cm)	50	60	70	80	90	100	110	120	130	140	150	200
Concrete area (cm^2)	1,964	2,868	3,849	5,027	6,362	7,854	9,504	11,310	13,274	15,394	17,672	31,416
Minimum area of steel (cm^2)	9,82	14,14	19,25	25,14	31,81	39,27	47.52	56,55	66,37	76,97	88,36	157,08
Examples of minimum rebar	9 Ø 12 or 7 Ø 14 or 5 Ø 16	10Ø14 8Ø16 5Ø20	10Ø16 7Ø20 5Ø25	13Ø16 8Ø20 6Ø25	13Ø16 11Ø20 7Ø25	13Ø20 8Ø25	16Ø20 10Ø25	18Ø20 12Ø25	22Ø20 14Ø25	25Ø20 16Ø25	18Ø25 11Ø32	33Ø25 20Ø32

(2.540 cm = 1 in.)

The second role is to resist the shearing forces.

The third role is to prevent any longitudinal cracks from opening up in the concrete; the transverse steel increases the resistance of the concrete compared to that of the same non-reinforced concrete, independently of the effect of the longitudinal reinforcing. In fact, the transverse steel provides containment for the concrete and acts to increase the factor of safety when large compression stress is present.

From the point of view of construction, transverse steel in association with other reinforcing steel (hoops for maintaining the diameter of the cage, for example, § 5.3.1) acts to stabilize the cage against distortion during handling and maintains the position of the longitudinal steel during the lowering of the rebar cage into the borehole and during concreting.

5.2.2. Geometry of the transverse steel

In practice, the strength of the transverse steel does not play a role of resistance to normal stresses; the bars are nearly always smooth for convenience, (strength Fe E 24 or possibly Fe E 22). However, when the shearing stresses are very significant (exceptional cases), recourse may be taken to deformed bars so that the transverse bars can have satisfactory spacing for the placing of the concrete.

The bars for the transverse reinforcing are installed in successive increments. Each increment may be composed of one hoop. In the case of piles, taking into account the fact that the longitudinal bars are distributed in a circle, the transverse reinforcing is generally composed of a helix also called a spiral (Fig. 76). To facilitate placing, a wire machine is often used for small diameters of bars (up to approximately 10 mm - 0.40 in.). For larger diameters (above 12 mm - 0.47 in.), steel rebar is used.

Theoretically, from the point of view of reinforced concrete, the fashioning of hoops and spirals should comply with the guidelines shown in the sketch in Fig. 79. The anchorage of a hoop should not be in the same vertical plane with that of the preceding hoop. Mechanical continuity of various spiral sections should be achieved by means of lapped joints and the ends of the bars are to be fitted with hooks; thus, the anchorage of the ends of the bars is created by embedment in the concrete mass.

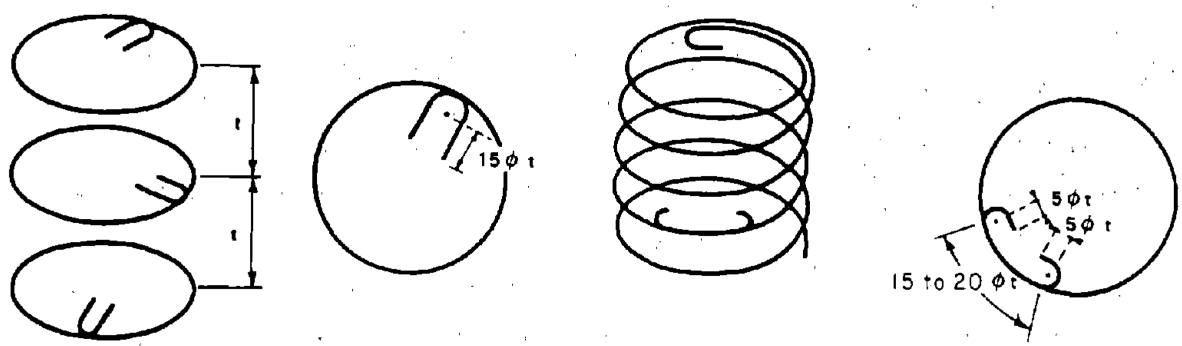


Fig. 79. -- Theoretical guidelines for construction given by reinforced concrete regulations.

Important Comment. -- The anchorage of hoops, just as the anchorage and lapping of spirals discussed above, functions properly when the concrete has hardened. The anchorage is achieved by means of the phenomena of steel-concrete adhesion. It should be noted that the anchorage and lapping function poorly during the concreting phase. Under the effect of the weight of the fresh concrete and the pressure which it exerts on the reinforcing steel, the anchorage tends to unroll and the lapped portions may slide past each other.

During the concreting phase, the transverse bars cannot prevent the buckling of the longitudinal steel if special precautions are not taken. The two sketches in Fig. 80 show some of the problems which may arise.

Finally, the guidelines for anchorage and overlapping of bars cannot always be followed because turning of bars toward the inside of the rebar cage can interfere with the placing of the tremie used for the pouring of the concrete.

In order to avoid significant deformations of the cage, precautions with the transverse reinforcing may be taken. Examples of methods appear in Figs. 81 and 82.

5.2.3. Diameter of bars

The application of Clause 36.1 of Article 68 and of current regulations for reinforced concrete leads to a slightly small diameter for the transverse steel. These regulations are not appropriate for consideration of the practical problems of construction posed by bored piles.

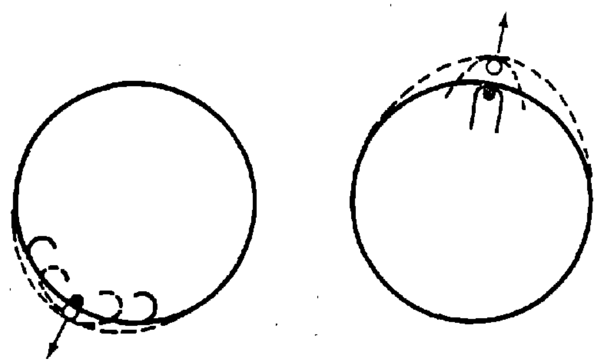


Fig. 80. -- Risks of uncoiling of transverse bars during concreting.

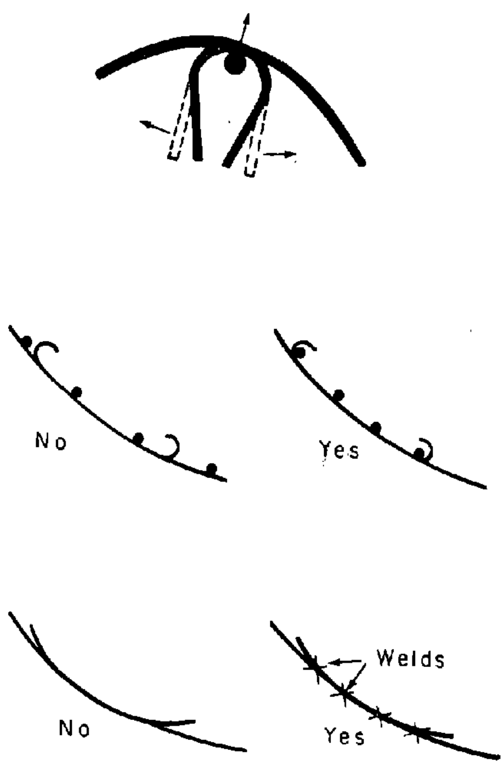


Fig. 81. -- Practical construction guidelines.

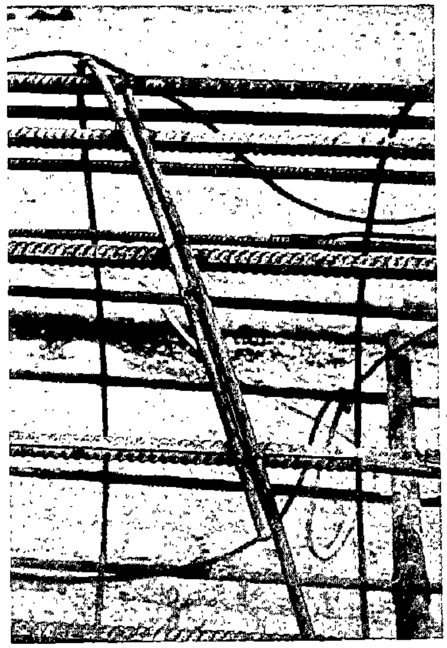


Fig. 82. -- Example of overlapping transverse bars with weld. The weld limits the risk of sliding at the time of concreting.

Taking into account observations made at the job site, we think it is reasonable to adopt for $\emptyset_t$ the *minimum values* given in Table III, until possible modifications are made in Article 68.

The average quantity of transverse steel in current use is around 13 kg per cubic meter (0.8 lb/ft³) of concrete. We think that it is necessary to increase this quantity to at least 20 kg per cubic meter (1.3 lb/ft³).

Figure 83 is a good illustration of the poor behavior of overly weak transverse reinforcing.

TABLE III

Longitudinal reinforcing steel $\emptyset_t$ (mm)	12	14	16	20	25	32
Transverse reinforcing steel $\emptyset_t$ (mm)	6-8	6-8	8-10	12-14	12-14-16	16

(25.40 mm = 1 in.)

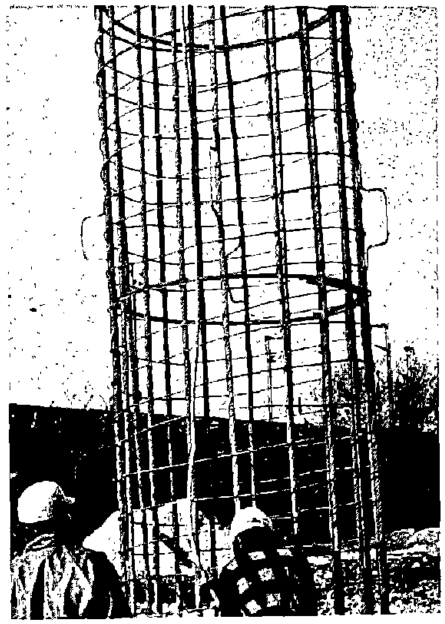


Fig. 83. -- Transverse reinforcing bars with too-small diameter. They are deformed before being placed.

5.2.4. Diameter of hoops and spirals

The diameter of hoops or spirals, and more precisely the diameter of the outside of the cage, should be equal to:

-- the nominal diameter of the finished pile, less 10 cm (2 x 5 cm of coating - 4 in.) in the case of a pile constructed without temporary casing and unlined (Fig. 84);

-- the inside diameter of the temporary casing minus 10 cm (4 in.) when the pile is built by using a temporary casing, but is not lined. (In this case, the rule is determined with the essential aim to avoid the lifting of the cage when the temporary casing is pulled, Fig. 85).

-- the inside diameter of the lining minus 6 cm (2.4 in.) when the pile is lined.

5.3. THE REINFORCING STEEL AND SPECIAL DEVICES

5.3.1. Sizing hoops

To facilitate the fabrication of the cage and to obtain the finished diameter of the cage and the appropriate distribution of the longitudinal steel, it is necessary to use special reinforcing called sizing hoops or gauge hoops (Figs. 86 and 87).

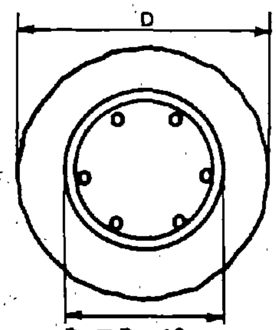


Fig. 84. -- Exterior diameter of cage.

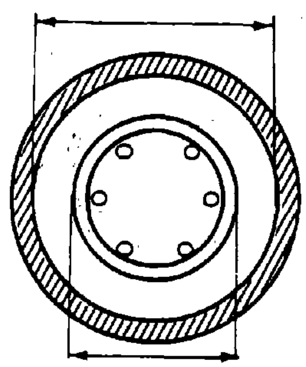


Fig. 85. -- Exterior diameter of cage (pile constructed with temporary casing).

Fig. 86. -- Gauge hoop with overlapping welds with markings for position of longitudinal steel.

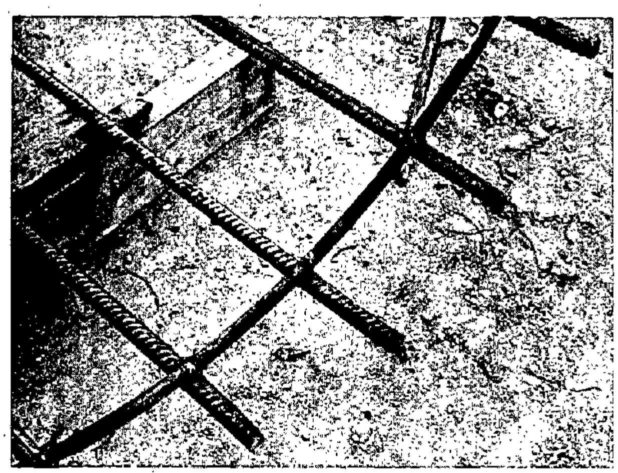


Fig. 87. -- Fabrication using gauge hoop.

These hoops should be ***rigid***, and thus the diameter of the bar should be large (example: Ø 20 to 25 mm for a 1 m pile - 0.79 to 0.98 in. for a 3.3 ft pile).

The sizing hoops enhance the rigidity of the cage during transport and handling by maintaining the position of the longitudinal bars. In addition to these functions, they accommodate devices for making an acoustic investigation and tubes for investigating defects in concrete placement (see § 5.3.7). Figure 88 shows the deformations a reinforcing cage is subjected to without such gauge hoops.

The sizing hoops may be made by welding end to end or, better, by welding overlap (Fig. 89). They are fashioned from smooth rods and generally prepared at the fabrication yard. ***Their spacing varies between about 2 and 3 meters (6.6 and 9.8 ft).***

5.3.2. Devices for centering the cage

It is necessary to prevent the rebar cage from rubbing along the wall of the borehole during its placement. It is also necessary to center the cage properly in the borehole and to ensure a suitable cover of the rebar. For these purposes, use of special devices is required.

-- *Guides:* these special units, constructed of smooth bars, are welded onto the longitudinal steel (Figs. 90 and 91). Considering their shape, they are sometimes called "skis". They number four per level, eight in the first level in the case of piles of large diameter, and the spacing of the levels is about 2 meters (6.6 ft). They should be rigid and their diameter should be equal to that of the longitudinal reinforcing less one increment. The guides shown in Fig. 92 are attached to the cage in a manner resulting in too much flexibility.

-- *Plastic or Concrete Spacers:* to ensure that there is adequate cover of concrete over the rebar and that the cage is centered in the borehole, circular spacers are used which may be made of mortar or plastic. As a rule, plastic spacers are rejected because they do not ensure an efficient centering due to their flexibility (Figs. 93 and 94). Concerning the attaching of these spacers, precautions should be taken to avoid the penetration of water which may cause corrosion of the rebar.

Shown in Fig. 95 are two method of fastening the spacers. Solution (b), which consists of fastening the spacer on the longitudinal steel, is not recommended because the concrete may not flow completely around the spacer allowing infiltration of water that could cause corrosion of

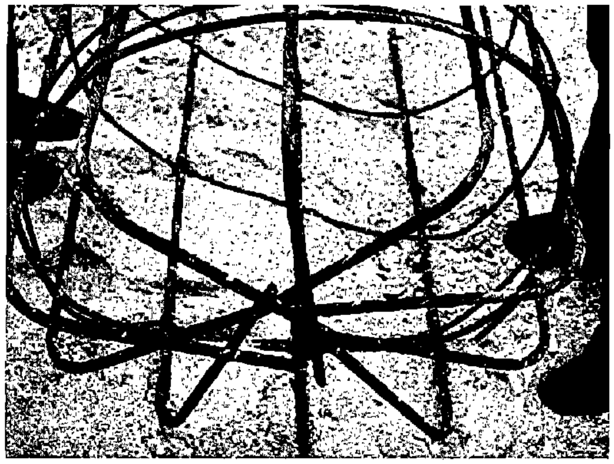


Fig. 88. -- Deformed cage due to absence of sizing hoop, note nonrecommended form of basket.

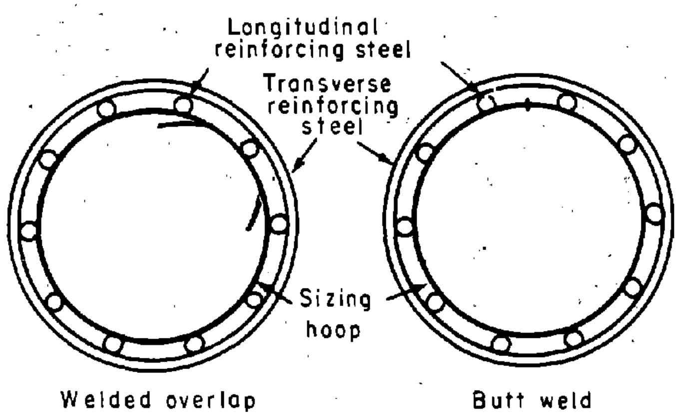


Fig. 89. -- Gauge hoop assembly.

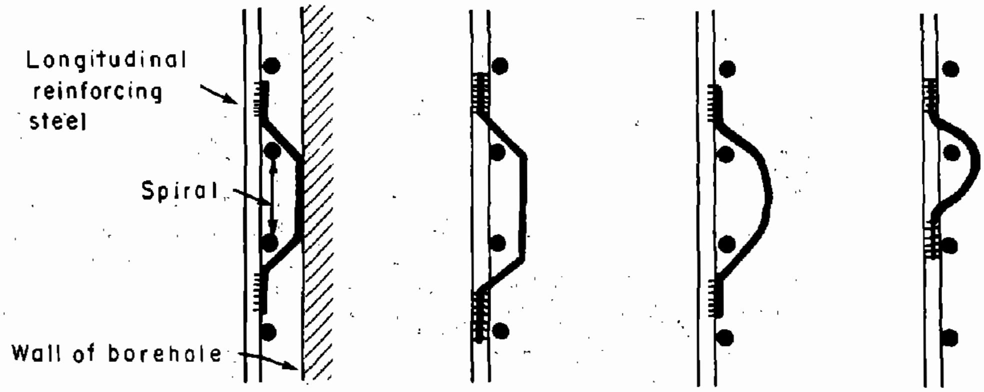


Fig. 90. -- Different types of steel spacers.

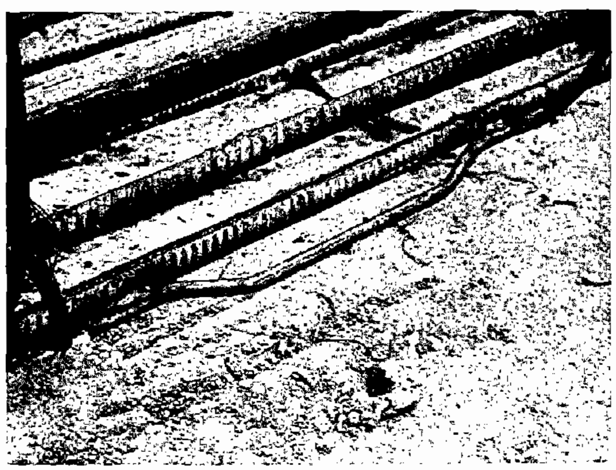


Fig. 91. -- Spacers with satisfactory dimensions.

Fig. 92. -- Incorrect solution. Spacer is welded to transverse bars.

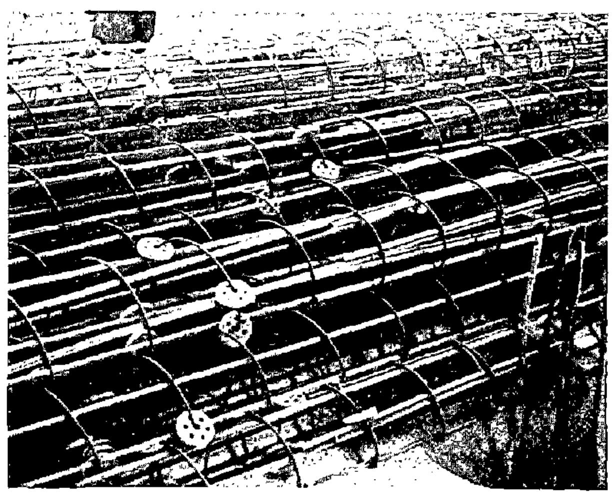


Fig. 93. -- Inefficient centering.

Fig. 94. -- Type of spacer to be rejected.

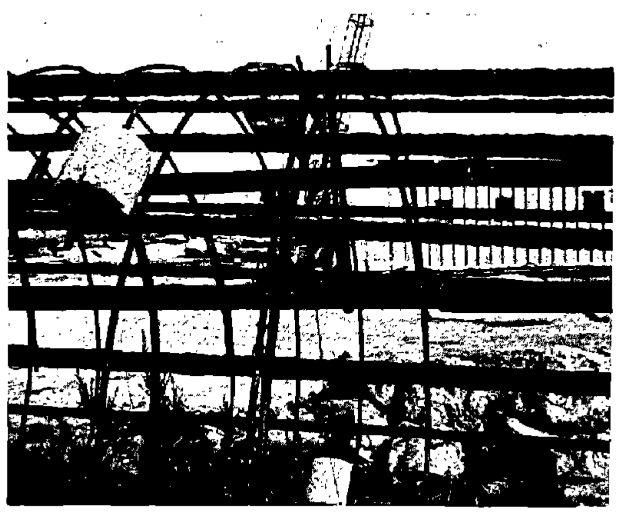


Fig. 95. -- Fastening of concrete spacers.

the main reinforcing steel. Also, when the rebar cage is being placed, these spacers may scrape the wall, causing contamination of the rebar and accumulation of loose soil at the base of the borehole that should be removed before concreting.

The best solution (a) consists of fastening the spacers to two longitudinal rods by means of a small welded rod.

5.3.3. Reinforcing for strengthening of the cage

The cage is flexible and may deform by bending and rotation. The transverse-hoop reinforcing system for sizing the cage is not sufficient. It is necessary to stiffen the cage by means of special reinforcing. These bars can be left in place or may be eliminated as the cage is lowered into the borehole where the bars interfere with lowering the tremie.

Among the devices used are:

-- *transverse stiffeners* that act to prevent the flattening of the cage (Fig. 96);

-- *longitudinal stiffeners* such as those shown in Fig. 97 which prevent the twisting and rotation of the cage.

5.3.4. Basket

At the lower part of the cage, it has been customary to turn the longitudinal bars towards the center of the section of the pile so as to form a "basket" (Fig. 98).

The basket is assumed to have two functions:

-- it prevents the penetration of the longitudinal steel into the bottom of the borehole;

-- it also prevents the possible lifting of the rebar cage due to the bouyancy within the fresh concrete, because of the load of the concrete on the basket. However, the basket prevents the tremie from touching the bottom of the borehole when the concrete pour is initiated and possibly becomes a barrier that prevents the removal of the water in the borehole. The result can be excess water in the concrete, segregation of the aggregate (Fig. 99), and a poor contact between the soil and the concrete.

To permit the tremie to reach the bottom of the borehole, it now seems preferable to allow only a slight curvature of the bars toward the interior of the cage (Fig. 100). Indeed, it is preferable to avoid all adjustments of this type (Fig. 101) especially if, in order to avoid the penetration of the bottom of the cage into the soil, it is possible to suspend the cage in the borehole.

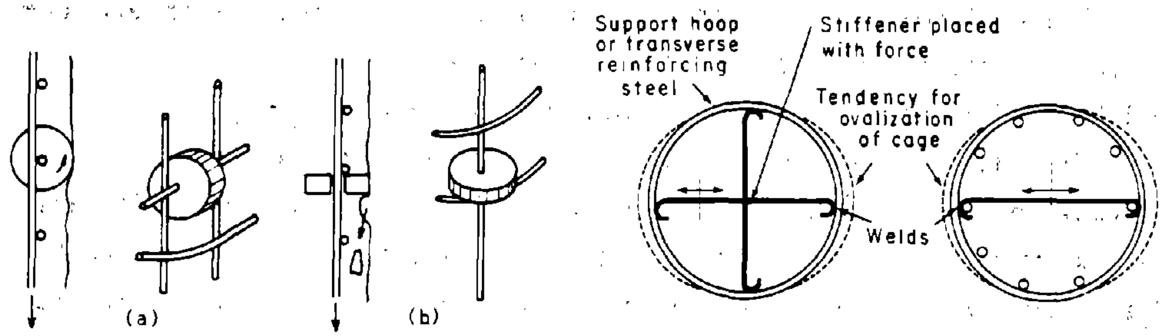


Fig. 96. -- Stiffeners for temporary rigidity.

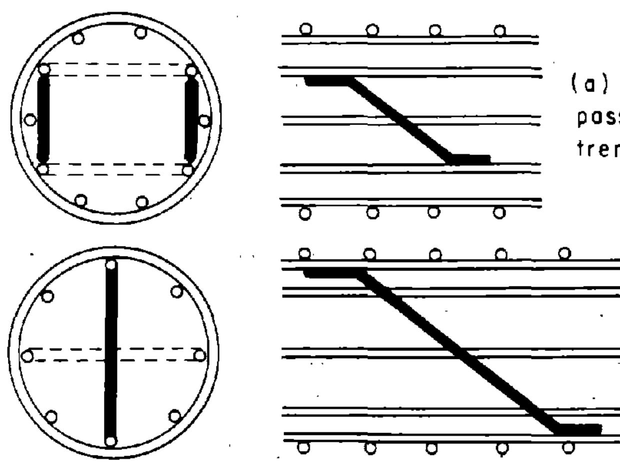


Fig. 97. -- Other types of special reinforcing.

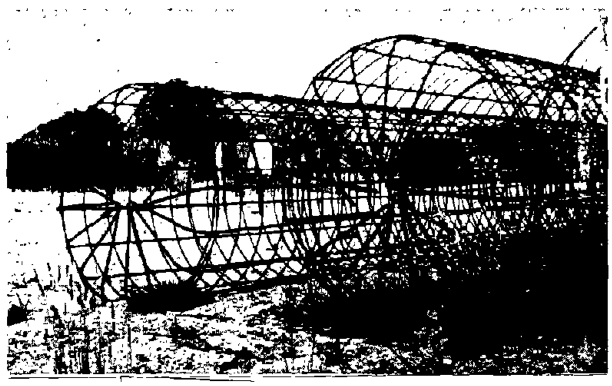


Fig. 98. -- Basket shape not recommended.

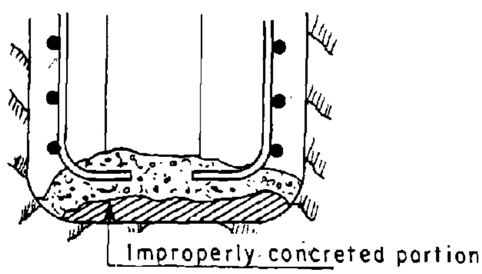


Fig. 99. -- Effect on concreting bottom of pile due to presence of basket.

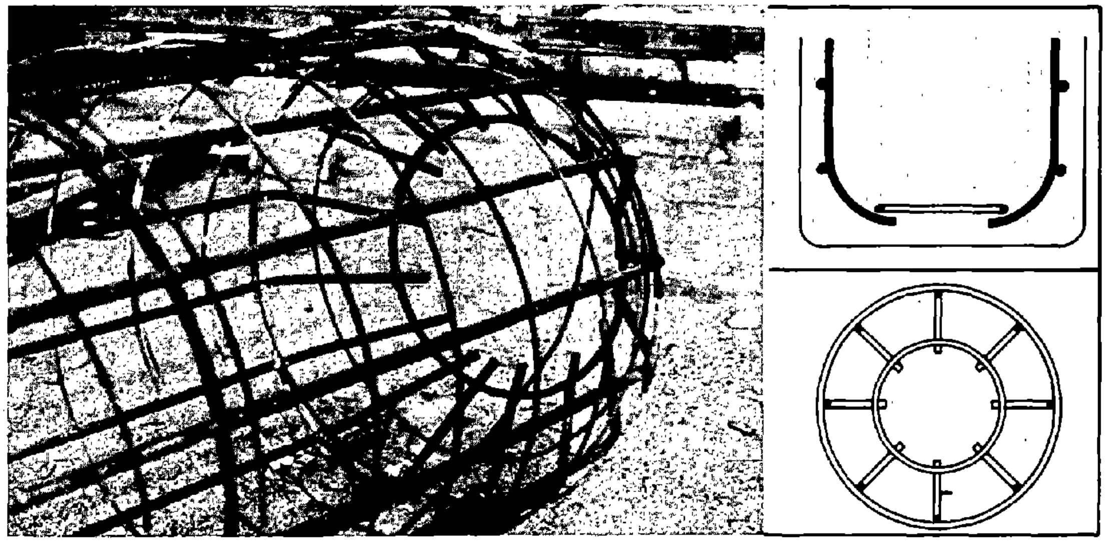


Fig. 100. -- Acceptable configuration of bottom of cage.

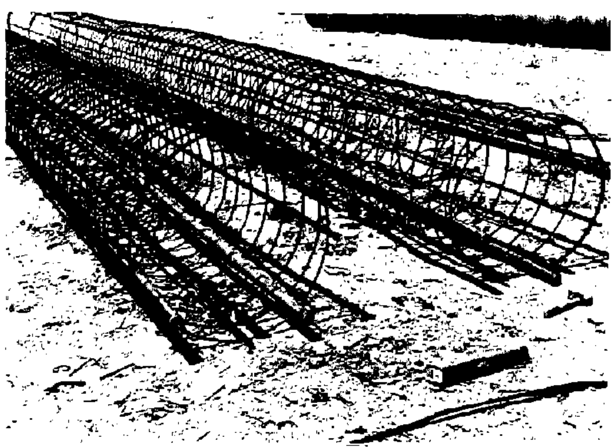


Fig. 101. Reinforcing cages without baskets.

The cage is thus held at a few centimeters from the bottom of the borehole (about 10 cm - 4.0 in.). It is further noted that such an arrangement permits better concreting of the base of the pile.

5.3.5. Arrangements for lifting cages

Unfortunately, on numerous job sites the rebar cages are lifted without taking proper precautions. The hook of the crane is attached at arbitrary places along the cage and the deformations that result from this lack of precaution are serious.

It is necessary to provide special lifting arrangements and to reinforce the cage in the vicinity of the lifting areas (for example by arranging a greater number of sizing hoops).

Figure 102 shows some arrangements for the lifting of a cage that may prove to be useful.

5.3.6. Special connectors

Assembly of the components comprising the cage is achieved by welded joints or by means of special connectors.

This second solution is more costly than the first but certainly safer due to the uncertainty of the quality of welds made on the job site and the rapidity with which the welds must be made.

5.3.7. Access tubes for non-destructive testing

The non-destructive test of finished piles requires that the rebar cages be equipped with metal access tubes for which the dimensions vary, according to the methods, from 50/60 to 102/114 mm (2.0/2.4 to 4.0/4.5 in.).

These tubes (see Chapter 7, § 7.2) may be welded directly to the sizing hoops (Fig. 104) but it is necessary to increase their fixity with arrangements of the sort shown in Fig. 105. Note that it is necessary to use special care in the positioning of the pipes over two consecutive sections of the rebar cage.

5.4. PRESENTATION OF DRAWINGS FOR REINFORCING OF PILES

Drawings of pile reinforcing should not be vague sketches in which only the longitudinal and transversal bars are shown, but construction drawings in which all reinforcing appears including special bars and arrangements. It is necessary to show separately the finished pile and the various components which comprise the rebar cage.

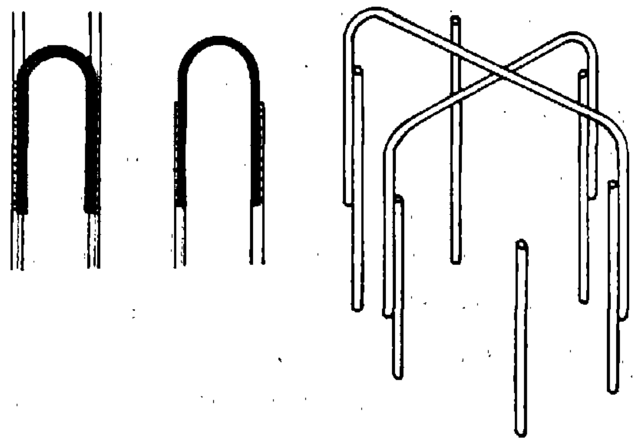


Fig. 102. -- Various types of lifting reinforcements.

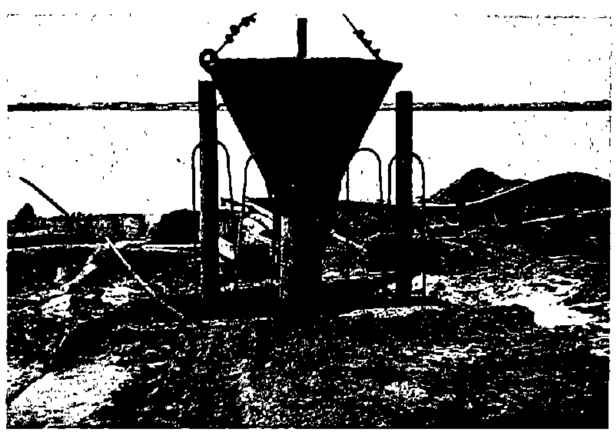


Fig. 103. -- Lifting loops for reinforcing cage for a barrette.

Fig. 104. -- Acoustic investigation tube simply welded to sizing hoop. Not recommended!

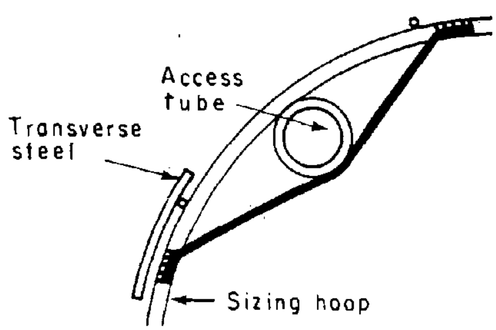


Fig. 105. -- Correct mounting of acoustic investigation tube.

In the following pages, an example is given that illustrates the minimum information a drawing of pile reinforcing should contain (Fig. 106).

5.5. LOADING - TRANSPORT - UNLOADING - STOCKING OF REBAR CAGES

Handling of the cages during their loading, unloading, and placement in the boreholes should be carried out with concern for limiting deformations as much as possible and to prevent failure of the welds. A cage which is mistreated during these operations will remain permanently deformed. It is thus necessary to take the essential precautions to avoid consequences which would result from deformations (difficult lowering of the cage in the borehole, scraping of the walls, poor cover of the rebar, etc.).

5.5.1. Loading with transport in mind

It is necessary to avoid lifting the cage by one or two points with simple connector links especially when the cage is long and of a large diameter.

-- Lifting at the center (Fig. 107a) creates significant deformations and requires a rope support at each end.

-- Lifting at two points (Fig. 107b) without a stiffening beam is no more desirable because the deformation of the cage is accentuated by the effect of the compressive stresses exerted between the points of attachment of the connectors.

The most satisfactory method is that which consists of lifting the cage with a stiffening beam (Fig. 108). The cage is thus supported at several points, which results in the reduction of deformations.

5.5.2. Transportation

The transportation of the rebar cages is not cost effective if it must be treated as a "non-standard" load by the highway codes.

The highway codes in France do not require special provision for hauling if the length of the vehicle, all projections included, does not exceed the values given below:

-- 11 m (36 ft) for automobiles and trailers, not including coupling devices,

-- 15 m (49.2 ft) for vehicles joined by moving parts (tractor + trailer)

-- 18 m (59 ft) for a group of vehicles (truck + trailer).

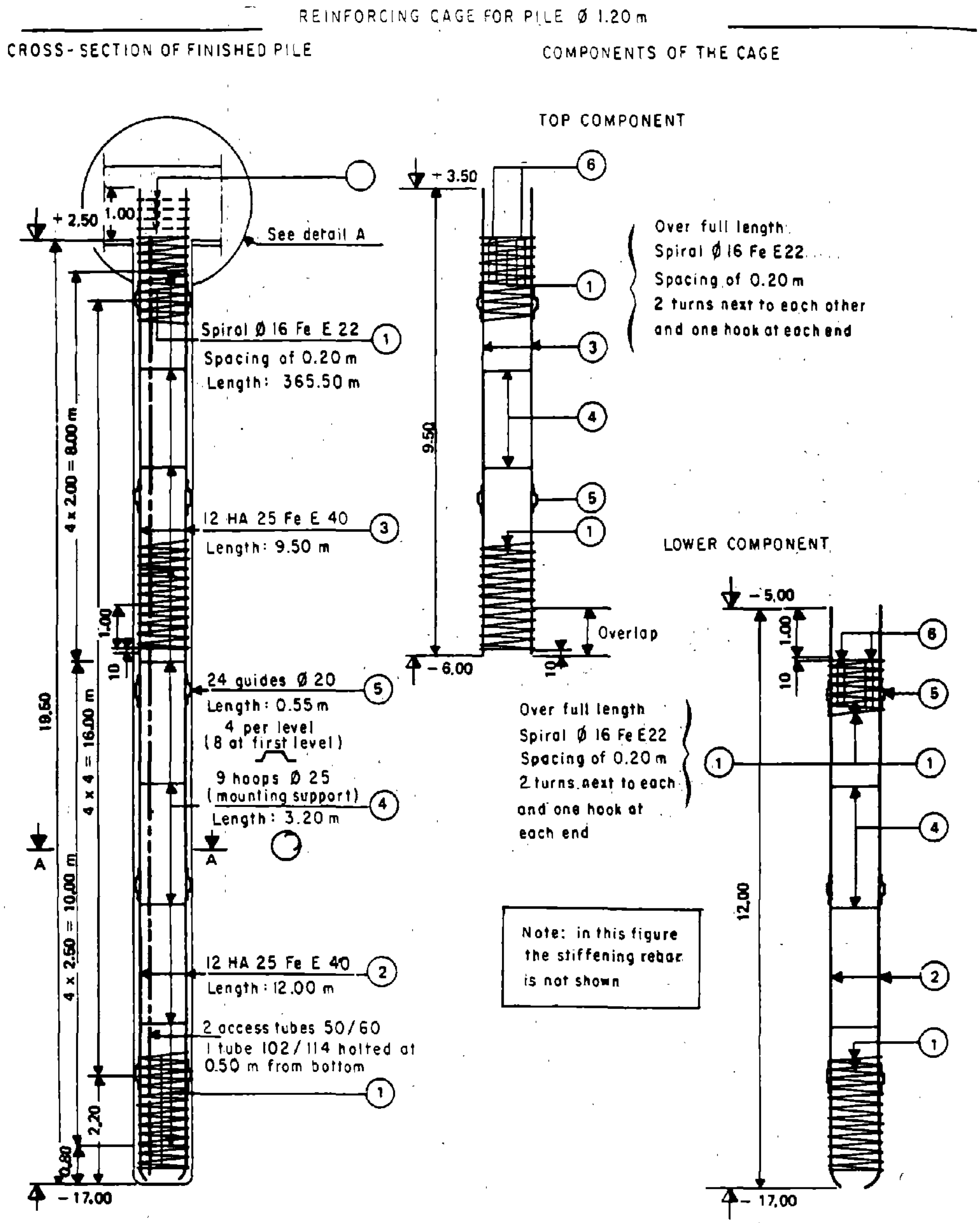
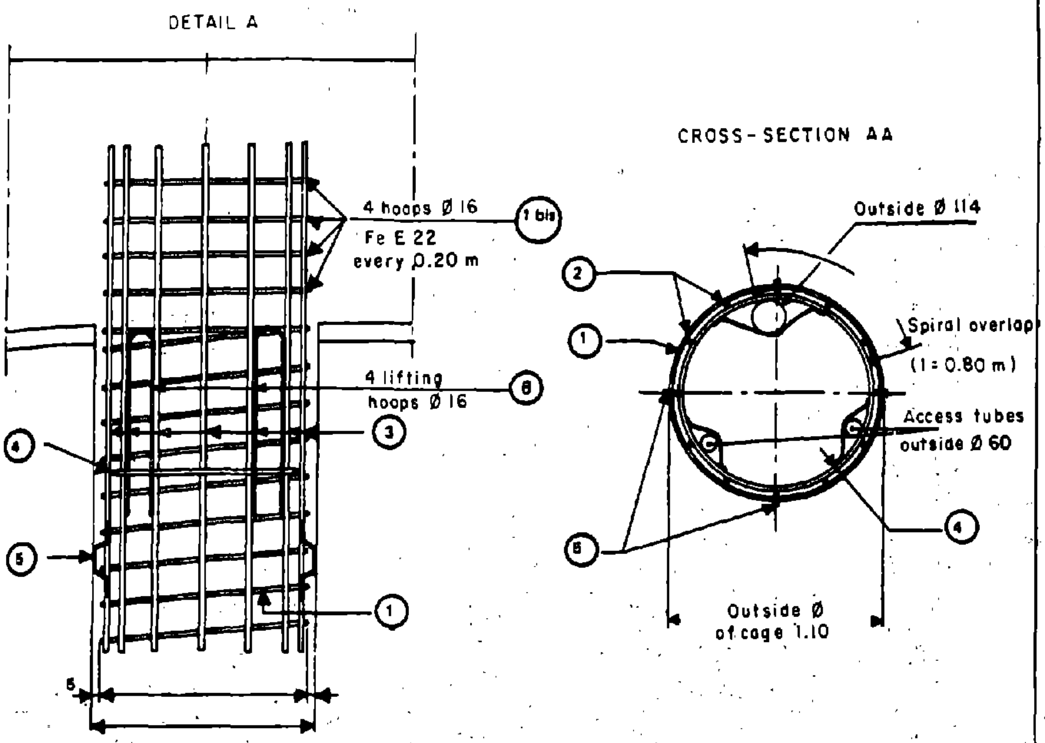


Fig. 106. -- Example sketch for pile reinforcement.



N°	Diameter Deformed	Diameter φ	Number of Bars	Formation Remarks	Length per bar	Total Length	Wt. per m	Tot. wt.
1		16	1	Inside Ø 1.06[4] 20 20,50	386,5	365.5	1.578	577
1 bis		16	4	Inside Ø 1.06[4]	3.44	13.76	1.578	22
2	25		12		12.00	144.00	3.854	554
3	25		12		9,50	114,00	3.854	439
4		25	9	Inside Ø 0.97	3.20	28.80	3.854	111
5		24	20	10 10 15 10 10	0.56	13.20	2.466	32
6		16	8	25,5 1.00	1.77	14.20	1.578	20

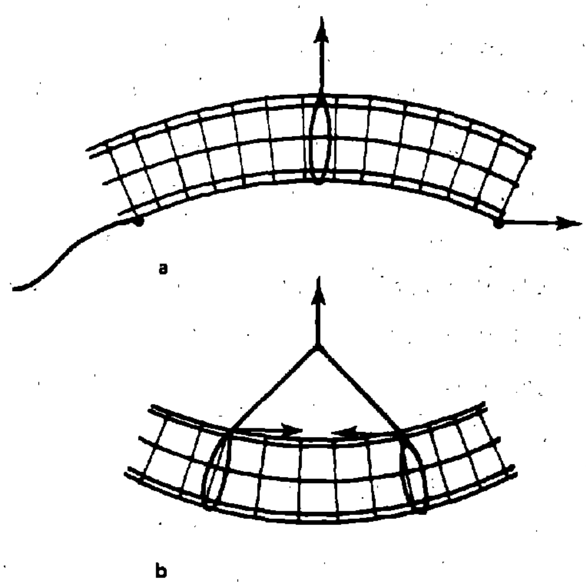


Fig. 107. -- Deformations of cage in lifting.

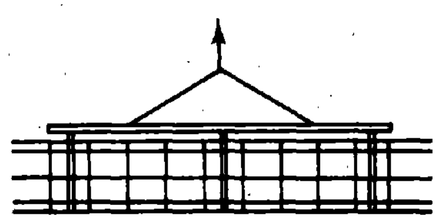


Fig. 108. -- Recommended use of stiffening beam.

In addition, the load should not exceed the rear extremity of the vehicle by more than 3 meters (9.8 ft).

By respecting the above requirements which apply to trailer lengths varying from 9 to 11 m (29.5 to 36 ft), it can be seen that it is possible to transport cages of lengths from 12 to 14 m (39.4 to 45.9 ft), which correspond to current commercial lengths. Under exceptional cases (modification of transportation equipment, proximity of job site), it is possible to transport cages of slightly greater lengths.

5.5.3. Unloading and stocking on the job site

Unloading should be performed with the same precautions as loading. This operation is unfortunately often poorly conducted. The unloading is done at the start of the job when the preliminary excavations are being performed and when the lifting machines are few and often absent. The unloading unfortunately too often consists of pure and simple dropping of the cages on the ground; they are consequently soiled and deformed.

The rebar cages should be stocked in clean areas (Fig. 109), on concrete if possible; they should be set down on wooden blocks to limit deformation and to separate them from the soil. As much as possible, it is necessary to avoid piling the cages on top of each other. Figures 110 (a) and (b) show exactly what not to do.

5.6. ERECTING OF THE CAGE AND PLACING IN THE BOREHOLE

5.6.1. Erecting of the cage

This involves the lifting up of the stocked cage from a horizontal position and bringing it to a vertical position over the borehole. This is the most critical operation. The cage is generally hooked at one or two points (Fig. 111). Before the cage leaves the ground, it rests on its end and is subjected to significant bending deformations. To these bending deformations torsional deformations are added when the cage, suspended by the crane, swings above the ground. To perform this maneuver in a proper manner, it is desirable to employ a rigid support to which the cage is attached at several points.

5.6.2. Placing of the cage in the borehole (Figs. 112 through 117)

The operations of erecting and placing the cage in the borehole should be done without loss of time so as to limit the sedimentation, floculation, or settling out of the soil in suspension that may occur before concreting.

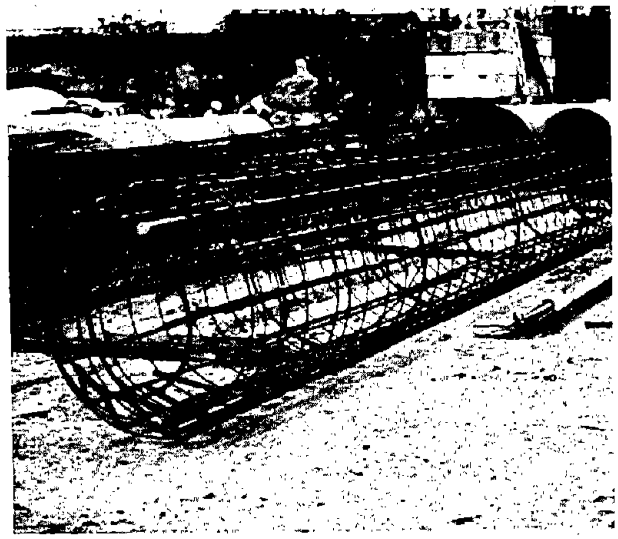
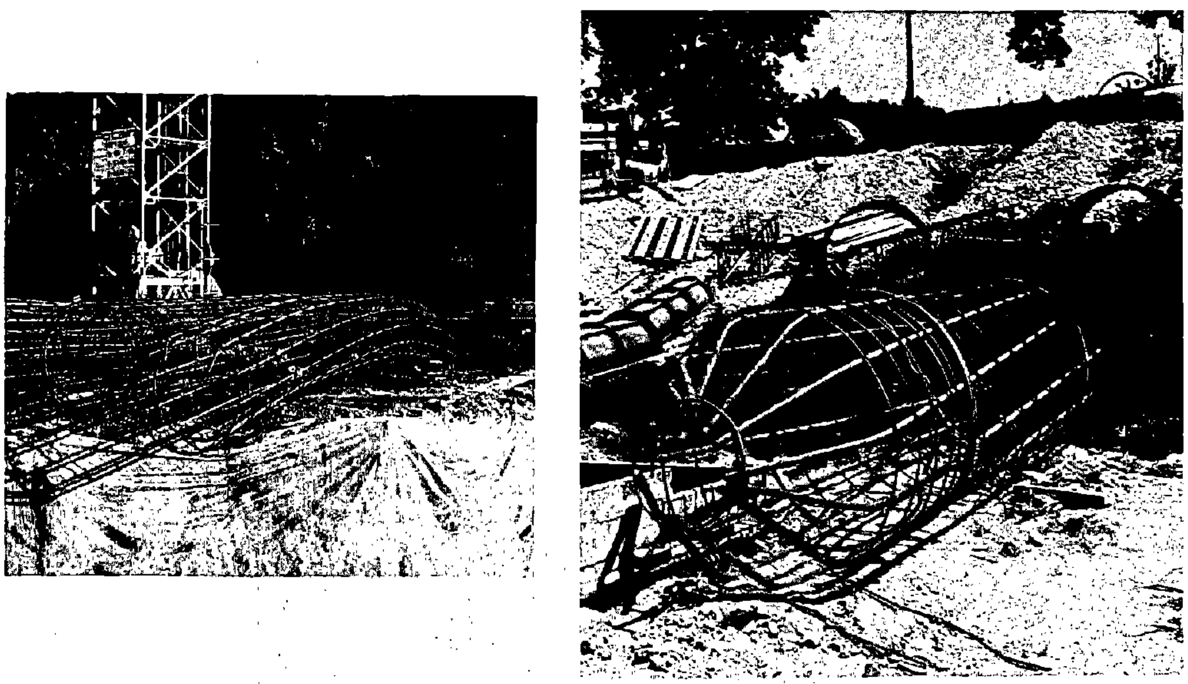


Fig. 109. -- Proper storage environment for reinforcing cages.



(a) (b)

Fig. 110. -- (a) and (b) Unacceptable storage.

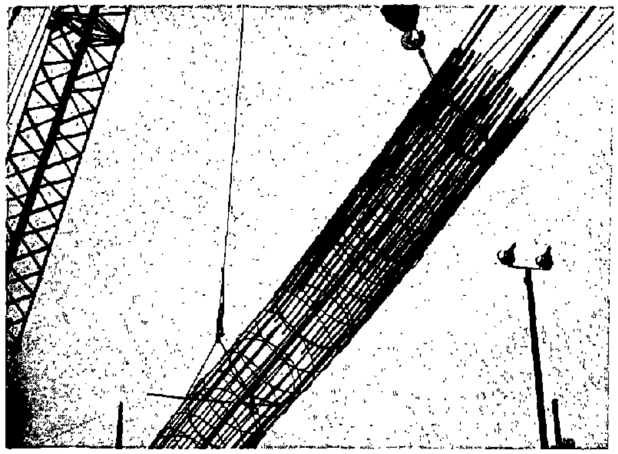


Fig. 111. -- Lifting of reinforcement cage.
Note deformations.

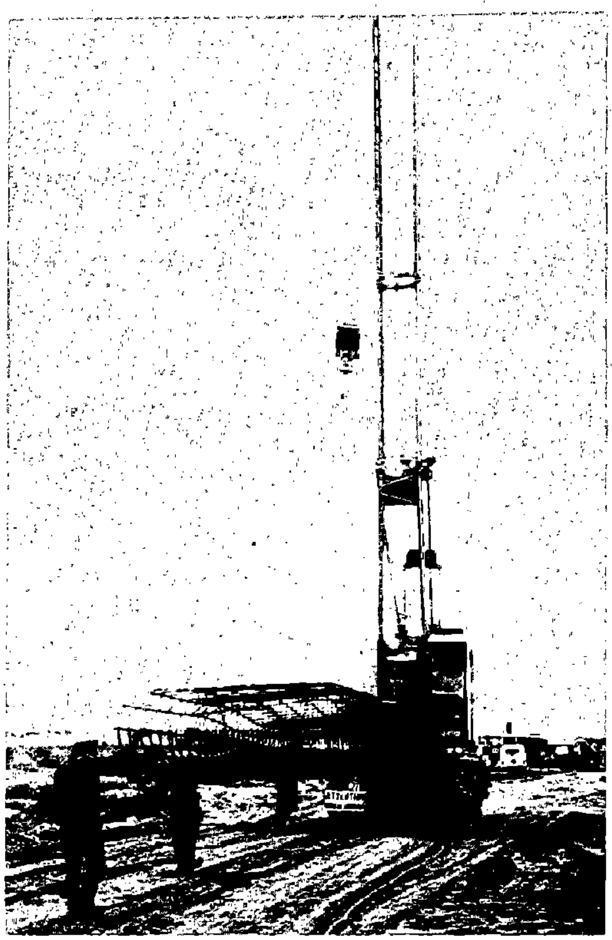


Fig. 112. -- Handling of cage for barette.

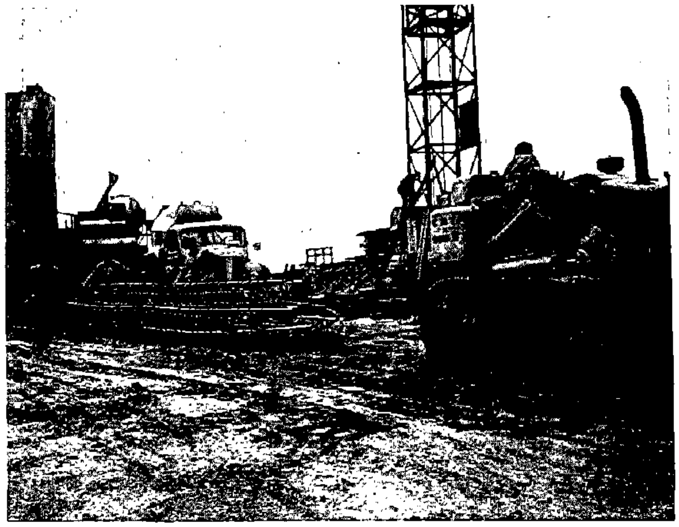


Fig. 113. -- Transportation of cages on job site by sled.

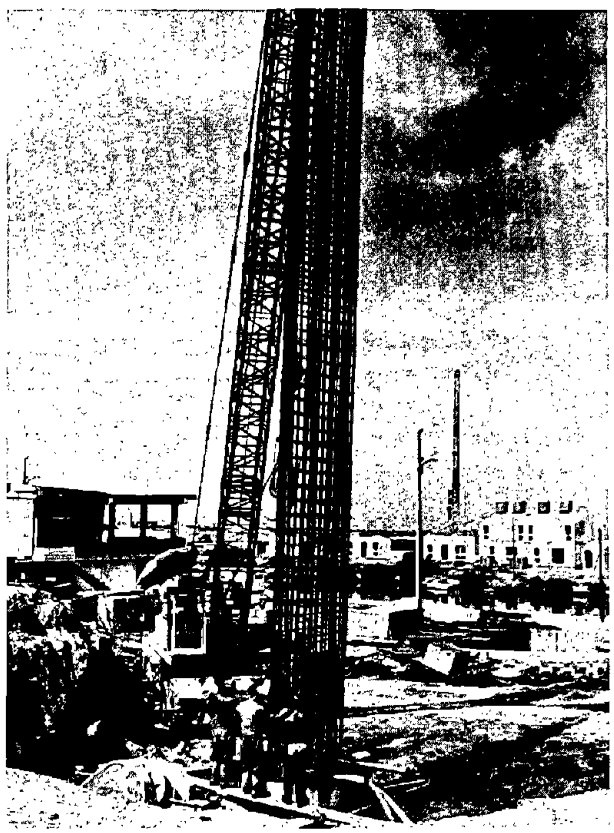


Fig. 114. -- Lifting of reinforcing cage.

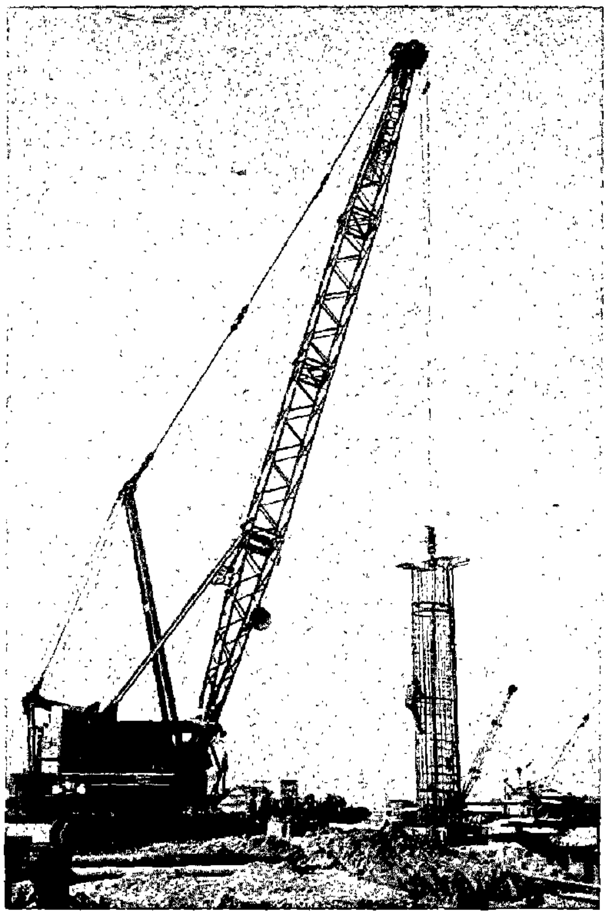


Fig. 115. -- Lowering of cage into borehole.

Fig. 116. -- Device for maintaining cage in borehole.

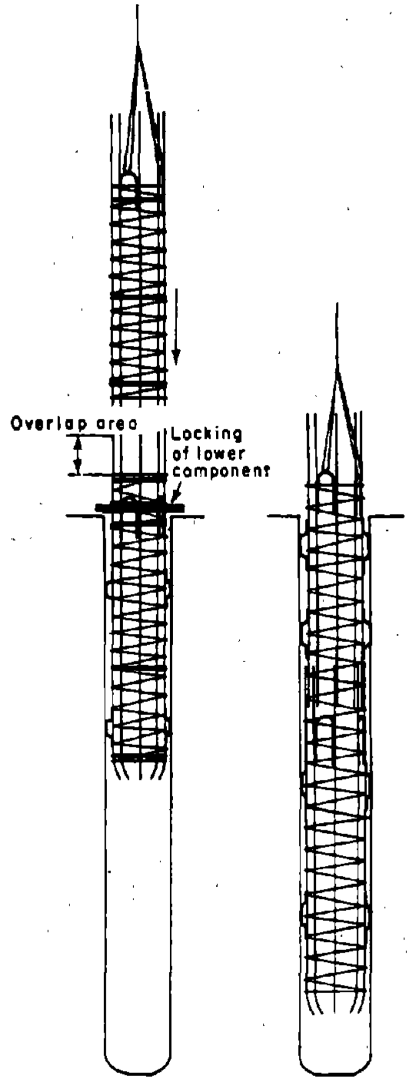


Fig. 117. -- Phases of placement of two sections of reinforcing cage.

-- Cleaning of bottom of hole.

-- Slow lowering of first component into borehole up to its upper level (overlap area) or to level of a man.

-- Locking with large brackets.

-- Introduction of second component and achievement of overlap (welding of longitudinal bars placed side-by-side or placed with special connectors).

-- Slight raising of two integral sections.

-- Unlocking and lowering of outfit.

-- Repetition of operations in such case when other components require joining.

-- Verification of the upper level of the placed cage.

-- Verification of the bottom of the borehole.

-- Locking of cage at upper portion to avoid its descent or its rising during concreting (weld on collar, or anchorage).

-- Concreting of pile.

There are few methods to find out if caving is produced at the time of the lowering of the cage. In the special case where the borehole is dry, the mirror procedure may be used. With bentonitic mud, the only method of detecting caving is the plumb line. In any respect, when the presence of significant caving is discovered (surface caving, obstacles impeding the lowering of the cage to its final position...), the construction supervisor should not hesitate to have the contractor pull the cages and then to clean out the bottom of the borehole, or to require the bottom of the excavation cleaned to conform to the recommendations presented in Chapter 3.

To guard against the possible penetration of the rebar into the bottom of the borehole, and against risks of buckling, it is preferable to suspend the rebar cage from the top of the temporary casing, or of the surface casing, rather than setting it down on the bottom of the borehole (Fig. 116).

5.7. PROBLEMS - OBSERVATIONS - REMEDIES

5.7.1. Soiling of the rebar

Some precautions should be taken during stocking and handling of the cages in order to avoid soiling of the rebar. Cleaning with a water jet may be required. Contamination of the rebar during lowering of the cage into the borehole (scraping of the wall) is inhibited by the use of guides and spacers.

5.7.2. Centering errors

Errors in the centering of the cage are due to an excessive flexibility of the cage and above all to the insufficient use of guides. In the case of battered piles it is necessary to increase the number of levels of centering guides and spacers on the lower face which has a tendency to lean against the wall of the borehole.

5.7.3. Waterlogging of the concrete due to the presence of the "basket"

We have previously indicated that, at the base of the pile, the longitudinal reinforcements should not be turned in toward the center. This "basket" prevents the tremie from touching the bottom of the borehole during the initial placement of the concrete and the "basket" can ultimately behave like a grate that leads to segregation of the aggregate and weakening of the concrete in the presence of water.

5.7.4. Raising of the rebar cage during concreting

During concreting, the reinforcing steel is submitted to the uplift forces from the fresh concrete (Fig. 118); these forces are increased if there is some setting-up of the first concrete. The uplift forces are even greater when, in the commendable effort to prevent contamination of the concrete, the tremie is embedded too deeply in the column of fresh concrete.

To avoid this sometimes significant lifting of the rebar cage (up to 2 m - 6.6 ft), which is possible regardless of the weight of the cage and even if it is restrained at the head, it is desirable then to limit the space between the base of the tremie and the level of the concrete in the borehole. This should be a reasonable distance and is compatible with the security required in light of the risks of separation of the tremie pipe from the surface of the concrete (see § 6.4.1.1 use of tremie to place concrete).

For information only, a single shortening of 6 to 7 m (19.7 to 23 ft) of the tremie after concreting of 10 m (32.8 ft) generally permits achieving, without difficulties, piles of the order of 20 m (65.6 ft) and above (Fig. 119). On the other hand, it has been seen that the presence of a "basket" at the base of the rebar cage presents little advantage in solving this problem and, on the contrary, risks disturbing the quality of the concrete-soil contact at the base of the pile (see § 5.3.4).

5.7.5. Lifting of the rebar cage during the pulling of a temporary casing

If the space between the rebar cage and the temporary casing is small, there is a possibility of lifting the cage by simple friction or the locking of particles of coarse aggregate between the rebar cage and the wall of the temporary casing. These disadvantages are accentuated by poor centering and by excessive flexibility of the cage (Fig. 120). They are also more frequent when the pile is lined.

5.7.6. Settlement of the rebar cage into the concrete

Due to forces from fresh concrete that moves downward with respect to the rebar cage, the cage can be subjected to significant compressive stresses. Special precautions should thus be taken to allow proper performance of the cage during the concreting phase (see especially in § 5.2.2, the recommendations for transverse reinforcing). If these precautions are not taken, the longitudinal rebar can buckle, the cage

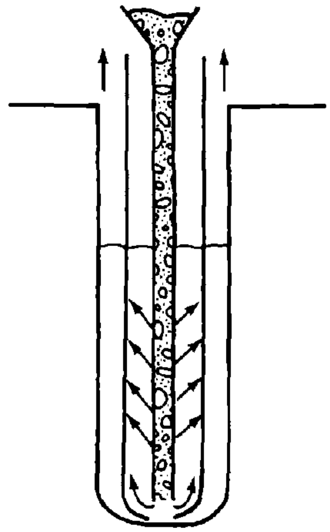
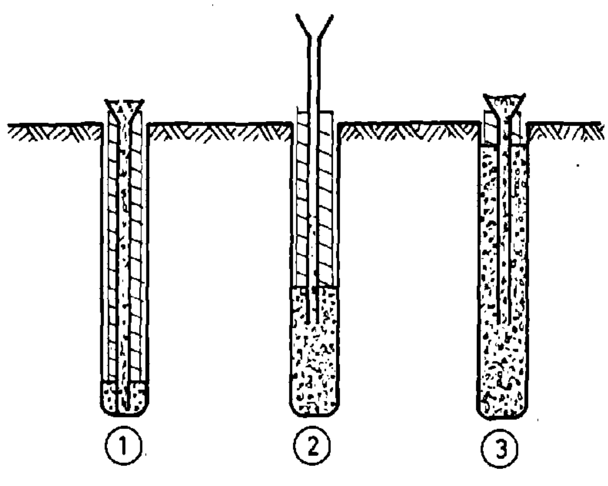


Fig. 118. -- Uplift pressure on reinforcing cage
due to tremie concreting.



1 Priming and start of concreting.

2 Halt of concreting and raising of pipe.

3 Continuation of concreting after dismantling of one or two sections of the pipe.

Fig. 119. -- Proper tremie-concreting method
to avoid raising of cage.

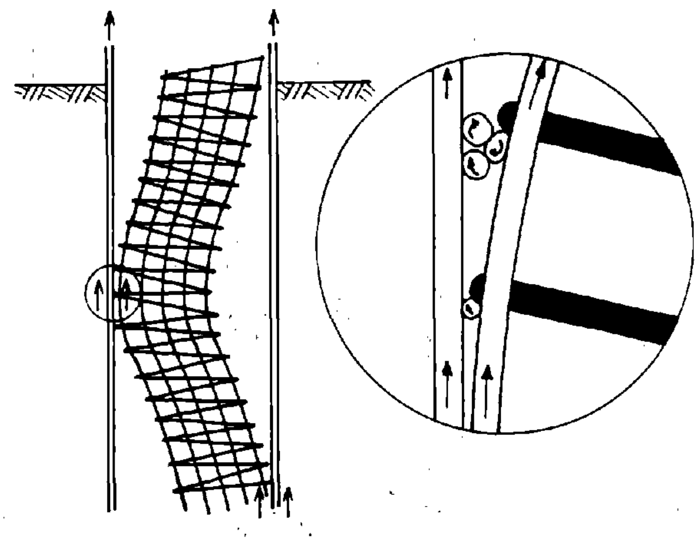


Fig. 120. -- Mechanism resulting in pulling cage along with raising of temporary casing.

undergoes large deformations and distorts, and the welds may break (especially at the lap joints).

The problems are more frequent when the pile is constructed with a temporary casing. Especially at the time of the extraction of the casing, the concrete must fill not only the space occupied by the casing but must fill all gaps. Of special concern is *when the extraction of the casing is accompanied by vibration*(vibration-installed piles for example). It is advisable in such cases to extract the first meters of the casing in stages (halt of vibration for one minute approximately every 20 cm - 8 in.), as various observations on an important highway job have proven [60].

Figures 121 and 122 illustrate to what extent a cage may deform. The components encountered by the coring are the longitudinal rebar in one case and a pipe for acoustic investigation in another case.

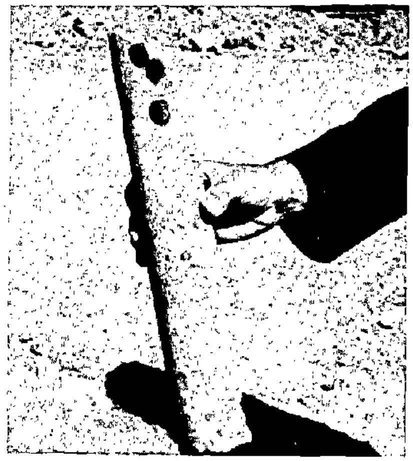


Fig. 121. -- Core taken from axis of pile ϕ 100 mm. (4 in.)

Fig. 122. -- Coring through acoustic-investigation tube that followed deformation of reinforcement cage.

CHAPTER 6

CONCRETING

A concreting job for cast-in-place piles is different from a concreting job for superstructures. The concrete should, in effect, have very different qualities. Even though usually lightly stressed, it should continuously transmit to the soil the loads that it supports and thus cannot contain any discontinuities within its mass. The concrete used in cast-in-place piles must have very special qualities, its elements should compact by their own weight and not by vibration as is the case with concrete used in superstructures. The concreting of a pile is a very precise operation that must be carried out with extreme care.

For large projects, it is possible to use advanced methods of construction and to carry out sophisticated construction tests. These methods often become very expensive and greatly increase the cost of small jobs. Therefore, it must be kept in mind that the consequences of a pile defect do not depend on the importance of the project.

Experiences from large-scale testing of piles, confirmed by field observations on job sites, indicate that *if the state-of-the-art is followed, the rate of success will be very high, but if it is neglected construction defects will arise.* Therefore, the contractor should ensure that the concreting operations are carried out under the best possible conditions.

This chapter examines:

-- concretes for piles,

-- quality control and progress samples,

-- production and transportation of concrete,

-- placement, and

-- organization of a concreting test.

6.1. CONCRETES FOR CAST-IN-PLACE PILES

Today, there are several mixes called "deep foundations" (or QF) that have been studied for the purpose of obtaining the special characteristics needed for concrete used in piles. These concretes can be placed by the tremie method or by the use of a pump, depending on the project. Some ready-mix concrete plants also provide these mixes. These concretes may be adapted to site conditions by the possible additions of plasticizers or set retarders.

Unfortunately, in current practice, there is too much improvisation in attempts to adapt a reinforced-concrete mix by adding sand and water and reducing the amount of gravel. The generally ineffective result leads to a difficult and uncertain placement which might adversely affect the quality of the pile.

The problem of concretes for piles is examined from two angles:

-- basic characteristics of concretes for piles, and

-- conditions necessary to obtain those characteristics.

6.1.1. Basic characteristics of concretes for cast-in-place piles

These characteristics are as follows:

-- fluidity, good flowing property, and ability to compact under self-weight,

-- resistance to segregation and to leaching,

-- slow and controlled setting,

-- resistance to harsh environments through high density and good impermeability, and

-- good mechanical performance.

It should be noted that, unlike the concrete for superstructures, high mechanical performance is not the main objective.

6.1.1.1. Fluidity, property of flowing and compacting under their own weight

The fluidity (or workability) of concrete should be sufficient to allow the concrete to fill all the gaps when it rises in the borehole due to the difference in pressure between the levels of the concrete in the tremie pipe and the excavation. Fluidity also depends on the amount of time spent on the concreting operation.

It is essential that an acceptable fluidity be obtained by means that will not adversely affect other concrete characteristics.

In this manner, an increase of the amount of mixing water should be avoided because this easy way of improving the fluidity also increases segregation and decreases compactability, impermeability, and strength of the concrete.

6.1.1.2. Resistance to segregation and leaching

All falling concrete tends to segregate. The large elements reach the bottom first. This phenomenon is accentuated by the presence of obstacles and by a lack of cohesion in the concrete due to a high water content (concrete too soft). The mortar adheres to the reinforcement and to the walls of the excavation. The presence of water in the borehole also increases this segregation problem: there is a separation of the cement and the aggregates by washing of the latter. The leaching is thus one form of segregation to which piles cast under water are particularly susceptible.

Segregation causes very serious defects within the pile shaft: gravel pockets, caverns, porous concretes. These zones are particularly susceptible to groundwater flow which again accentuates the leaching of fresh concrete and the harshness of the environment in terms of the compactibility (§ 6.1.1.4).

6.1.1.3. Slow and controlled setting

Certain operations, such as extraction of the temporary casing and the tremie, especially when these operations are not done until directly after placement, involve a rapid concreting operation and a properly retarded set. In other respects, it may be useful in the case of certain concreting setbacks, to extract the already-poured fresh concrete.

It is thus imperative to control the beginning of concrete set and its timing through judicious choice of cement and the possible use of a carefully proportioned set retardant.

6.1.1.4. Resistance to an aggressive medium through high density and good impermeability

Density and impermeability go hand in hand. The concrete may be subjected to aggression of the environment by movement of water that is loaded to varying degrees with salts and organic materials or, on the contrary, by very pure water that may mechanically or chemically attack the concrete. It is evident that the concrete has a better resistance when these waters do not penetrate it, and in particular when water movements do not reach the reinforcement. A thick and dense concrete cover can prevent damage to the reinforcement by the corrosion-cell effect.

Unfortunately, proper density and impermeability are difficult to obtain because the high water content of concrete for piles, which cannot be vibrated to an optimal density, counteracts the start and growth of a dense crystalline network at the time of the setting of concrete. In piles the fresh concrete compacts under its own weight and this results in an increasing density with depth. This has been further evidenced by gammametric studies (see § 7.1.1.3).

6.1.1.5. Good mechanical performance

Even though the principal objective is not to obtain high mechanical strength it must be remembered that proper performance should allow for reduction of the dimensions of the piles or shafts and thus their cost. This reduction of size (or optimization) of the shaft should depend on the construction conditions and the current recommendations of the directives. However, the tendency must be avoided to require in the CCTP strengths that are proposed in the commentary of Article 9, § 7 of Volume 61, Title VI. It could appear that reinforced concrete is involved because rebar cages are present.

High cement proportions (350 to 400 kg/m^3 - 590 to 675 lb/yd^3) are justified here by the need to obtain a good flowing property and, as a result, a good density in place without the aim of attaining superior mechanical strengths. A high cement proportion actively contributes to confidence in the different operations of placement. This is the way to neutralize the detrimental action of an excessive amount of mixing water.

Practically speaking, in the absence of specific rules concerning the concrete for piles and considering the maximum stress allowed by the present CPC (Article 38.2 of Volume 68), satisfaction is obtained with a 28-day compressive strength of about 250 bars (see [3], § 8.973). However, the special case of pile columns must be the subject of a specific review.

6.1.2. Conditions necessary to obtain those characteristics

These conditions are expected to be met if there is a careful selection of the concrete components and an in-depth study of its design.

6.1.2.1. Selection of components

The selection of the components of concrete cast against soil should be carefully made. This pertains to cement, aggregates, and admixtures:

-- *The cement* should be capable of resisting the chemical aggression of the environment: waters loaded with organic matters or mineral salts, acid waters, selenitic, or pure. It should be noted that the aggressiveness of different waters will vary depending on the nature or the concentrations of the elements dissolved in it. For example, natural-acid waters loaded with carbonic gas or with humic acids may be encountered. This carbonic gas is very detrimental to cements because it can combine with free lime to form very soluble lime salts that decalcify the concrete. Equally of concern is the presence of sulfated waters (selenitic, magnesian waters) that attack cements by formation of highly expansive, complex salts that may cause concrete disintegration. Overly pure waters are dangerous because of their decalcifying character.

To select the cement, it is necessary to consult a specialized laboratory that is familiar with regional products and current literature on the subject:

- Volume 3 of the CPC: Supplies of hydraulic cements,
- list of hydraulic cements intended for marine works and in selenitic waters, published and annually reviewed by the "Ministere de l'Equipement," and
- practical guide for the use of cements by M. Adam (collection of the ITBTP [73]).

Cements that meet the physical and chemical characteristics given in Volume 3 are in general those with large slag percentages: CPF, CHF, CLK, CPMF, and only some CPA, CPAL, CPALC, having a good resistance to aggressive waters.

It is necessary to eliminate those cements known for their quick setting and, for those chosen, to ensure by means of acceptance testing that they are not subject to false or overly rapid setting.

-- *The aggregates* should preferably be rounded and possess the qualities specified in Volumes 65 and 23 of the CPC: in particular, a continuous, granulometric curve and a good shape coefficient are necessary to facilitate good concrete flow. A siliceous round sand is desirable.

When the use of crushed sand and gravels is inevitable, a constant percentage of fine elements should be required for the sands (the composition study will give the percentage that should not be exceeded) and a good shape coefficient for the gravel.

Porous aggregates must be rejected.

-- *The admixtures,* setting retarders or plasticizer-retarders, should be selected from the list in the ministerial circular "approved list of concrete admixtures." This list is updated annually.

6.1.2.2. Study of the mix design of the concrete

The study of the composition of the concrete should be performed as related to the required qualities of concretes for piles. It should particularly take into account the aggressiveness of the environment, as well as the means of production, transport, placement, and concrete controls. This composition should not be improvised. Rather, its related study and testing should be entrusted to a specialized laboratory.

It is necessary to seek fluidity (good flowing property) and density at the same time.

To obtain a good uniformity in the composition of concrete and the best possible density and workabilities, it is necessary to use at least three sizes of aggregates.

Therefore, it is a proper balance between the proportions of the different sizes of aggregates and not an inappropriate addition of sand in place of gravel that will best reconcile these two apparently contradictory qualities. The optimum proportions are determined by a thorough study of the workability of concrete.

Density and resistance to segregation (and to leaching) may be improved by increasing the proportion of very fine elements (between 80 and 160 microns) when a very "coarse" sand is used. These very fine elements may be provided through the addition of fines (crushed limestone, fly-ash, ...) or by use of a sufficiently high quantity of cement (400 kg/m^3 - 675 lb/yd^3). The latter method is particularly desirable due to a resulting tighter crystalline network for the concrete, without additional risk of cracking since it involves "buried" concrete.

Any significant increase in the water content to increase the fluidity of concrete should be considered with much reservation because of its detrimental effect on the density and mechanical strength. It is better to use a plasticizer. It should be noted that when special cements are employed (those with large proportions of additives) that are commonly used in foundations, the effect of approved additives must be verified by an appropriate study.

The use of retarders to control the set and to avoid any stiffening of the concrete before the end of operations has a positive effect, even more so because those admixtures generally have a secondary plasticizing effect. However, the set-retarding action should be verified and standardized in the laboratory at the time of the concrete-composition study. When a significant set retarder is not required, a plasticizer may be used that will have a secondary retarding effect.

The general outline for the study of the concrete composition consists of five main phases:

-- *Theoretical study* in which the mixtures with continuous grain size curves, that favor the flow of concrete (Fig. 123), are recommended.

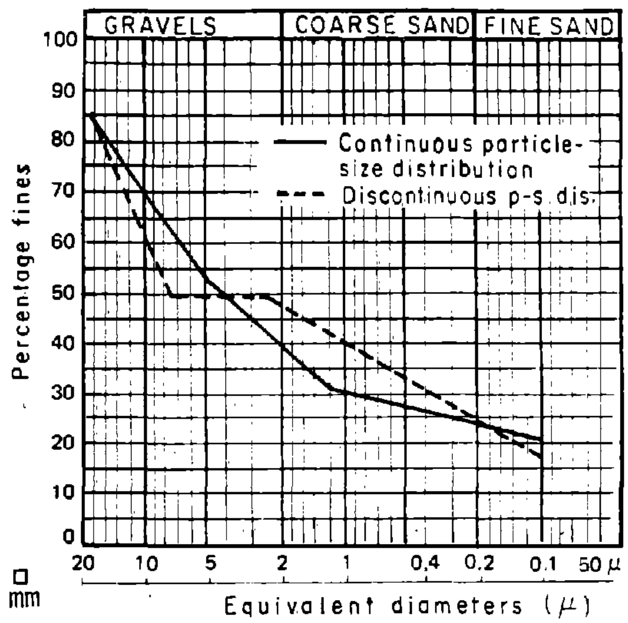


Fig. 123. -- Examples of well-graded and gap-graded grain-size distribution curves.

-- *Investigation for an optimum workability,* with a workability meter LCL[1]. This must be performed with a constant water content (W/C = 0.46) and by varying the sand percentage in comparison to that of the gravel.

-- *Investigation for the optimum water content* necessary for proper flow of the concrete in the tremie pipe and a good placing operation. This is done with a fluidometer (Fig. 124) from the design established in the previous phase. The use of the slump cone in the laboratory is not recommended because this procedure, in cases of high slump (18 to 22 cm - 7 to 8.7 in.) looses all sensitivity. The slump is not representative of the fluidity of the concrete.

[1] LCL: Laboratoire Central-Lesage.

-- *Study of the setting times of the concrete*, when using a set retarder or a plasticizer-retarder in the concrete, with a concrete setmeter (Fig. 125).

-- *Measurement of the mechanical strengths* and possibly the density and permeability of samples of hardened concrete.

6.2. TESTS OF CONCRETE SUITABILITY

When the study of the design of the mix of the concrete is completed, it is necessary to verify that the contractor has the means to produce a concrete having the same characteristics as that used in the study and to carry out a satisfactory placement of the concrete in the borehole. For this, a truly large scale test is carried out, also known as "control concrete," as defined by Article 8.33 of Volume 65 of the CPC (Construction of Works in Reinforced Concrete). The following methods are perfectly adaptable to concrete for bored piles:

-- *Examination of the arrangements made by the contractor concerning the preliminary verification of production equipment, transport, and placement:*

Production plant:

- storage of equipment (cement silos, aggregate bins, conveyors, etc.);
- measurement apparati (scales, flowmeters, etc.);
- control devices (automatic mechanisms, humidimeter, etc.).

Transportation equipment:

- mixing trucks;
- dumpers;
- concrete pumps;
- other means.

Placement equipment:

- tremie pipe with accessories;
- tremie pipe cross-head for concrete pump;
- buckets;
- tools for extraction of concrete;
- pumps;
- etc.

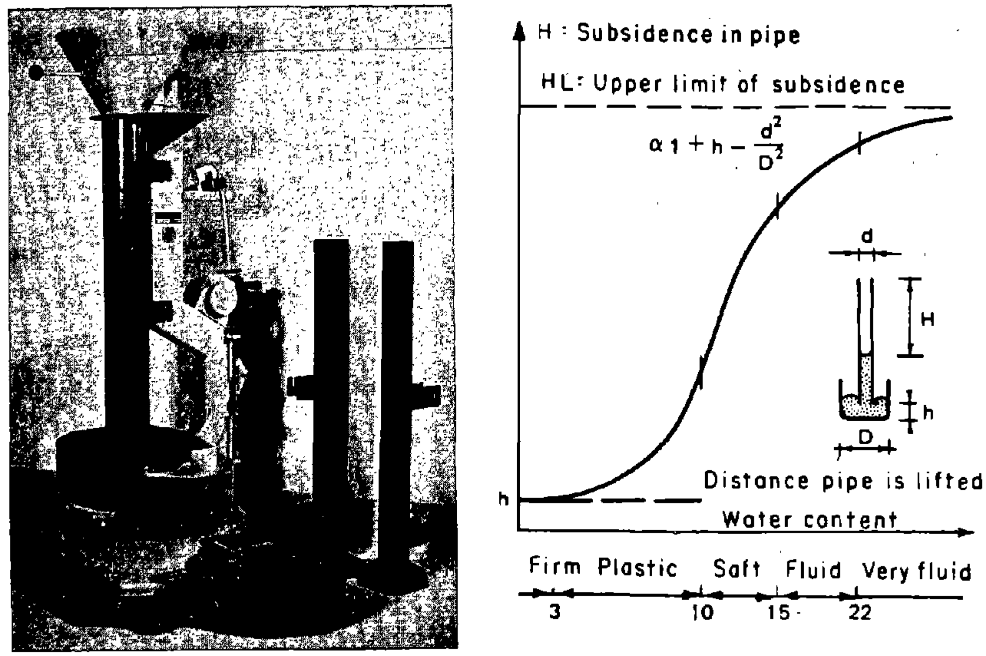


Fig. 124. -- The fluidometer permits determination of optimum water content of a concrete for a pile.

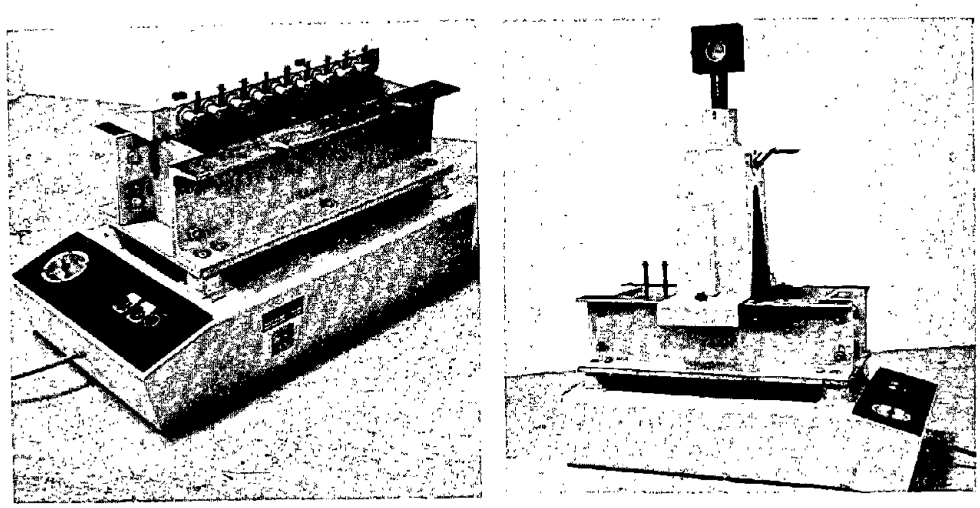


Fig. 125. -- Concrete setmeter for study of proportion of set retarder.

It is desirable to verify that energy sources at the site (electric connections, generators, compressors, etc.) are adequate and that they are reliable in case of a breakdown.

-- *Batch production, with the means at the site and with facilities for verification of the batch characteristics:*

On fresh concrete:

• flow characteristics using an Abrams slump cone in the current absence of a better method,

• analysis of the composition of the fresh concrete, and

• yield (the composition that produces a given volume of concrete on the job).

On hardened concrete:

• physical and mechanical characteristics (density, impermeability, strength).

-- *Placing of one or more batches.*

This placement test should be performed with the setup that will be used at the job site: supply system, tremie pipe, concrete pump...

It is necessary to anticipate the arrangement for a test borehole. This may be one of the boreholes of the project because construction equipment for piles, especially on the site for a small job, is generally not available except for a very limited time. In this case it is preferable whenever possible to make use of one of the least stressed piles and it is wise to determine beforehand the possibility of its replacement if it is rejected due to problems. In the case of large jobs, a special test borehole may also be prepared to receive the test concrete. This is constructed under the same conditions and at the same depth as the production piles. These test boreholes are obviously equipped with rebar cages that are fitted with the required acoustic-investigation devices (see Chapter 7).

6.3. PRODUCTION AND TRANSPORT

6.3.1. Production

It is useless here to review the rules for production of the concrete that are exactly the same as those applicable to structural concrete which are summarized in:

-- Volume 65 of the CPC,
-- Circular No. 73-91 relative to the use of factory-manufactured concrete (updated periodically), and
-- the Guide, level 2 of the GGOA 70 [3].

It is necessary, however, to give attention to the need of combining good fluidity with proper density when dealing with concrete for piles (see § 6.1.1).

The selected consistency will greatly influence the density. Therefore, it is necessary to monitor the plant very closely and to ensure a permanent control of the plant's production. For this reason the use of unreliable concrete mixers or site plants should be absolutely avoided. If a properly equipped production plant cannot be supplied on the job site, it is preferable to have access to an approved ready-mix plant, conforming to the rules issued in Circular No. 73-91 (quoted earlier).

It is essential that the plant produce the concrete with the consistency required during the pouring operation or with a slightly higher fluidity in the case of high temperatures or where transport time is considerable. *Any systematic addition of water to the truck upon arrival to the job site should be prohibited because it causes segregation* of the concrete. The truck is no more than a rudimentary mixer that is very effective when the fluidity of the concrete is increased. Preventing the addition of water is particularly difficult to accomplish for plant-manufactured concretes, because plants prefer to deliver very dry concretes for the following four reasons:
-- better resistance at the time of the control approval (which is done upon departure from the ready-mix-concrete plants),
-- possibility of loading larger amounts of the mixture, which allows a limited number of rotations of the drum,
-- easier unloading, and
-- greater overturning security during transport.

In this regard, mention may be made of the appearance in the Paris vicinity of a new system capable of avoiding the inconvenience inherent to transportation. It involves a mixing-pump which, on site, raises the water content of an almost dry mix that is transported from a plant by a mixing truck.

Finally, it is necessary to emphasize the special importance of precise control of the proportions of admixtures. This is especially important because there is a perfectly justified tendency to work more and more with longer set delays; in other words, to work with very near to an overdose of the retarder that may cause permanent blocking of the concrete setting.

6.3.2. Transport

It may be that the concrete plant is sufficiently close to the pile project for the concrete to be transported by means of buckets or be pumped. Transport in dumpers should be prohibited because of the segregation that results in the fluid concrete. In most cases, however, transport is carried out by transit-mix trucks.

As a general rule, very special attention should be given to organization of the transport of concrete in order to satisfy the two following conditions:

-- short transit time, and

-- no supplying interruptions, nor waiting of the concrete mixers at the site.

These two conditions imply the installation or selection of a plant located close to the job and good organization of the circulation of the trucks.

Any unanticipated and significant delay in transport, any supplying discontinuity and any truck waiting increases the risks of premature hardening of the concrete. These setbacks are generally catastrophic for the pouring operation. Because unloading is often slow when using the tremie-pipe technique, it is thus desirable to have access to concrete mixers of low to medium capacity (4 to 6 m^3 - 5 to 8 yd^3).

Because the concrete for piles is very fluid and sensitive to segregation, there is an increased need for the transit-mix trucks to be always in the very best condition. It is also necessary to have access to a good system for transmitting orders and information (radio for example).

Finally, it is necessary to consider the case of a concrete pump used as a relay between the concrete mixer and the classic tremie pipe (see [3], level 3) and the use of pumps for the transport of concrete. This use is particularly critical because the concrete should be fluid enough to descend easily in the tremie pipe and form the pile under its own weight. It is then at the limit of "pumpability." Characterized by a great fluidity, the concrete risks segregation and clogging inside the pump lines. Therefore, in this case it is necessary to ensure an excellent uniformity in the concrete production even more carefully than is customary.

6.4. PLACING CONCRETE

The quality of the foundation depends on a good pouring procedure as well as on good quality of the concrete. In the case of bored piles, pouring of the concrete is an extremely critical operation and every mistake risks ruining the job. It is wise to be familiar with and to follow the state-of-the-art.

We will successively examine:

-- techniques of placing concrete in the borehole,

-- influence of the presence of water or drilling mud and sensitive aspects of the operation, and

-- operations performed after concrete placement: withdrawal of a temporary casing, striking off, and the possible extraction of the already placed fresh concrete in the case of a serious setback.

6.4.1. Techniques of placing concrete

The technique of pouring concrete by discharge from the surface cannot be allowed except in dry, shallow (less than 10 m - 32.8 ft) boreholes without reinforcement.

Three techniques can be practiced:

-- tremie method,

-- use of a pump to deliver concrete directly to the base of the pile, and

-- bucket with remote-controlled opening, used only for large diameter shafts.

6.4.1.1. Concreting by the tremie method

a) DESCRIPTION

The tremie method is intended to avoid leaching, segregation, and pollution of the concrete by guiding the discharge down to the bottom of the borehole and by maintaining a continuous concrete feed to the middle of the mass of fresh concrete already in place.

It is composed of a pipe and a funnel-shaped bucket (the tremie, Fig. 126). The pipe must have the following minimum characteristics:

-- the inside must be smooth;

-- it must be made up of short (about 3 m - 9.8 ft), easily disconnected sections (square or trapezoidal threadings);

Fig. 126. -- Cone and pipe that form a tremie.

-- it must be strong (minimum thickness: 8 mm - 0.31 in. standard sections); and
-- it must have a diameter approximately six times that of the maximum coarse aggregate, but allowing a distance between pipe and reinforcement (including pipes for acoustic investigation) of at least four times the dimension of the maximum coarse aggregate. Furthermore, these specifications might, in certain cases, reduce the size of the maximum coarse aggregate.

In addition to these minimum characteristics, it is recommended:
-- to use a tremie pipe with a smooth exterior, or to prohibit projecting connection pieces (couplings, flanges, attachment brackets) that may become hooked onto the reinforcing cage. The connections between sections must be done by threads cut into the pipe wall. The thickness of the pipe should be sufficient at the shoulder;

-- to make an optimum combination of the sections of different lengths according to the lifting schedule for the tremie pipe, avoiding overly long sections at the top (this lifting schedule may be set at the time of the concreting of the test pile);

-- to provide efficient tools for rapidly connecting and disconnecting the sections;

-- to plan for the use of a device to centralize and stabilize the tremie pipe at the head of the borehole;

-- to place, when necessary, a system that allows evacuation of the confined air trapped under the plug at the time of starting the concreting (breather pipe arranged on the interior of the pipe, for example, Fig. 127). This precaution is especially useful when the water level in the pipe is relatively low in relation to that at the head of the borehole;

-- to notch or provide lateral openings at the bottom of the pipe (Fig. 128) to ensure the evacuation of the water (or mud or air) during the descent of the first concrete, allowing the base of the pipe to rest at the bottom of the borehole. This arrangement allows for control of the initial concreting because the plug is not freed until the tremie pipe is raised some distance (thickness of the plug);

The tremie should have the following characteristics:

-- a truncated cone shape in preference to a truncated pyramid shape;

-- angle α (Fig. 129) at the top of the cone between 60° and 80° (above 80° the concrete might arch over the hole stopping the flow).

b) USE

The tremie pipe must rest at the bottom of the borehole before the starting of the concreting operations. For this reason it is necessary to leave a central space at the base of the reinforcement, large enough to allow the tremie pipe to pass through and rest at the bottom of the borehole (see Chapter 5).

If the tremie pipe does not rest at the bottom when concreting under water or mud, there is leaching or pollution of the first concrete. Contrary to common thought, the first concrete does not all rise up to the surface at the end of the operation. Some of the leached concrete covers the bottom while some coats the walls of the borehole, decreasing base resistance and lateral friction (Fig. 130).

It is important to note that there is an absolute need for careful cleaning of the bottom of the excavation (see § 3.4.4).

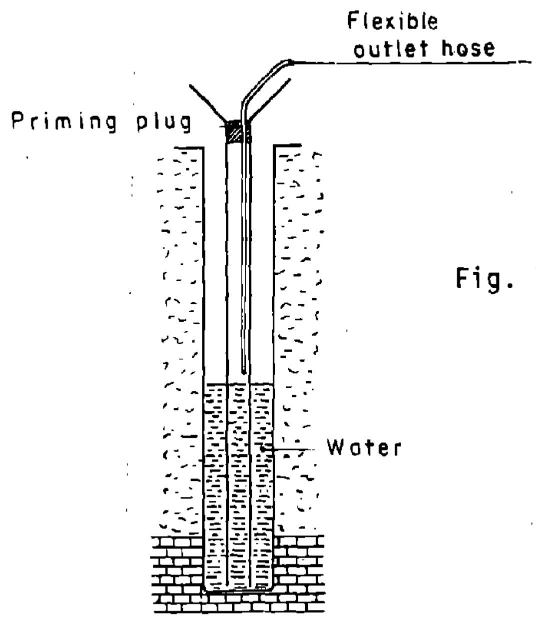


Fig. 127. -- System for evacuation of air or water by outlet hose.

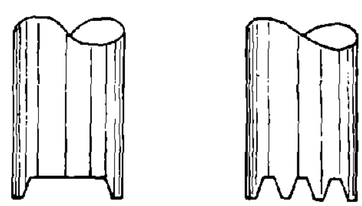


Fig. 128. -- Toothing of lower portion of tremie.

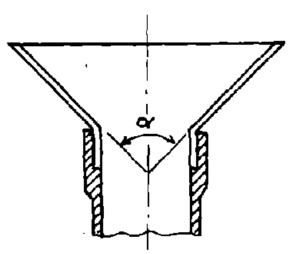


Fig. 129. -- Angle α of tremie cone should be between 60° and 80°.

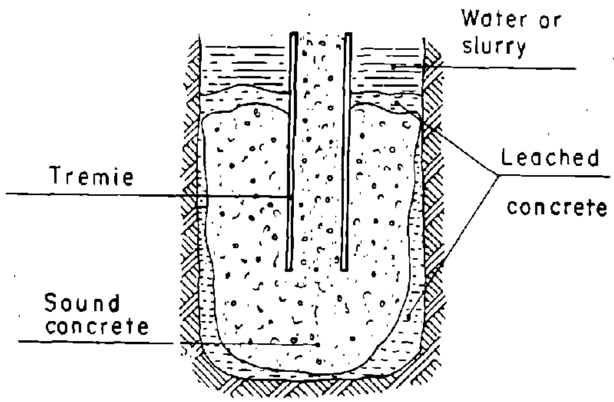


Fig. 130. -- Distribution of leached concrete after faulty priming of tremie.

For placing concrete for bored piles one tremie pipe is generally used, but if the diameter of the boring is large (shafts) or when working with pile-barettes, it might be necessary to use several tremie pipes at the same time. In this case, it is necessary to study carefully their location in order to obtain a uniform rising of the concrete over the complete bored or excavated area. Poorly or improperly placed pipes might lose a large amount of their effectiveness or may obstruct proper rising of the concrete. It is also necessary to prepare and place the reinforcement in such a way that it will allow proper pouring and easy manipulation of the pipes.

The ACI (American Concrete Institute) recommends not exceeding the following surfaces per tremie pipe: 28 m^2 (300 ft^2) for rectangular and 14 m^2 (150 ft^2) (4.50 m - 15-ft diameter) for circular caissons. However, these recommendations imply a technique which employs the horizontal displacements of the pipes and does not concern the concreting of piles. Therefore, it is advisable not to concrete shafts larger than 2 m (6.6 ft) in diameter with only one tremie pipe. There are risks of caving of the concrete along the tremie pipe with entrapment of mud (Fig. 131) or leaching by water.

b1) Starting of concreting

An overly rapid descent of the first concrete in the tremie pipe causes its dispersion and segregation. In the presence of water or mud,

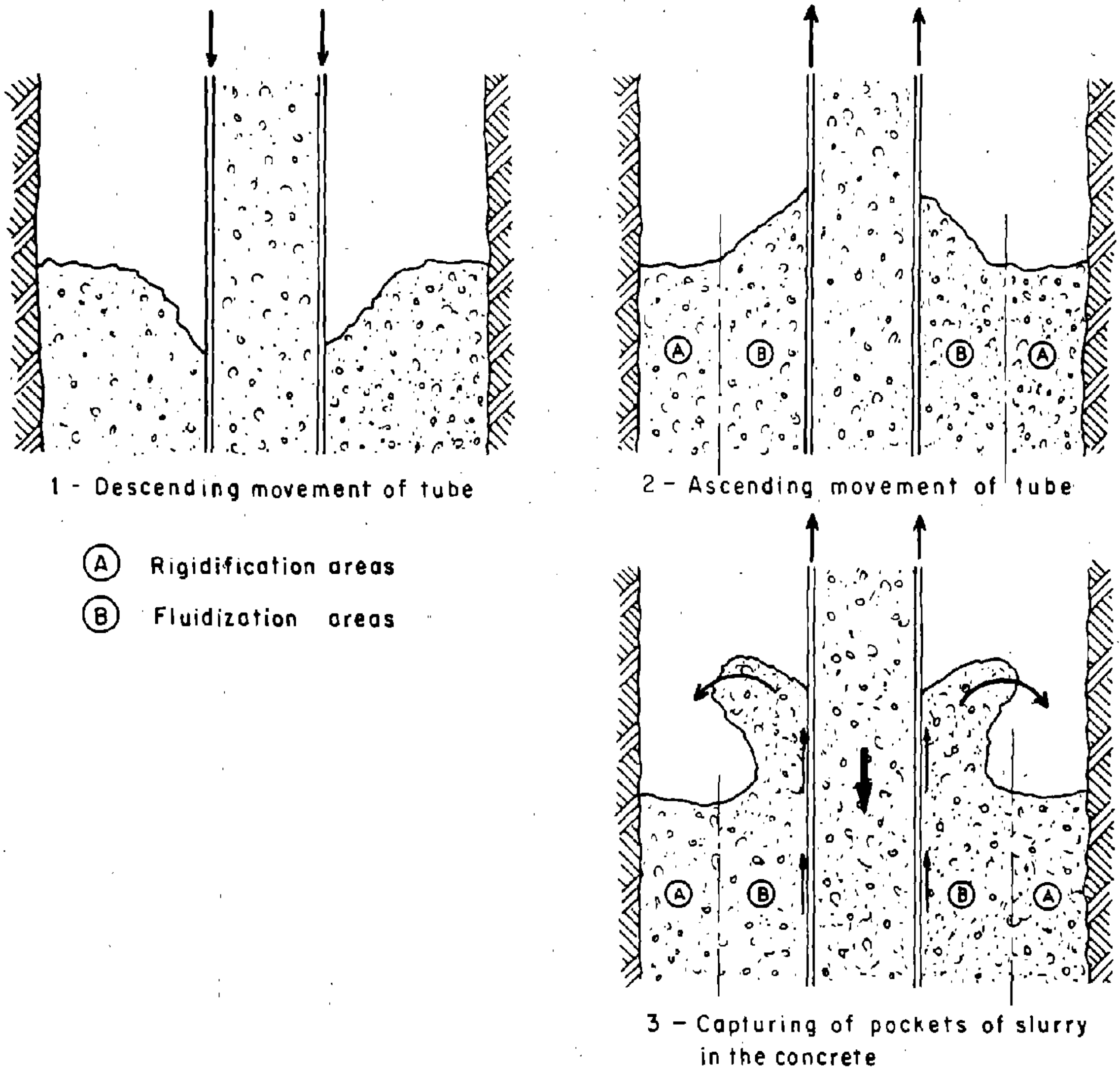


Fig. 131. -- Initiation of fluidization areas during up and down movements of tremie for piles of large diameter.

1. Descending movement of pipe
2. Ascending movement of pipe
3. Trapping of mud pockets within concrete

the leaching or pollution that results is more serious since it increases the segregation. Part of this poor quality concrete is left at the bottom and on the walls of the borehole (Fig. 130). To avoid this problem, it is necessary to carry out successfully the critical operation of priming. This consists of first filling the tremie pipe with a homogeneous mass of concrete and second, of flushing or scouring of the bottom of the hole. To this end, it is essential that a plug be placed at the top of the pipe before the concreting operation.

The role of the plug is to slow down the descent of the first concrete and to promote the formation of a homogeneous, continuous column that, due to a piston effect, flushes the water or the mud ahead of it, without mixing.

The flushing action will be facilitated by an arrangement of the extremity of the tremie pipe: notching or small lateral openings (Fig. 128) are recommended in place of the usual practice of lifting the pipe prior to priming (an imprecise operation that may result in leaching if the movement of the tube is considerable.

It is also necessary to eliminate some other practices, unfortunately very widespred, such as:

- the shovel technique, in which a shovel temporarily obstructs the opening at the bottom of the cone or funnel and then is removed (with difficulty) when the cone has received the first load of concrete (Fig. 132). This method does not in any way prevent segregation, leaching nor pollution of the initial concrete;
- the paper or tissue plug does little to slow down the descent of the concrete and generally remains enclosed in the concrete at the bottom of the pile or on the wall; and
- the first batch of pure paste or concrete poured in bulk in the pipe: there is leaching and trapping at the bottom of the borehole.

A technique that sometimes works well consists of placing a plug of expanded polystyrene inside the top opening of the tremie pipe. This plug is split in four and has a diameter slightly larger than that of the pipe (Fig. 133). The descent may be initiated with a slight push on the plug through the concrete in the tremie.

This plug performs the job of brake and piston. At the bottom, it splits in four pieces each of which is brought to the surface with the

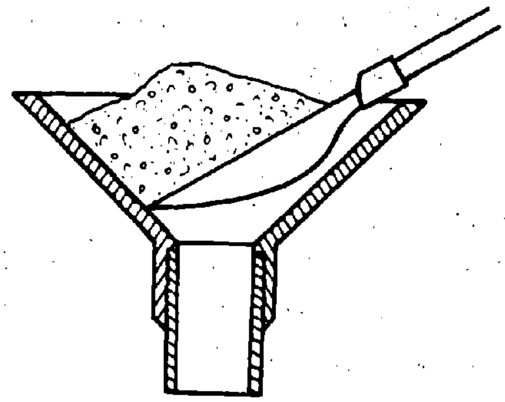


Fig. 132. -- Technique of priming with shovel - prohibited.

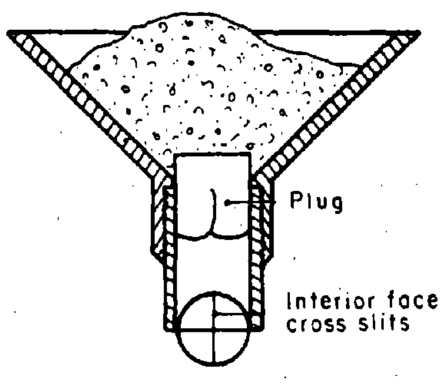


Fig. 133. -- Priming using polysterne plug.

concrete. Unfortunately, the polystyrene pieces may remain hooked to the reinforcing cage. This technique should therefore be used with caution (with or without the option of allowing a small defect in the pile shaft).

The best technique consists of preparing a plug of pure-cement paste of a firm consistency. This plug, as in the preceding (polystyrene), must have a diameter slightly larger than that of the tremie pipe. It is then desirable to wait for the onset of the setting of the cement to avoid an immediate deformation that would limit the braking effect (Fig. 134). Such a plug remains at the bottom of the pile, and does not present any inconvenience.

The new types of "elastic" plugs, composed of pure cement paste and metallic or polypropylene fibers or of steel shavings (lathe residue), combine the advantages of polystyrene (elasticity, braking effect) and of the pure-cement paste. They may be molded and given an appropriate shape (Fig. 135).

In addition, there is another priming technique which uses a *cap at the extremity* of the pipe. This watertight cap, located in the lower part of some tremie pipes, allows the descent of the initial concrete within a dry tube and thus prevents leaching. Unfortunately, it does not prevent some amount of segregation during the fall of the concrete in the case of a long tube. *This bottom cap must therefore be used only for concreting of piles of short length.* In addition, this system requires a hose positioned in the interior or exterior of the tremie pipe (Fig. 136) that allows air to escape.

There are two principal types of caps:

-- *the hinged closure* (Fig. 137) that is composed of a hinged plate kept closed by a hook and opens under the weight of the concrete when the pipe is raised. A plastic joint placed over the plate assures watertightness. In addition to risks of deterioration of the valve, this system presents the major inconvenience of causing a considerable segretation during the up and down vertical movements of the tremie pipe;

-- *the "hat" closure* (Fig. 138), composed of a hollow plug capping the extremity of the tremie pipe. Imperviousness is obtained by use of a plastic friction joint. When the pipe is raised the plug will be left at the bottom of the hole under the weight of the concrete.

Finally it must be noted that with these end caps it is necessary to use a pipe heavy enough to withstand the hydrostatic pressure of the

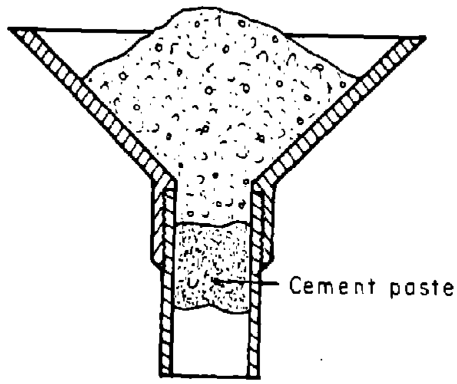


Fig. 134. -- Another type of plug.

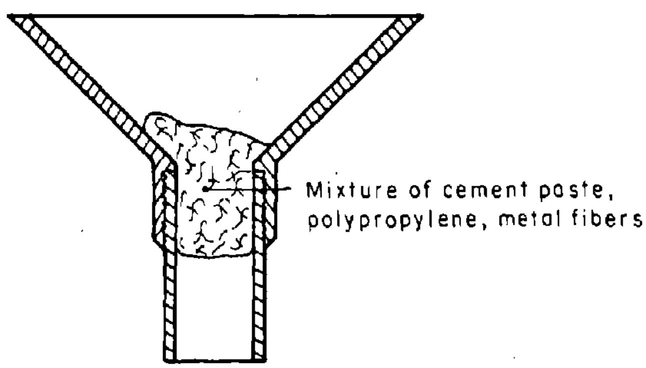


Fig. 135. -- New type of molded plug
(being studied).

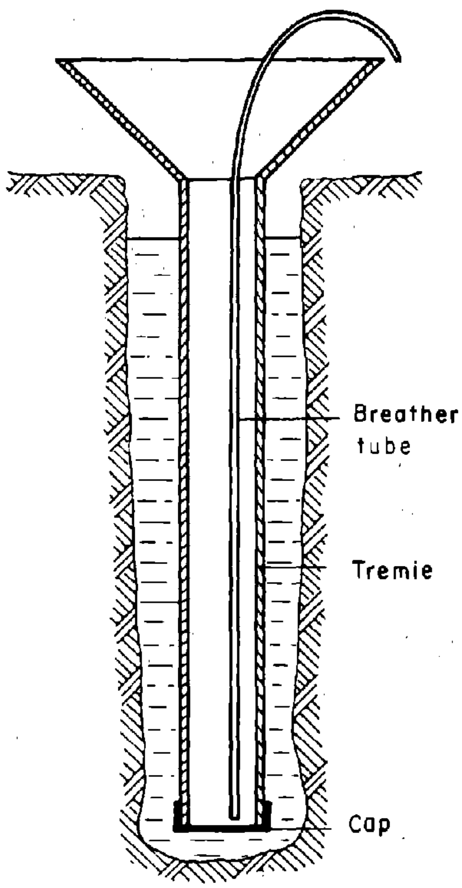
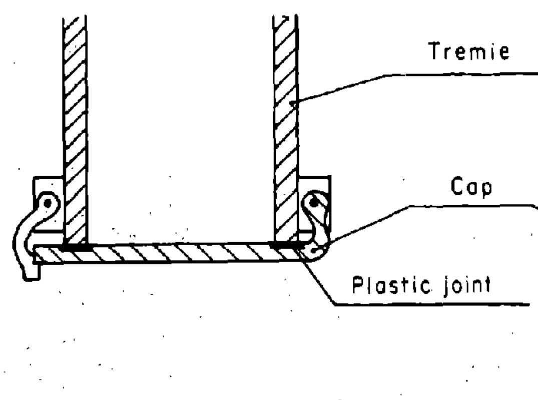


Fig. 136. -- Capped tremie pipe with breather tube.



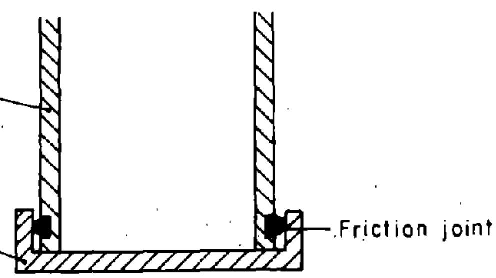


Fig. 137. -- Hinged closure.

Fig. 138. -- "Hat" closure.

mud or the water. For example, for a 200 mm (7.87 in.) interior diameter tube, it is necessary to use a steel pipe with a minimum thickness of 8 mm (0.31 in.).

b) Concreting by the tremie method (Fig. 139)

Once the priming is done, the concrete flows to the lower end of the tremie pipe and ascends within the borehole under its own weight. If feeding of the concrete to the tremie is stopped, an equilibrium is established between the column of concrete within the pipe and that in the borehole. The level of concrete within the pipe stabilizes at a higher level than that within the excavation due to internal friction. Renewed pouring of concrete in the tremie increases the level of the concrete in the excavation.

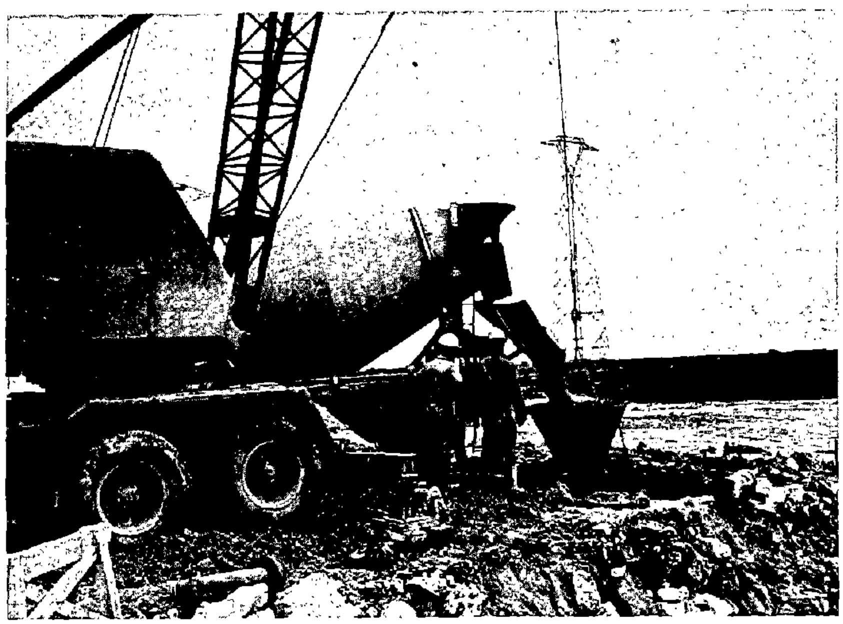


Fig. 139. -- Concreting of a pile with tremie with concrete from a transit-mix truck.

This difference in levels increases as concreting progresses because of the increase in internal friction as concreting advances and eventually the concrete canot descend further. It is then necessary to raise the tremie pipe to a different height and to remove one or more components at the top, just below the cone or funnel. However, it is absolutely necessary to keep a certain length of pipe embedded within the concrete.

This length can vary from 2 m (6.6 ft) for small diameter piles (60 to 80 cm - 23.6 in. to 31.5 in. in diameter) to 4 m (13.1 ft) for the larger ones (note that these values are larger than those appearing in the commentaries of Article 38.1.4 of Volume 68).

The best concrete is that which has a normal water content and fills the borehole with a small difference in the levels of concrete when equilibrium is reached. This characteristic permits concreting of the pile without frequent raising of the pipe.

A common practice to facilitate the descent of the concrete consists of transmitting a vertical up and down movement to the pipe which is more or less drastic and intense. This must be avoided. Actually, experiments performed in locations of tests have shown that the concrete within the pile tends to rise along the tremie pipe but not along the periphery of the borehole (§ 6.4.1.1b). The alternating vertical movement attracts mortar to the wall of the tremie with an increase in the fluidity of concrete along the tremie pipe (wall effect). Thus, there is a greater possibility of movement along this pipe (see Fig. 131) and improper concrete placement.

This practice is often used inadvisedly in order to offset the lack of fluidity of the concrete and may unprime the tremie pipe, especially at the beginning of concreting when the depth of the tremie in the concrete is small.

Hence, emphasis must be placed on the seriousness of any accidental loss of the priming and damage to the concrete. There is the possibility of enclosing in the pile shaft concrete of poor quality (leached or segregated), water, mud, sediments, or even caved soil.

It is especially important to reject the mistaken idea that the vertical movements of the bottom of the tremie pipe helps to coat the reinforcing. In fact, the quality of the coating depends more on the fluidity of the concrete than on the construction techniques used on the job.

Finally, care must be taken to ensure a regular and continuous feeding of the concrete and that the tremie bucket is always kept as full as possible.

6.4.1.2 Concreting by the pump method (Fig. 140)

The concrete pump is used more and more frequently for "direct" concreting of bored piles instead of as a relay between the mixing truck and the tremie assembly. For those aspects which concern the general rules of the use of concrete pumps, it is necessary to consult technical information memo LCPR/SETRA [72].

a) DESCRIPTION

The delivery-pipe of the pump is introduced vertically within the borehole and serves as the tremie pipe.

There are two advantages for the use of this technique:

-- a concrete with a lower water content as compared to the classic tremie method may be used, and therefore has better compactness; and

-- there are benefits during the placing of concrete, not only from the weight of the column of concrete in the pipe, but also from the pressure provided by the pump that may be very high (up to 120 bars). It follows that the various operations intended to make the concrete descend are eliminated. However, there may be a need during the concreting to raise the pump line if the rebar cage starts to rise.

The pump used must be a powerful piston pump with a large capacity (conduit diameters of 100 to 120 mm - 4.0 in. to 4.7 in.) and must be in excellent condition (Fig. 141). Pumps which work by deforming a flexible diaphragm do not generally yield good results in this case (insufficient pressure).

The pump line must be rigid (metal) , smooth in the interior and exterior, and thick enough to present a sufficient resistance to oppose the pulsating lateral movements produced by the piston stroke. It must be composed of screwed components with the threads cut into the wall of the pipe. The bottom of the pump line, as for the tremie pipe, must be provided with openings that allow the flushing of air, water, or mud.

This pump line must be fitted at the top with a special arrangement (Fig. 142) composed of an uptake pipe for concreting and priming and a cross-head. The pipe for concreting and priming is closed at its upper end by a screwed cover that may be replaced by a tremie bucket, and at its lower end it is connected to the pump line. In the case of a pump breakdown this arrangement allows the attachment of a tremie bucket at the top and concreting by the classic tremie method.

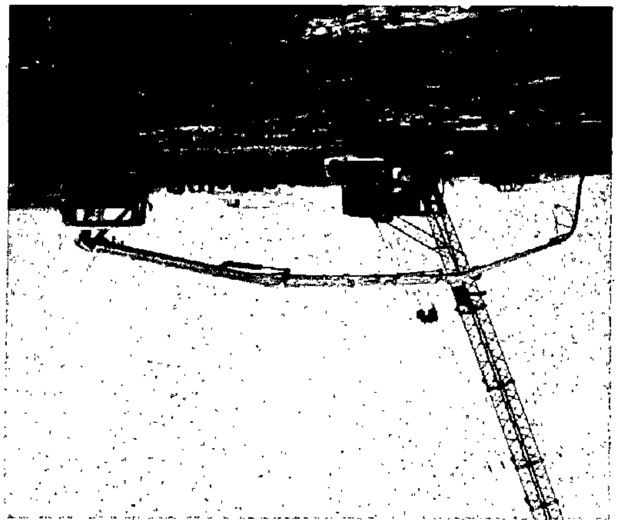


Fig. 140. -- Concreting by pump method.

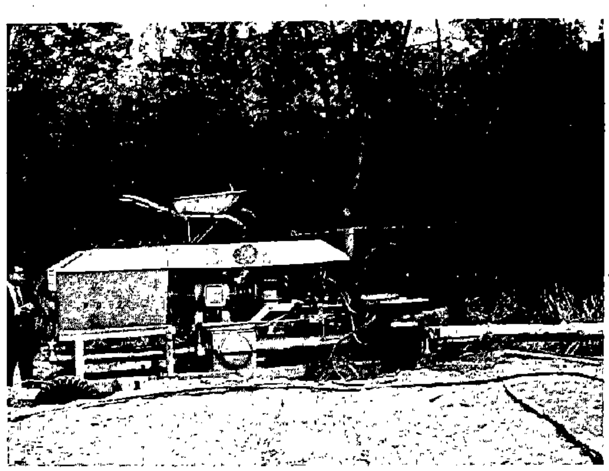


Fig. 141. -- Concrete pump.

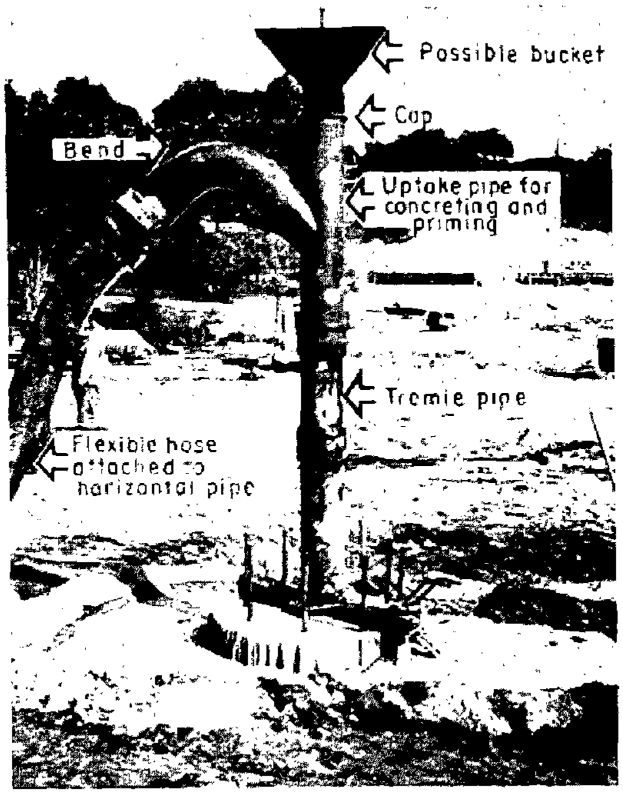


Fig. 142. -- Metal cross-head of tremie for concreting with pump.

The cross-head allows the filling of the pump line so that pressure may be maintained in the concrete column and inclusions of air are avoided that are detrimental to pumping. The cross-head is connected by flexible tubing to the horizontal line from the pump.

b) USE

b1) Priming

This is one of the greatest difficulties with use of the pump due to the fact that it is impossible to introduce directly a plug of mortar inside the pump. Therefore, it is advisable to use the following method (Fig. 143):

1. Opening of the screwed cover on the "Y" of the tremie pipe and introduction of a priming plug (paste-of-cement-metal-fibers type),

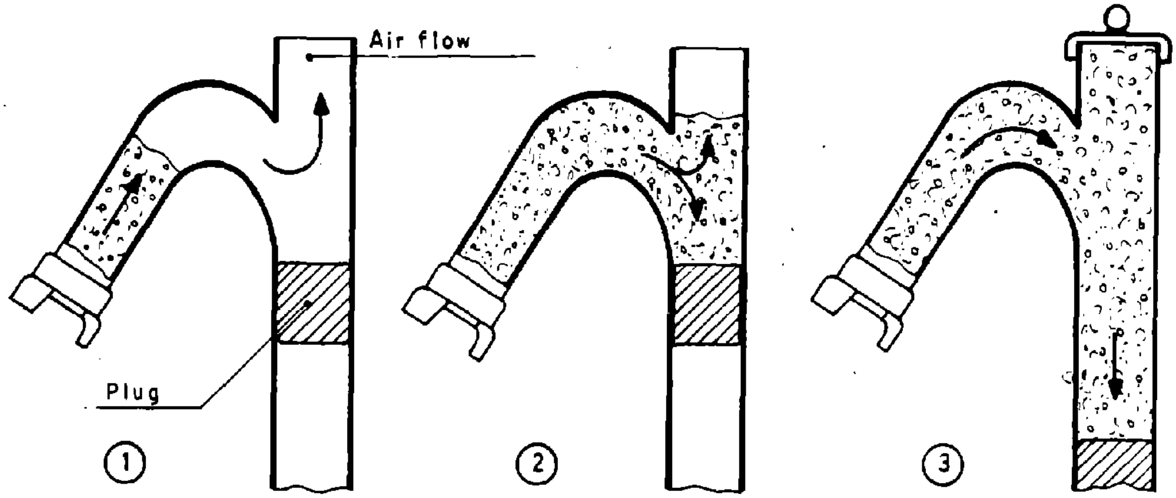


Fig. 143. -- Technique of priming with concrete pump.

2. Starting of the pump. The cap is left off to allow air to escape during filling of the conduit, and

3. Closing of the cover and normal pumping when the concrete has reached the plug. The flushing of the water or mud is carried out as in the tremie-pipe method.

b2) Concreting

Supplying of concrete to the pump should be done in a regular and continuous fashion. The concreting can, therefore, be carried out in a single process without raising nor alternate movements of the pump line and thus without risk of loss of the priming.

6.4.1.3. Valve-bucket concreting

This third method is only mentioned here because it is useful mainly for concreting caissons with large cross-sections. In unusual circumstances it can be used for large-diameter-shaft foundations.

Concrete buckets are equipped with a water-tight release valve which is opened by one of the following methods:

-- automatically operated by contact with the bottom of the borehole (or the level of concrete already in place, Fig. 144), or

-- manually operated from the surface.

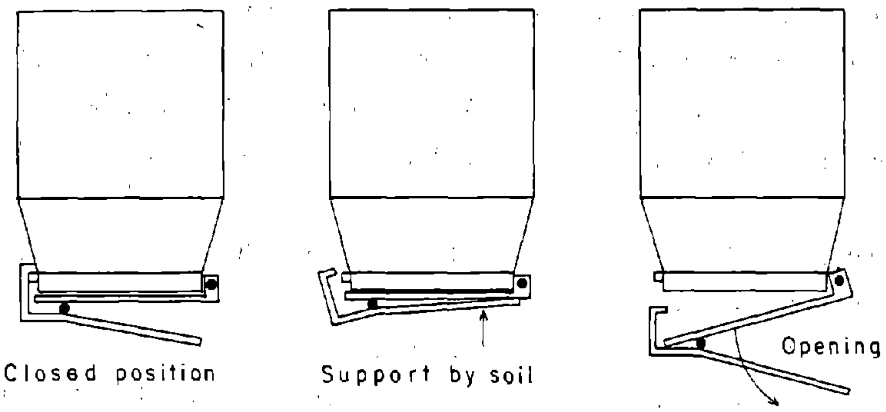


Fig. 144. -- Bucket with automatic valve.

The opening of the valve generally presents difficulties during the course of the job and numerous incidents may occur. Frequent checks and care of the equipment that is involved are essential.

6.4.2. Influence of the presence of water or drilling mud. Critical aspects of the operation.

6.4.2.1. Soil-concrete contact at tip and sides

It is re-emphasized here that a good contact is absolutely necessary; at the tip in bearing, to ensure a good load transfer to the substratum, and laterally to mobilize side friction.

In this repsect, it is not only necessary to check the cleanliness of the bottom of the excavation after cleaning out (see § 3.4.4) but also to prevent leaching of the first concrete or its pollution by drilling mud in the case of faulty priming. This first concrete, as previously discussed, almost completely covers the bottom and the lower parts of the walls of the borehole.

6.4.2.2. Inclusions

The entrapment of mud or debris within the concrete can occur:

-- in the case of faulty priming of the tremie pipe during concreting (see § 6.4.1);

-- when the ascension of concrete in the borehole occurs irregularly. This phenomenon is dangerous for large diameter piles (and barrette piles) and also when the tremie pipe is moved too vigorously. Around this pipe the concrete then has the tendency to become more fluid and to rise faster than in the periphery of the pile where it remains more inactive (see § 6.4.1). In this way a central roll or wave is created, which soon collapses entrapping water, mud or sediments (wave phenomenon, Fig. 131); and

-- as a result of local caving of the wall in the absence of a temporary casing, sheath, or lining.

6.4.2.3. Distribution of concrete layers

It is generally accepted that the concrete establishes itself in the borehole by chronological horizontal layers, the first layer reaching the surface at the end of concreting. *This is an erroneous idea.*

Experiments performed at test sites have shown that layer distribution is, on the contrary, concentric with the tremie pipe such that one layer pushes the previous one toward the periphery. The movement of concrete is centrifugal. Figure 145 illustrates this phenomenon very well and has been verified by full-scale concreting at testing locations where concretes were dyed different colors.

6.4.2.4. Flushing by overflow

It is necessary by overflowing the borehole to clear out the sediments, mud, silt, and bleeding water that rises on top of the initial concrete reaching the surface. This initial concrete is in itself polluted or partially leached despite the precautions taken during priming and therefore must also be eliminated. Further, it is worth noting that in other cases (especially withdrawal of a temporary casing by vibration) water movement to the surface from small "springs" has often been noted after the end of concreting (Fig. 146). There is nothing alarming about this. These migrations are quite small and do not constitute any danger to the pile.

When the level at which the concrete will be cut off is clearly lower than that of the working level (Fig. 147) this cleaning by overflow might seem excessive. If this is not done, concrete should however be placed to a level high enough so that there is no weakened concrete below the cut-off point. Then the concrete can be trimmed away to the cut-off level.

6.4.2.5. Overpouring of concrete

It is difficult to define and measure an overpouring of concrete. The needed data for the calculations include the following:

Fig. 145. -- Experimental determination of the distribution of layers at bottom of pile using concretes of different colors. (Experiments by Bordeaux Regional Laboratory.)

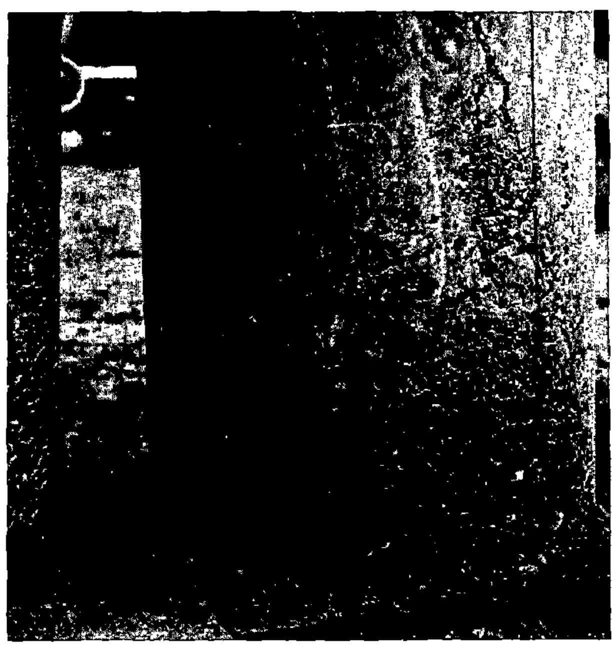


Fig. 146. -- Water migration verified at test site.

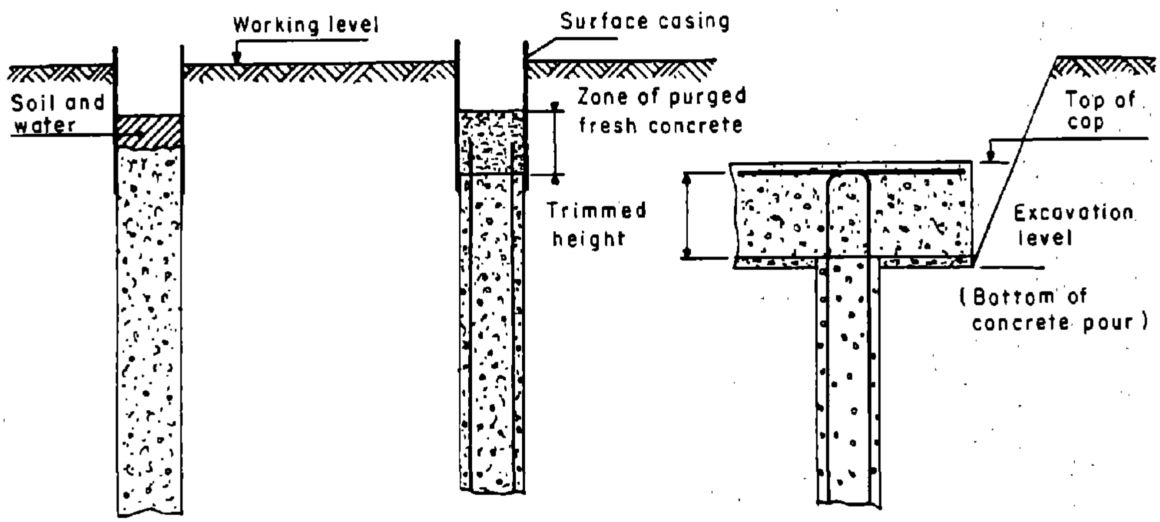


Fig. 147. -- Cleaning by flushing followed by trimming.

• the theoretical volume, V_t, of the borehole according to the dimensions of the tools used and not the dimensions shown in the plans which frequently differ,

• the volume, V_1, of the concrete delivered by the production plant,

• the volume, V_e, of the excess concrete in the last truck,

• the volume, V_p, discarded during the trimming process.

The overpouring, V_s, is therefore calculated as:

$$V_s = V_1 - V_e - V_p - V_t$$

This calculation is only an indication. The quantity evaluated in this way is not representative of the effective overpouring since it includes an overpour, ΔV, relative only to the enlargement of the borehole. Therefore, the real volume, V_r, is larger than the theoretical volume.

$$V_t (V_r = V_t + \Delta V).$$

Regardless of the method used, the diameter of the borehole is always larger than the diameter of the tool.

The true, accidental overpouring is caused by fissures (cracks), karsts, or to the flow of soft soils and cannot be calculated with precision unless the overpour ΔV can be estimated. Unfortunately, experience shows that ΔV varies in an uncertain manner according to the machines and tools used, the drilling depth, and the consistency of the soils. In addition, it is of greater interest to be able to localize the levels of overpouring and as a result to deduce the causes. To this effect, it is desirable to establish a meaningful concreting curve [3] from the continuous measurement of the volume of concrete delivered and of the elevation attained by the concrete in the borehole.

Due to technological reasons, such measurements are still impossible to obtain and one must be satisfied with spot measurements with rudimentary probes (as a general rule, one check of the level of concrete per truck). More precise instruments are presently in the experimental stage.

Furthermore, in the case of piles constructed in place with temporary casings, it would be desirable to be able to perform measurements of the levels not during concreting, which have no bearing, but when the casing is withdrawn, which is practically impossible.

Finally, even under ideal conditions, the concreting curve does not always allow location of the overpouring zone. Indeed, in very soft soils or in the presence of a partially obstructed karst system, the creeping or emptying into the voids only occurs under a certain head of concrete, in other words, well above the level of the anomaly for which the position cannot be detected. In this case, however, the shape of the concreting curve gives information as to the nature of the incident and, from the geotechnical data, determination may be made of the layers susceptible to being affected (Fig. 148).

6.4.3. Operations following placement

The two common operations that complete the construction of a pile after concreting are the withdrawal of the casing, if necessary, and the striking off of the pile.

It is necessary to emphasize that in the case of serious setbacks during concreting, it might be necessary to proceed to the extraction of the fresh concrete already in place.

6.4.3.1. Withdrawal of the temporary casing

When the technique of boring and concreting using a temporary casing (used as a form during these operations) has been employed, it is necessary to remove this casing during or at the end of concreting and in any case before the setting of the concrete. A late attempt at extraction might involve the loss of the casing or very serious mishaps:

-- development of voids in the concrete with the possibility of caving-in and inclusions;

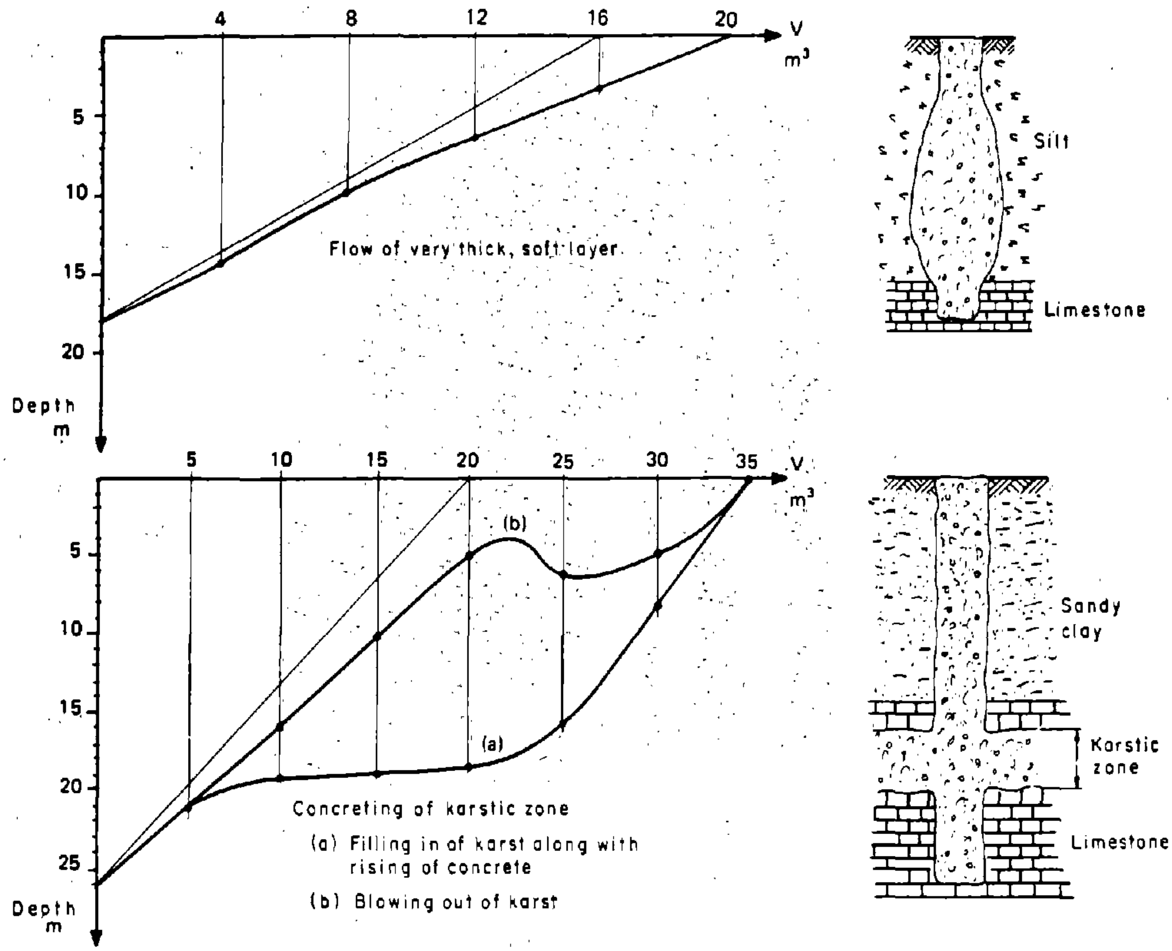


Fig. 148. -- Examples of concreting curves.

-- misalignment of the reinforcement,
-- lifting of the shaft with the loss of base contact and reduction of lateral friction, and
-- possible lifting of the entire pile.

Actually, due to the complex distribution of the concrete layers, it is possible to create surfaces of discontinuity during the setting of concrete: a "recent" layer may be located next to an old layer where the setting has already started. These surfaces constitute zones susceptible to rupture by stress at the time the casing is pulled out.

A concrete layer in which setting has begun might develop a considerable adhesion with the temporary casing on one hand and with the reinforcement on the other hand. At the time the casing is pulled out, the reinforcement may also be pulled out and, in many places, the pile may "tear."

Therefore, one sees the benefit of:

-- a quick, continuous, regular and systematic concreting operation, and
-- total control of the retardation of the setting of the concrete, so that the set does not occur overly early nor heterogeneously.

6.4.3.2. Striking off or trimming the pile

Generally, piles are concreted to an elevation higher than the level of strike-off. It is then necessary to trim the pile (Fig. 149).

The purpose of this operation is to eliminate over a certain height the concrete at the surface which may contain zones of questionable quality (polluted, leached, segregated) when flushing of the concrete cannot be performed either by overflowing or by cleaning out. Trimming is also necessary when the risks of caving at the surface, following withdrawal of the surface casing (or the temporary casing), prohibit extraction of the fresh concrete down to the level of striking off which must be protected from any contamination before setting.

In fact, flushing and striking-off operations may be complementary (Fig. 150), with the height to be struck off being dependant on one hand on the extent of possible cleaning, and on the other hand, on the conditions at the job site (terrestrial or aquatic site, relative levels of operations and cutoff...). Taking appropriate action at the job site will avoid the chipping away of excessive concrete (151).

Trimming performed using a concrete-breaker or explosives is an operation that should be performed with great care so that the rebar is thoroughly exposed, cleaned, and prepared to receive the concrete from the superstructure.

The technique of trimming with the aid of explosives uses a complex arrangement of numerous microcharges with explosions delayed a few microseconds, in order to avoid any shock detrimental to the environment. This procedure is not commonly used at present and is mainly for shafts with large diameter.

In any case, the operation must not be performed too late nor too early. If performed too late, the concrete will develop a considerable strength requiring a large expenditure of energy. If performed too early,

Fig. 149. -- Trimming of a pile.

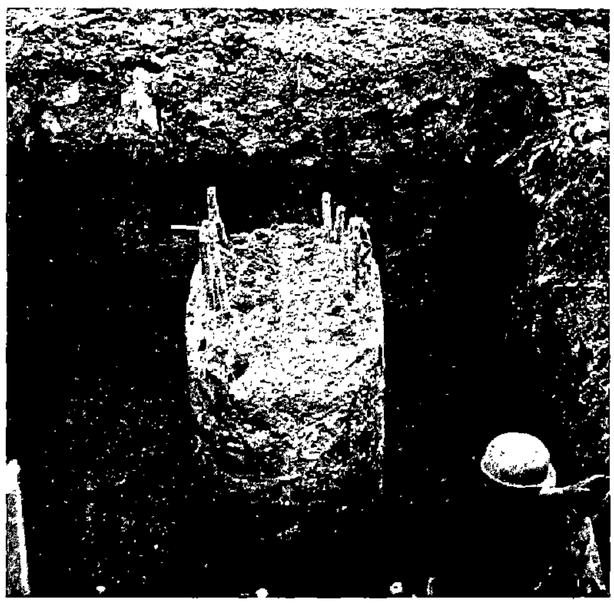


Fig. 150. -- Reasonable trimming height due to preliminary flushing of fresh concrete.

Fig. 151. -- Example of excessive trimming height.

there is a risk of microfissuring the concrete at the pile head, mainly due to a lack of tensile strength.

6.5. ORGANZIATION FOR THE CONTROL OF CONCRETING

A detailed, strict plan for the control of concreting cannot be established due to the large diversity of job sites from the point of view of their importance, their organization, and construction conditions.

It is better to devise a control plan adapted to each job, just as each site will have its own construction plan.

It is obviously desirable at the time the CCTP is drafted to entrust the establishment of a control plan to a specialized laboratory in order to avoid as much as possible any subsequent difficulties.

However, examination will be made here of the major aspects of the control of concreting for bored piles.

6.5.1. Control of concrete and its components

-- *At the plant,* where it is necessary to:

• proceed to sample the concrete components (aggregates, cements, admixtures) that will be subjected to the tests according to specifications of the CCTP;

• to establish a system that allows attention to production: summary of weighing operations, proportions, information on recording devices (printer), and others (humidity meter), and the general working order of the plant;

• to carry out some tests of the fresh concrete (particularly fluidity) to ensure that the concrete has the same characteristics upon arrival at the job as at the plant; and

• to verify some measurements of the setting times of concrete.

-- *At the concreting job,* where it is necessary to:

• verify the fluidity of the concrete by use of the fluidometer or in the absence of this equipment by using the slump cone, which is not well suited for the application (see § 6.1.2.2). To facilitate the control of the water content, it is of interest to correlate the measurements that are made with the water content of concrete. Even though the adding of water to the concrete at the construction site is generally thought to be undesirable, if the appropriate correlations are available and if the concrete is too dry[1] when it arrives at the site, the addition of mixing water to the truck can be made quickly and in the proper amount;

• to prepare the samples of concrete which will be subjected to mechanical tests and to density determination; and

• to record the times of transportation and arrival of the concrete before placement.

---

[1]Any addition of water to the mixing truck must be followed by a very rapid mixing of at least five minutes.

6.5.2. Control of placement

The principle of a preliminary study of the control plan adapted to a site applies particularly to this part of the operations. The plan involves the monitoring of each of the successive operations and verifying that they are carried out properly. The operations at the job site that must be controlled are:

-- the preparation of the concreting material (tremie pipe assembly, concrete pump, etc.);

-- the priming of the tremie pipe (traditional pipe or pump). This operation is particularly critical and extremely important because it controls the quality of the concrete that will be in contact with the soil at the bottom of the pile;

-- the concreting operation in general and during which attention must be given to:

• regularity of the concrete supply (rotation of trucks, ensuring that the concrete is on time, radio communication, etc.);

• strict compliance with the operations and instructions that are described in § 6.3, and especially to the establishment of the concreting curve; and

• the proper behavior of the reinforcing steel; and

-- to certain operations which follow the concreting: flushing by overflowing, concrete behavior during the extraction of the temporary casing, etc.

For each concreting operation, it is necessary to note in an official report the operations carried out for control, the observations made concerning the environment and the working conditions, and the small details, incidents, and anomalies that are observed. A standard form is proposed in Appendix B.

The record keeping involves the use of four sheets with the conents of the forms indicating a reminder (or check list) of that which must be controlled and observed during the concreting of the piles. The use of these four sheets is very flexible and depends on each operation and it is not necessary to fill them all out every time concreting of a pile is performed.

SHEET No. 1: control of the components of concrete: cements, aggregates, water, admixtures.

SHEET No. 2: control and transport of fresh concrete, characteristic tests on fresh concrete, sample collection, and truck rotation.

SHEETS Nos. 3 and 4: placement control: climatology, drilling, cleaning and reinforcing (taking into account the characteristics that influence the concreting operations), actual placement methods (priming technique, supplying of concrete, placing concrete, serious incidents, possible extraction of the temporary casing, overpouring, and trimming).

This official report is primarily destined for the laboratory in order to accumulate the data needed to prepare the final project-record report. However, the report may be sent to the construction supervisor if he wishes in order that the information on the construction schedule proposed in Appendix A may be completed.

CHAPTER 7

TEST FOR COMPLETED PILES

The tests for completed bored piles are performed in order to ensure the quality of the concrete in the shaft and the quality of the contact at the base between the concrete and the soil. These tests are unrelated to the static loading test (see [1], § 3.5.5, clause 3) which is a test for the overall performance or adequate sizing of the pile.

In performing the tests on the completed pile, methods can be employed that allow detection of the majority of imperfections in the shaft or at the base of the pile [22] [27].

7.1. TEST METHODS

7.1.1. Geophysical methods of investigation

The most popular methods used by the LCPC are the following:

-- Investigation of foundations by wave transmission using the reflection method (October, 1974).

-- Investigation of piles and diaphragm walls by wave transmission using lateral transmission (September, 1974), and

-- Investigation of piles and diaphragm walls using radioactive material for transmission of waves (to be published).

There are other methods including the mechanical-impedance method used by the Center for Experimentation in Buildings and Public Works (CEBTP) [47][52].

7.1.1.1. Investigation by reflection of waves where a mechanical impulse is applied to the pile

This method is simply noted here because its use is disappearing in favor of other methods. Nevertheless, it is useful in some cases: short piles without a rebar cage or of small diameter ($\emptyset \leq 60$ cm - 24.0 in.); and battered, precast-concrete piles of reinforced concrete [14].

a) PRINCIPLE

This method (Fig. 152) has been established according to the laws that govern the propagation and reflection of waves in heterogeneous materials. It consists of:

-- creating a vibration at the top of the pile,

-- recording this vibration after reflection, and

-- measuring the travel time from the instant of emission and reception, in order to calculate the distance traveled by the wave, knowing its velocity of propagation.

Disregarding the effect of the distance between the transmitter and the receiver, the height (h) of the pile, or the distance to the first anomaly, is given by the relation:

$$h = 1/2\ VT$$

where:

V is the velocity of propagation of the wave,

T is the travel time of the wave.

The velocity (V) is measured either over core samples or over cylindrical samples made in advance.

The time of propagation (T) is determined from the signal that appears on the calibrated screen of an oscilloscope. The time is found by measuring the distance on the record between peaks of the oscillations (Fig. 153).

The impulse is applied at different points at the top of the pile and separate sets of readings are taken, according to the procedure set forth by the LCPC. Automating the analysis allows a significant increase in the speed of evaluating the measurements.

The piles can be investigated no sooner than seven days after concreting, or after the top is trimmed.

b) CONCERNS AND LIMITATIONS

The investigation of the quality of a pile by the transmission of waves using the reflection method with the wave generated by a mechanical impulse has two advantages:

-- the measurements are rapidly performed because the investigations can be performed on as many as 20 piles per day, depending on conditions at the field site; and

-- it is not necessary to have placed previously any access tubes.

Nevertheless, this method presents a number of inconveniences:

-- Beyond 15 meters (49.2 feet) of depth, the reflected wave is dampened so much that only a weak reflection will appear on the oscilloscope. Therefore, no defect may be detected beyond 15 meters (49.2 feet) with the present equipment.

-- A decision cannot be made about the quality of the first two meters (6.6 feet) of the top of the pile.

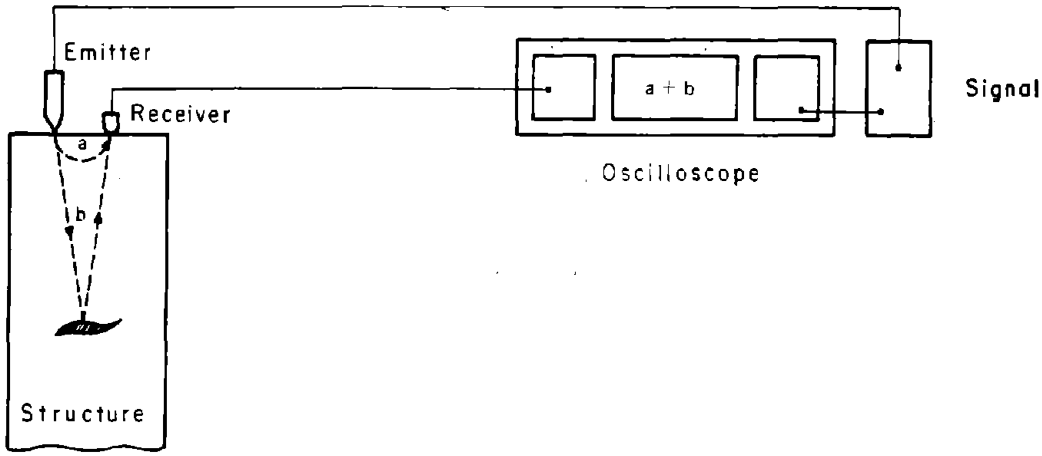


Fig. 152. -- Principle of investigation by reflection of waves.

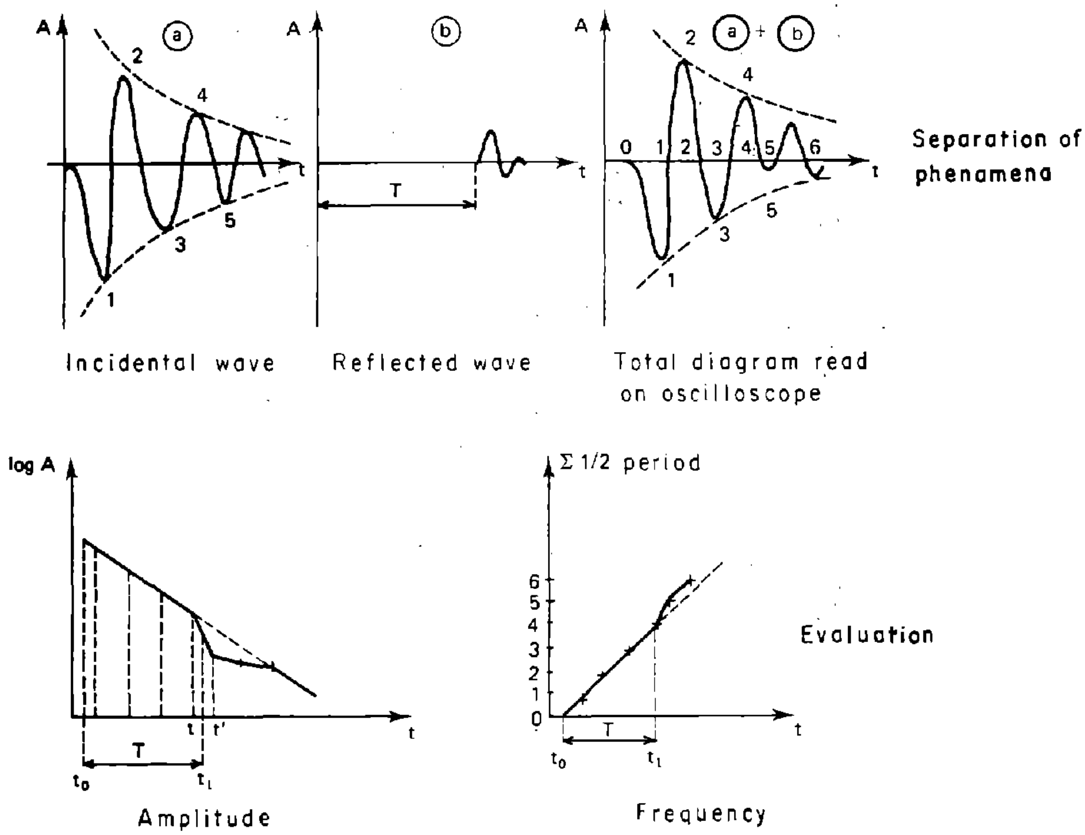


Fig. 153. -- Investigation by reflection of waves: representation of results.

-- The locating of defects is not very precise as to the position in a plane in a cross section of the pile nor as to depth, where the relative error varies between 6 and 30% depending on the level of the reflection zone.

-- This test does not permit investigation of the quality of the contact at the tip of the pile unless the substratum is a sound rock for which the mechanical impedance is at least equal to that of the concrete. Otherwise, (as in the case of marl, for example), it is not possible to reach a conclusion.

-- Using this method, an enlargement of the section may be interpreted as if it were a defect such as a narrowing of the diameter of the shaft.

-- Finally, a defect near the top of the pile might obscure other defects if the upper defect is extensive, or the upper defect may cause lower defects to appear to be minor.

7.1.1.2. Investigation method employing lateral transmission of waves

This method permits the study of variations in the quality of concrete over the entire length of the pile and the locating of the possible defects. Therefore, this method is qualitative.

a) PRINCIPLE

The method of investigation using the lateral transmission of waves (Figs. 154 and 155) consists of:

-- emission of an ultrasonic vibration in an access tube filled with water;

-- capturing this vibration at the same level in another tube filled to the same level with water after passing the vibration through the concrete of the shaft;

-- measuring the travel time and the amplitude of the oscillations that are captured first.

This operation is repeated at a high frequency and at levels sufficiently close to each other so that recording of the measurements may be considered to be continuous over the entire length of the shaft.

The measurement is only valid for the central part of the shaft between the access tubes (Fig. 156). For this reason, a defect at the edge of the pile might pass unnoticed (inadequate covering of the reinforcing steel with concrete, for example).

The results of the measurements (Fig. 157) are illustrated by two curves as a function of depth:

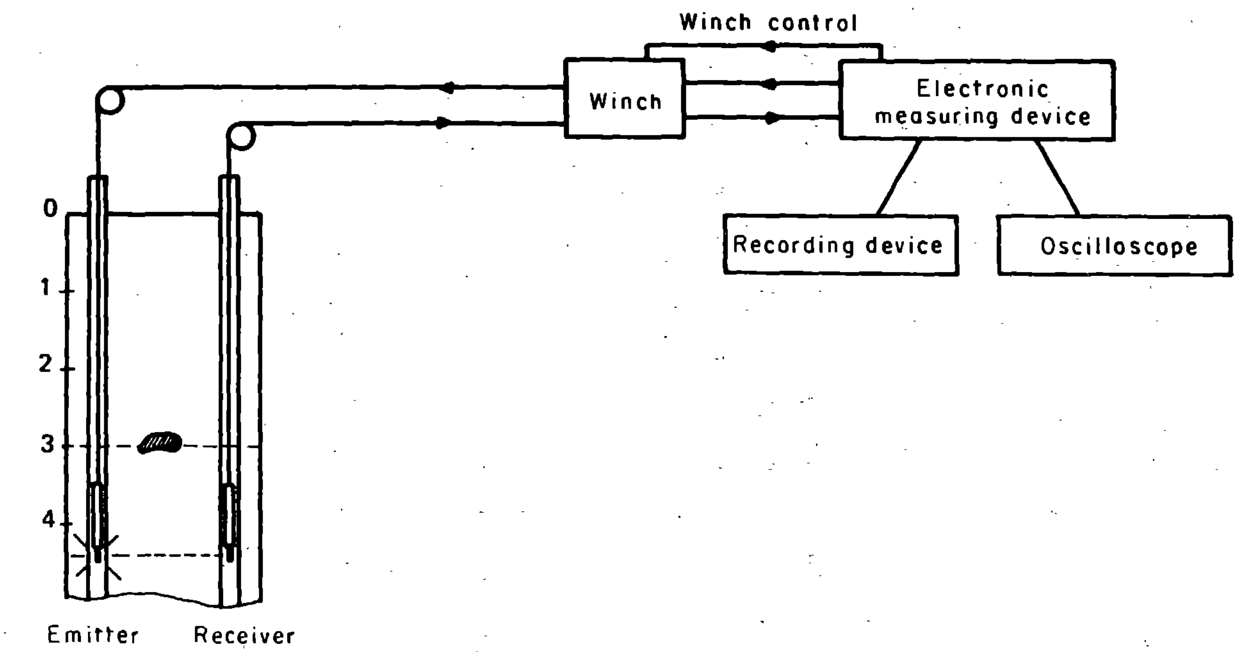
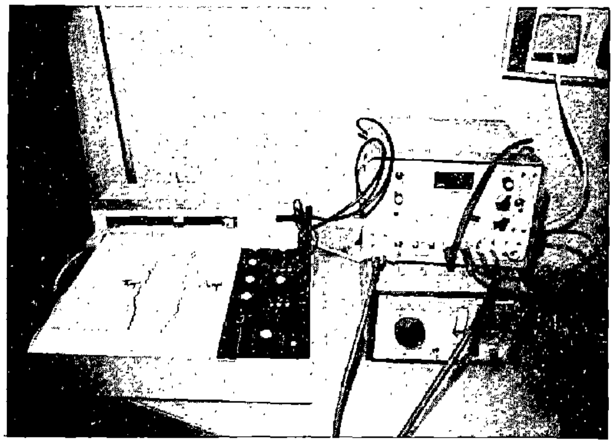


Fig. 154. -- Investigation using lateral transmission of waves, flow chart.



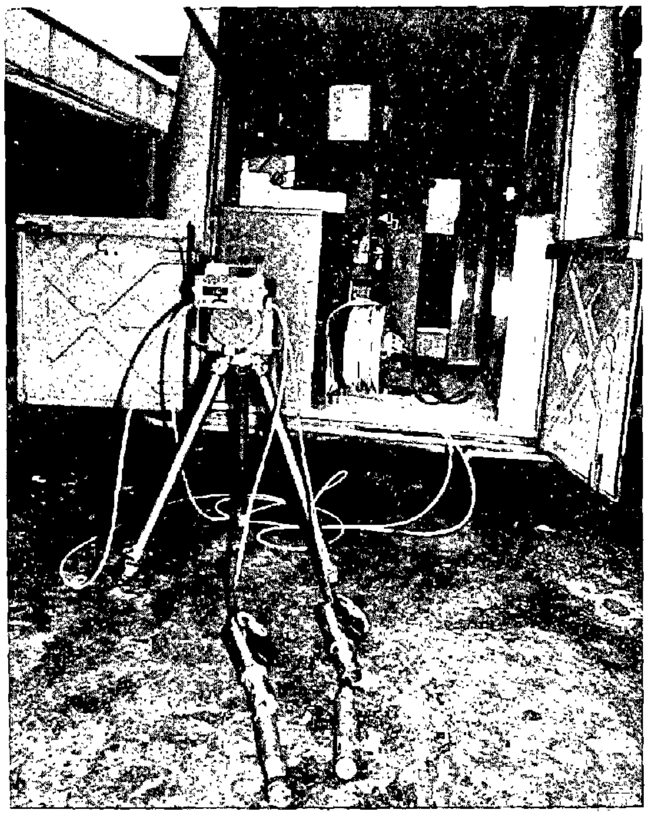
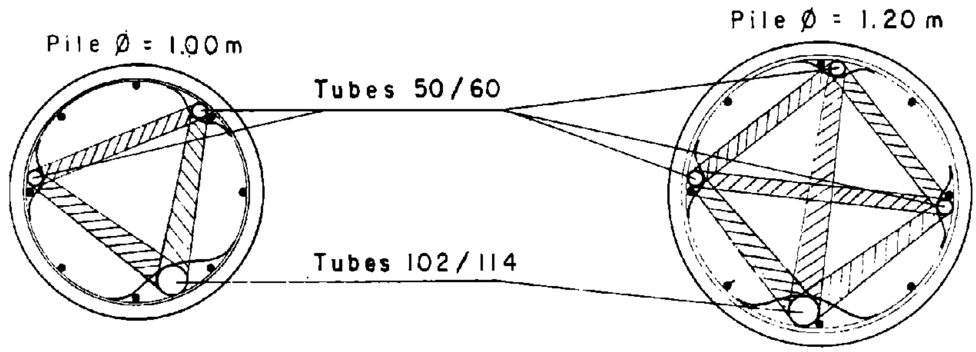


Fig. 155. -- Views of equipment for investigation using lateral transmission of waves.



(0.0254 m = 1 in.)

Fig. 156. -- Only the marked areas are investigated using lateral transmission (where no symbol is given, the dimension is mm, 25.40 mm = 1 in.).

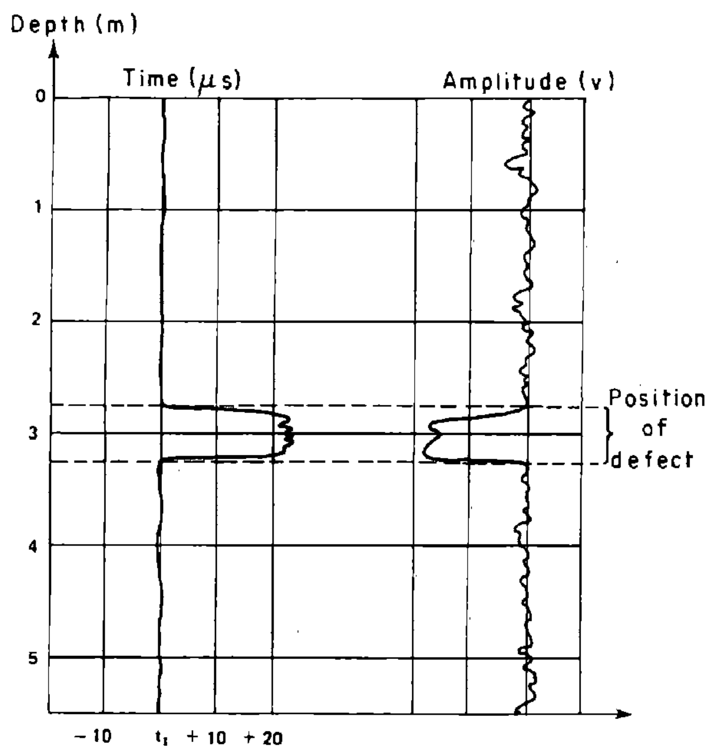


Fig. 157. -- Investigation using lateral transmission of waves, representation of results.

-- the time curve of the wave propagations, and
-- the curve of the variation of the amplitude of the captured waves. Graphically, each detected anomaly is characterized by a sharp decrease in the amplitude and an increase in the travel time.

b) TEST CONDITIONS

It is necessary to provide metal access tubes that vary in number according to the dimensions of the pile (see §7.3).

These tubes must be carefully cleaned before the operation starts to eliminate sediments or mud which might have been deposited. An efficient method is proposed in TN No. 12 for washing the tubes.

The minimum age for the concrete in order to apply this test under good conditions is two days. However, trimming the top of the pile should not be performed before the measurement is made. The trimming of concrete from the top of the pile may deform the tubes, preventing passage of the probes. It may also cause separation of the tubes from the concrete, making measurements impossible to obtain, at least over the first few meters.

c) PRODUCTION

Five to twelve piles may be investigated every day depending on:
-- the number of access tubes per pile (see § 7.2.7), and
-- the accessability and spacing of the piles or groups of piles. For piles that are difficult to access and/or are spaced large distances from each other, down times due to relocation of the equipment (winch, generating units, van, etc.) considerably limits the speed of the operation.

d) ADVANTAGES AND DISADVANTAGES

Advantages:

-- Good results in locating the anomalies as to depth as well as to transverse position in a plane in the shaft (if the number of tubes distributed along the circumference of the pile is adequate).
-- Immediate interpretation of the results.
-- Continuous recording over the entire length of the shaft of the pile.

Disadvantages or limitations:

-- This method also does not permit an investigation of the quality of the contact at the base; the testing must be halted, even in the most favorable case, at approximately 10 cm (4.0 in.) above the bottom of the pile. However, coring may be employed to investigate the quality of the contact at the base of the pile after the lateral-transmission method has been completed but the cost of the study will increase significantly [see § 7.4.1],

-- Placement of the access tubes must be anticipated before the concrete is cast, and their influence on the cost of the pile is not negligible.

-- The maximum distance between the access tubes is in the order of 1.5 meters (5.0 feet) using currently available instrumentation.

7.1.1.3. Method of investigation using gamma rays

This method permits the locating of the defects in the shaft, and possibily at the base, and the evaluation of the significance of the defects. In addition, the method indicates the degree of homogeneity of the concrete.

a) PRINCIPLE

This method is based on the phenomenon of the absorption of a parcel of gamma rays by the material they penetrate (Fig. 158).

If N represents the number of gamma photons detected after passing through a thickness x of a material with density γ, then:

$$N = N_o e^{-K\gamma x}$$

where:

-- N_o represents the number of gamma photons emitted per unit of time (N_o is, therefore, a function of the activity of the radioactive source), and

-- K is a coefficient that depends on the radiation used and on the nature of the material being investigated.

By means of a previous calibration, this procedure (Fig. 158) permits measurement of the density of the concrete over the length of the pile by recording the number of photons captured per unit time (Fig. 159).

b) TEST CONDITIONS

The test conditions are identical to those for the investigation method using the lateral transmission of waves (see § 7.1.1.2c and § 7.2).

c) PRODUCTION

Rates at which this test may be performed are limited to 4 to 8 piles per day, considering the speed of lifting the probes and precautions required when employing radioisotopes.

d) ADVANTAGES AND DISADVANTAGES

Advantages:

-- Good ability to locate anomalies, as to depth as well as to the position in a plane perpendicular to the axis of the pile.

-- Instantaneous interpretation of results at the site.

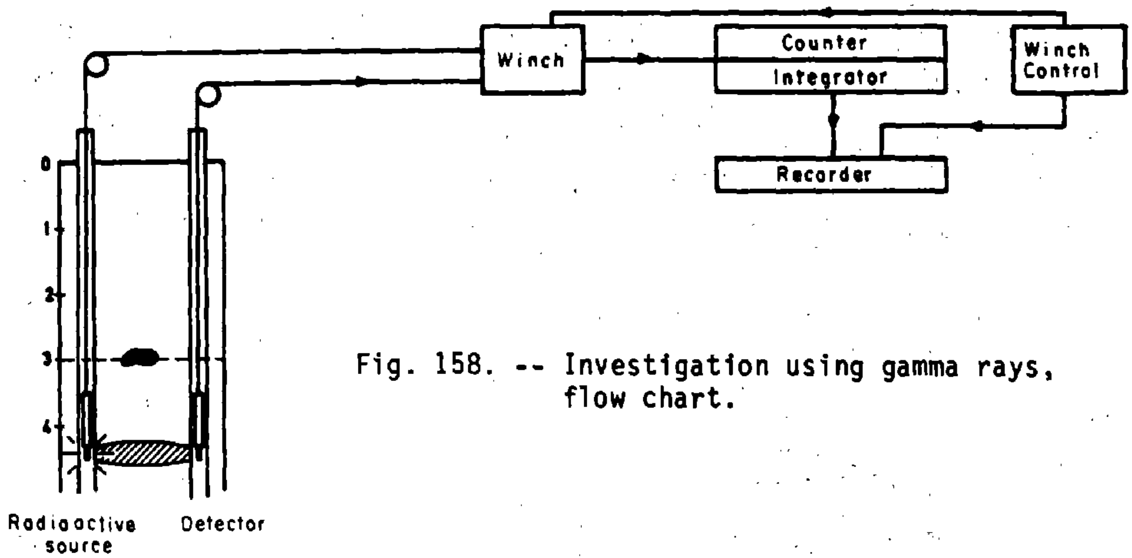


Fig. 158. -- Investigation using gamma rays, flow chart.

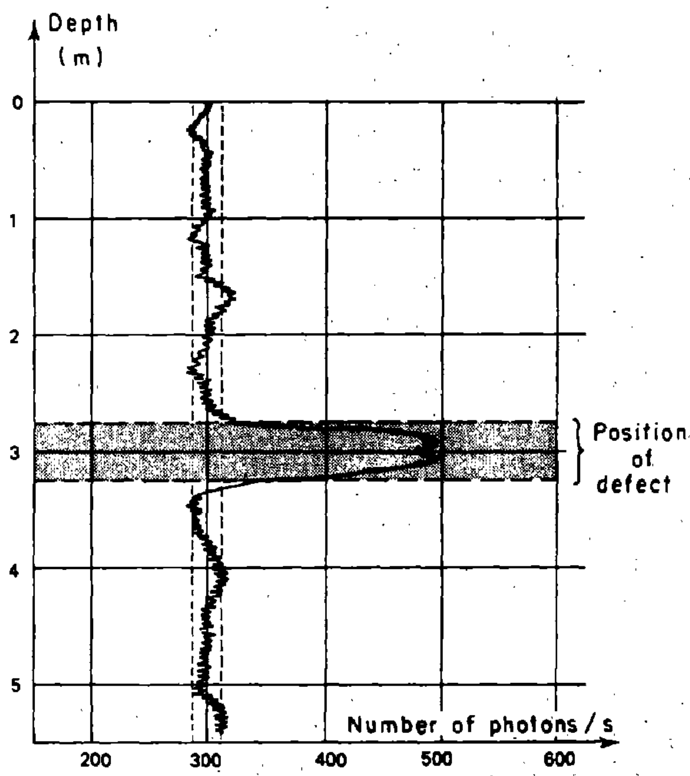
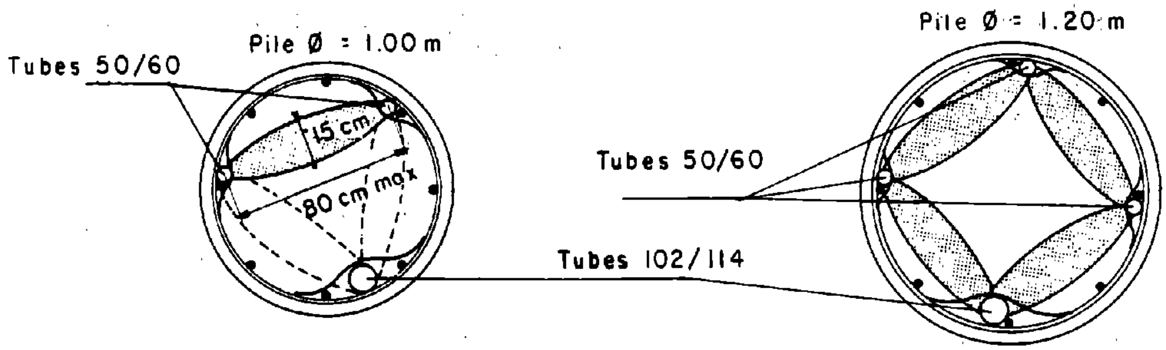


Fig. 159. -- Example of record and detection of a defect by investigation using gamma rays.

-- Continuous recording over the entire length of the shaft.

-- Possibility of detecting imperfections in the contact between concrete and soil (rock) at the base if the access tubes descend close enough to the bottom of the pile (< 5 cm - 2.0 in.). At each instant, the measurement gives results over a volume of concrete in the form of an ellipsis of revolution with the major axis equal to the thickness of the material being penetrated and with the minor axis equal to about 15 cm (6.0 in.) (Fig. 160).



(2.540 cm = 1 in.; 0.0254 m = 1 in.)

Fig. 160. -- Plan view of areas investigated using gamma rays.

Disadvantages:

-- The maximum distance between the access tubes is 80 cm (31.5 in.), a distance appropriate for three tubes for piles with a diameter of one meter (3.3 feet).

-- The method requires the taking of precautions at the site in regard to the use of radioactive sources.

-- The method also requires that the pile be equipped with a sufficient number of access tubes (see § 7.2.6).

7.1.2. Coring

The use of coring as a method of investigation is costly if continuous coring is performed over the length of the pile; nevertheless, it is practically the only method that permits assurance of the quality of the pile-soil contact at the tip of the pile.

In order to be cored, the concrete should be at least four days old. In addition, consideration should be given to the size of the coarse aggregate in the concrete (possibly from 15 mm to 25 mm - 0.60 in. to 1.0 in. in size). Theoretically, the cored hole should measure at least 75 mm (3.0 in.) in diameter. In general, a double core barrel of 80 to 100 mm (3.15 in. to 4.0 in.) in diameter is used.

7.1.2.1. Coring at the tip of the pile

Prior to concreting, a metal tube with a diameter of 102/114 mm (4/4.5 in.) (see § 7.2.1) is installed into the borehole so that the bottom of the pipe is approximately 50 cm (20.0 in.) from the bottom of the excavation. This permits the cost of the coring operation to be relatively modest. When a defect at the tip has been detected, the tubing further permits the cleaning of the concrete-soil interface by use of compressed, air-entrained water (see Technical Note No. 12 and [29]).

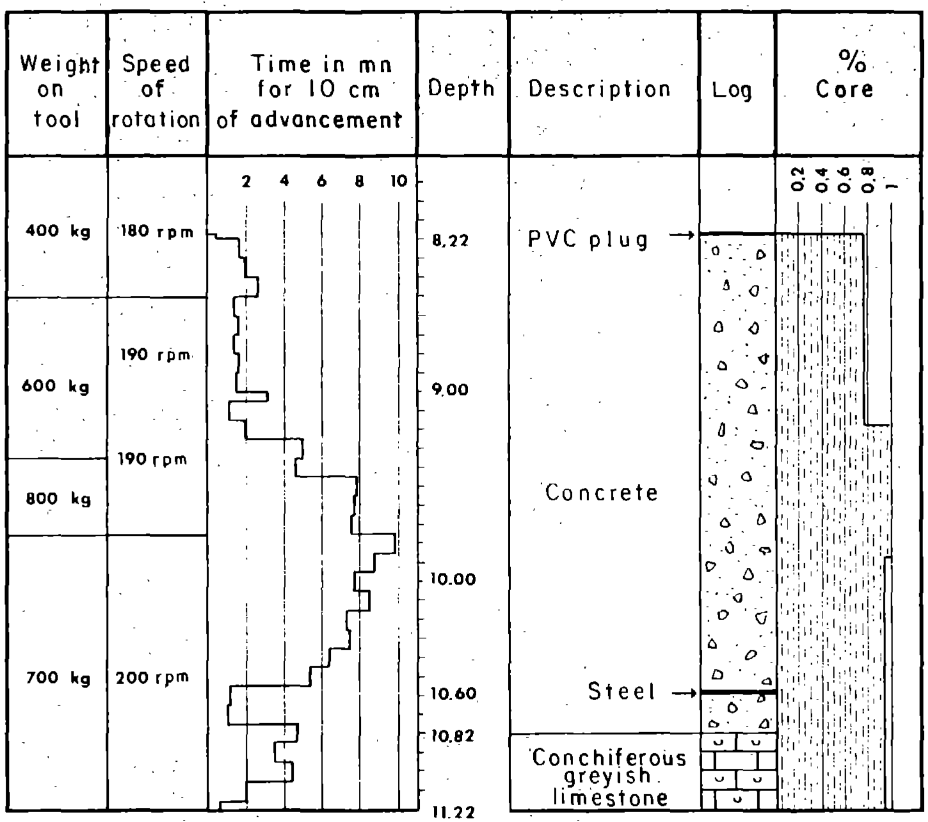
Coring of the last fifty centimeters (twenty inches) of concrete at the base of the pile should be supplemented by additional coring of the substrata layer for at least the same length (50 cm - 20.0 in.). During this operation, the rate of advancement for each pass (5 cm - 2.0 in. in general) in noted along with any observations that might help the specialist in his interpretation (Fig. 161).

The examination of the cores is done to determine the heterogeneity of the concrete (segregation, porosity, discontinuity in the concrete) and the quality of the concrete-soil contact at the tip. Obviously, a distinction must be made between cracks due to core drilling and discontinuities in the concrete. It is recommended that photographs be taken of the core samples (Fig. 162).

The speed of propagation of the ultrasonic waves may also be measured along with the resistance to simple compression of the samples obtained.

However, such an arrangement has the disadvantage of preventing the acoustic investigation by transmission of waves over a portion of the bottom of the pile because the access tube used for core drilling does not reach the tip.

When pipes of sufficient diameter are not specified for use in the coring of the tip, then, a possible defect that is detected using geophysical techniques can only be confirmed by coring, starting from the top of the pile. To minimize the cost of such a core boring, consideration should be given to making use of destructive methods (a wagon-drill, with a tricone bit) up to a depth of approximately one meter (3.3 feet) above the suspected zone, and then to proceed with the coring method.



(0.453 kg = 1 lb; 2.540 cm = 1 in.)

Fig. 161. -- Example of coring log of pile tip.

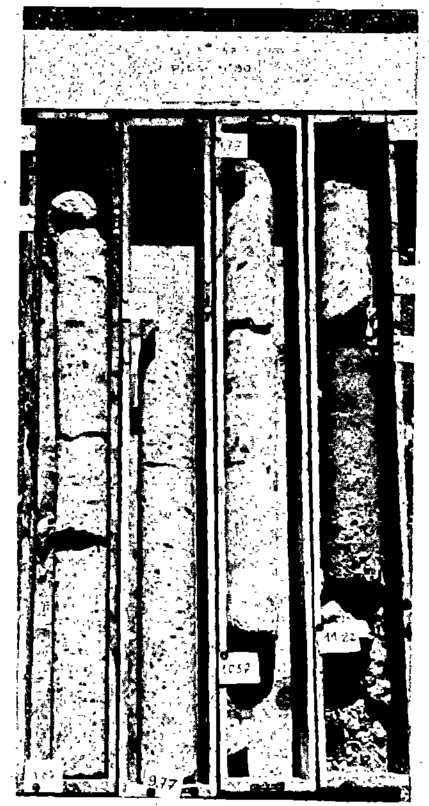


Fig. 162. -- Core of tip.

In general, this type of coring cannot be performed except in the central portion of the pile. Taking into consideration the probable form of the tip of the pile (Fig. 163) which is directly related to the method of placing concrete, especially at the base of shafts of large diameter, for which the "flushing" effect is misleading (see § 6.4.1 and § 6.4.2), central coring may not confirm peripheral defects revealed by acoustic-investigation methods. Coring, which is considered in principle to yield irrefutable evidence, does not actually reflect the true condition of the bottom of a pile, and may turn out to be dangerously optimistic. This limitation is especially significant because the results first obtained geophysically are more apt to be questioned than visual results from coring.

Therefore, the tendency to depend heavily on the results of coring must be avoided and the geophysical methods should be recognized as better revealing the condition of the pile than central-coring methods.

7.1.2.2. Coring of the pile shaft

Depending on the significance of a defect detected in the shaft of the pile by the methods just described, it is sometimes necessary to perform coring operations (Fig. 164).

The cores permit a visual verification of the nature of the concrete at the level of the defect, and the pile can be repaired if necessary (see Chapter 8). To avoid coring the high quality concrete of the tremie column, it is recommended if at all possible to core away from the center.

The techniques used and the rate of the advancement of the coring are identical to those for coring at the tip.

7.1.3. Miniature television camera

This method of observation, an adjunct to the primary reason for making core borings, permits the viewing of the sides of a core hole on a television screen.

7.1.3.1 Material

This includes:

-- a watertight, cylindrical camera, 48 mm (1.90 in.) in outside diameter, for which different "heads" may be used to allow axial or lateral views (Fig. 165);

-- a cable of 100 meters (328 feet) in length that connects the camera to the control center; the cable also serves to lower the camera into the hole.

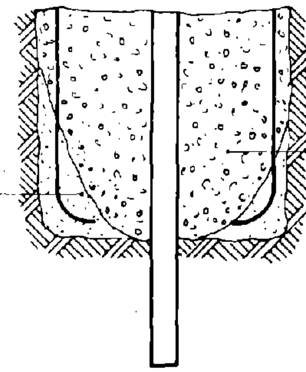


Fig. 163. -- Central core of pile tip; peripheral anomaly is not detected.

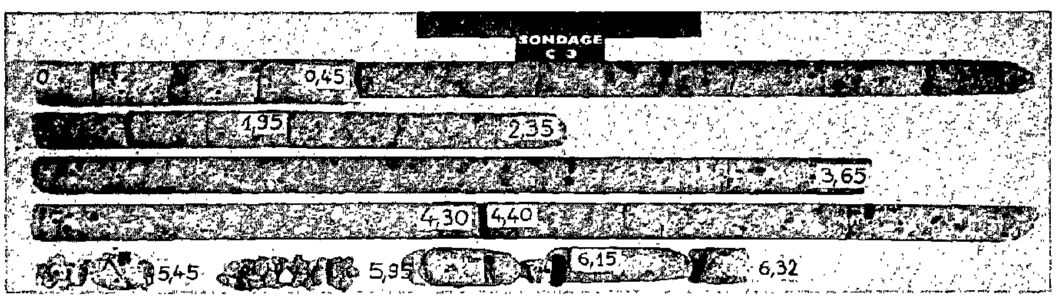


Fig. 164. -- Core of pile shaft confirming presence of defect previously detected between 5.45 and 5.95 m deep by investigation using lateral transmission of waves. (0.0254 m = 1 in.)

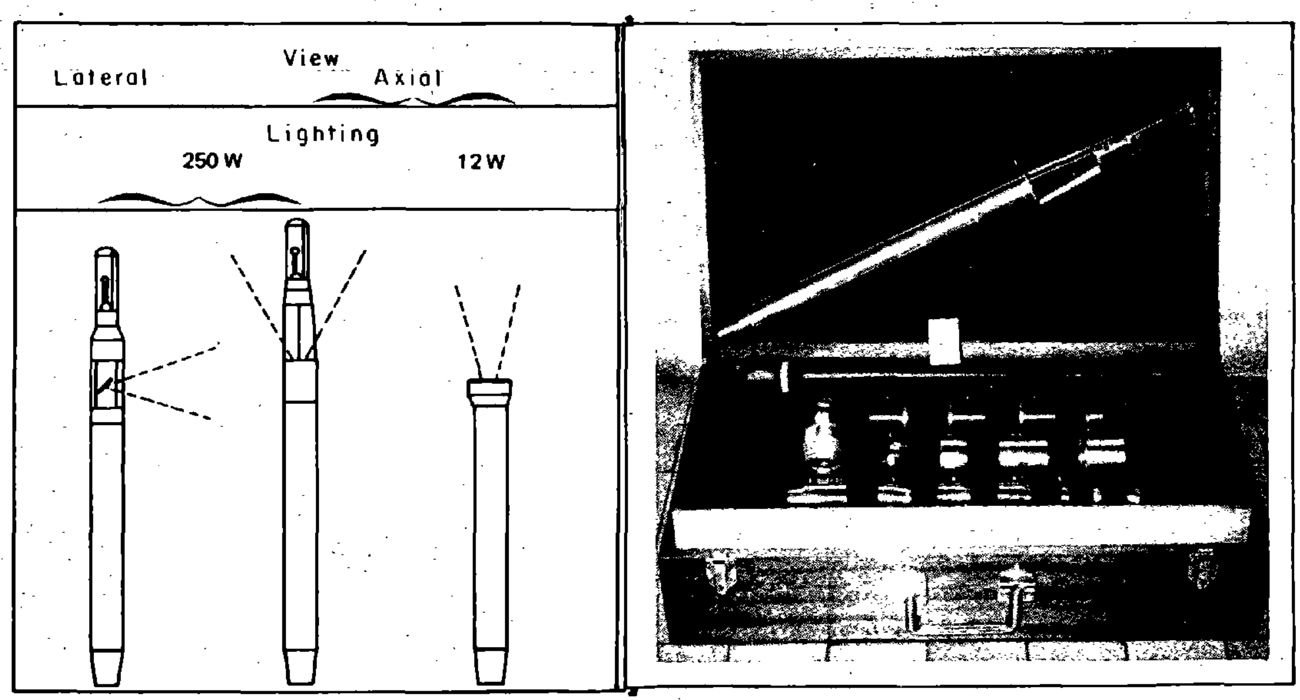


Fig. 165. -- Various heads for miniature television camera.

-- a control center;

-- a closed-circuit television receiver (video monitor); and

-- a high-performance, video-tape recorder.

Figure 166 shows a sketch of the principle of the television-camera set, which, for site operations, is mounted in a specially arranged vehicle.

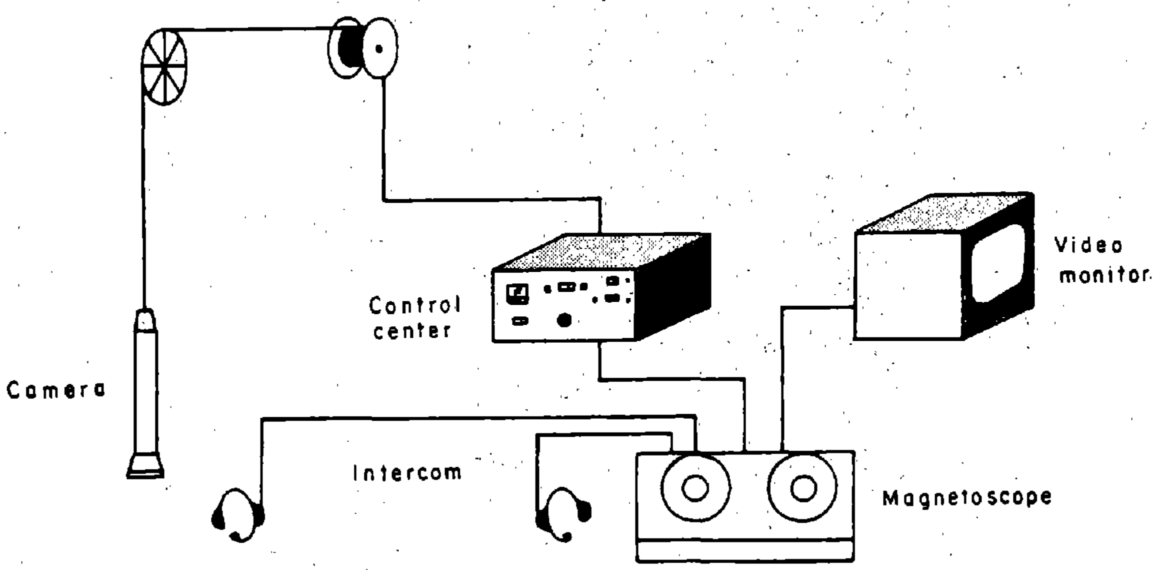


Fig. 166. -- Diagram of components of a miniature television camera.

7.1.3.2. Uses

This method is particularly well adapted to an evaluation of the quality of the area at the concrete-soil contact at the tip of a pile and of the quality of the concrete along the length of the coring.

In the case where a defect has been revealed at the base of a foundation by geophysical methods, observation with the television camera after coring permits an estimation of the extent of the defect as limited by the ability to provide lighting.

Figures 167 and 168 indicate the quality of some of the observations that can be made.

7.1.3.3. Conditions for operation

It is not advisable to lower the camera into boreholes with a diameter of less than 65 mm (2.6 in.).

Fig. 167. -- Axial view: bottom of defective pile. Seen are a vertical rebar of reinforcing cage (on top) and a horizontal rebar of basket (below). The central black circle is lighting support.

Fig. 168. -- Lateral view: bottom of defective pile with vertical rebar of reinforcing cage protruding from concrete.

Because it is necessary to clean carefully the core hole before operating the camera, clean water should be available on the site. This cleaning can be performed quite easily using an emulsifier (see Technical Note No. 12).

Furthermore, to improve the image it is sometimes necessary to inject either pure, clear water under low pressure in front of the camera, or to cause flocculation of the suspended particles using appropriate products.

7.1.3.4. Production

Rates of work are extremely difficult to predict because the rates depend primarily on the difficulties that are encountered: site conditions, possibility of caving, clearness of water, etc. Nevertheless, under the right conditions, the following rates may be expected (not including coring time):

-- for observation of the concrete-soil contact: 3 to 4 piles per day;

-- for observation along the full length of the pile (shaft + tip): 2 piles per day; and

-- for observation of a defect at a given height: 2 piles per day.

7.2. RECOMMENDATIONS FOR ACCESS TUBES

It has been shown that most well performed post-construction testing requires that the piles are initially equipped with vertical access tubes in order to permit:

-- examination of the shaft by means of the lateral tansmission of sonic waves or by the use of gamma rays;

-- coring of the concrete-soil contact;

-- possible examination of the tip using a miniature television camera;

-- after coring, grouting of the tip by injection.

These tubes must meet certain requirements that influence the success of wave-transmission methods, interpretation of recordings, and performance of coring at the tip.

7.2.1. Nature and diameter of tubes

-- *Tubes for investigation of the shaft using wave-transmission methods:* These are metal tubes that should have a minimum inside diameter of at least 50 mm (2.0 in.). Tubes of the type used for heating systems, commonly 50/60 mm (2.0/2.4 in.) in diameter, delivered in 6 m (19.7 ft) lengths, fitted with threads at both ends, are perfectly suitable.

-- *Tubes for investigation of the shaft using wave-transmission methods and coring of the tip:* these tubes are metal and should have a minimum inside diameter of 100 mm (Ø 102/114 mm - 4.0/4.5 in. - for example), and also are threaded.

If necessary, the tubes should be cleaned with a non-greasy product before use in order to prevent oil films from causing problems of adherence between the tube and the concrete. In fact, in the case of investigation by the lateral transmission of waves, a bad joint between the metal tube and the concrete will cause a significant reduction in the sonic waves, causing the recording to show a time variation of propagation and amplitude. This variation might be incorrectly interpreted as a significant defect in the pile.

7.2.2. Connecting the access tubes

It is *necessary* that the tubes be connected to each other by a screwed coupling.

In any case, welding must not be used for connecting the tubes because it does not necessarily ensure a free and open access for a probe and often causes protrusions inside the tubes that could interfere with the lowering of the instruments.

7.2.3. Plugs

Plugs should seal the bottom of the access tubes to prevent grout or concrete from entering the tubes during the placing of the concrete.

In order to allow the maximum depth of investigation by the wave-transmission method and to permit easy drilling of the concrete at the tip (wagon-drill) or coring, the following closures *must not be used:* wooden plugs that are fitted by force, welded metal plugs, mortar plugs, or packed fabric, etc. The preferred method is to use screwed, PVC caps, for example of the Armosing type, also referred to as:

-- BBG 2 in. or BG 60 for tubes 50/60 mm (2.0/2.4 in.);

-- BBG 4 in. or BG 114 for tubes 102/114 mm (4.0/4.5 in.).

In order to prevent delays in the delivery of supplies, the plugs must be provided by the CCTP at the same time as the access tubes are provided.

The upper ends of the tubes must be closed to prevent debris or concrete from causing an obstruction.

7.2.4. Attachment to rebar cage

Systems for attaching the tubes to the reinforcing should be solid to resist the "pressure and bouyancy" of the concrete on the tubes at the time of the concreting. Brackets for attaching the tubes should be sufficiently close together (approximately 3 m - 9.8 ft) to limit deformations of the tubes during placement of the rebar cage or during concreting.

7.2.5. Length of access tubes

The access tubes should meet the following requirements (Fig. 169):

-- tubes for investigation of the shaft by wave-transmission methods should reach the base of the rebar cage; and

-- tubes for coring of the tip should stop at 0.50 m (1.60 ft) from the base of the cage.

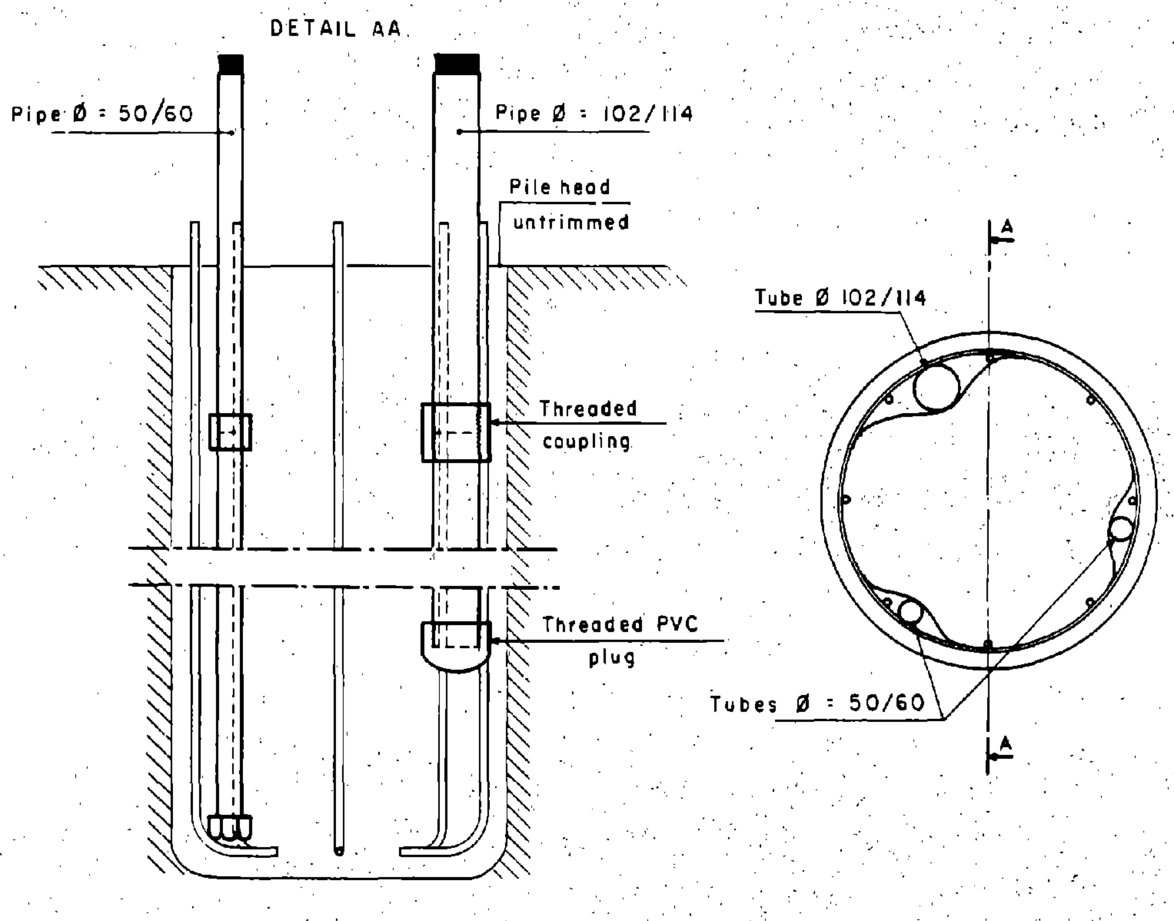


Fig. 169. -- Positioning of access tubes for wave transmission and coring of a pile of diameter less than or equal to 1 m. (1 m = 3.28 ft)

Above the untrimmed top of the pile, the access tubes should extend for at least 0.50 m (1.60 ft) to permit the instruments to be referenced to a datum prior to testing and to prevent bits of mud or concrete from falling into the tube.

7.2.6. Arrangement of access tubes

The arrangement of the tubes and their number vary in relation to the diameter of a pile. The diagrams in Fig. 170 represent the arrangements that are desirable in respect to the diameter of a pile in order to be able to apply the test methodology described in Section 7.3.3. The recommended spacing is given for information only and can be modified according to the desired accuracy of the measurement of detected anomalies (accuracy increases with the number of tubes).

Pertaining to barrettes, the arrangement of the tubes and their number should be determined in relation to the cross-sections and dimensions of the barrettes. Agreement should be worked out with the laboratory in charge of testing. The following principles should be observed:

-- the distance separating two tubes should not exceed 1.50 m (4.9 ft) in the case of investigation by wave-transmission methods, and 0.80 m (2.6 ft) for investigation by gamma-ray methods, in order to accomodate the limited capabilities of the equipment, and

-- for investigation by the wave-transmission method, the space between the tubes, taken two by two, should involve the maximum of the volume of the barrette.

The diagram in Fig. 171 represents the arrangement that is generally used for a barrette 2 m (6.6 ft) long and 0.80 m (2.6 ft) wide.

7.3. TEST ORGANIZATION

This section discusses only the testing that should be described at the time of the preparation of the plans for the project. Additional tests may be required by the construction supervisor for a pile that is considered to be defective as a result of a recognized incident; for example, an interruption in concreting.

7.3.1. Problem location

The construction supervisor often finds himself facing a difficult decision. On one hand, it is desirable to ensure the quality of the foundations of the structure to guarantee good ultimate behavior, and, on the other hand, the total cost of testing must be limited, or at least be in proportion to the cost of the foundations. The recommendations

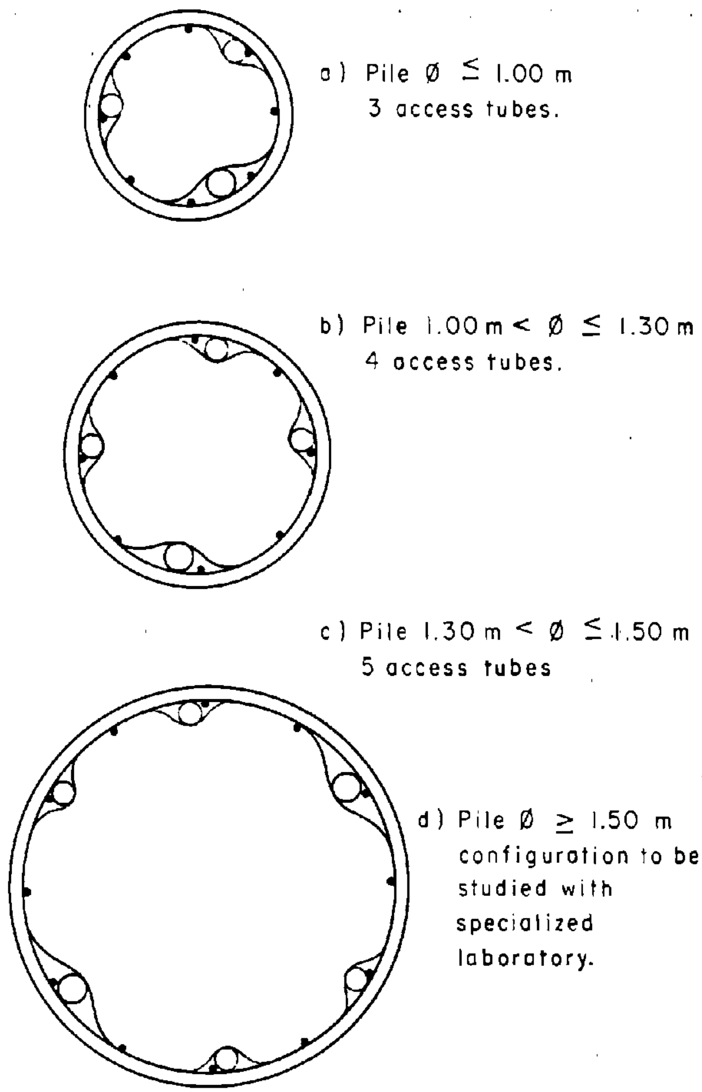


Fig. 170. -- Arrangement of access tubes in relation to diameter of piles when using sonic or gamma transmission waves. (1 m = 3.28 ft)

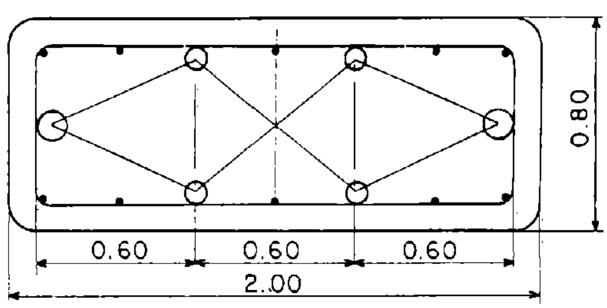


Fig. 171. -- Example of distribution of access tubes in barrette of 200 x 80 cm. (2.540 cm = 1 in.)

given in the following paragraphs show that a definitive and thorough testing may cost a great deal. However, what is the risk if defective construction techniques have been used?

In general, improper construction in a pile constructed in place may either result in poor contact at the tip with the foundation soil or in heterogeneities in the shaft.

a) *If the contact at the tip is defective,* and if the pile is designed to perform mainly as an end-bearing pile (little resistance due to skin friction), the risk is obvious. At the time of its loading, the pile may move downward a relatively significant amount if an actual void separates the base from the soil. This may result in excessive settlement for a given pile, or excessive differential settlement for piles in a group, resulting in higher stress levels in certain piles that exceed the criteria for which they were designed. When a given structure is loaded and if several piles are defective, an overall settlement of the foundation may result. The bending stiffness of the superstructure, an abutment, or a mat foundation will dictate how tolerable this settlement is.

b) *Poor construction of the shaft* may have similar consequences. For example, if at a given level the diameter of a pile is clearly less than its theoretical diameter, the stresses in the concrete are increased in relation to the calculated stresses and, in extreme cases, may surpass the concrete resistance to failure. Longitudinal rebars that have an insufficient cover of concrete may buckle in the weak section, and a settlement of the top of the pile will result. Additionally, such weak sections resist bending poorly; they often behave like joints, such that if the foundation is subjected to significant horizontal forces (abutments for example), greater displacements than anticipated will result.

Thus, it may be seen that poor construction of the piles in a foundation may have serious consequences concerning the performance of the structure, depending on the nature and significance of the affected zones. The question of durability of a pile has not been discussed but is also very important. Possible defects in a pile may not appear instantly but may arise several years after construction, when any repair to the foundation is virtually impossible.

The selection of the testing that is to be done should rely on an analysis of the technical aspects of each test method and on the degree to which risks are eliminated. This analysis may be done according to certain criteria that are discussed more fully below.

7.3.2. Criteria for test selection

7.3.2.1. Technical criteria

The technical criteria for use in selection of a method of testing to locate and describe possible defects in a pile are linked to the technological limitations of the test methods and to the geometry of the pile. For example, the wave-transmission method where the wave is generated by a mechanical impulse and the reflection of the wave is captured by an oscilloscope is incapable of detecting a defect beyond 15 meters (49.2 feet). Such limitations are stated for each method in Section 7.1 of this chapter and are not repeated here.

First, the construction supervisor should ensure that the testing program that has been planned is capable of being carried out by the testing laboratory that has been selected. If the testing involves the Regional Laboratories of Bridges and Roads, (see § 7.4.2) the majority of these laboratories can carry out the wave-transmission methods, some are capable of performing the gamma-ray techniques, and one of them has the capability of an investigation with a miniature television camera.

However, it is noted that in general when one of the Laboratories of Bridges and Roads is not capable of performing one or another of the geophysical methods of investigation, the laboratory may require an associated laboratory to provide the equipment needed for performance of all or part of the procedures.

Pertaining to the soil-pile contact at the base of the pile, only the coring method at the tip of the pile is adequate for this investigation. The gamma-ray method cannot provide more than an indication under the best of circumstances. In case of litigation between the construction supervisor and the contractor, the use of the television camera may then prove useful.

7.3.2.2. Criteria associated with structural design

Requirements for the quality of foundations may vary according to the nature of the structure being supported. For simplification, two categories of structures are used:

-- structures that are not standard, and are very rigid, and

-- other structures.

This terminology is taken from Circular No. 75-146 of September 24, 1975 by the Ministery of Equipment. An example of a very rigid, non-standard structure would be a bridge, with a continuous deck of constant thickness, that will be subjected to significant loads as from a railroad.

For structures that fall within the first category, it seems reasonable to avoid any risks concerning the foundation even though the number of supports for the structure would appear to be ample. Generally, for an exceptional structure (bridge that is cable-stayed, suspended, arched, etc.), the number of supports is limited and each pier that is pile-supported is over barrettes or piles of large diameter. A systematic test of all the piles will have little financial bearing on the overall cost of the structure and it would be absurd to attempt to take shortcuts in the testing. In the same way, if the structure is very rigid, any differential settlement is generally not allowed so it is also necessary to plan on an extensive geotechnical investigation for the foundations.

For structures in the second category, a reasonable risk seems acceptable. However, because the tendency is to construct piles of large diameter and of limited number, the decision as to whether or not to investigate the quality of the piles as constructed should be made after considering each individual case.

7.3.2.3. Criteria related to design and method of construction of foundations

Certainly, it is criteria related to design and construction that require the greatest attention on the part of the construction supervisor.

Related to the design plans, it is clear that risks resulting from improper construction of a pile are more significant when the foundation simply consists of a few large piles than if it consists of many piles of small diameter.

Another important point is that it is known that construction problems vary according to whether the piles are concreted in the dry, with mud, with a temporary casing, or with or without a permanent casing or lining, etc.

The geotechnical conditions are also very important with respect to the quality of construction. For example, for cases where piles or barrettes are constructed through a soil in which water is moving, there is a considerable risk of segregation of the concrete.

Thus, decisions about a testing program are complex and the following section gives only general recommendations that may be modified or adapted to each particular case.

7.3.3. Recommendations for selection of testing methods for completed piles

7.3.3.1. Case of testing of piles for structures in the first category

As previously stated, the testing should be carried out for all the piles when such structures are involved.

Furthermore, these tests should generally be carried out for piles that sustain their load exclusively or partially in end bearing and rarely solely by friction. It is thus desirable to provide for the possibility of verification of the quality of the shaft as well as the concrete-soil contact at the tip.

As a result, all the piles supporting a structure of the first category, independent of the length of the piles, should be equipped with access tubes with a diameter of 102/114 mm (4.0/4.5 in.) for coring at the tip; The number of these tubes is a function of the diameter of the piles (see § 7.2.6).

With access tubes in place, the principles to be employed in the testing may be considered in an orderly fashion, as in the following:

- *Systematic* investigation employing wave-transmission method.
- Gamma-ray investigation of any questionable piles that were identified by the wave-transmission method, either to identify the significance and nature of defects that were not clearly revealed by the wave-transmission method, or to confirm possible anomalies suggested by mishaps during the course of construction (quality of the concrete, concreting problems, etc.), but not detected simply by using the wave-transmission test.
- *Systematic* coring of the tip through the tubes 102/114 mm (4.0/4.5 in.).
- Partial coring at the base of borings made by a wagon-drill to confirm possible significant defects located in the shaft by wave-transmission or gamma-ray methods.
- Viewing of recognized defects in cored boreholes by television camera.
- Wave-transmission or gamma-ray investigations of the repaired piles to verify the efficiency of the treatment (see § 8.4.3).

7.3.3.2. *Case of foundations for structures of the second category*

For these structures, Table IV shows a test methodology that may be used as a starting point. The testing can be modified depending on the procedure used for constructing the piles.

Once the amount of the testing is defined, the construction supervisor may easily determine the possible quantities of access tubes that will be needed. He is then responsible for the work and should see to the testing, *particularly of the piles that were constructed first* (especially the test pile), those that require testing the most (inclined piles for an abutment for example), and those that may be constructed under the most complex conditions (piles at an aquatic site, for example).

TABLE IV

TABLE TO INDICATE PERCENTAGE OF PILES TO BE EQUIPPED AND TESTED ON CURRENT JOBS

in relation to:
-- the manner in which the loads are resisted by the piles
-- the total number of piles (N)
-- the number of piles per foundation element (n)

Load distribution method	N	n ≤ 4				n > 4			
		Access tubes		Tests		Access tubes		Tests	
		Tubes 50/60	One Tube 102/114 (1)	Investigation of shaft	Coring at tip	Tubes 50/60	One tube 102/114 (1)	Investigation of shaft	Coring at tip
Skin friction only	≤ 50	100	0	100	0	100	0	50 - 100	0
	> 50	100	0	100	0	50 - 100	0	50 - 100	0
Skin friction + tip	≤ 50	100	≥ 50	100	≥ 30	100	≥ 30	50 - 100	≥ 20
	> 50	100	≥ 30	50 - 100	≥ 20	50 - 100	≥ 20	50 - 100	≥ 10
Tip only	≤ 50	100	100	100	50 100	100	50 - 100	50 - 100	≥ 30
	> 50	100	50 100	50 - 100	≥ 30	50 - 100	≥ 30	50 - 100	≥ 20

(1) See Section 7.2.6 for details on number and arrangement of access tubes in relation to pile diameter.

7.3.4. Specifications concerning test operations

The administrative texts generally lack specifications governing the control of tests for piles. Only Clause 36 of Article No. 1 of the CPC recommends, if the CCTP prescribes tests or controls in greater number than those defined in the special clauses of the CPC, the manner in which these extra tests are to be administered and paid for. Clause 26 of the CCAG, § 1, calls for demolition of the structures or portions of the structures presumed defective, and the resulting expenses of this operation are charged to the constractor once the construction defects are verified and recognized.

This clause does not apply in the case of foundations of bored piles because a pile that is poorly constructed cannot be demolished; it is repaired or replaced by several supplementary piles.

In view of the absence of detailed specifications, an analysis is proposed that seems logical. Two cases are studied: that of the test defined in the contract, and that of the supplementary test that may be required by the construction supervisor of a pile that may have been constructed improperly.

7.3.4.1. Case of test defined in the contract

The supply and installation of possible access tubes should appear in the bidding documents. Their cost is included in the overall estimate by the contractor.

The test is performed by a laboratory that is remunerated directly by the construction supervisor.

If the test results do not indicate any poor construction, there is no problem. On the other hand, if the results of the first test are bad, several cases may arise:

a) *The contractor accepts the results of the test.* The construction supervisor then asks him to make proposals for repair of located defects (these proposals require either repair or construction of supplementary piles). If the construction supervisor accepts the proposals, their construction and cost are the responsibility of the contractor, possibly under control of the laboratory, and the test operations should be overseen by the construction supervisor.

If the construction supervisor does not accept the proposals by the contractor, there is litigation. The construction supervisor may employ coercive measures to obtain the type of remedy he desires and repair is also at the expense of the contractor.

b) *The contractor refutes the results of the first test.* Independent of questions of bad faith, it may be that interpretation of the control test is not obvious. The doubt may not be removed except by a second test using more extensive methods (for example, coring and, if the case arises, use of a miniature television camera). This second test will be performed by the laboratory that performed the first test or by another laboratory agreed upon by both the construction supervisor and the company.

If the second test does not reveal any defect, it seems appropriate that its remuneration be taken care of by the construction supervisor. On the contrary, if a defect is found, the expense of this supplementary test should be charged to the contractor and referral to the preceding case is made.

7.3.4.2. Case of supplementary test not specified in the contract

The steps may be the same as in the preceding. However, it should be realized that a supplementary test that has not been specified in the contract always costs more. Be that as it may, if the test pile is satisfactory, the construction supervisor should assume the resulting expenses of this test. On the other hand, if the pile is defective, the response is not obvious. Certainly, the repair expenses will be charged to the contractor, but the test expense may require discussion. We believe that if the defect in the construction of the pile is the result of an unpreventable delay, the test expense should be charged to the construction supervisor.

7.4. COST AND PRACTICAL INFORMATION

7.4.1. Cost of tests on finished piles

The prices quoted are the TTC prices of 1977. They do not include supply nor installation of access tubes for which the average sum may vary from 30 to 55 F/m depending on the diameter.

These prices are given, for information only, with a significant margin of error that includes the following factors:

-- *The distance* from the laboratory in charge of testing and the pile job site. The greater this distance is, the greater is the time allotted to personnel and vehicles for travel time. It is thus desirable, for job sites located far from the laboratory, to limit the number of operations by grouping the piles to be tested in an amount that is compatible with the rates of geophysical investigation and coring (see § 7.1.1). However, these tests should be performed sufficiently early on the first piles of a job to allow, when needed, for correction of construction procedures for subsequent piles.

-- *The general conditions of the job site* that involve:

- distance from the piles or groups of piles to be tested. Placement, calibration, and removal of the equipment require the most time and are the most complex operations;
- accessibility to the piles for the same reasons; and
- proximity of water.

-- *Preparation of the piles,* entrusted to the construction company and consisting of:

- verification that the access tubes are not obstructed and, if the case arises, their cleaning; and
- filling of the tubes with water in the case of investigation by lateral transmission of waves.

Insofar as these operations are performed before arrival of the investigation crew, the anticipated productivity may be achieved.

Taking these criteria into account, the cost of tests on finished piles may be evaluated on the following basis:

a) *Geophysical investigation of the shaft* (despite the method used)

- Price for one pile: 600 F ± 200 F. this price does not depend on the length to be investigated, but varies as a function of the number of access tubes per pile.

b) *Coring at the tip*

- Price for one pile: 2,000 F ± 500 F with coring diameters from 86 to 96 mm (3.38 to 3.78 in.), over 1 to 1.5 m (3.3 to 4.9 ft) of depth.

c) *Miniature television camera*

Due to the fact that only the Bordeaux Laboratory is equipped with this device, the operation cost varies according to the number of days of operation (see rates in § 7.1.3.4) and according to the distance from the job site to the Bordeaux Laboratory.

- Average price for one day: 4,200 F,including interpretation.
- Additional cost for transportation of equipment between Bordeaux and the operation location: 750 F per 100 km (62 miles) beyond the first 100 km (62 miles).

d) *Coring of a shaft (86 mm - 3.38 in. diameter)*

- Average price by the meter: 750 F ± 150 F.

It is noted that scarcely more than one pile per day may be anticipated.

e) *Use of wagon-drill* (generally by the construction company)

-- For perforation of the tip of the piles at the base of the geophysical investigation tubes:

• Average price per day: 2,000 F ± 300 F.

-- For pre-boring in the shaft, up to the required level for coring or injection:

• Average price by the meter: 300 F ± 100 F.

7.4.2. Addresses of the Laboratoires des Ponts et Chaussées (LPC)

Table 5 summarizes the capabilities of the various Laboratoires des Ponts et Chaussées (central laboratory and regional laboratories) pertaining to geophysical investigation and tests for finished piles.

TABLE V

Laboratories	Address - Telephone	Tests Performed
AIX-EN-PROVENCE	Zone industrielle, rue Einstein B.P. n° 39 13290 LES MILLES Tél. : (42) 27 98 10	Reflection of waves Lateral transmission of waves Coring Wagon-Drill
ANGERS	Avenue de l'Amiral-Chauvin B.P. n° 66 49130 LES PONTS-DE-CE Tél. : (41) 66 86 43	Lateral transmission of waves Radioactive wave transmission Coring
AUTUN	Zone Industrielle B.P. n° 141 71406 AUTUN CEDEX Tél. : (85) 52 02 12	Lateral transmission of waves Coring Wagon-Drill
BORDEAUX	472, avenue du Maréchal-de-Lattre-de-Tassigny B.P. n° 57, Bordeaux-Caudéran 33019 BORDEAUX CEDEX Tél. : (56) 47 14 24	Reflection of waves Lateral transmisstion of waves Radioactive wave transmission Television camera Coring Wagon-Drill
BLOIS	Rue Laplace 41000 Blois Tél. : (54) 74 29 50	Reflection of waves Lateral transmission of waves Coring
CLERMONT-FERRAND	8-10, rue Bernard-Palissy Zone Industrielle de Brézet B.P. n° 11 Saint-Jean 63014 CLERMONT-FERRAND CEDEX Tél. : (73) 91 22 70	Lateral transmission of waves Coring
LILLE	1, route de Séquedin B.P. n° 99 59320 HAUBOURDIN Tél. : (20) 50 40 00 et 50 44 33	Reflection of waves Lateral transmission of waves Coring
EST-PARISIEN (Centre du Bourget)	Rue de l'Egalité B.P. n° 34 93350 LE BOURGET Tél. : (1) 837 61 00	Reflection of waves Lateral transmission of waves Coring
ROUEN	Chemin de la Poudrière B.P. n° 247 76120 LE GRAND-QUEVILLY Tél. : (35) 63 81 21	Lateral transmission of waves Radioactive wave transmission Coring Wagon-Drill
OUEST-PARISIEN	12, rue Teisserenc-de-Bort B.P. n° 108 78195 TRAPPES CEDEX Tél. : (1) 050 09 27	Lateral transmission of waves
LYON	109, avenue Salvador-Allende B.P. n° 48 69672 BRON Tél. : (78) 26 88 25	Lateral transmission of waves Coring Wagon-Drill
NANCY	50, rue Grande-Haie B.P. n° 8 54510 TOMBLAINE Tél. : (28) 29 52 09	Lateral transmission of waves Coring Wagon-Drill
TOULOUSE	1, avenue du Colonel-Roche Complexe Aérospatial 31400 TOULOUSE Tél. : (61) 53 35 35	Reflection of waves Lateral transmission of waves Coring Wagon-Drill
STRASBOURG	Rue Jean-Mentelin B.P. n° 9 Strasbourg-Kœnigshoffen 67035 STRASBOURG CEDEX Tél. : (88) 30 41 12	Reflection of waves Lateral transmission of waves Coring
SAINT-QUENTIN	151, route de Paris 02100 SAINT-QUENTIN Tél. : (23) 67 01 29	Lateral transmission of waves Coring Wagon-Drill
SAINT-BRIEUC	12, rue Sully 22000 SAINT-BRIEUC Tél. : (96) 33 40 32	Coring
Laboratoire Central des Ponts et Chaussées	58, boulevard Lefebvre 75732 PARIS CEDEX 15 Tél. : (1) 532 31 79	Lateral transmission of waves

CHAPTER 8

POOR CONSTRUCTION AND REPAIRS FOR BORED PILES

The tests described in the preceding chapter show that bored piles can frequently be subject to defects. It is likely that the overdesign of the piles in most cases prevents the defects from being noticed. It is also true that in some cases defects in a pile cause serious problems for the structure. The problem is even more serious when the foundation is heavily loaded and supported by a small number of piles.

8.1. CAUSES

Defects are numerous and basically attributable to:

-- a partial or total ignorance of the nature of the soils and hydrology of the site;

-- an error or improper control of the job on the part of the construction supervisor or the company;

-- the existence of a very rigid contract or a tight schedule that imposes construction rates that are incompatible with careful construction;

-- the incompetence or lack of care by the company concerning the construction of structures that appear more complex then they are; and

-- the fact that construction of a pile involves certain operations that are simple but demanding so that the practitioner, despite good intentions, does not always have good control.

In the following paragraphs the only causes of defects to be examined will be those linked to the principal phases of pile construction: boring and concreting.

8.1.1. Boring phase (see Chapter 3)

Defects that may be attributed to this phase are the result of:

-- the technique, boring equipment, or type of pile that is poorly adapted to the soils involved;

-- accidental loss of mud (karstic soils, gypsum soils,...) or sudden rising of the fluid surface in the borehole, resulting from cave-ins. These two incidents cause the borehole to lose its intended dimensions;

-- poor control of the boring as a result of a slurry composition that is poorly selected or poorly controlled;

-- accidental or systematic deviations of the boring from its intended position due to occasional blocks or boulders or sloping bedding planes.

These deviations lead to poor alignment of the borehole with the vertical or with the inclination called for in the plans; and

-- insufficient cleaning of the borehole to allow a layer of sediments to remain at the bottom of the borehole that is fairly thick, causing poor contact at the base of the pile and contamination of the concrete.

8.1.2. Concreting phase (see Chapter 6)

Certain defects may be related to the following causes:

-- concreting equipment that is poorly selected or in poor condition;

-- poorly conducted concreting procedure, error in initiating the concreting operation, discontinuity in concreting as a result of lifting the tremie too rapidly;

-- irregular supplying may in certain cases cause a premature setting of concrete (between the temporary casing and the liner for example); and

-- placement of concrete having an improper mix, unsuitable workability, or with a tendency to segregate.

It must be added that there are other causes that are also related to the boring and concreting that may cause defects or a reduction in the capacity of the pile. Included are:

-- significant water circulation that locally leaches the fresh concrete;

-- remolding of the soil, causing a decrease in skin friction or in end bearing;

-- an overly long delay between boring and concreting that causes caving or sedimentation at the bottom of the borehole. *These setbacks occur frequently on jobs* where piles are being constructed too rapidly and without careful control; and

-- using percussion drilling too close to a pile in which the concrete has not had time to set.

8.2. NATURE AND SEVERITY OF DEFECTS

The causes mentioned in Section 8.1 may be at the root of defects affecting the tip, shaft, or upper portion of the pile (Fig. 172).

8.2.1. Defects at the tip

These defects are probably the most frequent. They are obviously more serious for end-bearing piles (especially in the case of lined or cased piles). A defect at the base may result in a noticeable decrease in the total resistance of the pile, its end-bearing capacity, and can cause significant settlement. The lack of proper performance may well be due to the tip of the pile as well as the underlying soil.

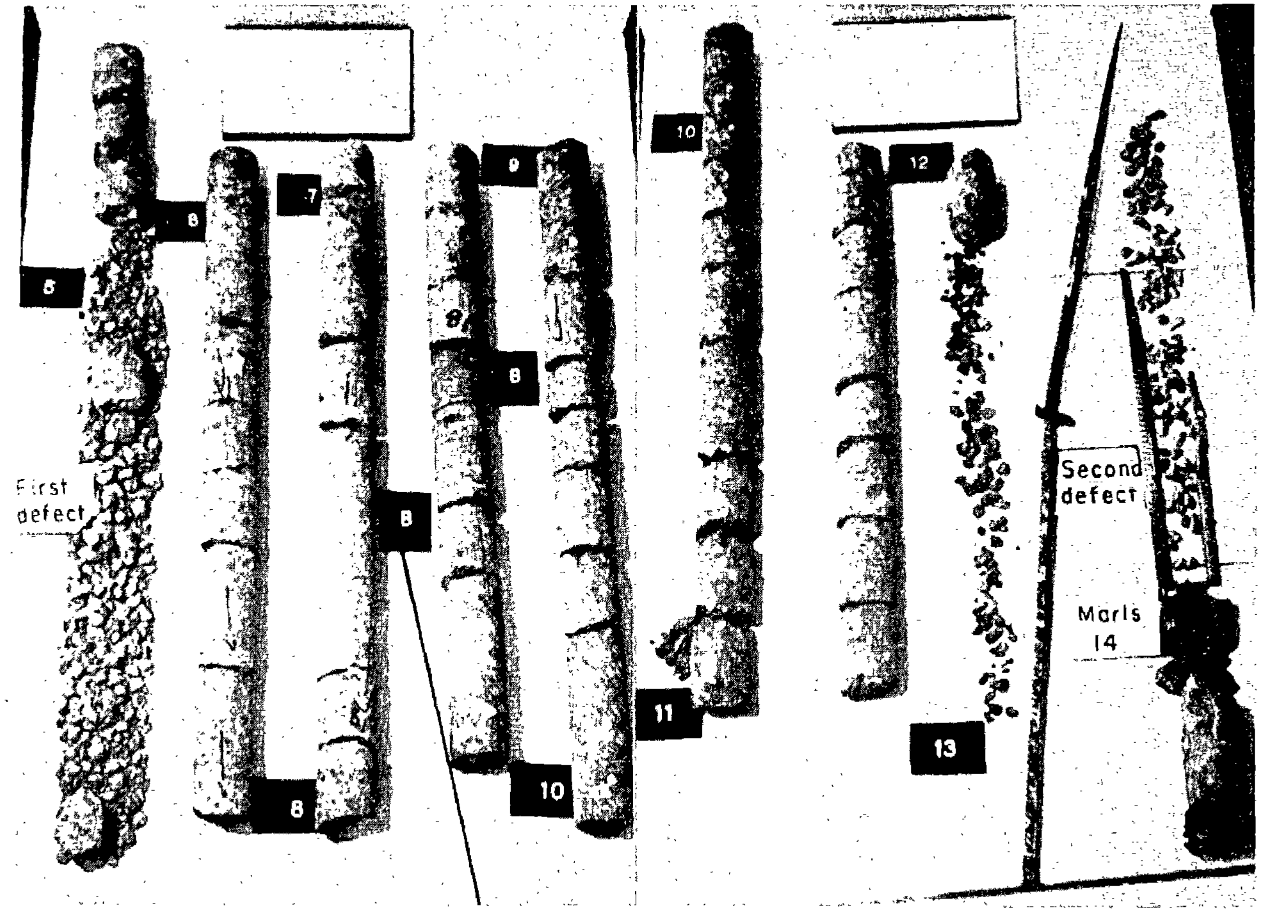


Fig. 172. -- Test coring taken from pile shaft. Note between 5 and 6 m presence of segregated concrete and leached concrete at bottom of pile (12 to 14 m). (1 m = 3.28 ft)

-- *In the first case* (Figs. 173 and 174), the base of the pile is constructed using concrete of poor quality (segregated, or contaminated by inclusions of mud). As a rule, such a defect will not cause a failure of the pile because it is rare that the bearing stress on this segregated or contaminated concrete is large enough to cause a problem when the service load is applied to the pile. The risk incurred is even less serious when the pile is socketed into a strong formation (especially in rock), in which even the concrete aggregate with no binder is securely contained. However, taking into account the subjective character of these considerations, the defective zone should always be reinforced by injection. Serious repair and testing of the performance of the pile are mandatory when the defect may lead to an instability of the foundation due to horizontal loads or to a loss of axial capacity (lightly socketed piles in rock at an aquatic site, for example).

-- *In the second case* (Figs. 175 and 176), the defect may involve an improper contact at the tip due to inefficient cleaning of the bottom of the borehole (resulting in a mixture of mud and sediments between the concrete and the soil). Or, the defect may involve a weakening of the in situ soil that is attributable to the use of boring techniques that are poorly adapted to the nature of the soils (see § 3.4). There may be a settlement that can prove to be unacceptable for the structure, or indeed, a decrease in carrying capacity that may completely invalidate the design computations. For this type of defect, the use of injection or grouting is a generally satisfactory remedy.

8.2.2. Defects in the shaft

As a rule, discontinuities are involved in defects in the shaft:

-- bulging due to flow of a soft layer under the pressure of the fresh concrete, or due to irregularities in the borehole (caving, cavities, etc.);

-- contraction of the diameter of the borehole caused by horizontal forces from the soil;

-- fairly significant inclusions of mud (possibly to the point of the complete replacement of concrete at some point: Figs. 177 and 178) as a consequence of errors in concrete placement; a local trapping of sediments, etc.;

-- leaching attributable to horizontal water flow or to interruptions in concreting; and

Fig. 173. -- Core of bottom of pile showing area of leached concrete at soil contact.

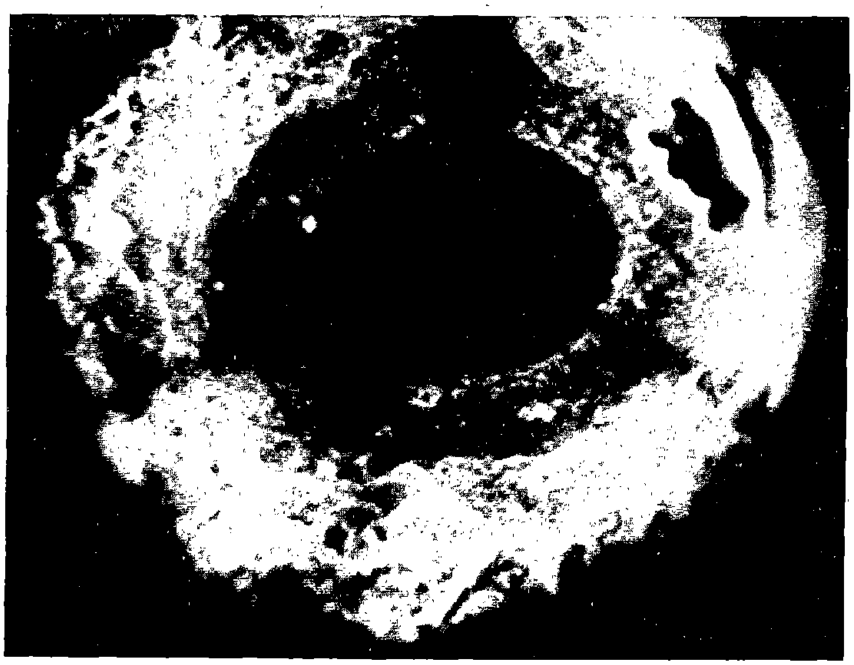


Fig. 174. -- View by television camera of same defect. Note clear lack of cohesion of aggregate.

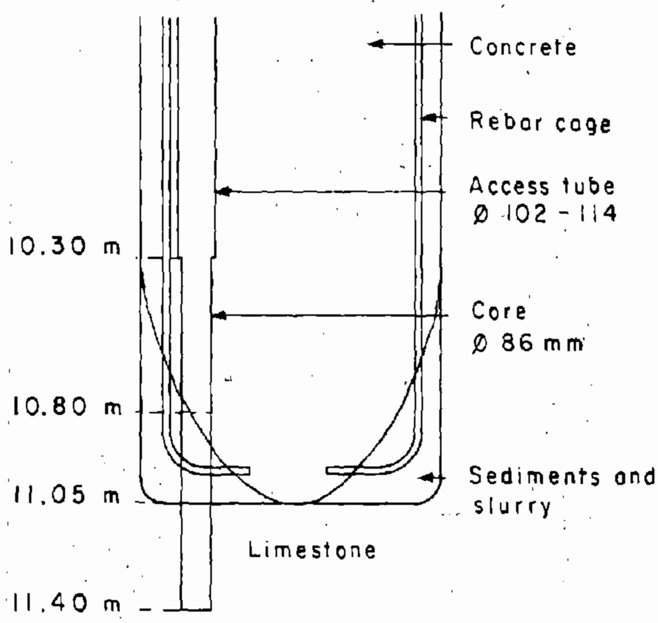


Fig. 175. -- Cross section of pile base. Bevelled-shape form is due to interposition between concrete and soil of mixture of mud and sediments. (25.40 mm = 1 in.; 1.0 m = 3.28 ft)

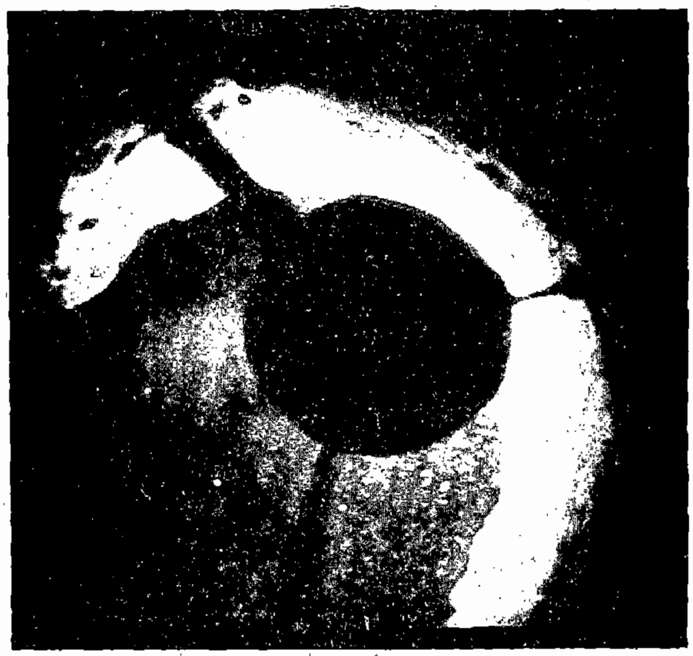


Fig. 176. -- Axial view in corehole of a cavity at depth of 10.80 m; concrete on 2/3 of circumference toward inside of pile, empty toward outside. (1 m = 3.28 ft)

Fig. 177. -- Cavities left after cleaning of mud pockets in barrette. (From Institute of Research on Bridges and Roads at Varsovie)

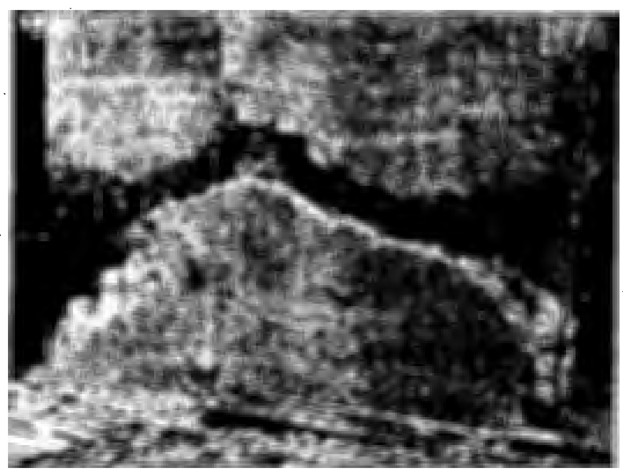


Fig. 178. -- Discontinuity in section of barrette. (From Institute of Research on Bridges and Roads at Varsovie)

-- errors in alignment inherent to deviations in boring (Fig. 179).

While protrusions or bulges and certain limited inclusions of mud may not compromise performance, contraction, errors in alignment, significant pockets of mud, and complete breaks (Fig. 180) are serious problems when the number of piles is small and the foundation is heavily loaded.

Errors due to the use of insufficiently workable concrete may also be of concern. The placement of concrete with only a small slump is incompatible with a proper covering of the rebar cage. Furthermore, a defect is difficult to detect by geophysical investigation (or by coring) because it involves the periphery of the pile while the core remains normal. It must be noted that difficulty with low-slump concrete is even greater when the diameter of the pile is small, because the access tubes are fewer in number (geophysical investigation limited to a smaller area).

Figure 181 illustrates the consequences of the use of a concrete with a slump that does not exceed 11 cm (4.3 in.) for a pile of 80 cm (31.5 in.) in diameter. The pile was judged to be satisfactory on the basis of investigation by wave transmission using three access tubes. All that was concreted was the interior space of the reinforcing cage.

Repairs to the sides of a bored pile (the shaft) are generally more difficult than repairs to the base and, from an economic point of view, shaft repairs depend on the dimensions of the pile. Thus, it may at times be advantageous, in the case of a pile of large diameter, to grout or to make use of special solutions (see § 8.5, examples 5 and 6). In the case of a pile of small diameter ($\leq$ 80 cm - 31.5 in.), it may be preferable to replace the pile with one or more piles that are placed nearby.

8.2.3. Defects in the upper portion of the pile

The inadequacy or lack of care in being sure that mud and contaminated concrete are ejected at the top of the pile by the rising column of good concrete is the most frequent defect and results in inclusion of mud or sediments (Fig. 182). The relatively simple and well known remedy for this defect consists of eliminating the defective portion of the shaft by trimming; concrete of good quality is then cast at the upper portion of the pile. As shown in Fig. 183, poor covering of the rebar cage also leads to defects which may compromise the durability of the foundation. As in the preceding solution, repair is made by trimming away the defective concrete and reconstruction is performed with the use of concrete

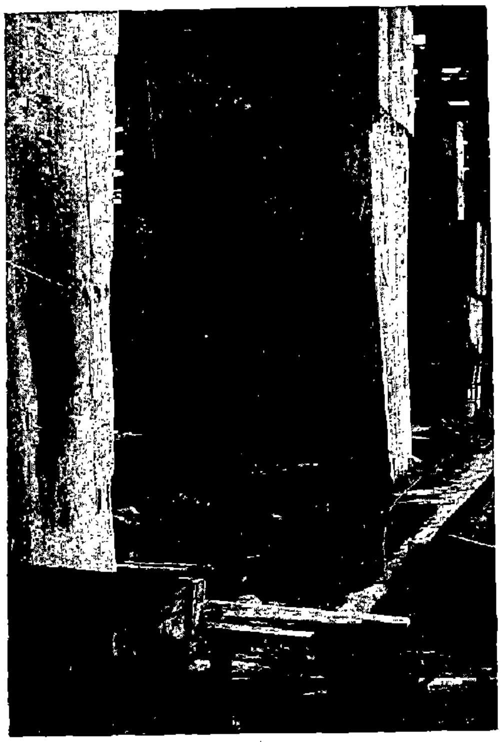


Fig. 179. -- Alignment defect of this barrette does not compromise foundation stability.

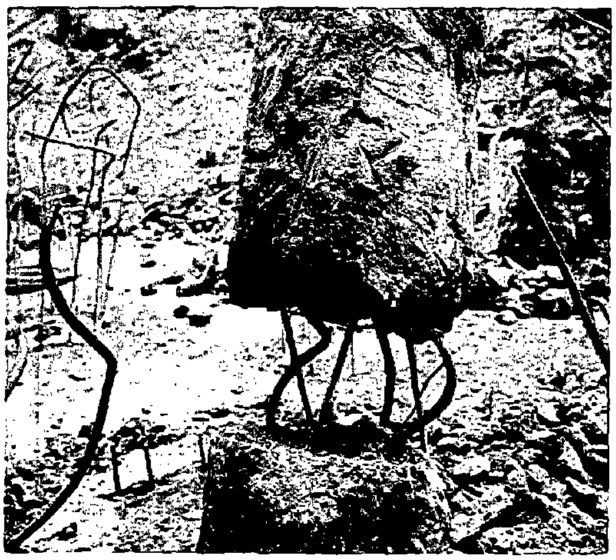


Fig. 180. -- Complete break of section of pile shaft.

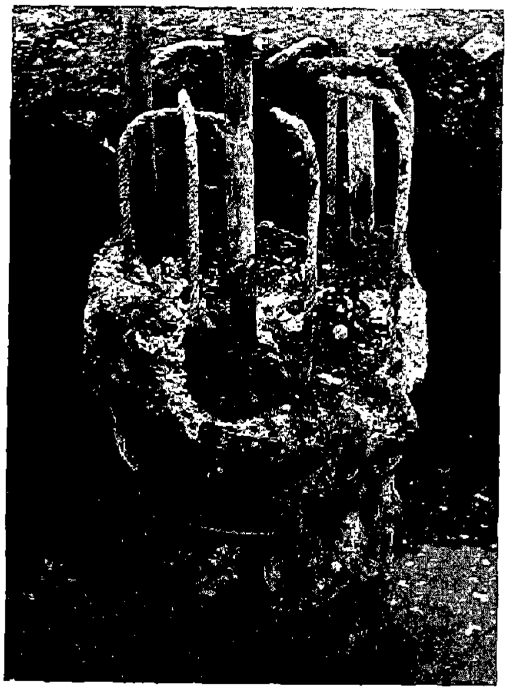


Fig. 181. -- Poor coating of reinforcing due to placement of insufficiently workable concrete.

Fig. 182. -- Inclusion of mud at pile head.

poured in the dry. If the defect is extensive, it may also prove more economical to construct a new pile and to modify the reinforcing of the pile cap if necessary.

Fig. 183. -- Poor centering of reinforcing cage at head.

It is of interest to note that in the case of piles with a liner at the top, an annular gap is inevitable between the shaft and the wall of the borehole (see § 4.3.6). If negative skin friction (downdrag) is not a consideration and if there is a desire to mobilize the lateral resistance of the soil, this annular space should be filled by gravity or, better, by grouting. Obviously, such an operation does not constitute a repair of a defect but rather a modification that should be anticipated at the planning stage.

8.3. APPROPRIATENESS OF REPAIR

Independent of the nature of the defect, repair always constitutes a special case.

Once a defect is detected by geophysical investigation or coring (see Chapter 7), an estimation of its extent must be made and then an evaluation is made of the risks incurred by the structure.

As a rule, there are not grounds for becoming concerned about anomalies such as protrusions or bulges in the shaft and minor alignment errors (Fig. 179) inasmuch as they do not compromise the stability nor the durability of the foundation.

Similarly, local reduction in the resistance of the concrete or of the diameter of the shaft may be allowed when the location and the amount of the reduction remains compatible with the forces exerted at the levels involved (see § 8.5, example 2). Estimation of the severity of such errors is made through the progressive use of various methods of geophysical investigation and through possible recourse to coring, compression testing, and television viewing if necessary.

On the other hand, it was seen in § 8.2.1 that there are grounds for imperative repair of any defect that affects the tip (leached concrete, mud or sediment pocket, loosening or weakening of the soil). Also, repair is essential if there are serious defects involving the shaft such as concrete breaks, reduction of the diameter, pockets of concrete with voids, porous concrete without flexural strength, and inclusions of boring mud. Such defects are especially serious in the upper portion of the shaft where the stresses are maximum.

It should be noted that for those cases where there are defects in the shaft of the pile, repair or complete replacement of the pile is mandatory when the pile is subject to large axial loads or must sustain lateral loads.

In conclusion, depending on the degree of the severity of the defects, three possibilities exist:

-- ***not to repair*** because the defect is judged to be of no consequence and does not compromise the carrying capacity of the foundation;

-- ***to make repairs***, in consideration of the efficiency, economy, and speed of the treatment;

-- *to replace the defective pile*, when repairs are neither technically nor economically feasible or do not offer a sufficient guarantee concerning durability.

8.4. GROUTING AS A MEANS OF REPAIR

In practice, grouting offers the possibility for repairing numerous defects.

It is of interest to note that grouting permits:

-- repair of defective concrete for which the principal characteristic is lack of binder;

-- improving the nature of soil by in-filling or compaction of soil that has been loosened or weakened;

-- sealing of fissures or voids in the soil.

In all cases, a definition of the grouting parameters is essential: nature of grout used, pressure, and quantities to be injected.

8.4.1. Grout

Generally, a distinction is made between:

-- unstable grouts (cement + water with possible addition of fine sand) with low cement content (W/C from 10 to 1);

-- stable, two-phase grouts (cement + water) with high cement content (W/C around 0.5) or a compound mixture (cement + water + bentonite) with, if the need arises, the addition of fine sand; and

-- chemical grouts (synthetic resins, silicate-based gels, hydrocarbonated products) that are rarely used due to their high cost.

In relation to the problem to be solved, a selection is made that is the most suitable noting that there may sometimes be reason to use two grouts successively.

Thus, when the repair involves the filling-in of gaps or strengthening of very leached and, therefore, very permeable concrete (permeability should be verified by a preliminary water test from boreholes or access tubes), use of a stable, two-phase grout of high cement content (100 kg - 220.5 lbs of cement to 40 to 50 liters - 10.6 to 13.2 gallons of water, W/C around 0.5) is recommended.

On the other hand, when the repair involves the compaction of soils at the base of the pile that are loosened and disturbed by the boring, there is a possibility for use of stable, compound grouts (cement + water + bentonite), that are more fluid and have proven to have a better range of action than two-phase grouts.

Of course, it is necessary to ensure the compatibility of the cement used with the aggressiveness of the subsurface conditions (soil + water). Finally, it is noted that, in general, additives such as bentonite, fine sands, or chemicals are also used in small amounts in the grouts. The grouts should always be prepared in mixers at a high turbulence ($\omega \geq 1{,}500$ rpm, Fig. 184).

Fig. 184. -- Mixer with high turbulence.

8.4.2. Pressure of injection and quantity of grout

As is the nature of grouts, the pressure to be used and the quantity to be placed depend on the type of repair to be performed. Therefore, it is necessary to distinguish between injections for filling-in of gaps or regeneration of leached concrete, and injections for compaction of the soil at the base of the pile.

In the first instance, the quantity of grout should correspond to the volume of the gaps to be filled-in and may as a rule be preliminarily estimated from the results of geophysical investigations. When used to fill-in gaps, grouting pressures are intentionally limited to 2 or 3 bars (29 or 43.5 psi) at the end of the operation. On the other hand, the grout must attain higher pressures (up to 20 or 30 bars - 290 or 435 psi if the case arises) in order to fill properly the areas of leached concrete.

In the second instance (see § 8.5, example 4), the injection pressure should adhere to two contradictory requirements:

-- sufficiently high to penetrate and *sqeeze or compact as well as possible*, and

-- remain low enough to prevent hydraulic fracturing which could cause an uplift of the pile or surface movement of the soil.

The quantities of grout to be injected may vary from several liters to several cubic meters. Stable grouts (cement + water + bentonite) that are often used for repair of defects in piles, behave in effect *like fluids that can be pumped indefinitely if the increased viscosity due to the setting of the grout did not cause an abrupt increase in the injection pressure* [8]. A preliminary study should be made to decide on the quantities of grout to be injected and to determine the injection pressure in relation to the permeability of the soil. The flow of grout to the exterior of the pile being repaired may be proof that the area being treated will not absorb more grout. It is then necessary to halt the injection immediately and to restart if necessary after the grout has set.

In general, it is difficult to set precise rules for injection pressure and grout quantities. Therefore, both must be controlled throughout guidelines from observations made during the process and there must be provisions in the arrangement of the injection equipment for recording these two parameters.

8.4.3. Performance of grouting

The procedure to be employed obviously depends on the problem presented. However, nearly all of these treatments include:

-- placement of a minimum of two conduits allowing formation of a circuit between the area to be treated and the surface (Fig. 185);

-- washing of the defective zone;

-- injection of the grout; and

-- test for efficiency of the treatment.

The *conduits* may be affixed to the inside of core holes previously constructed within the foundation or directly into the soil around the outside of the foundation. The first arrangement is used when injection is made in pockets, in breaks in concreting, or at the tip. Use is made of the lateral conduits for injection of the tip as well as to improve the soil/shaft friction. Depending on the type of treatment to be performed, the conduit may be equipped with packers.

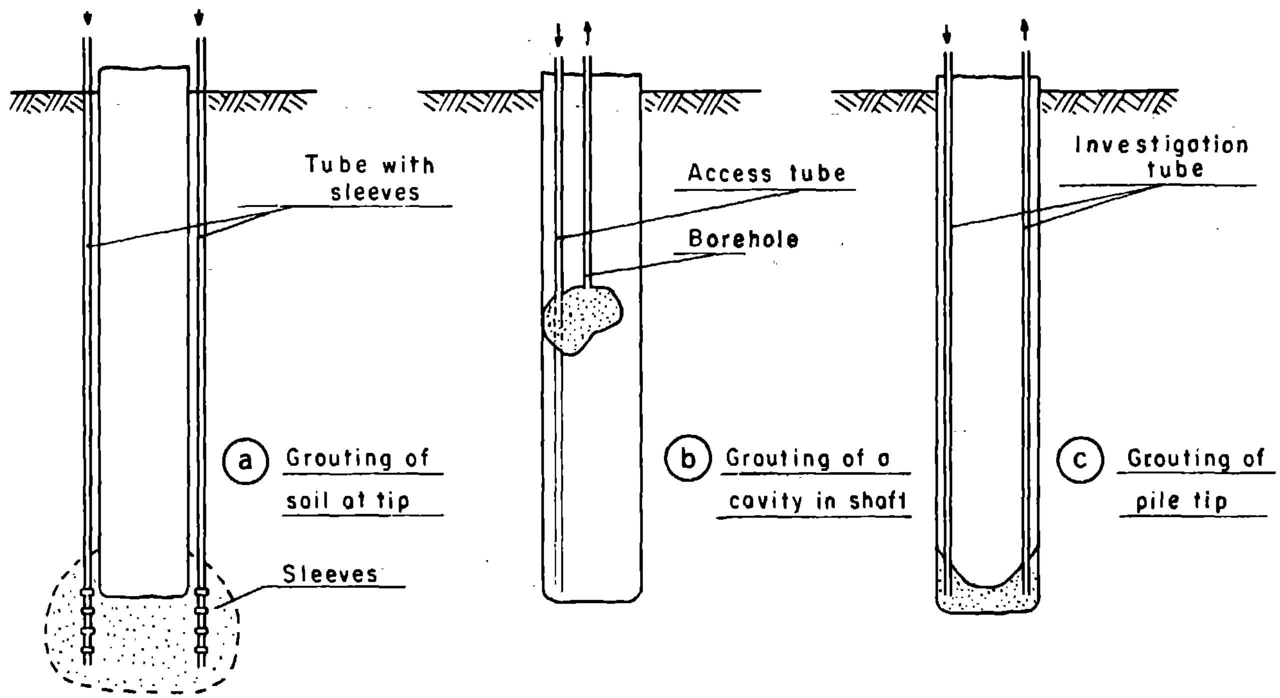


Fig. 185. -- Three types of grouting arrangements.

Construction of boreholes may be superfluous if the pile or the barrette has been previously outfitted for geophysical investigation with access tubes that extend to the area involved:

-- For a treatment at the tip, the plugs obstructing these tubes are broken by a wagon-drill.

-- For a local repair of the shaft, the tubes themselves are destroyed by an oxygen torch over the entire length of the area to be treated. For this, a special spear may be lowered to the level of the portion to be cut. This spear is provided with an electric, self-igniting device (Soletanche patent). The lance is composed of a steel pipe that is packed with rods of fusible metal and a jet of oxygen under pressure is sent through the steel pipe. The very high temperature given off during combustion melts the surrounding steel and a molten liquid flows from the end of the lance and destroys the plastic pipe.

The washing of the pockets of defective concrete and gaps in concrete of the shaft is done in the same manner as is used for elimination of inclusions of mud and sediment at the bottom of the pile. The technique is to use water that is emulsified with compressed air (see TN No. 12). However, this technique may not be suitable for cleaning the base of the pile except for piles socketed in cohesive soil or in rock. In the case of cohesionless soil, the emulsifier may dangerously undermine the socketing area.

Without an emulsifier, the washing operation may be performed with an open system under low pressure (3 to 5 bars - 43.5 to 72.5 psi). However, the efficiency of the cleaning is then difficult to control and does not depend on the amount of water injected which should be limited.

Injection of the grout is achieved with a pump, most often with a double piston, called a "squeezer" (Fig. 186).

The pump is driven in various ways depending on the kind of pump and in relation to the nature of the defect:

-- In the case of the improvement of the soil at the base of the pile, tubes with packers may be installed in the peripheral boreholes (Fig. 185a). The grouting can be performed in stages with a packer placed above and below each level where grouting is being done. At the end of each stage, the grout tubes must be carefully cleaned so as to eliminate the grout that has descended due to gravity when the packers are deflated.

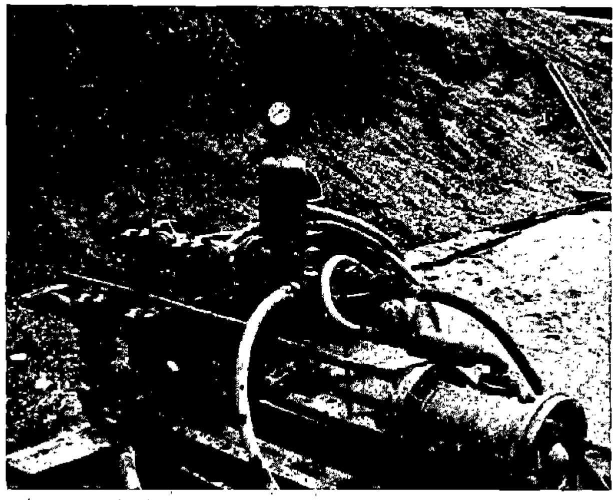


Fig. 186. -- Grouting pump.

-- When grouting of the pile itself (shaft or tip) is involved, injection is made through the use of boreholes constructed for this purpose (Fig. 185b) or by access tubes (Fig. 185c). One of the grout lines is connected directly to the pump and the other serves as a return. Pressure gauges can be installed in the system and the injection pressure can be controlled by restricting the return line. Before the grout starts setting, it is desirable to clean the various core holes or access tubes so that they may be used again. Additional injections can be made after fracturing the previously placed grout. In this respect, a small diameter hose is used and is lowered in each opening up to about 2 m (6.6 ft) from the injected area; maintaining the 2 m (6.6 ft) distance prevents leaching. This "hose" operation prevents breaking of the grout over the entire length of the grout tubes. In order to limit the residual amount of grout in the access tubes or the boreholes, small diameter tubes may be used that are equipped with a packer at the end (Fig. 187). Such a device also facilitates cleaning of the grout openings. After releasing the packers, the small tubes may be withdrawn and cleaned with clear water.

The indispensible *test* of the efficiency of the grouting is performed by the usual geophysical-investigation methods (see Chapter 7). The tests can be made after opening the access tubes by a wagon-drill because the

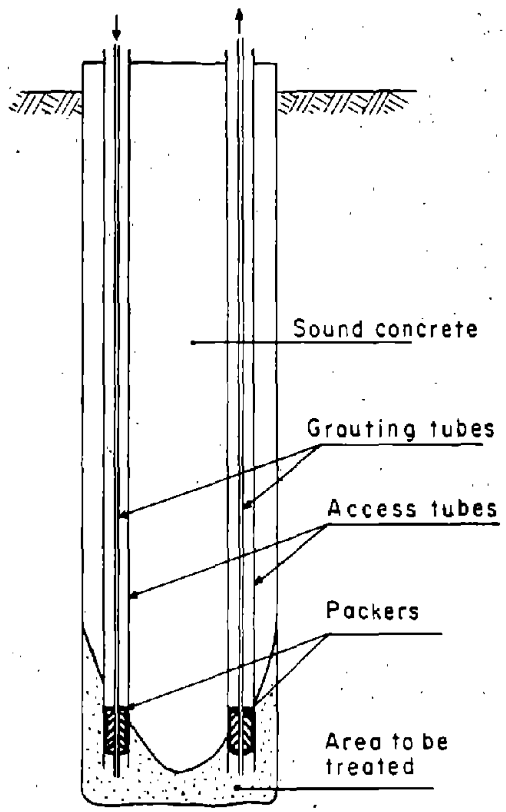


Fig. 187. -- Grouting of tip with use of packers at end.

access tubes were more or less blocked if they had been used for injection of the grout. The opening of the access tubes would be much easier if they had been partially cleaned following the grouting. In case the access tubes had to be cleared of grout, the use of the gamma-ray method is preferable. The accuracy is better than with the wave-transmission method and a larger volume of material can be invstigated. Another means of testing the quality of the grouting is to put a core boring through the treated zones; the possible viewing of the sides of the boring with a television camera could also be done.

8.5. EXAMPLES OF REPAIRS

The practical examples presented here illustrate the solutions that have been employed for repair of certain types of errors affecting piles or barrettes.

EXAMPLE NO. 1

A barrette of 28 m (91.8 ft) in length (cross-section 2.20 x 1 m - 7.2 x 3.3 ft) was placed into a hard, fissured limestone. At the time of concreting the tremie was too short and caused leaching of the concrete at the base over a 1.50 m (4.9 ft) distance. This defect was detected by geophysical methods performed in four access tubes (Ø 50/60 mm - 2.0/2.4 in.) cast into the barrette. Two tubes, diametrically opposed, were destroyed by an oxygen torch over the entire length of the leached area; these tubes were used for the cleaning of the defective zone. The pressure in the cleaning system was 4 to 5 bars (58 to 72.5 psi) and washing was continued until clear water was obtained. Then, cement grout was injected, at first very diluted (W/C = 3), then very rich in cement (W/C = 0.5). The injection was performed under low pressure, 3 to 4 bars (43.5 to 58 psi), to avoid overconsumption of grout. A new test using geophysical methods revealed the efficiency of the treatment.

EXAMPLE NO. 2

On another barrette of the same job, geophysical investigation allowed detection of a defect at about 6 m (19.7 ft) of depth. The defect seemed to be located near one of the access tubes. Coring confirmed the existence of a pocket of concrete that was contaminated by bentonite. Despite the divided opinions about the severity of the defect, it was decided to make repairs. Washing was performed and then grouting of the defective concrete was performed. The low quantity of grout that was ultimately injected (1 to 2 liters - 0.26 to 0.53 gallons) confirmed the insignificance of the defect.

This example, which is not unique, shows the extent of difficulty, in some cases, in making a decision about the severity of a defect.

EXAMPLE NO. 3

On piles 1.50 m (4.9 ft) in diameter that were socketed in a hard, marly limestone and equipped with two access tubes, geophysical investigation did not allow detection of any error along the shaft. On the other hand, corings revealed the existence, at the base of the piles, of gaps 10 to 60 cm (3.9 to 23.6 in.) in height. After washing with water through the two tubes, the repair consisted of injecting, under a pressure of 5 to 6 bars (72.5 to 87 psi), a cement grout of W/C equal to 0.5. The amounts injected were variable, at times reaching 800 liters (211 gallons) per pile. It must be pointed out that settling of several millimeters was reported during the subsequent construction of the superstructure and that was under loads for which the piles had not been designed.

EXAMPLE NO. 4

This example is taken from an important study that is based on load tests for bored piles of 90 cm (35.4 in.) in diameter, socketed to 11 m (36.0 ft) in a clayey, compact silt, that was, however, susceptible to disturbance from boring. Tests in such soils [28] have shown that construction of piles with an auger or by the Benoto Method lead to a significant disturbance of these clayey silts. The disturbed soil extends to over 60 cm (23.6 in.) below and laterally from the base of the pile. Considering the behavior of the base of the pile, the limited capacity found at the time of the first testing was practically identical (130 to 135 t - 143.3 to 148.8 tons) but very far from loads estimated from soil tests (200 to 250 t - 220.5 to 275.6 tons). After a delay of nine months, deemed necessary for recovery of the soil, grouting was performed at the tip of the pile constructed with the auger. The grouting occurred 15 days before the second load test and the program was designed so as to estimate the efficiency of the injection. This injection was limited to 1 m^3 (35.3 ft^3) of stable, compound grout (400 kg of cement of CPA 325, 200 liters of water, 10 kg of bentonite - 882 lbs, 53 gallons, and 22 lbs) under a pressure of around 10 bars (145 psi). The new load tests performed on the two piles indicated a limited load at the tip of 195 t (215.0 tons) for the augered and injected pile and 150 t (165.4 tons) for the pile that was not injected. If the gain in capacity of 15 t (16.5 tons) is excluded as being due to the recovered strength of the soil during the delay of nine months, the increase in the resistance at the tip of 50 t (55.1 tons) as a result of injection permits attainment of approximately the values of load that were expected.

Finally, it must be emphasized that injection, which reduced the settlement very appreciably, permitted a significant increase in the service loads for the "augered" pile (Fig. 188).

EXAMPLE NO. 5

The originality of the solution and methods used in this example of repair of an defect in a shaft show that each case is special.

This involved a foundation on cased piles of 1.50 m (4.9 ft) in diameter, bored at an aquatic site in a complex marl-limestone.

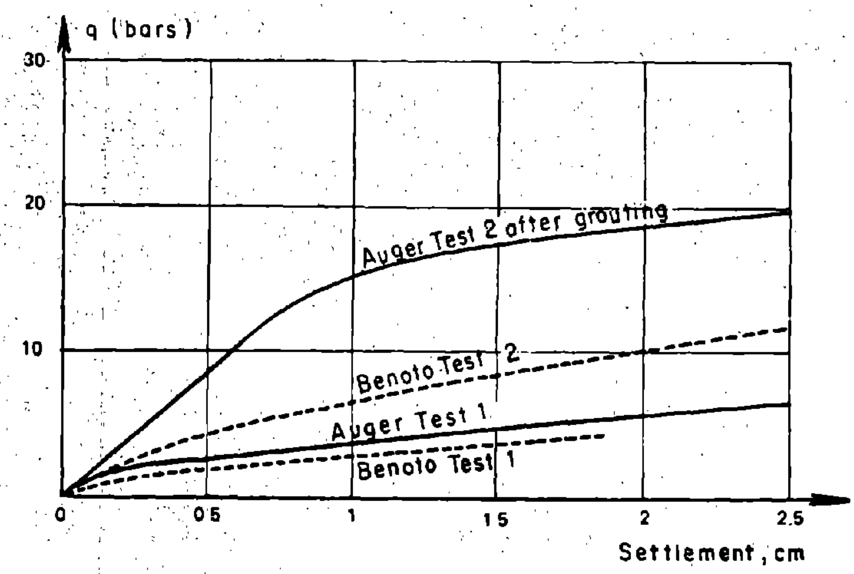


Fig. 188. -- Mobilization of load at tip. Load-settlement curves show increase due to grouting at tip. (2.54 cm = 1 in.)

Some difficulties arose during the course of concreting when pumping caused rupture of the pump line. A significant leaching resulted along with a severe segregation of the concrete evidenced by geophysical investigation and confirmed by corings (Fig. 189) and viewing with a television camera (Fig. 190).

With the envisioned possibility of a complete removal of concrete to 11 m (36.0 ft) of depth on the inside of the casing (15 mm - 0.59 in. in thickness), an underwater, local repair was preferred. Divers made an opening in the casing, permitting cleaning of the segregated and leached material that prevailed over 4 m (13.1 ft) in height. This opening was then plugged with a metal plate and concreting of the defect in the pile was made by using one of the core holes to inject the grout and the other as a return passage. The special concrete had the following mix:

Aggregate:

6/10 mm (0.24/0.39 in.): 1,090 kg (2403.4 lbs)

2.5/6 mm (0.10/0.24 in.): 205 kg (452.0 lbs)

0/2.5 mm (0/0.10 in.): 610 kg (1345.0 lbs)

Fig. 189. -- Core showing sand and gravel resulting from segregation and from leaching of concrete over height of 4 m. (1 m = 3.28 ft)

Fig. 190. -- Photograph of television view at depth of 9.50 m: Cavity with diameter significantly larger than borehole diameter. (1 m = 3.28 ft)

Cement:

CPAL: 400 kg (882 lbs)

Water: 210 liters (55.5 gallons)

Additive:

TTB 5477: 0.2% (set- and hardening-retarder).

Other:

slump: 16 cm (6.3 in.),

7-day strength = 221 bars (3204 psi)

28-day strength = 373 bars (5408 psi).

The subsequent geophysical investigations attest to the efficacy of the repair. However, it must be emphasized that this particular site presented an exceptional problem.

EXAMPLE NO. 6

This example relates an incident that arose immediately after concreting when removing casing 1 m (3.3 ft) in diameter by vibration. There was a breakdown of the electric-generating equipment when only 4 or 5 m (13.1 or 16.4 ft) of the temporary casing remained to be extracted.

The repair of the electric-generating equipment required more than two hours during which setting of the concrete was begun. Then, at the time of the resumption of the extraction of the casing there was a superficial but fairly extensive desegregation of the concrete in the final meters of the shaft.

After the concrete had set, a core boring in the central portion of the pile had revealed a satisfactory quality of the concrete located on the inside of the rebar cage. It was decided to reinforce only the outside of the shaft.

In this repsect, the reinforcing was uncovered by a jack hammer for about 0.70 m (2.3 ft) of length and the head of the pile was then capped with a metal casing 1.50 m (4.9 ft) in diameter and 6 m (19.7 ft) long that was placed around the shaft by using a vibratory hammer (Fig. 191).

To the inside of the casing were welded 22 Adx rebars Ø 25. The bars were uniformly distributed and were designed to improve the connection between the pile and the pile cap. After being cleaned by hosing, the annular space between the pile and casing was concreted in the dry.

EXAMPLE NO. 7

In order to illustrate the proper type of device to offset the consequences of poor construction on the behavior of the upper portion of a shaft, an example is cited of lined, inclined piles (Ø 1 m - 3.28 ft). The boring methods were poorly selected and there was a considerable over-break near the top of the pile. The location of the top of the pile was shifted as was the angle of inclination.

In this case, a classic type of repair was used for these defects by grouting the annular space between the soil and the pile. This was done by gravity injection of a cement- and sand-based grout (50 kg - 110.2 lbs of cement CPF, 15 kg - 33.1 lbs of sand 0/1, 30 liters - 7.9 gallons of water, 0.75 kg - 1.6 lbs of bentonite). The average amount injected per pile was around 2.4 m^3 (84.7 ft^3).

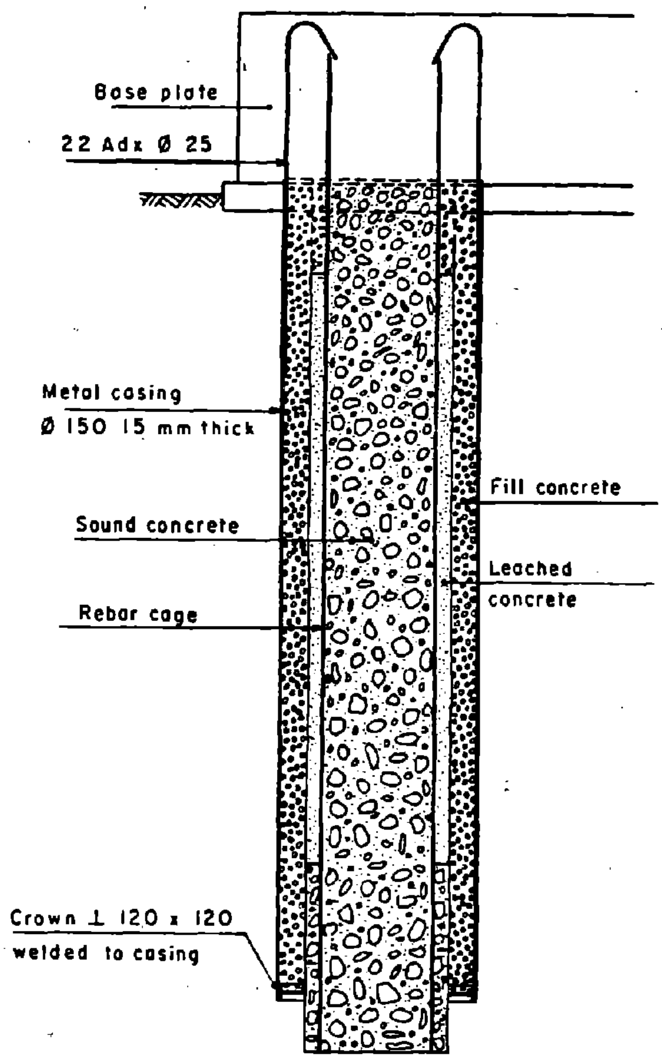


Fig. 191. -- Principle of repair at pile head after problems during extraction of temporary casing. (25.40 mm = 1 in.)

TECHNICAL NOTES

SUMMARY

TN No. 1. -- Machines for Percussion Drilling
TN No. 2. -- Drilling Tools for Percussion Drilling
TN No. 3. -- Temporary Casings
TN No. 4. -- Boring by Benoto Method
TN No. 5. -- Use of Hammers for Placing Casing
TN No. 6. -- Vibratory-Drivers for Casings
TN No. 7. -- Auger-Type Drilling Machines
TN No. 8. -- Tools for Rotary Drilling
TN No. 9. -- Drilling Slurry Station
TN No. 10. -- Drilling Tools for Barrette Piles
TN No. 11. -- Drilling Machings with Reverse Circulation
TN No. 12. -- Drilling by Use of Airlift

MACHINES FOR PERCUSSION DRILLING

1. WOOD OR METAL TRIPODS WITH WINCH

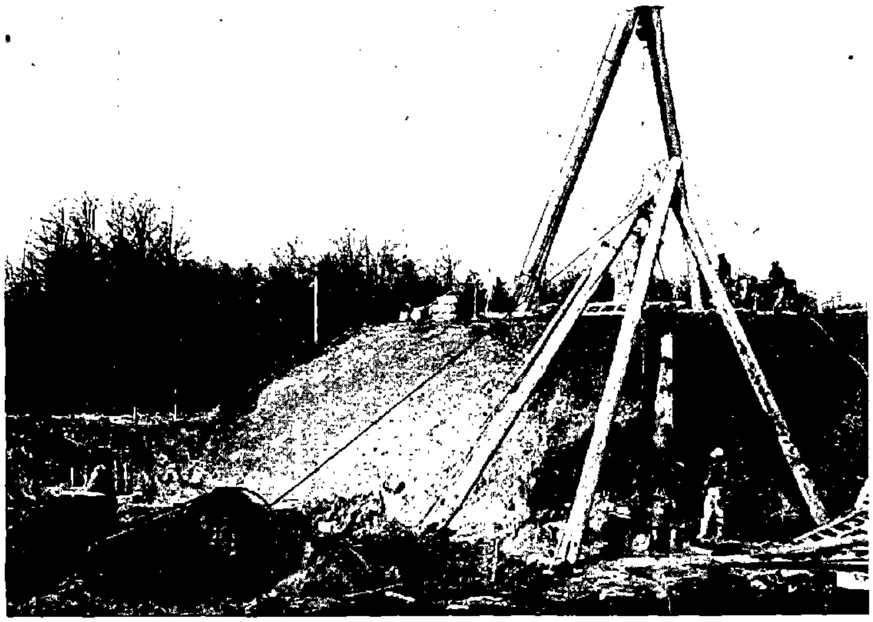


Fig. 1.1. -- Example of wood and metal tripods with driving winch.

The boring outfit simply includes: 1) a tripod or hoist positioned over the borehole, 2) a winch used for the drilling tools as well as for placement and extraction of the temporary casing.

TABLE I

EXAMPLE CHARACTERISTICS OF DRIVING WINCHES WITH A SINGLE DRUM

Model	Weight without motor (kg)	Required Power (HP)	Maximum weight of tool (kg)	Winding capacity (m)	Diameter of cable (mm)	Maximum diameter of pile (*) (cm)
TPA 100	460	18/25	1,000	90	12	50
TPA 251-B	1,000	25/40	2,500	200	16	60
TPA 350	1,600	60/80	3,500	200	18	120

(*) Given by the constructor.
(0.453 kg = 1 lb; 0.0254 m = 1 in.; 25.40 mm = 1 in.; 2.540 cm = 1 in.)

Fig. 1.2. -- Migal machine MB 6 C.

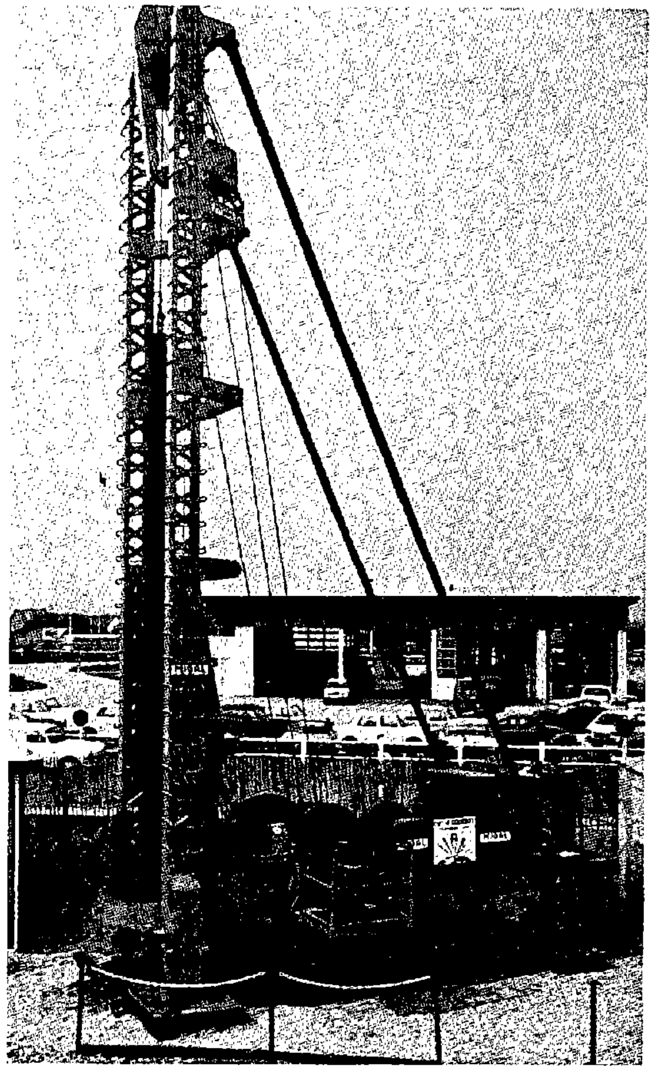


Fig. 1.3. -- Migal machine MB 10 C.

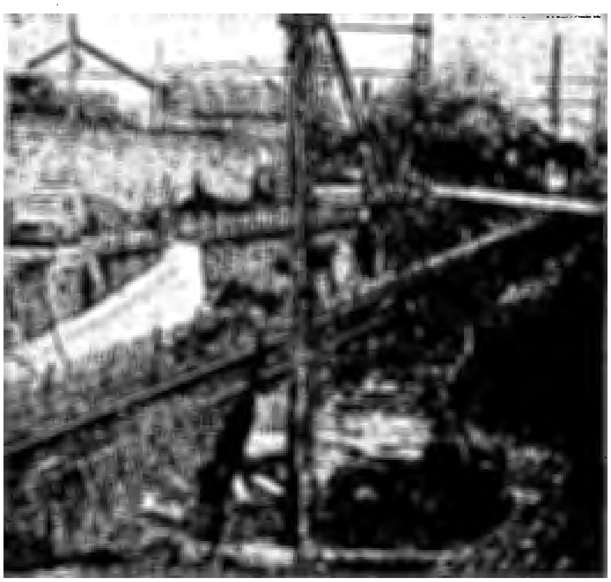


Fig. 1.4. -- Another example of winch and hoist outfit.

2. INCLINABLE BORING MACHINES WITH DERRICKS

The machines that are available on the market are varied and some contractors manufacture the equipment themselves. As an example, described below is the range of Migal drills.

The machines are composed of hoists with two folding derricks, a winch-carrying frame and a motor that rests either on rollers, tires or on tracks that permit increased mobility in poor terrain.

The winches on these drills are double or triple drum with a force of 2.5 to 10 tonnes (2.8 to 11 tons) according to the purpose of the derrick. The use of pullies for extraction of casings may reach 80 tonnes (88 tons) of force allowing construction of bored piles in excess of 1.20 m (4.0 ft) in diameter.

Table II summarizes the main characteristics of the machines.

It is also noted that the MB 10 C includes several types of equipment that make the machine very versatile. Thus, a standard machine may accommodate:

-- a twister for the temporary casing,

-- a rotary table (4,000 kgm - 29,000 ft-lbs) with a telescopic kelly of 3 x 7 m (10 x 23 ft) (Ø maximum: 800 mm - 32 in.),

-- a revolving head with or without concrete injection under pressure for construction of continuous-auger type piles such as Tecvis (components of 9 m - 29.50 ft, Ø maximum: 800 mm - 32 in.).

TABLE II

Name	Weight of machine (t)	Height of boring mast (m)	Name of winch outfit	Maximum extraction force (t)	Maximum opening size for drill (mm)
MB 5 - M 7	10	4.40	2 TPA 251 B	30	1,100
MB 6	11	8.10	2 TPA 251 B	25	1,200
MB 7 C	15	7.00	2 TPA 251 B	40	1,400
MB 10 C	18	11.00	TPA 350 E	80	800 - 1,400
MB 12 C	21	11.00	2 TPA 350 S	40	1,100

(0.0254 m = 1 in.; 25.40 mm = 1 in.; 0.91 t = 1 ton)

DRILLING TOOLS FOR PERCUSSION DRILLING

1. PERCUSSION DRILLS

These tools are used to penetrate blockages or fallen rock and for socketing in rock or very resistant ground such as calcarious soils and operate by freefall. They are adaptable to all drilling setups or cranes with winches and equipped with a control to ensure the freefall of the tool. Their form is varied: with central chopper, with a double chopper arranged asymetrically, or with triple or cruciform choppers. The diameter of the tools varies between 0.30 to 1 m (1.0 to 3.28 ft) and their weight may attain several tonnes by the addition of extra weights.

Table I shows several examples of the characteristics of percussion tools of the cruciform type.

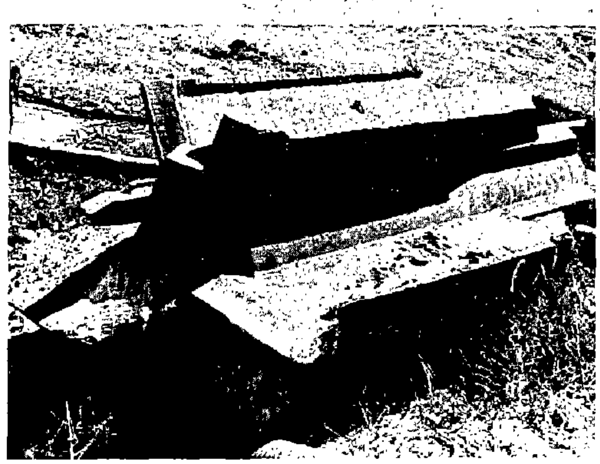


Fig. 2.1. -- Example of multiple-bladed percussion tool.

2. CLEANING BUCKETS OR GRAB BUCKETS

After the use of the percussion tools, the clean-out tools permit the raising of the loose debris or cuttings.

The lower portion or cutting shoe includes a grab that opens at the time of the descent and closes itself after the debris has entered. With the proper operation of the winch, the grab can be lowered with a high velocity so that the interior of the tool will fill up (Fig. 2.4).

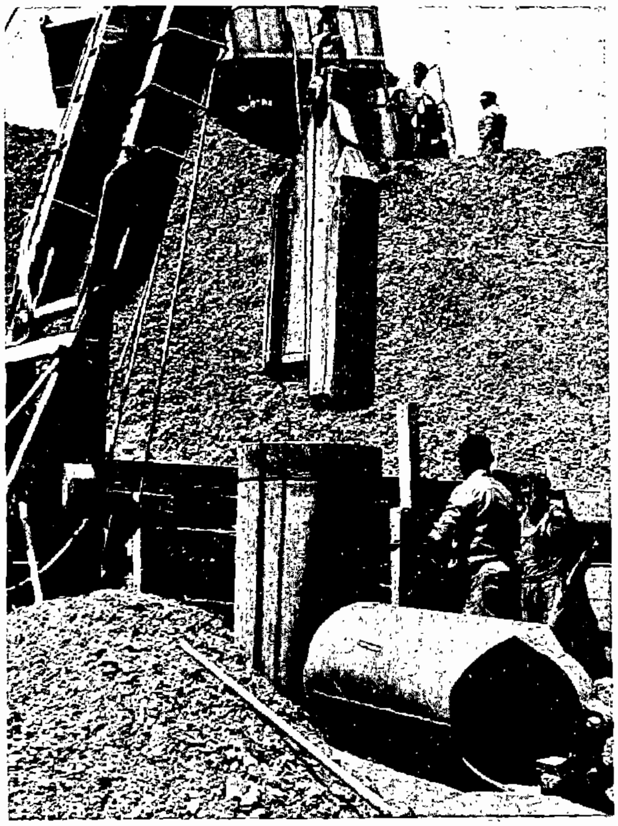


Fig. 2.2 -- Two-bladed cutter, asymetrical percusion tool. Grab in foreground.

Fig. 2.3. -- Cruciform percusion tool.

TABLE I

EXAMPLES OF CHARACTERISTICS OF CRUCIFORM PERCUSSION TOOLS

Diameter (mm)	Inside diameter of casing (mm)	Play (mm)	Thickness of cutters (mm)	Weight of tool without extra weight from drill string (kg)
300	340	20	50	400 - 600
320	360	20	50	500 - 600
350	390	20	50	550 - 650
360	400	20	50	700
390	430	20	80	700 - 750
410	450	20	80	750
460	500	20	80	750 - 800
500	550	25	80	800 - 900
550	600	25	80	800 - 1,000
600	660	30	90	900 - 1,100
620	680	30	90	950 - 1,300
700	800	50	90	1,000 - 1,500
800	1,000	100	90	1,800
1,050	1,250	100	90	2,000

(25.40 mm = 1 in.; 0.453 kg = 1 lb)

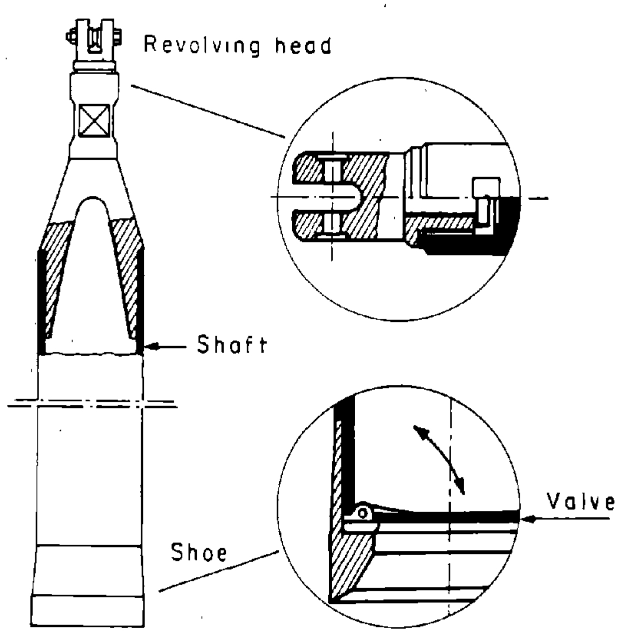


Fig. 2.4. -- Chopping tool with valve.

For boring diameters of 60 cm to 1 m (24.0 in. to 3.28 ft) and above, the diameter of the grab must be less by 10 to 15 cm (4.0 to 6.0 in.) than the inside diameter of the temporary casing in order to facilitate the operation of the tool.

The Benoto Company manufactures a special tool (sand grab) for penetrating sandy soils that are wet. The tool has a unique foot that opens to empty the soil (Fig. 2.5).

In selecting a winch for a particular job, an important point is that the required lifting force is from 2 to 2.5 times the weight of the empty tool.

3. HAMMERGRABS

A hammergrab is a bucket that can work in percussion, owing to the presence of two chopping bits at the bottom of the tool which is dropped (with tool open) and retrieved with the debris (with the tool closed). The hammergrab can be operated by any machine that can lift the tool (and soil) and can drop the tool in freefall.

The Benoto Company manufacteres five basic models of hammergrabs, CP4, CP5, CP6, CP7 and CP8 (Table II), to which various types of chopping bits can be attached. The particular chopping arrangement should be selected depending on the nature of the soils (see Technical Note No. 4).

The Casagrande Company also markets eleven models of hammergrabs for which the main characteristics are shown in Table III.

4. GRAB BUCKETS

For the construction of cylindrical, bored piles all of the grab buckets are of the cable type and their closing mechanism is composed of two series of pullies located on the upper portion and base of the body of the bucket.

There are also hydraulic buckets that may be used with boring machines of the Poclain type, for example (see Technical Note No. 10).

Tables IV, V and VI summarize the main characteristics of buckets on cables that may be employed for construction of bored piles with the percussion method.

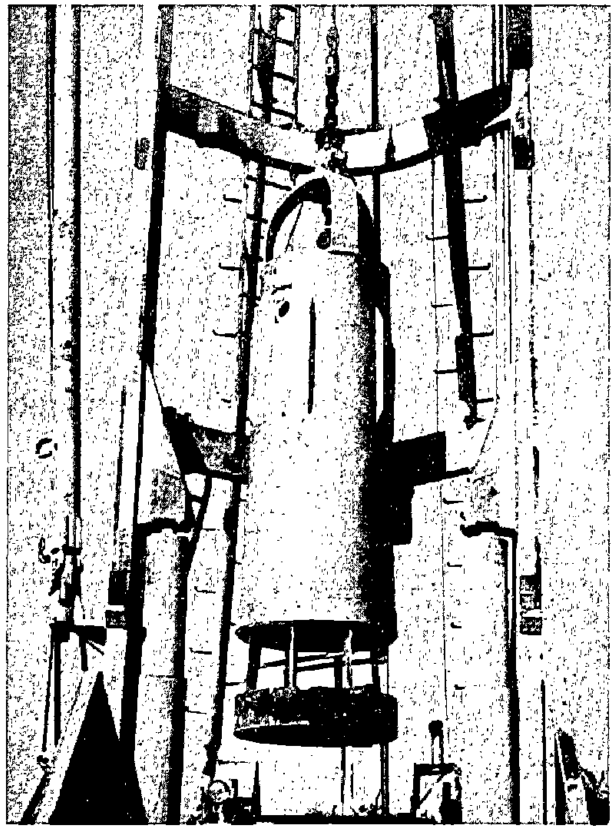


Fig. 2.5. -- Sand valve, open position (Document from Benoto).

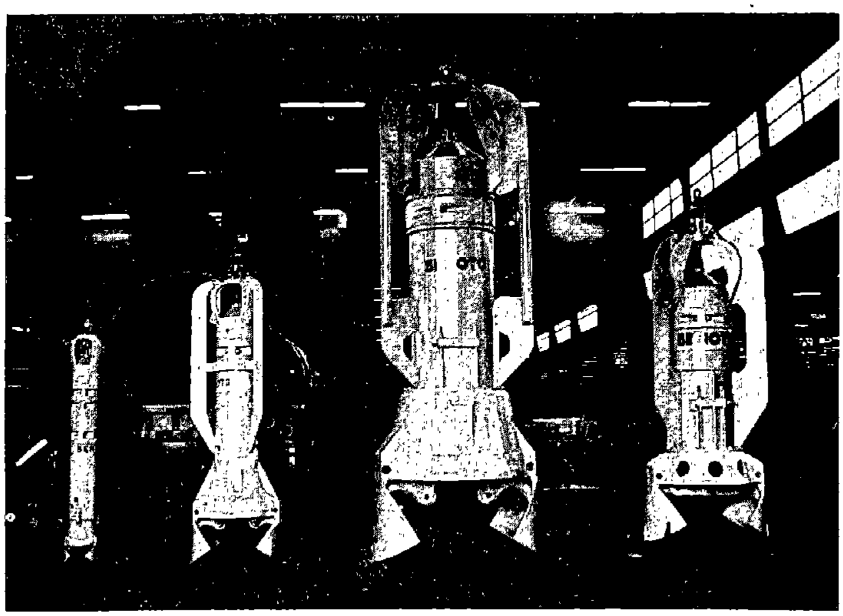


Fig. 2.6. -- View of group of four models of Benoto chopping buckets (Hammergrab); from right to left: CP 4, CP 5, CP 8, CP 6.

TABLE II

MAIN CHARACTERISTICS OF BENOTO HAMMERGRABS

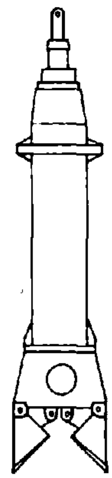
Type of bucket	Lifting Force (t)	Maximum weight with chopping bit (t)	Diameter of chopping bit (mm)	Adaptable chopping bits or buckets				
				Hemispherical				super-reinforced
				normal	with claws	for clay	semi-long	
CP 4	1	0,360	360	×		×		×
			440	×	×	×		×
			520	×	×	×		×
CP 5	2	1.450	570	×		×		×
			750	×	×	×	×	×
			850	×	×	×	×	×
			950	×	×	×		×
			1,050	×				×
CP 6	5	2.850	950	×		×		×
			1,200	×	×	×		×
			1,400		×			
CP 7	5	3.500	1,050	×				
			1,250	×				
			1,400	×				
CP 8	7 5	5,700	1,400	×				
			1,650	×				
			1,750	×				

(25.40 mm = 1 in.; 0.91 t = 1 ton)

TABLE III

MAIN CHARACTERISTICS OF CHOPPING BUCKETS BY CASAGRANDE COMPANY

Model	Open bucket diameter (mm)	Height (m)	Weight (kg)
BELR 600	520	2.87	1,700
700	580	2.87	1,750
800	730	3.00	1,820
900	790	3.00	1,870
1000	850	2.95	2,050
BEL 1000	850	2.80	2,750
1250	1,050	2.90	2,800
1300	1,150	3.00	2,850
1500	1,250	3.20	2,900
1800	1,550	3.60	4,100
2000	1,750	3.80	4,600



(25.40 mm = 1 in.; 0.0254 m = 1 in.; 0.453 kg = 1 lb)

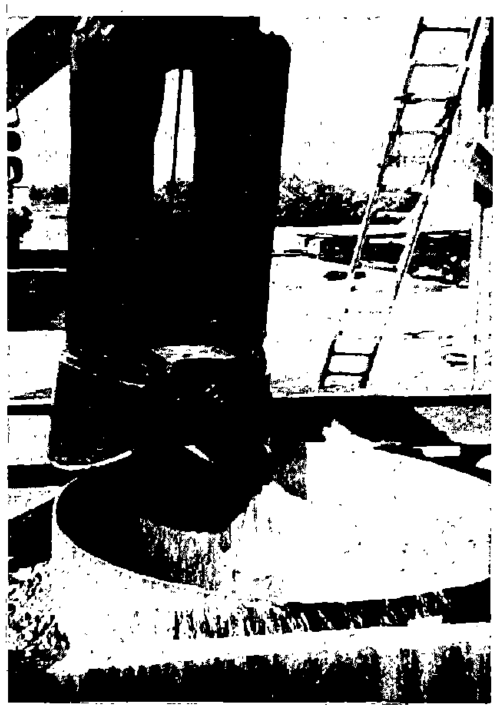


Fig. 2.7. -- Type SC bucket.

TABLE IV. -- BENOTO BUCKETS, SC TYPE

Code	Crown	Borehole diameter (mm)	Lifting force (kg)	Weight (kg)	Capacity (liters)	Height	
						Closed bucket (m)	Open bucket (m)
SCO	CP 4	600 - 660	1,000	800	20	1.700	1.750
SCD	CP 4 - CP 5	800 - 880	1,250	1,050	40	1.900	1.950
SCE	CP 5	1,000 - 1,000	2,000	1,750	90	2.400	2.475
SCE	CP 5	1,100 - 1,200	2,300	1,850	140	2.450	2.550
SCB	CP 6	1,400 - 1,540	3,000	2,550	300	2.920	3.025

(25.40 mm = 1 in.; 0.453 kg = 1 lb; 3.8 liter = 1 gal; 0.0254 m = 1 in.)

TABLE V
GALLIA, HEMISPHERICAL BUCKETS

Weight (kg)	Model	Empty weight (kg)	Capacity (liters)	Width (m)	Open height (m)
800	H 800	550	150	1.35	1.76
1,000	H 1000	700	200	1.60	1.85
1,250	H 1250	875	250	1.75	1.82
1,400	H 1400	970	290	1.85	1.95
1,500	H 1500	1,030	300	1.85	1.95
1,750	H 1750	1,150	370	1.95	2.12
2,000	H 2000	1,200	400	1.95	2.12
2,500	H 2500	1,500	500	2.30	2.37
3,000	H 3000	1,950	600	2.40	2.40

(0.453 kg = 1 lb; 3.8 liter = 1 gal; 0.0254 m = 1 in.)

TABLE VI
MASSANZA-GALINET GRAB BUCKETS

Model	Diameter of bucket (cm)	Weight (kg)
750	75	2,100
870	87	2,200
1100	110	2,300

(2.540 cm = 1 in.; 0.453 kg = 1 lb)

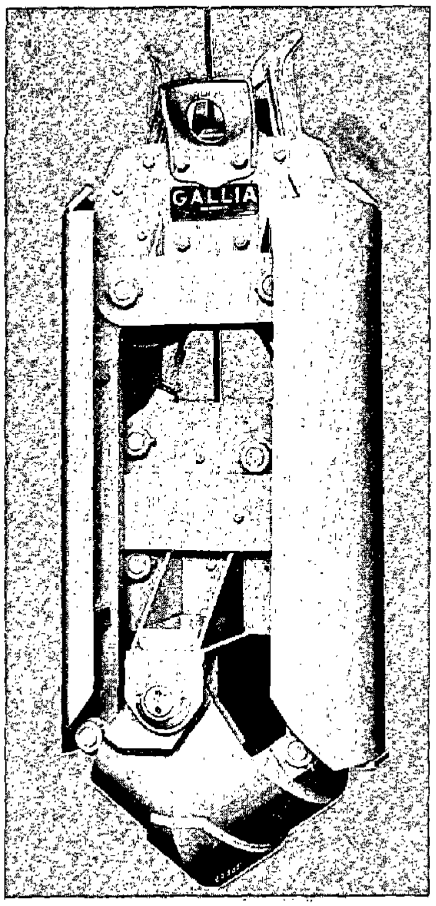


Fig. 2.8. -- Heavy, sinking bucket, hemispherical type (Document from Gallia).

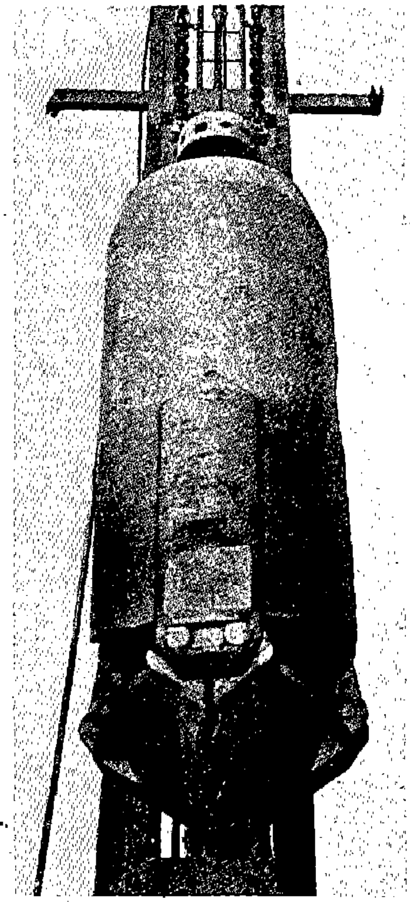


Fig. 2.9. -- Heavy bucket, "Massenza Galinet" (Document from Galinet).

TEMPORARY CASINGS

1. BENOTO TEMPORARY CASINGS

Two types of temporary casings are manufactured by the Benoto Company:

a) **Welded, metal casings**

A distinction must be made between the pipe that make up the main portion of the casing from the "base" pipe to which the cutting teeth are welded.

Most of the pipe used for casing is delivered in uniform lengths of 1.5 m (5.0 ft) with the heavy-walled pipe for the bottom of the casing coming in lengths of 2 meters (6.56 feet). Table I lists the main characteristics of casings.

b) **Quick-lock casings** (Table II)

-- the casings are *double walled* and perfectly smooth inside and outside;

-- they are delivered in uniform lengths of 6 m (19.69 ft), 4 m (13.12 ft), 2 m (6.56 ft) and 1 m (3.28 ft);

-- assembly is ensured by one type of lock regardless of the diameter of the casings; and

-- pipe for the bottom of the casings is adaptable to cutting teeth of various types that are selected in relation to the soils.

2. GALINET TEMPORARY CASINGS

Two models are manufactured by the Galinet Company:

a) **Casings with square-face threaded connections**

These casings are made of steel, without welds, or rolled sheets that have a male and female threaded connector welded to each end. Table III lists the main characteristics of these casings.

b) **Quick-lock casing**

These casings of rolled sheet metal consist of a system of socketing that is male and female, with an extendable lock similar to Benoto casings. They are delivered in the dimensions given in Table IV.

Again, it is necessary for construction of a pile and especially for passage of the reinforcing cage to take into consideration the inside diameter at the connectors.

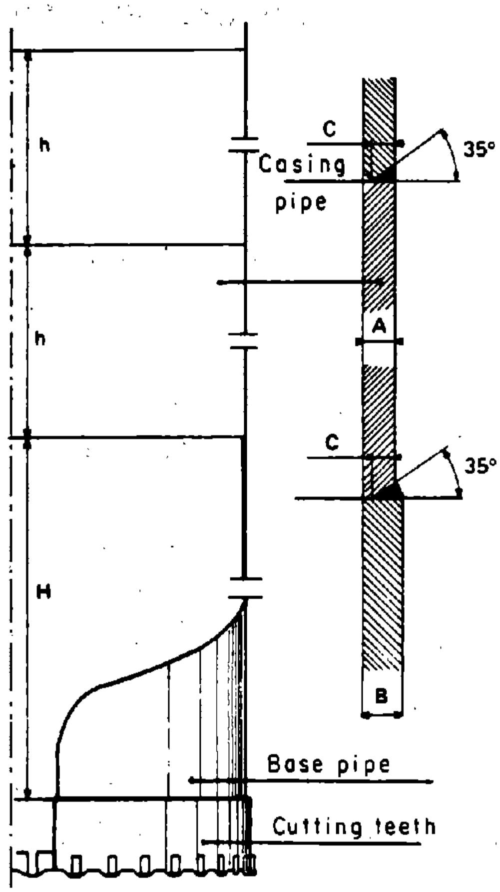


Fig. 3.1. -- Welded metal casing.

TABLE I

TEMPORARY CASINGS						
A (mm)	inside Ø (mm)	Expanded length (mm)	Height (mm)	Weight per meter (kg)	Segment (1.5 m) Weight (kg)	C (mm)
6	400	1,274	1,500	60	90	4
6	500	1,588	1,500	75	112,5	4
6	600	1,902	1,500	90	135	4
8	800	2,537	1,500	159	238	6
10	1,000	3,171	1,500	248	272	8
10	1,250	3,956	1,500	309	464	8
12	1,500	4,747	1,500	445	668	10
PIPES FOR BOTTOM OF CASING						
B (mm)	inside Ø (mm)	Unrolled length (mm)	Height (mm)	Weight per meter (kg)	Segment (2.0 m) Weight (kg)	
8	400	1,281	2,000	80	160	
8	500	1,595	2,000	100	200	
10	600	1,915	2,000	150	300	
10	800	2,543	2,000	198	397	
12	1,000	3,177	2,000	298	595	
12	1,250	3,962	2,000	371	742	
14	1,500	4,753	2,000	520	1,039	

(25.40 mm = 1 in.; 0.453 kg = 1 lb)

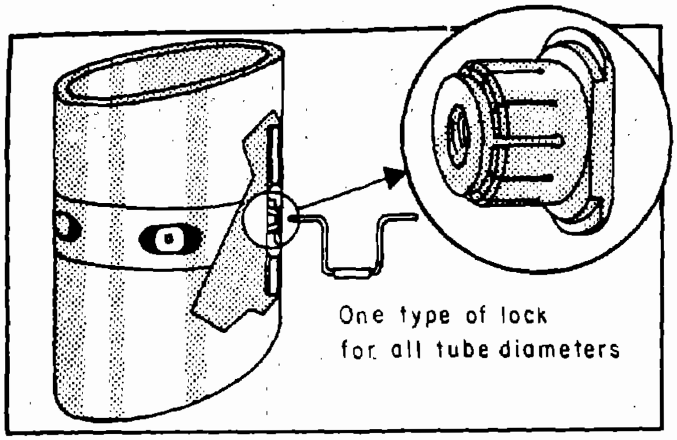


Fig. 3.2. -- Locking system.

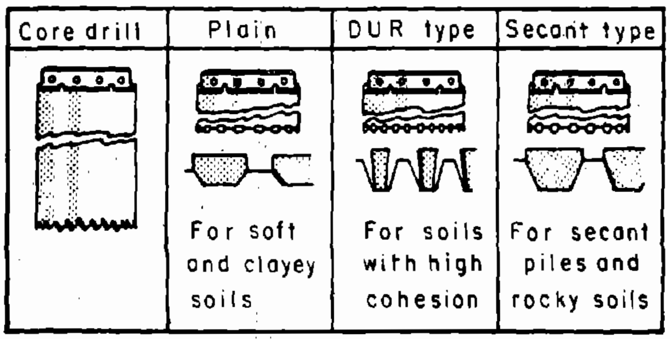


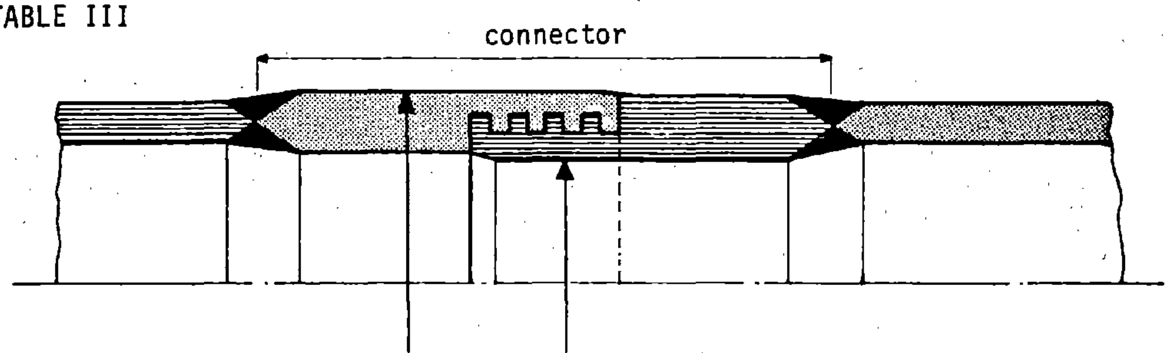
Fig. 3.3. -- Cutting teeth.

TABLE II

Inside diameter of casing	Outside diameter of casing	Number of locks	Diameter of locks	Diameter of cutting head
400	470	6	40	480
460	530	6	40	540
540	610	8	40	620
590	670	12	55	680
800	880	12	55	890
890	970	15	55	980
1,000	1,080	15	55	1,090
1,100	1,180	16	55	1,190
1,190	1,270	20	55	1,280
1,300	1,390	24	55	1,390
1,500	1,580	24	55	1,590
1,600	1,680	32	55	1,690
1,750	1,830	32	55	1,840
1,880	1,960	32	55	1,970
Dimensions expressed in mm				

(25.40 mm = 1 in.)

TABLE III



Diameter of casing (mm)	Thickness (mm)	Outside diameter (mm)	Inside diameter (mm)	Length (mm)	Total length (mm)	Weight (kg/m)
■● 207 - 219	6	228	198	250	2.50 - 4.00	35
■● 261 - 273	6	281	251	250	2.50 - 4.00	45
■● 309 - 323	7	332	300	250	2.50 - 4.00	60
■● 339 - 355	8	365	333	260	2.50 - 4.00	75
■● 352 - 368	8	375	343	250	2.50 - 4.00	83
■● 388 - 406	9	409	377	250	2.50 - 4.00	95
■● 400 - 419	9	428	396	250	2.50 - 4.00	110
■ 405 - 425	10	428	396	250	2.50 - 4.00	115
■ 430 - 450	10	459	427	250	2.50	118
■ 450 - 470	10	478	442	250	2.50	125
■ 505 - 525	10	589	497	250	2.50	138
■ 550 - 570	10	577	545	250	2.50	152
■ 605 - 625	10	636	600	300	2.60	176
■ 655 - 675	10	686	650	300	2.60	185
■ 680 - 700	10	712	676	300	2.60	192
■ 805 - 825	10	837	801	300	2.60	230
Standard dimensions		Dimensions at connector				

(25.40 mm = 1 in.; 1 kg/m = 0.672 lbs/ft)

TABLE IV

Diameter (mm)	Thick-ness (mm)	Inside diam-eter (mm)	Number of Slots	Total length (m)	Weight (kg/m)
800/ 820	10	740	16	3.00	280
900/ 920	10	840	16	3.00	320
1,000/1,024	12	940	20	3.00	400
1,250/1,274	12	1,190	24	3.00	500
1,500/1,530	15	1,440	24	3.00	700

(25.40 mm = 1 in.; 0.0254 m = 1 in.; 1 kg/m = 0.672 lbs/ft)

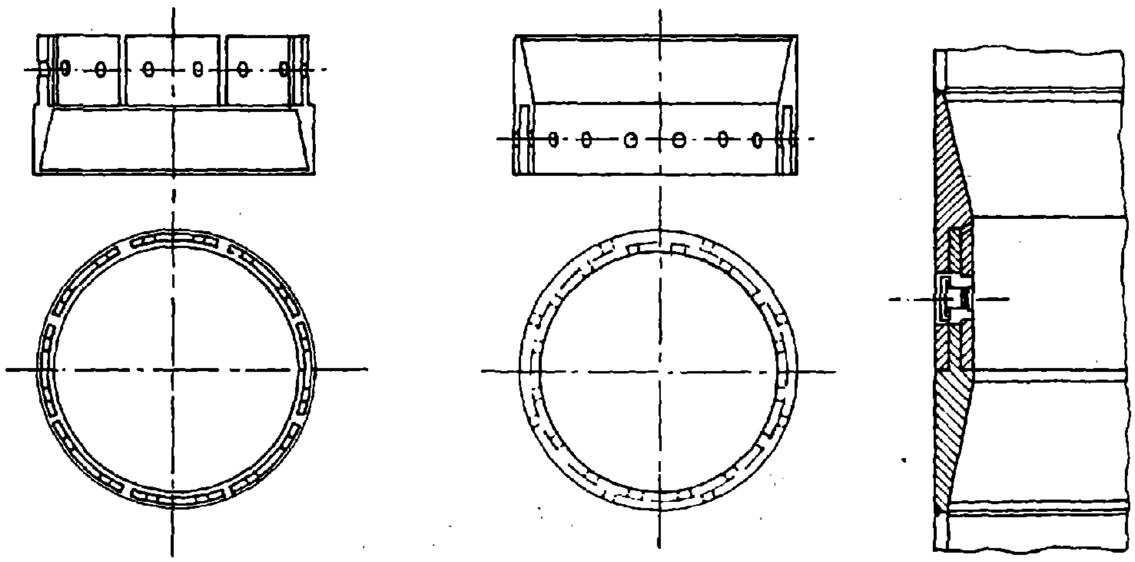


Fig. 3.4. -- View of quick-connecting joints.

BORING BY BENOTO METHOD

The Benoto method of boring is based on two principles: the use of a special tool (the hammergrab), see TN No. 2, that performs the function of the drill bit and or a bucket, and on the placement of a temporary casing by a twisting and rocking movement.

After the creation of drills No. 5 and No. 6, the Benoto Company built the EDF 55 machine, combining the operations of excavation and placement of the casing. These operations had been independent until then (1955). This machine, limited to boreholes cased to a maximum diameter of 970 mm (38.2 in.) and making special use of the hammergrab CP 5, was rapidly followed by the Super EDF, also called EDF 1180 (max. Ø: 1,180 mm - 47 in.), the EDF 1580 associated with hammergrabs CP 6 and 7, and finally the EDF 2000 (borehole of 1,960 mm - 77 in.) with hammergrabs CP 7 and 8).

1. EDF SERIES MACHINES

These are composed of the boring system (winch, excavation system) and of the hydraulic system for driving, twisting, and extraction of the temporary casing.

The main characteristics of interest to the construction supervisor, in addition to the boring diameters of each type of machine in relation to the corresponding tool, are summarized in Table I.

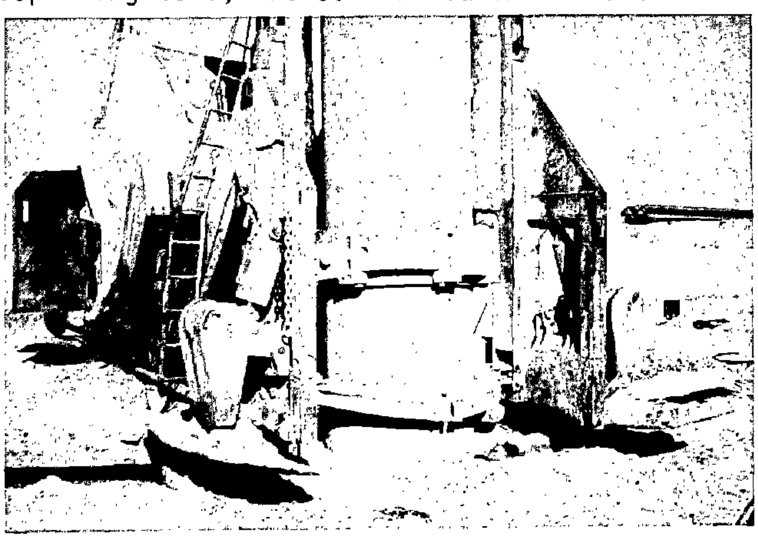


Fig. 4.1. -- Detail view of twisting system.

TABLE I

MAIN CHARACTERISTICS OF EDF MACHINES
(possible inclinations 6° and 12°)

Model	Borehole diameter (mm)	Tool	Height (m)	Lift (tonnes)	Weight Job site (tonnes)	Weight Road (tonnes)
EDF 1180	670 - 880 980 - 1,080 1,190	CP 5	13.50	60	32	34
	470 - 530 610	CP 4				
EDF 1580	1,180 - 1,270 1,380 - 1,580	CP 7	16.45	90	50	45
	670 - 880 890 - 1,080 1,180	CP 5				
EDF 2000	1,580 - 1,680 1,830 - 1,960	CP 8	21.24	120	105	65
	1,180 - 1,270 1,380	CP 7				

(25.40 mm = 1 in.; 0.0254 m = 1 in.; 0.91 tonne = 1 ton)

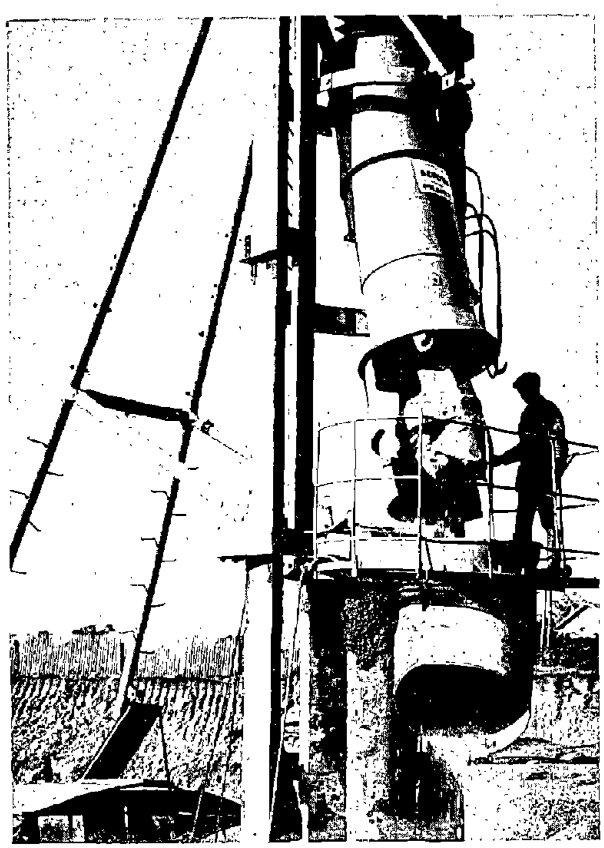


Fig. 4.2. -- Detail view of debris-emptying system.

2. FONCEX DRIVER-EXTRACTORS NO. 4 AND NO. 5

These machines, used for driving and extraction of temporary casing, are coupled with an independent drilling machine (a crane for example). They make use of the techniques employed by the EDF machines, ensuring the placement of the casing by twisting motion (Ø 470 to 1,180 mm - 18.5 to 46.5 in.), (Table II).

The machine consists of (Fig. 4.3):

-- the apparatus for handling the casing that places, twists, and recovers the casing; and

-- a group of controls that allows the machine to be operated remotely.

The Benoto Company has developed for commercial use a machine that permits the placement of casings of 1,960 mm (77.2 in.) in diameter.

Note -- There is other equipment, similar in design to the Foncex, that is not yet commercially available in France.

It is also of interest here to mention the Casagrande driver-extractors of the MKT type for casing diameters of 700 to 1,000 mm (27.6 to 39.4 in.), and those of the GC 72 type for diameters from 1,000 to 1,500 mm (39.4 to 59.0 in.).

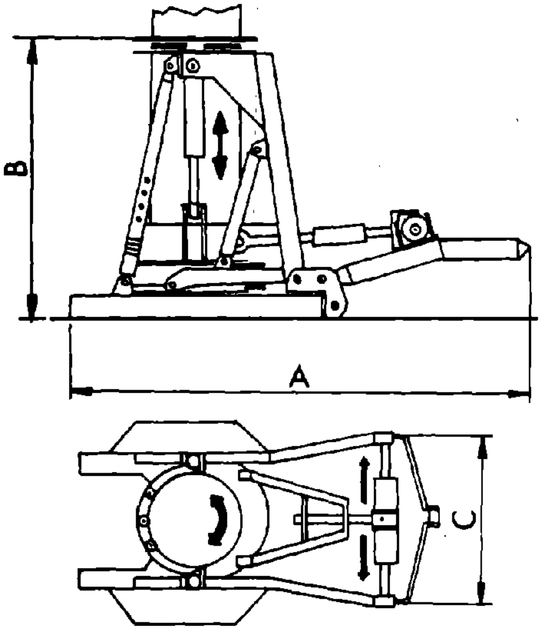
Salzgitter also manufactures VR-type machines for diameters of casing of 600, 800, 1,000, 1,250, and 1,500 mm (23.6, 31.5, 39.4, 49.2, and 59.0 in.) (Fig. 4.4).

All of these tools function on the same principal: the placement of the casing is achieved by driving and twisting. Their main advantage over machines of the EDF type is their light weight and the independence of the boring system that involves a simple crane that maneuveres a tool adapted to the nature of the soil.

TABLE II

Characteristics	Foncex 4	Foncex 5
Borehole diameter (mm)	480 - 540 620 - 680	680 - 890 - 980 1,090 - 1,190
Lifting force (tonnes)	20	61
Outside dimensions (meters):		
A	3.75	4.80
B	2.40	3.15
C	1.95	2.30
Inclination	up to 12^{o}	

(25.40 mm = 1 in.; 0.0254 m = 1 in.; 0.91 tonne = 1 ton)



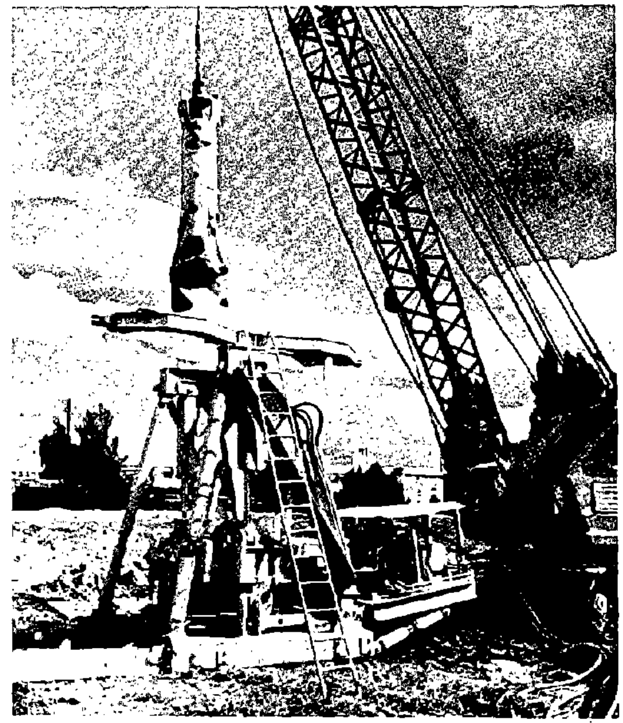


Fig. 4.3. -- Foncex driver-extractor (Document from Benoto).

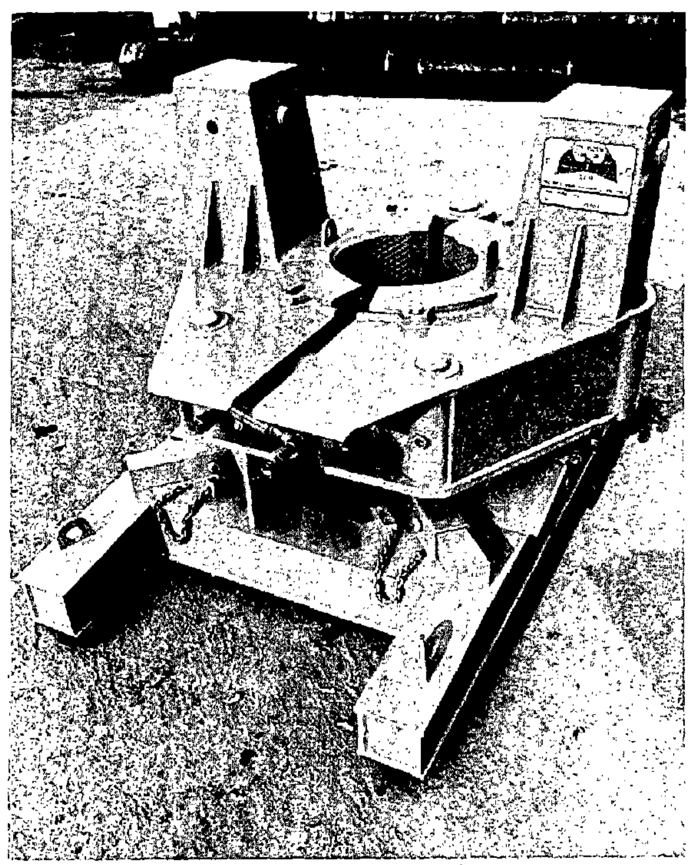


Fig. 4.4. -- Casagrande driver-extractor (Document from Casagrande).

USE OF HAMMERS FOR PLACING CASING

1. DRIVING HAMMERS

There are three main types of hammers:

-- hammers that operate in freefall,

-- hammers that are operated by compressed air or steam, and

-- diesel hammers, for which the striking mass is raised by the energy provided by the explosion of the diesel fuel.

Placement of casings of large diameter requires a driving energy that is relatively large (generally above 3,000 kgm - 21,720 ft-lbs).

Table I summarizes the characteristics of the main hammers used for placement of metal casings (energy ≥ 3,000 kgm - 21,720 ft-lbs).

Fig. 5.1. -- BSP diesel hammer on sheet-wall piles.

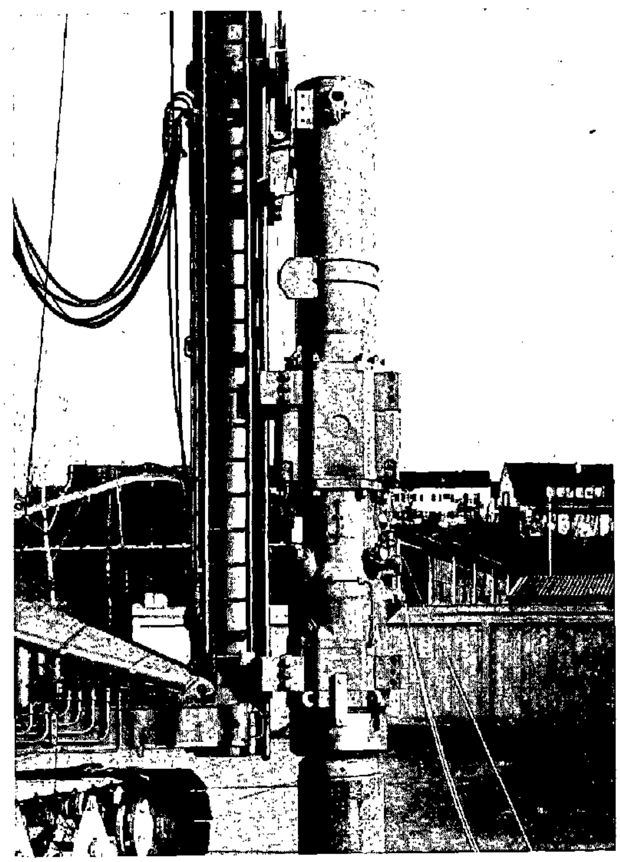


Fig. 5.2. -- Delmag D 44 diesel hammer (Document from Delmag).

2. DOUBLE-ACTING HAMMERS

Double-acting hammers strike with a faster rate (120 to 160 blows per minute) than that of driving hammers.

They are also less powerful and thus are used for driving casings of smaller diameter.

Table II summarizes the charcteristics of some working hammers.

TABLE I

CHARACTERISTICS OF MAIN TYPES OF HAMMERS

Brand	French Representative	Model	Nature of energy	Total weight (kg)	Weight of ram (kg)	Energy per strike (kgm)
DELMAG	DELMAG-FRANCE	D 12 D 22 D 30 D 36 D 44 D 55	Diesel	2,750 5,030 5,600 8,000 10,200 11,960	1,250 2,200 3,000 3,600 4,300 5,400	3,125 5,500 3,300 - 7,500 4,200 - 10,200 6,000 - 12,000 8,650 - 16,200
DEMAG	DEMAG	BB 3000	Air	5,800	3,000	3,750
KOBE	ROLBA S.A.	K 13 K 25 K 35 K 45	Diesel	2,900 5,200 7,500 10,500	1,300 2,500 3,500 4,500	3,750 7,500 10,500 13,500
TIFINE	TIFINE	N° 6 N° 7 N° 8 N° 9 N° 10 N° 11 N° 12	Air or steam	4,400 4,500 5,500 6,500 7,600 8,500 10,500	3,500 4,000 5,000 6,000 7,000 8,000 10,000	3,600 3,600 4,500 5,700 6,300 7,200 9,000
All of these hammers strike at a rate of 40 to 60 blows per minute						

(0.138 kgm = 1 ft-lb; 0.453 kg = 1 lb)

TABLE II

CHARACTERISTICS OF DOUBLE-ACTING HAMMERS

Brand	French Representative	Model	Nature of energy	Total Weight (kg)	Weight of piston (ram) (kg)	Energy per strike (kgm)
BSP	BSP FRANCE S.A.	B 15 B 25 B 35	Diesel	3,820 6,230 8,640	1,500 2,500 3,500	3,790 6,320 8,850
DEMAG	DEMAG	VR 19 VR 40	Air	1,980 3,840	395 910	—
NILENS	MAÎTRAP	T 4	Air	3,400	1,400	3,350
PAJOT	PAJOT	3600 4700 6600	Air or Steam	3,600 4,700 6,600	— — —	1,425 1,950 3,000
TIFINE	TIFINE	T 4 T 5	Air	2,400 3,500	380 590	1,240 2,100

(0.138 kgm = 1 ft-lb; 0.453 kg = 1 lb)

VIBRATORY-DRIVERS FOR CASINGS

1. PRINCIPLE

These tools for driving or extraction of casings (case of vibratory-driven piles) operate on the principle of unidirectional vibration in the vertical plane.

A vibration-driver is composed of one or several motors causing rotation of eccentric masses, an oscillating system (on springs), and a helmet that permits, by tightening with hydraulic rams, the joining of the vibratory-driver and the casing to be driven.

At the time of driving or extraction, vibrations destroy the lateral resistance and provide significant forces at the base. The devices differ mainly as to their weight and frequency of vibration which may be variable on some models. Some companies have designed their tools so that they can be used in tandem. Use in tandem is in fact advantageous.

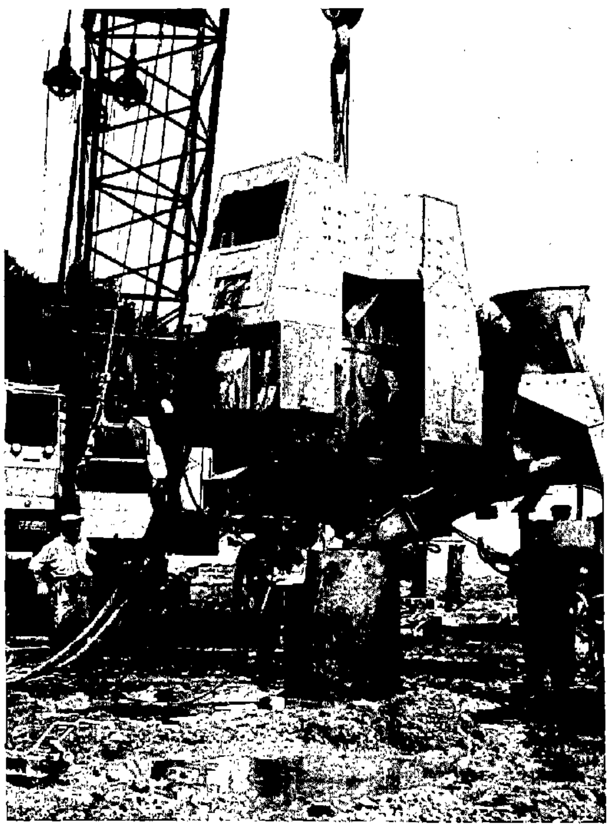


Fig. 6.1. -- PTC 40 A2 vibratory-driver over casing Ø 120, length 22 m.

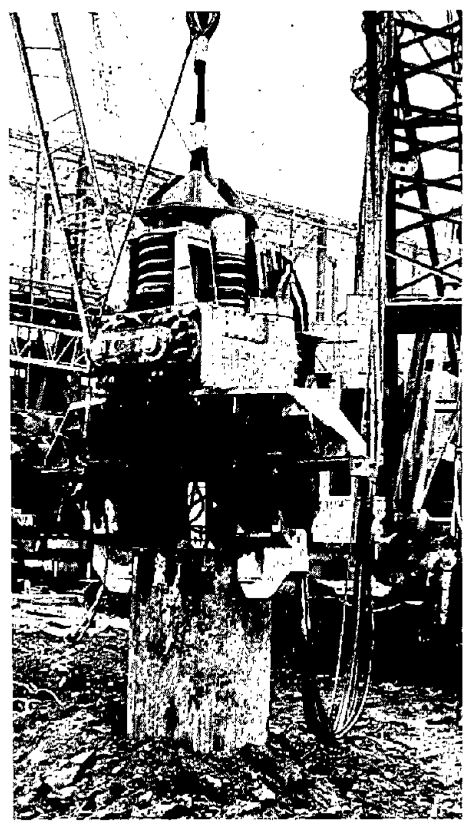


Fig. 6.2. -- Muller MS 26 vibratory driver (Document from Rolba).

2. BASIC COMPOSITION OF A VIBRATORY-DRIVER UNIT

This unit is composed of:

-- one or two vibratory-drivers in tandem,

-- a mechanical device for connecting the driver to the pile or a hydraulic device for that purpose,

-- a ground unit composed of a housing, a set of cables connecting the vibratory-driver to a push-button box and an power-generating outfit, and

-- a crane that is usually mounted on tracks.

For driving, the lifting force of the crane should be greater than the weight of the vibratory-driver outfit, the helmet, and the casing. The force available for extraction is equal to the difference between the maximum acceptable value for the hook and the total weight of these components. The height of the boom on the crane is obviously a function of the total height of the components of the casing and the vibratory-driver.

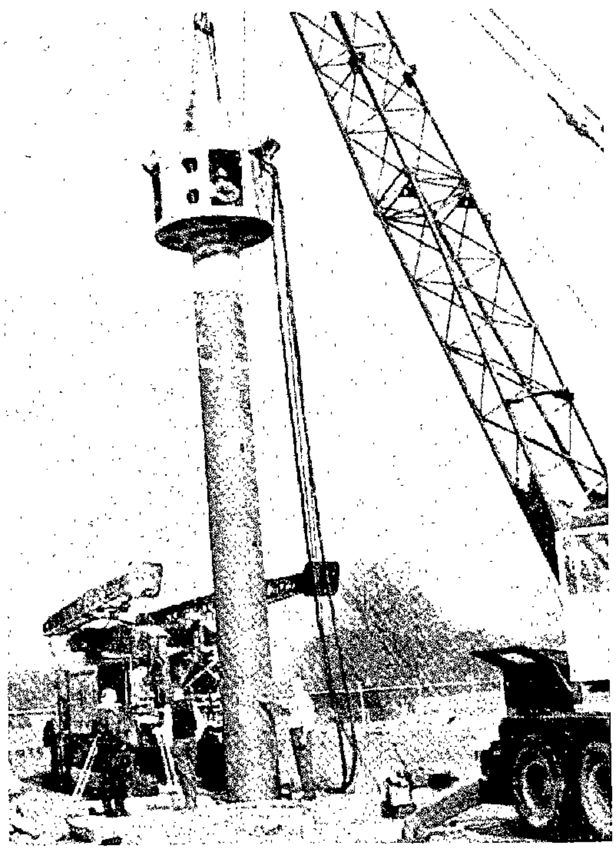


Fig. 6.3. -- Twin, Schenk DR 60 vibrators.

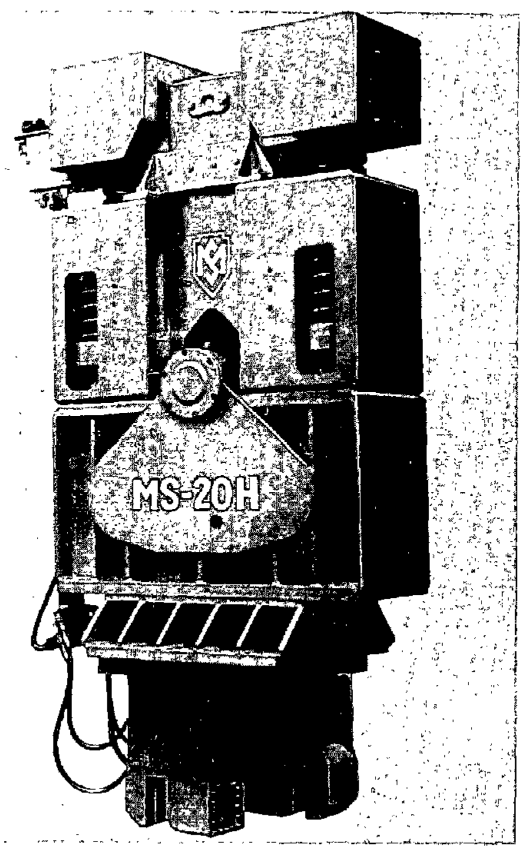


Fig. 6.4. -- Muller MS 20 H hydraulic vibratory driver.

3. TABLE OF VIBRATORY-DRIVERS

For information only, the table accompanying this technical note lists various vibratory-drivers that seem most characteristic for ensuring driving and extraction of temporary casings required for construction of vibratory-driven piles.

VIBRATORY-DRIVERS FOR PLACEMENT OF METAL CASINGS

Brand	French Representative	Model	Weight (kg)	Vibrations per minute	Motor (kW)	Power (HP)	Installation power	Lifting force for extraction (tonnes)	Diameter of placed casings (cm)
Muller	Galinet	MS 26	4,700	1,465	2 × 27	40	125 kVA	15 - 25	275 max. with special helmet (triform)
		MS 20 H	3,200	1,600	hydraulic		diesel	24	
	Rolba	MS 26 D	7,600	1,465	4 × 27	80	250 kVA	40	
		MS 60 E	7,500	726 - 1,323	2 × 55		250 kVA	40	
PTC	PTC	10 A 2	2,350	720 - 1,140	2 × 15	20	80 kVA	8 - 9	27 to 95
		20 A 2	3,700	920 - 1,100	2 × 29	40	120 kVA	18 - 20	26 to 195
		20 H 4	4,500	1,450 max.	hydraulic	—	200 HP	20	26 to 195
		40 A 2	7,400	770 - 1 045	2 × 55	80	200 kVA	40	35 to 285
		40 H A	10,500	1,450 max.	hydraulic	—	550 HP	40	35 to 285
Schenk		DR 40	5,500	1,200 - 1,920	—	—	170 kVA	30	—
		DR 60	7,200	950 - 2,350	2 × 40	55	175 kVA	—	≤ 200
Tomen-Vibro	SPEC	VM 2 - 4000 A	3,628	860 - 1,300	60	40	115 kVA	> 25	40
		VM 2 - 5000	4,887	920 - 1,800	90	60	180 kVA	> 30	≤ 80
		VM 4 - 10000	8,300*	1,100	150	110	300 kVA	> 30	≤ 120
		KM 2 - 12000	4,510	500 - 600	90	60	200 kVA	—	≤ 120
		VM 2 - 25000	7,400	620	150	110	300 kVA	—	≤ 200
Vibro-Mac	LCM Equipement	Vibro-Mac 5	4,940	1,095 - 1,770	90	120	250 kVA	40	80 - 150
		Vibro-Mac 12	6,100	560 - 1,020	90	120	250 kVA	40	

* Weight with helmet; (0.91 t = 1 ton; 2.540 cm = 1 in.; 0.453 kg = 1 lb)

AUGER-TYPE DRILLING MACHINES

These machines, depending on the model, are mounted on heavyweight, all-terrain trucks, on cable-cranes, or on hydraulic shovels preferably on trucks. Their form is thus very diverse but they all employ a rotary table that turns the bar (kelly) that carries the drilling tools. The kelly is most often telescopic (double, triple or even quadruple), so as to increase significantly the depth of boring.

To our knowledge, the machines mainly used on jobs in France are:

-- English (B.S.P. McAlpine),
-- American (Hughes-Williams, Watson, Calweld),
-- German (Salzgitter),
-- Italian (Soil-Mec-Trevisani),
-- French (Galinet, Domine).

However, companies sometimes modify these by adding supplementary equipment.

1. TRUCK-MOUNTED MACHINES

These machines are divided into two categories according to capacity and use:

a) Lightweight machines

Within this group are a number of models that are limited to a boring diameter of 60 cm (23.6 in.) for maximum depths of 10 to 15 meters (32.81 to 49.22 feet). These are similar to Highway or Texoma machines that are used for geotechnical investigations for highways and they are used for the construction of less important foundations (installation of telephone poles or similar utilities, foundations for houses or light buildings at depths on the order of 10 meters - 32.81 feet).

b) Heavy Machines

Table I gives an extensive (but incomplete) list of various machines used in the construction of bored piles up to depths of 30 m (98.43 ft) and above. These are generally composed of (Fig. 7.1):

-- *a platform* (1) connected to the frame of the truck that supports the boring unit. A turntable (2) permits rotation of the machine to allow excavations on either side of the truck axle or more simply to allow the digging tools to be emptied. A translation unit (3) ensures a front-back

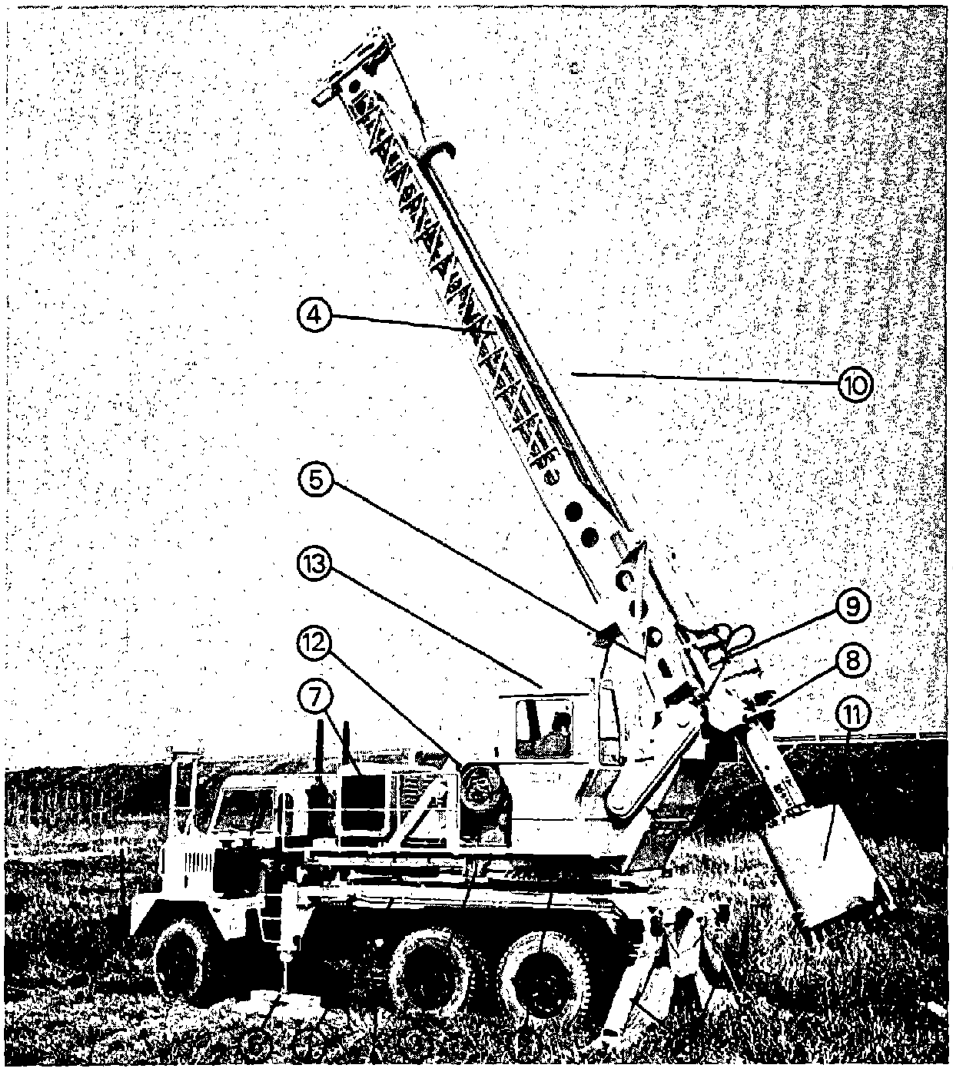


Fig. 7.1. -- RTA 10 drilling machine mounted on truck (after Document from Soil-Mec.).

displacement on the order of one meter that thus facilitates the correct positioning of the borehole. (It is of interest to note that these devices are not available on Calweld machines.)

-- *a derrick* 4 that is placed in a working position or transporting position by two hydraulic jacks 5 .

-- *a set of levelling devices* 6 composed of hydraulic rams attached with arms on the truck frame that provide the required stability during the course of drilling.

-- *a boring unit* including:

• *the power unit* consisting of a diesel motor 7 and a transmission that drives the rotary table 8 ;

• vertical *power system* 9 providing the "crowd" force on the kelly 10 and drilling tool 11 that is required during rotation to cause the tool to penetrate. This power device varies by model and is most often patented (Pull-down system);

• *two winches* 12 with several drums - one to support the kelly and one as the "service" winch that is used for miscellaneous work or when boring by percussion.

• *possibly a cab* 13 with control panel and levers.

The principal characteristics of various machines for which information from the sellers and the manufacturers is available are shown in Table I.

The values of torque as well as the dimensions of the equipment are given for information only, particularly concerning the *brake torque*. Some of the values in the table correspond to theoretical torque, assuming an efficiency of the transmission systems at 100%.

It is also noted that the somewhat large weight of these machines requires that they be mounted on trucks with a gross weight often exceeding 25 tonnes (27.5 tons), and thus job site roads are required for access (Fig. 7.2).

Finally it is noted that such units may easily be used in association with a drilling-slurry station (see TN No. 9).

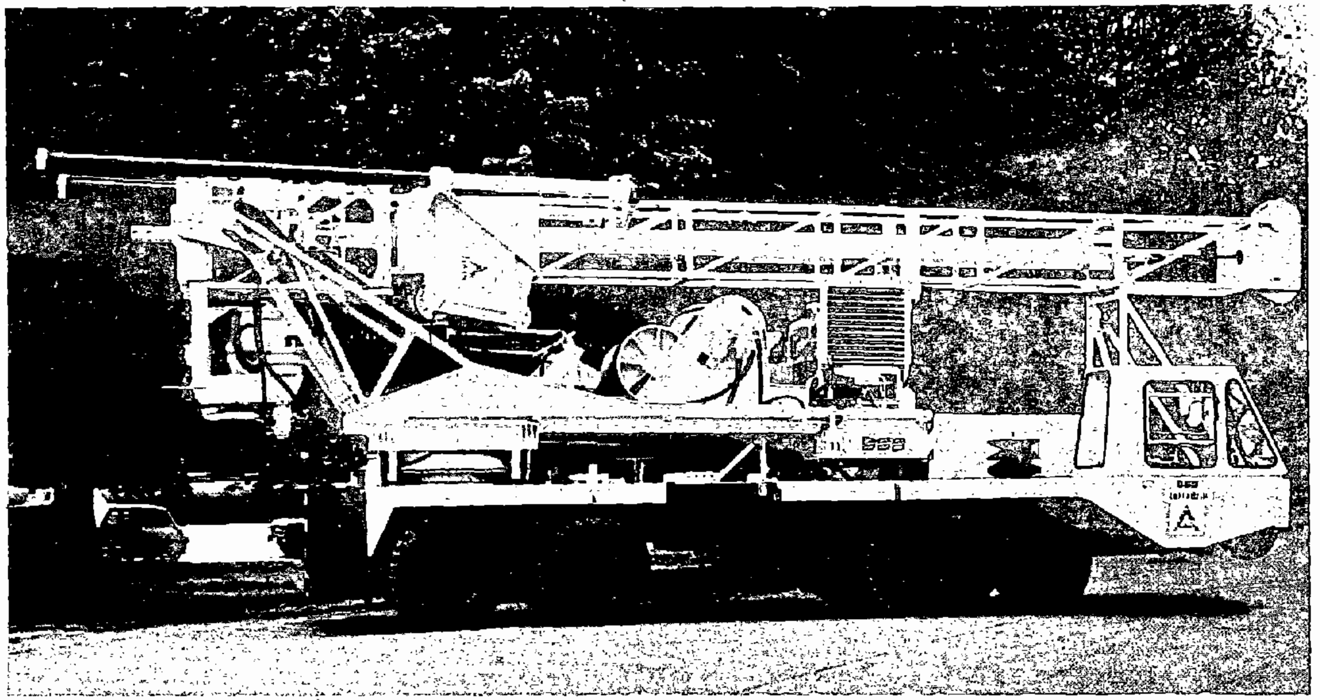


Fig. 7.2. -- Drilling machine, 1,250 T, mounted on truck in road position (Document from BSP France).

2. DRILLS THAT MAY BE USED WITH CRANES OR HYDRAULIC SHOVELS

a) Machines used with hydraulic shovels

This group includes BB series machines (Fig. 7.3) from Salzgitter, the 400 CM from BSP and the MPC 2 from Galinet. These make use of the hydraulic mechanism of the shovel.

Some units among these are distinct because they use their own drilling mast to serve as a guide for the rotary table (with Salzgitter models, the rotary table can shift on the mast a variable distance of 5.50 m to 8.50 m - 18.04 ft to 27.89 ft depending on the model) or may serve as a guide for the kelly with a fixed rotary table (case of the 400 CM and the MPC 20 for which the drilling masts may be broken down for transport).

b) Machines used with a conventional crane

The rotation unit composed of the motor and transmission system and the rotary table is mounted on a fixed or sliding platform that can be adjusted to variable distances from the crane. Distinction is made between:

-- *Drills for which the platform,* located at a fixed height in relation to the level of the natural terrain, is articulated to the frame of the crane (Figs. 7.4 and 7.5).

Positioning of the platform is ensured:

• by two vertical cables that are attached to the top of the boom that also serves as a guidance system during both the lowering and raising of the kelly (models 500 BSP, 55 CH Calweld, CEZ 300 Hughes...);

• or by two hydraulic rams. The kelly is thus guided only at the level of the rotary table and the "crowd" system (models BSP 1000 to 1250, Calweld 150 and Soil Mec RTH 10, RTC 10 and RT 315);

• or by both two hydraulic rams and two suspended cables. These cables are used mainly to ensure guidance of the kelly as is the case with the more powerful models of Calweld and Hughes (CEZ 450).

-- *Drills for which the platform slides vertically* on a twin-guidance device joined to the frame of the crane as is the case with the McAlpine models (Fig. 7.6) (EF 90 and 190) or the Soil-Mec (RT 03).

This latter system moves the kelly by use of two hydraulic rams and can lift the rotary table enough to permit work with equipment that can reach a height of 5 m (9.84 ft).

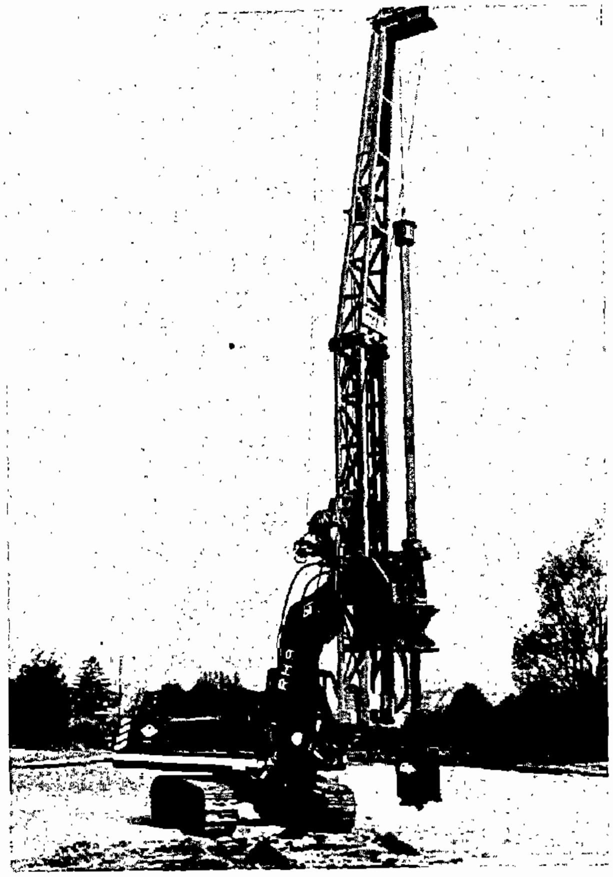


Fig. 7.3. -- BB 8 drilling machine on hydraulic shovel (Document from Salzgitter).

Fig. 7.4. -- Calweld drilling machine with fixed table operating simple auger.

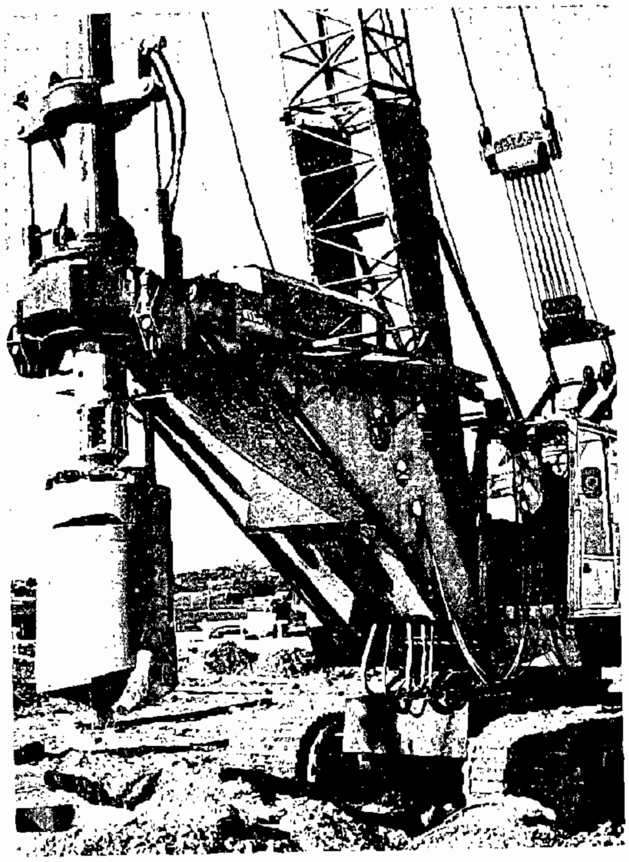


Fig. 7.5. -- RTH 10 drilling machine with fixed table operating drilling bucket (Document from Soil-Mec.).

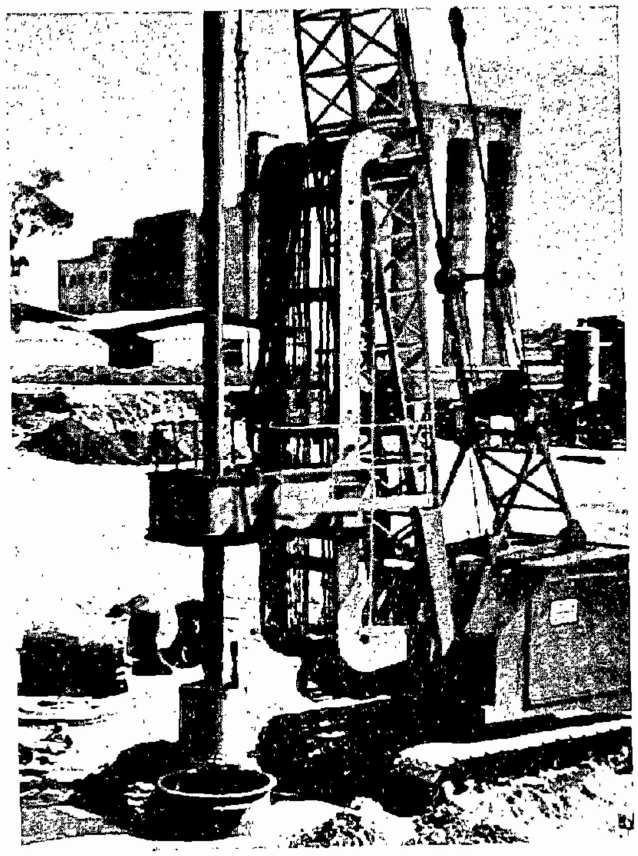


Fig. 7.6. -- Mc Alpine drilling machine with sliding table (Document from Mc Alpine).

Table II gives a partial list of various machines and their main characteristics. These characteristics are related to the power and type of transmission, to the boring depths and diameter of the tool, as well as to the lifting capacity of the crane.

As in Table I, and for similar reasons as for that table, the values of torque are given for information only as are the dimensions of the boring tools.

Of further importance:

- the use of cranes or shovels on tracks facilitates the movement of the drilling equipment on the job site;
- the presence of a derrick on a drilling machine or of some type of service crane aids in the placement of reinforcing, placement of a temporary casing, placement of heavy permanent casing, and in the handling of liners and tremies;
- such machines generally perform along with a slurry station (see TN No. 9), and the various equipment used is listed in TN No. 8.

TABLE I

DRILLING MACHINES FOR PILES MOUNTED ON TRUCK

BRAND	DISTRIBUTOR	MODEL	Weight of machine with kelly and boring tool (kg)	Power and model of motor used (HP)	Torque to rotary table (kgm)		Speeds of rotation (rpm)		
					brake torque	torque at min. speed	name	min.	max.
BSP England	BSP-France 58, rue Pottier, 78150 Le Chesnay Tél. : 954 81 40	1250 T	19,000	diesel Ford 2802 E, 160 HP to 2,500 rpm	7,450	5,530 to 10 rpm	3	35	125
CALWELD SMITH INTERNATIONAL INC. California, USA	Galinet-Paris Zone industrielle, R.N. 191 *bis*, BP 15, 78610 Le Perray-en-Yvelines Tél. : 484 86 01 484 86 10	100 B	5,000				4		
		150 B	5,400				4		
		175 B	6,100				4		
		200 B	6,350				4		
		250 B	9,000				5		
		1000 B	?						
GALINET	Galinet-Paris	MPB 20	?	diesel Deutz BF 6 L 913, 143 HP to 2,300 rpm	—	2,900 to 10 rpm	—	0	90
HUGHES TOOL COMPANY Machines Series Williams, Texas, USA	SEP Société d'études pétrolières 201, bureaux de la Colline-de-Saint-Cloud 92213 Saint-Cloud Tél. : 602 44 55	MF 60 T	13,000	diesel, 100 HP to 1,800 rpm	—	2,270 to 32 rpm	6	32	156
		LDH : 60 80 100	20,000	GMC 4.71 N diesel, 144 HP to 1,800 rpm	—	4,300 to 23 rpm	6	23	122 to 137
		LLDH : 80 100 110 T 120 T	28,000	diesel, 175 HP to 215 HP to 1,800 rpm	—	12,670 to 12 rpm	8	12	104
SOIL-MEC TREVISANI, Italy	LCM-Equipement Rue Ampère 95362 Pontoise Tél. : 030 38 38	RTA/10	16,800	diesel, GM 4/53, 130 HP to 2,800 rpm	9,250	9,250 to 5 rpm	2	0	103 to 140
WATSON Texas, USA	EHM 45, rue C.-Nodier BP 21, 93310 Le Pré-Saint-Gervais Tél. : 845 03 94	1000 TM	?	motor VB, 156 HP to 2,200 rpm	10,800	?	3	26	205

(0.453 kgm = 1 lb; .139 kgm = 1 ft-lb)

TABLE I (continued)

DRILLING MACHINES FOR PILES MOUNTED ON TRUCK

Transmission model for motor of the rotary table	Kelly model and dimensions	Height of mast apex/ above ground (m)	Characteristics for drilling: Max. depth of borehole (m)	Tool diameter (m) min.	Tool diameter (m) max.	Characteristics or examples of carrying truck for drilling machines
Hydrostatic transmission. Torque convertor. Reduction gearbox.	— quadruple telescopic 4x9.50 m	15.10	30.50	—	1.50	Truck 8×4 Tot. wt. 28.5 t with platform ≥ 8 m and motor with power > 170 HP
The rotary table here is a special type: the bucket or auger on top of a sliding stirrup in the kelly can completely pass to the inside of the rotary crown which is located at the back of the truck. The transmission unit of chains and pistons with reducer is an outfit patented by Calweld.	— double telescopic 2×7.50 m	10.00	14.30	0.30	0.70	
	— triple telescopic 3×7.50 m	(standard)	21.30	0.30	0.90	
	— double telescopic 2×9 m	12.00	17.40	0.30	0.90	
	— triple telescopic 3×9 m	(option)	26.00	0.30	1.20	
	— double telescopic 2×9 m — triple telescopic 3×9 m	13.00	17.40 26.00		1.20	
	— triple telescopic 2×13.5 m — triple telescopic 3×13.5 m	18.00	26.00 39.50		1.20	
See Table II, identical machine to drills MPC 20 and MPP 20.	— triple telescopic 3×8 m model 55 CH Calweld	12.50	20.00	0.40	1.20	Truck 6 × 4 Tot. wt. ≥ 25 t Example " Berliet" GBH 12"
The kinematic chain includes, after the friction clutches, a transmission with two selective engageable outflows to a gearbox with speeds of 3 or 4 ratios supplying a final range of 6 or 8 speeds to the rotary table. The Hughes rotary tables are Model DG 6000 for the LDH and RP 9000 for the LLDH.	— double telescopic 4" 1/2 and 3"	13.70	18.25	0.40	1.20	Truck 6 × 6 Tot. wt. 20 t Berliet, Mercedes, Man, Saviem, Mol-Faun...
	— double telescopic 6" and 4" 1/4	13.70 18.00 21.00	18.25 24.00 30.00	0.60	2.40	Truck Tot. wt 35 t, Mol-Faun, Berliet, Hendrickson, Willeme 8×6 and 8 × 4
	— double telescopic 7" and 5" 1/4	19.00 22.00 23.50 25.00	24.00 30.00 33.00 36.00	0.60	3.00	Truck Tot. wt 40 t, 8×4, 8×6, 8×8, Mol-Faun, Berliet, Hendrickson 1000 8 × 4
The rotary table is controlled through an Allison torque convertor and an Allison hydraulic gearbox.	— triple telescopic 3×9 m — quadruple telescopic 4x9 m	10.60 (telescopic)	23.90 32.00	0.40	1.60	Truck Tot. wt. 26 t
Engageable multidiscs. Torque convertor.	— simple 3" — double telescopic 4" - 3" 1/2	9.00	8.00 15.00	—	0.90	Truck 6 × 4

(0.0254 m = 1 in.; 0.91 t = 1 ton)

TABLE II

DRILLING MACHINES FOR PILES (ADAPTABLE TO A CRANE OR HYDRAULIC SHOVEL)

BRAND	DISTRIBUTOR	MODEL	Weight of machine without kelly nor drilling tool (kg)	Power and model of motor used (HP)	Torque to rotary table (kgm)		Speeds of rotation (rpm)		
					brake torque	torque at min. speed	name	min.	max.
BSP INTERNATIONAL FONDATIONS LIMITED (Great Britain)	BSP-France 58, rue Pottier 78150 Le Chesnay Tél. : 954 81 40	625 CA	3,098	diesel Ford 2704 E, 110 HP to 2,500 rpm	2,760	—	3		200
		1250 CA	5,770	diesel Ford 2704 E, 110 HP to 2,500 rpm	7,450	—	3		160
		2500 CA	8,480	two motors 2704 E of 110 HP	11,000	—	3		160
		80 CA	9,550	diesel 4 cylinders 148 HP	11,000	—	3		160
HUGHES WILLIAMS « Hugues Tool Company », Texas (USA)	SEP - Société d'études pétrolières 201, bureaux de la Colline-de-Saint-Cloud 92213 Saint-Cloud Tél. : 602 44 55	CEZ 300, model : 70 80 90 100 110	4,500	diesel GMC 4.71 N 65, 152 HP to 2,200 rpm	10,400 theoretical 6,000 actual	4,700 to 10 rpm	3	26	204
		CEZ 450, 70 80 90 100 120 150 180	about 6,000	diesel GMC 4.71 N 65, 152 HP to 2,200 rpm	17,000 theoretical 10,500 actual	6,500 to 10 rpm	3	18	142
		CLL DH Williams, model : 120 180	10,900	diesel 200 CV to 2,100 rpm (optional 240 HP)	22,900	9,500 to 15 rpm	2×3	15	85
CALWELD « Smith International Inc. » California, USA	Société Galinet-Paris 78610 Le Perray-en-Yvelines Tél. : 484 86 10	55 CH	3,400	diesel 100 HP to 2,500 rpm	—	3,500 service	3	22	149
		105 CH	9,500	diesel 106 HP to 2,100 rpm	—	7,000 service	3	16	133
		200 CH	20.000	2 diesels of 150 HP to 2,400 rpm	—	30,000 service	3	5	45

(0.453 kg = 1 lb; 1.488 kgm = 1 ft-lb)

TABLE II (continued)

DRILLING MACHINES FOR PILES (ADAPTABLE TO A CRANE OR HYDRAULIC SHOVEL)

Transmission Model	Kelly model and dimensions	Max. depth of borehole (m)	Tool diameter (m)		Crane			
			min.	max.	Min. required capacity (t)	Corresponding range (m)	Required derrick height (m) for max. kelly	Min. force on simple blade (t)
Twin Disc Convertor, reducer with gears.	Triple telescopic 3 × 9 m 3 × 15 m	25.00 45.00	—	0.90	5.6	4.60	14.00 20.00	4.6
Same as above.	Triple telescopic 3 × 12 m Triple telescopic 4 × 25 m	32.00 95.00	—	1.50	10	4.60	18.00 21.00	8
Two convertors, Twin Disc. Two reducers, two pinions on crown.	Triple telescopic 3 × 16.50 m Triple telescopic 3 × 25 m	44.00 70.00	—	2.00	17	6.10	21.00 30.00	13
Allison convertors. reducer with gears.	Quadruple telescopic 4 × 12.40 m Quintuple telescopic 5 × 25 m	44.00 120.00	—	2.00	17	6.10	18.00 30.00	13
Torque convertor with hydraulic torquer with three speeds. Hughes Rotary DG 6000 to gears with double reduction.	Double telescopic 6"×4"1/4 70 80 90 100 110	21.50 24.50 27.50 30.50 33.50	0.60	2.40	10 10.2 10.5 10.7 11	7.60	20.00 21.50 23.00 27.50 30.50	5 5.2 5.5 5.7 6
Same as CEZ 450, with entry reducer. Rotary table Hughes RP 11000 with conical pinions, spiral teeth.	Double telescopic 7" × 5"1/4 70 80 90 100 120 Triple telescopic 10"7/8 × 7" × 5"1/4 150 180	21.50 24.50 27.50 30.50 36.50 46.00 55.00	0.60	4.50	12.7 13.2 13.6 14 15 16.3 18.6	7.60	23.00 24.40 26.00 27.50 30.50 27.50 33.50	6.2 6.5 7 7.7 8.2 8.6 10
Self-adjusting, speed gearbox. Rotary table model CD 11000.	Double telescopic ⟶ (120) 10"7/8 × 7" Triple telescopic ⟶ (180) 10"7/7 × 7" × 5"1/4	36.50 55.00	0.60	4.50	22	6.00	30.50 33.50	12
Funk convertor. Calweld rotary table.	— simple, 17 m — double telescopic — triple telescopic	13.00 30.00 45.00	0.30	1.20	7	2.70	⩾ 24	6.5
Allison convertor, Oil-bath, Calweld rotary table	— simple, 21 m — double telescopic — triple telescopic	17.00 38.00	0.40	2.00	9to10	4.50	⩾ 31	13
Two Allison convertors, Calweld table.	On request, double kelly up to 80 m of depth.		0.80	3.00				

(0.0254 m = 1 in.; 0.91 t = 1 ton)

TABLE II (continued)

DRILLING MACHINES FOR PILES (ADAPTABLE TO A CRANE OR HYDRAULIC SHOVEL)

BRAND	DISTRIBUTOR	MODEL	Weight of machine without kelly or drilling tool (kg)	Power and model of motor used (HP)	Torque to rotary table (kgm)		Speeds of rotation (rpm)		
					brake torque	torque at min. speed	name	min.	max.
Mc Alpine Equipement (Great Britain)	SPEC Rue Guy-Moquet BP 101 95102 Argenteuil Tél. : 982 09 33	EF 90	8,200	diesel Ford, 90 HP to 2,500 rpm	max. torque of 0 to 15 rpm	3,500 to 15 rpm	4	15	62
		EF 190	10,700	diesel Rolls Royce, 190 HP to 1,800 rpm	max. torque of 0 to 16 rpm	7,750 to 16 rpm	4	23	91
Salzgitter Maschinen (Federal Republic of Germany)	Salzgitter 52, rue de Londres 75008 Paris Tél. : 292 26 97	BB 6	6,800	diesel Deutz 66 HP of the crane RH 6 O and K	—	1,480 to 12.5 rpm		12.5	31
		BB 8	13,000	diesel Deutz 100 HP of the crane RH 9 O and K	—	3,100 to 8 rpm		8	19
		BB 10	17,500	diesel Deutz 180 HP of the crane RH 20 O and K	—	5,000 to 7.5 rpm		7.5	14.4
Soil-Mec Trevisani Cesena (Italy)	LCM Equipement Zone industrielle Rue Ampère 95362 Pontoise Tél. : 030 38 38	RTO/3	2,500	60 HP diesel motor	3,100	3,100 5 rpm	2	0	40
		RTH/10	3,500	hydraulic motor Volvo FIO.C 78 giving 86 HP to work	11,000	9,800 to 5 rpm	2	0	75
		RTC/10	4,200	diesel GM 3/53, 97 HP to 2,800 rpm.	11,000	9,800 to 5 rpm	2	4.5	71
		RT 3/S	7,100	diesel GM 4/71 N, 175 HP	21,000	17,500 to 5 rpm	3	0	130
Domine 86530 Maintre-Chatelle-rault Tél. 21 23 04 (France)	Domine ou Société Bourlier Rue du Général-Leclerc 91420 Morangis Tél. : 909 18 30	TG 2000 Ground Explorer	8,800	diesel Deutz F 6 L 912, 90 HP to 2,200 rpm	15,800	11,100	3	0	125

(0.453 kg = 1 lb; 1.488 kgm = 1 ft-lb)

TABLE II (continued)

DRILLING MACHINES FOR PILES (ADAPTABLE TO A CRANE OR HYDRAULIC SHOVEL)

Transmission model	Kelly model and dimensions	Max. depth of borehole (m)	Tool diameter (m)		Crane			
			min.	max.	Min. required capacity (t)	Corresponding range (m)	Required derrick height (m) for max. kelly	Min. force on simple blade (t)
Hydrostatic transmission. One master hydraulic system supplies four motors with radial pistons, in star shape. Supplying of 1,2,3 or 4 motors gives 4 torques and 4 different speeds.	Double telescopic (circular pipe). Triple telescopic (circular pipe).	27.40 39.60	—	1.20	18	7.00	≥ 21.50	8.5
			—	2.10	20	7.00	≥ 24.40	11
One hydraulic pump with variable output supplies the hydraulic motor for which the power is adapted to the machine model. The hydraulic motor supplies directly the rotary table, along with the hydraulic winch.	Triple telescopic.	20.00	—	0.80	Orenstein cranes and Koppel RH 6, RH 9 and RH 20 used on models BB 6, BB 8 and BB 10 may be replaced with any other equivalent model, whether on a shovel with cable and supplementary hydraulic group or a hydraulic shovel.			
	Triple telescopic.	25.00	—	1.00				
	Triple telescopic.	30.00	—	1.50				
Rotary Group includes four motors with radial pistons.	Quadruple, telescopic kelly. Quadruple, special kelly	28.20 42.00	0.45	1.10	10	4.00	≥ 12.20	
Rotary group is controlled throu a compressor with Allison torque and an Allison, hydraulic gearbox. Reducer with gears.	Simple kelly to quadruple, telescopic kelly. Special quadruple kelly.	43.00 54.00	0.35	1.60	25	5.00	≥ 18.30	
	Same as RTH/10.		0.35	1.60	«	«		
	Quadruple, telescopic kelly.	42.00	0.45	2.20	11.5	6.10	≥ 18.30	
Convertor with Allison torque and "Power-Shift" gearbox.	— Double, telescopic kelly 5"1/4 —-Triple telescopic 4"1/4 — Quadruple telescopic 5"1/4.	21.00 48.00 64.00	—	2.00	16to 18	5.60	≥ 17 21.00 23.00	8 (crane GT 100 or 150)

(0.0254 m = 1 in.; 0.91 t = 1 ton)

TABLE II (continued)

DRILLING MACHINES FOR PILES (ADAPTABLE TO A CRANE OR HYDRAULIC SHOVEL)

BRAND	DISTRIBUTOR	MODEL	Weight of machine without kelly or drilling tool (kg)	Power and model of motor used (HP)	Torque to rotary table (kgm)		Speeds of rotation (rpm)		
					brake torque	torque at min. speed	name	min.	max.
GALINET-PARIS Zone industrielle, R.N. 191 *bis* BP n° 15 78610 Le Perray-en-Yvelines Tél. : 484 86 01 484 86 10		MPC 20	?	diesel Deutz BF 6 L 913, 143 HP to 2,300 rpm	—	2,900 to 10 rpm		0	90
		TR 18	50,000 (complete machine per contract)	diesel Deutz F 8 L 413, 175 HP to 2,000 rpm	—	3,000 to 35 rpm		0	40
		HF 25	11,000	diesel Deutz, 173 HP to 2,000 rpm	—	4,000 to 17 rpm		0	40
WATSON Texas (USA)	EHM (SA) 45, rue C.-Nodier 93310 Le Pré-Saint-Gervais, BP n° 21 Tél. : 845 03 94	5000 CA	4,600	diesel GMC, 241 HP	18,400		3		
CASAGRANDE (Italy)	BIP Diffusion (SA) Villebéon 77710 Lorrez-le-Bocage Tél. : 431 50 97	IRC 120	6,000	diesel motor 130 or 174 HP 2,000 rpm	12,000		3	8	98

(0.453 kg = 1 lb; 1.488 kgm = 1 ft-lb)

TABLE II (continued)

DRILLING MACHINES FOR PILES (ADAPTABLE TO A CRANE OR HYDRAULIC SHOVEL)

Transmission Model	Kelly model and dimensions	Max. depth of borehole (m)	Tool diameter (m)		Crane			
			min.	max.	Min. required capacity (t)	Corresponding range (m)	Required der-rick height (m) for max. kelly	Min. force for simple blade (t)
One pump with variable output supplies a SMV 25 hydraulic motor engaging directly the rotary table, model Calweld 55 CH.	— Triple telescopic 3 x 8 m.	20.00	0.40	1.20	The base carrying frame is a hydraulic shovel on tracks Poclain LC 28. A Poclain carrier LY 80 with twin wheels may also be used.			
Supply by pump, Poclain model HPOC 4 x 53. Hydraulic motor at bottom extremity of kelly (Galinet patent).	— Simple kelly. — Telescopic.	22.00 36.00	0.60	2.20	Carrying frame model Poclain GC 120. All hydraulic parts are standard components from Poclain.			
Supply by Poclain pump. Variable plus fixed output. Hydraulic motor (Galinet patent).	Standard version "simple kelly".	30.00	0.60	2.20	11	4.50		
Convertor with Allison torque. Watson 5 rotary table. Reducer with gears.	— simple — double telescopic	17.00 32.00	—	2.20	20 to 30	6.10	≥ 17	
Pump with variable ouput supplies 1 to 3 hydraulic motors. "Linde Guldner" pump.	Triple telescopic.	32.00	0.45	2.00	7	6.00 Link Belt 108 crane	21.00	7

(0.0254 m = 1 in.; 0.91 t = 1 ton)

TOOLS FOR ROTARY DRILLING

1. IN GENERAL

Drilling equipment that is rotated by machines with a rotary table are:

-- augers,

-- clean-out buckets or drilling buckets,

-- simple coring tools,

-- tools with extendable cutters (reamers) for use at the bottom of the borehole to enlarge the base (to cut a bell or underream), and

-- rotating drill-bits with teeth or blades, especially on machines that employ the principle of reverse circulation (see TN No. 10).

Each of these tools is manufactured or sold by companies that make the machines or their French representative, but it is not uncommon for contractors to make modifications or to manufacture their own equipment.

2. AUGERS

Figure 8.1 shows a number of profiles or types of augers. These are generally cylindrical in form and sometimes but rarely conical and are composed of one or two helices (flights). They are generally directed at their base by a guide tool (stinger) with teeth (stellite or tungsten-carbide) or of the fish-tail type. The helix has a series of teeth in its cutting edge which are often interchangeable (screwed on, fitted, or welded), or more simply one or two cutters are used when boring in loose soils. Also available are hinged augers that facilitate ejection of materials (Fig. 8.2).

3. DRILLING BUCKETS AND CLEAN-OUT BUCKETS

A bucket is a cylindrical drill with a cambered or flat base plate. This base plate, with or without a centralizing guide at the tip, creates an opening with a radius that depends on extendable cutters with interchangeable teeth.

By pressure and rotation, the teeth cut the soil that accumulates in the bucket. This is then raised and emptied by opening the hinged bottom plate (Fig. 8.3a and b).

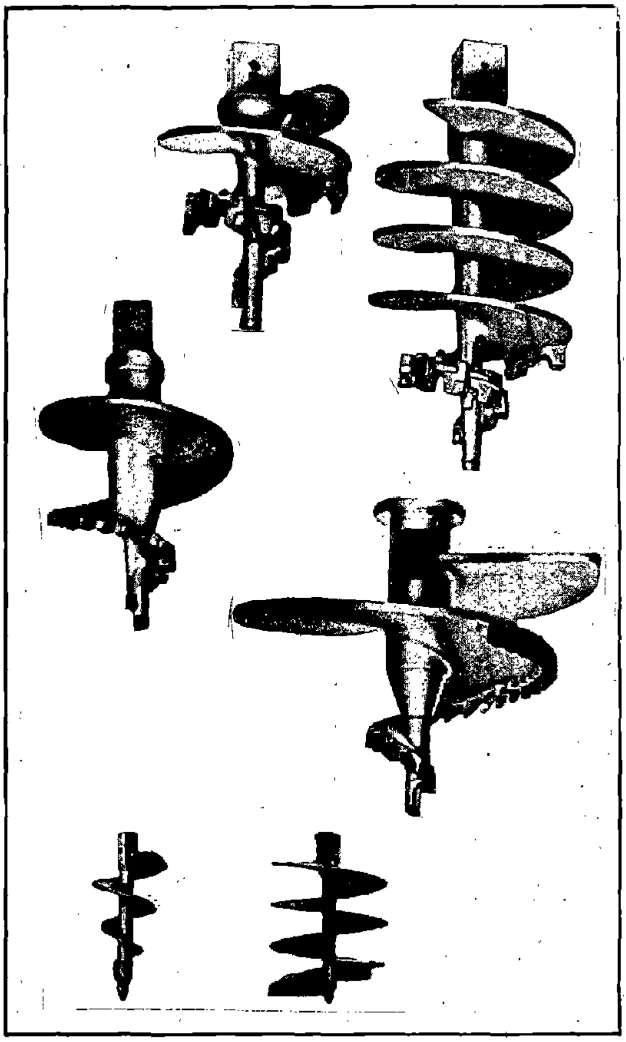


Fig. 8.1. -- Examples of simple augers.

Fig. 8.2. -- Type of articulated auger (Document from Salzgitter).

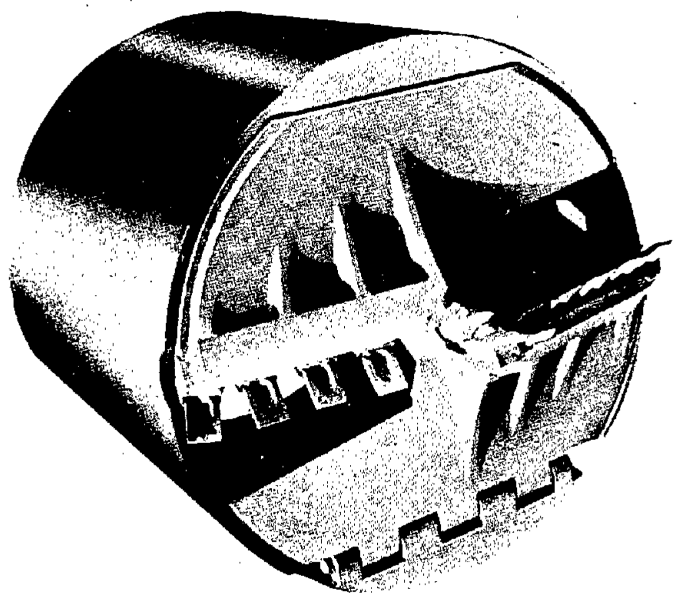


Fig. 8.3a. -- Bucket in closed position
(Document from HTC).

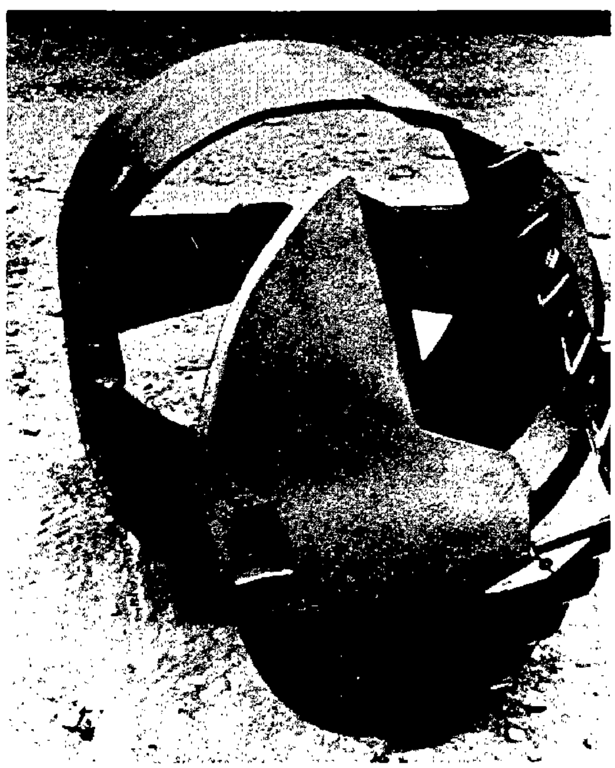


Fig. 8.3b. -- Bucket in open position
(Document from Massenza).

To empty the material that is extracted more easily and more rapidly, hinged buckets are sometimes used (Fig. 8.4), or more generally buckets with an automatic system for opening the bottom (Fig. 8.5). The opening device is attached to one side of the top of the bucket, and the releasing of the system is achieved by contact with the bottom of the rotary table when the tool is raised.

Finally, some manufacturers offer special buckets with flaps to close the opening in the bottom plate when excavating in wet or weak soils (Fig. 8.6). Also, buckets have been built with rotary drill bits at the bottom for boring hard rock (Fig. 8.7). These tools are, however, not often used and are advantageously replaced by reverse circulation and a rotating drill bit (see TN No. 11).

4. GRAB BUCKETS WITH EXTENDABLE ARMS

Such equipment is used for construction of bells at the base of the borehole. They are underreamers that can be used to enlarge the base of the pile by progressive drilling. The enlargement is in the form of a bell, which is where the angle-saxon name of "belling-bucket" is derived. The height of the bell is limited and is dependent on the design of a particular bucket.

As can be seen in Figs. 8.8 and 8.9, the tool includes one or two hinged cutters at the head or at the base and is rotated by the kelly. The cutters are fitted with teeth so as to facilitate the cutting of soil during rotation of the equipment.

It should be noted that this tool is rarely used for construction of bridges for the Ministry of Transport.

5. BOREHOLE ENLARGERS

These are generally fixed above a bucket and are composed of reaming cutters that are diametrically opposed which may or may not make use of stellite teeth. Each cutter is inclined so that the material that comes from the enlarging falls in the bucket. The borehole preliminarily constructed with a bucket or with an auger serves as a pilot hole.

Such tools have been used to construct boreholes of 3 m (9.84 ft) in diameter and above, as shown in Fig. 8.10.

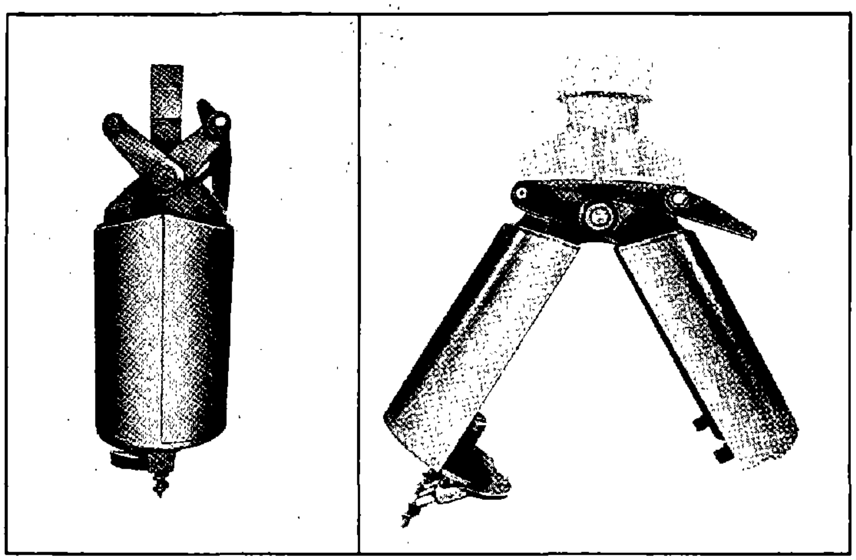


Fig. 8.4. -- Example of articulated bucket (Document from Salzgitter).

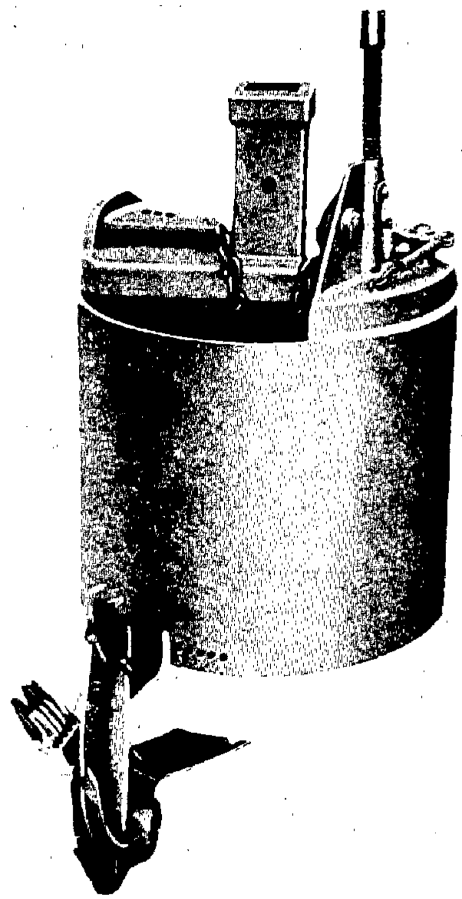


Fig. 8.5. -- Bucket with automatic opening system (Document from BSP).

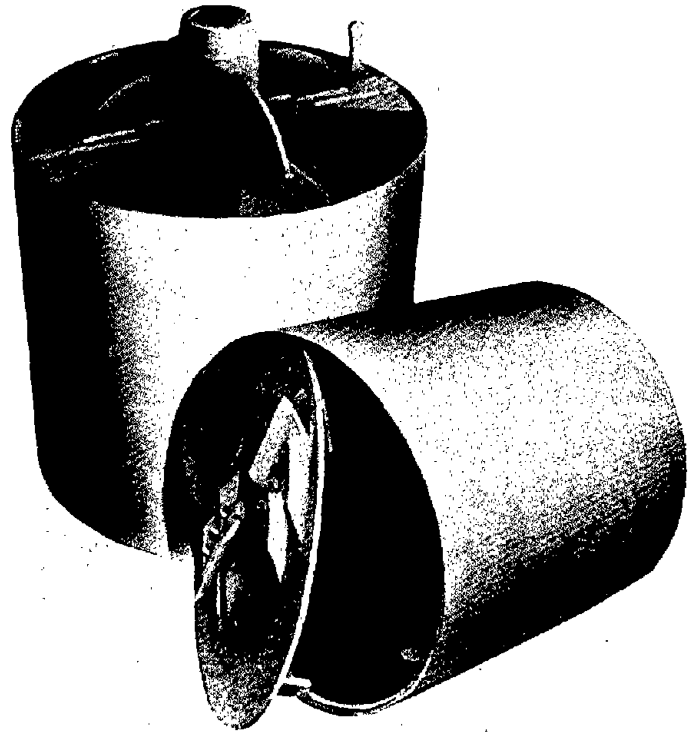


Fig. 8.6. -- Bucket with valves for cohesionless soils below water table (Document from HTC).

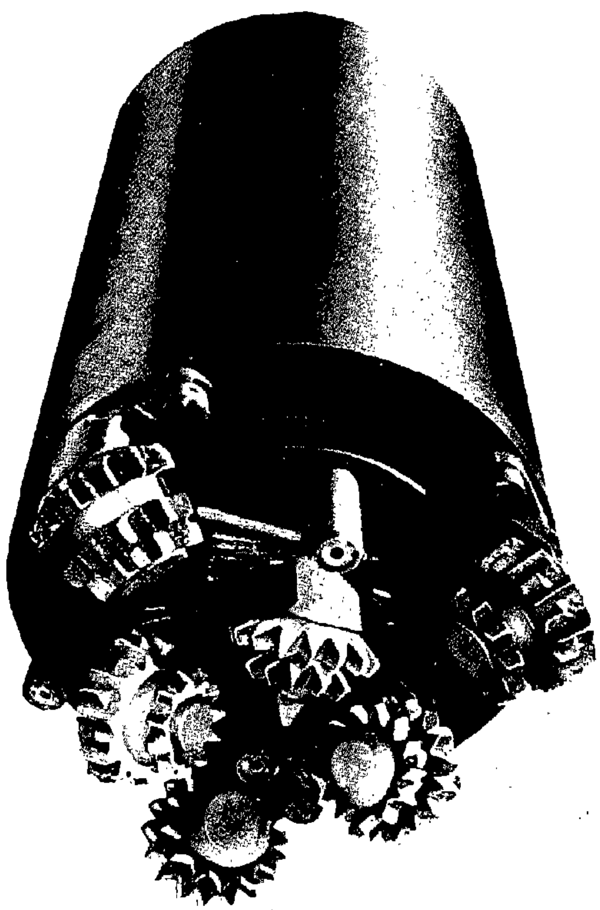


Fig. 8.7. -- Bucket with toothed-cutting wheels for hard rock (Document from HTC).

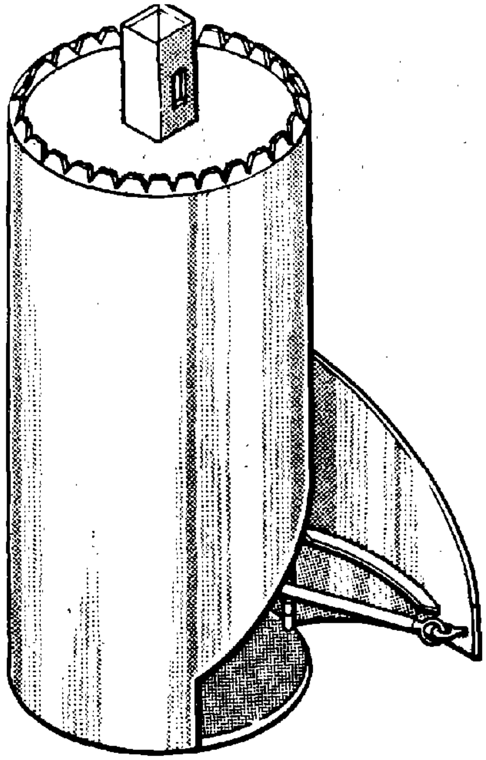


Fig. 8.8. -- Extendable bucket with door (Document from Watson).

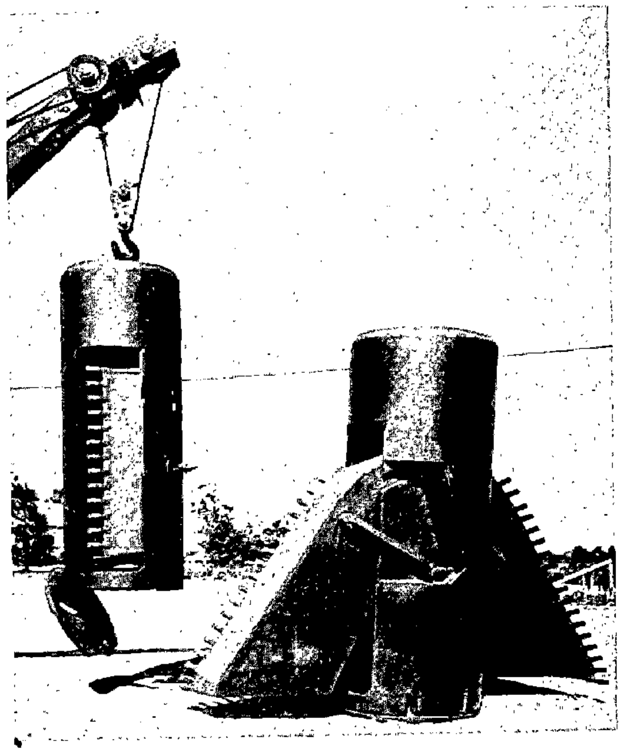


Fig. 8.9. -- Extendable bucket with two cutters (Document from Semafor).

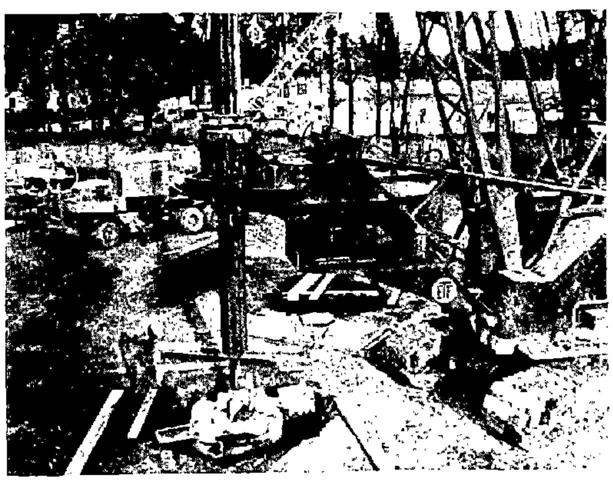


Fig. 8.10. -- Example of diameter enlarger to 3 m (Document from ETF).

6. CORING BUCKETS WITH RETAINERS OR SIMPLE CORERS

As may be seen in Figs. 8.11 and 8.12, the base of the bucket is fitted with teeth. The teeth are attached by welding directly to the bottom of the bucket (Fig. 8.11) or they are replaceable and fitted into holders that in turn are welded to the bottom of the bucket (Fig. 8.12).

Some corers such as the TEL.E.LECT have a cutting tool at the center of the bucket that permits easier splitting of the core by hammering when the core cannot be lifted (Fig. 8.13).

Some manufacturers provide, on demand, corers with a patented system that permits cutting and retaining of the core so that the core can be lifted from the bottom of the borehole (Fig. 8.14).

Despite everything, the lifting of cores of large diameter is not effective on the job site unless they have a diameter to length ratio of 1 to 5. Thus, the core must often be broken up at the bottom of the borehole with a drill bit.

Fig. 8.11. -- Core drills with stellite teeth.

Fig. 8.12. -- Core drill with changeable-teeth (Document from Semafor).

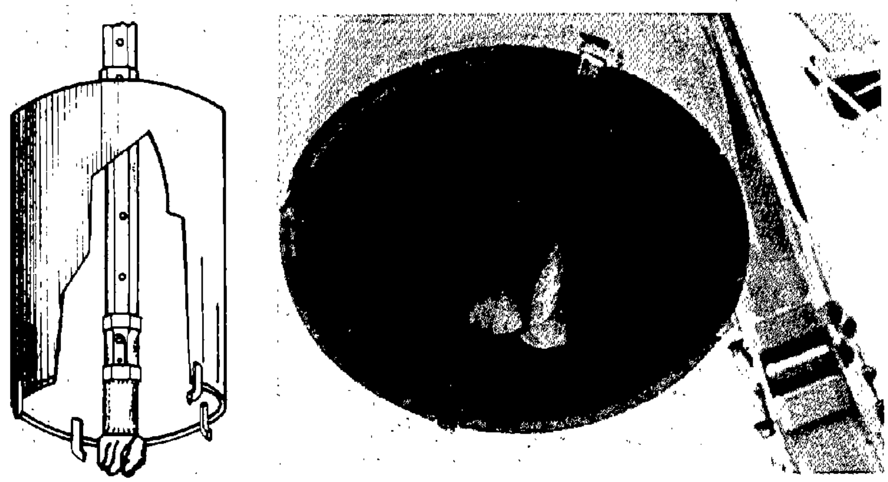


Fig. 8.13. -- TEL-E-LECT core drill with guide tool
(Document from SEP).

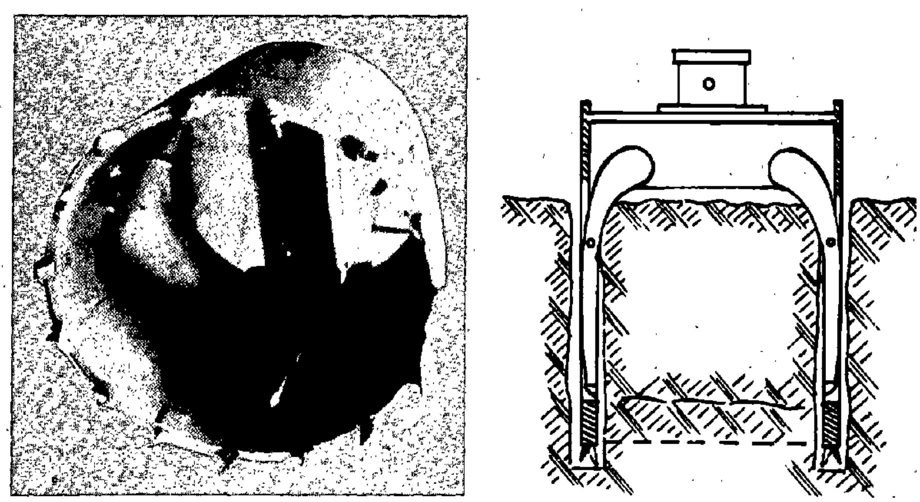


Fig. 8.14. -- Bourlier core drill with extractor
(Document from Salzgitter).

7. ROTATING DRILL BITS

a) Drill bits with arms

These usually have three or four arms with forged steel teeth attached by welds to the body of the drill bit. The drill bit is hollow in order to permit the lifting of sediments (Fig. 8.15a and b). Figure 8.16 shows a special drill bit called a "jumbo" that has a tool of multiple arms that may be used for the drilling of rock.

b) Drill bits with cones or roller bits (Toothed Wheels) (Fig. 8.17a and b)

These tools are derived from drill bits used in petroleum drilling (tricone or "rock-bit", also caled "roller-bit" when equipped with four rollers). Distinction is made between:

-- single-plate drill bits (Fig. 8.17a and b),

-- tiered or enlarged-plate drill bits.

As indicated in Fig. 8.18, these are composed of a tricone or a roller-bit as the cutter, mounted over several tiers and arranged in such a way to cut an excavation whose base is conical shaped. Each of these has four wheels on arms. The superposition of cutting positions permits attainment of numerous combinations of diameters.

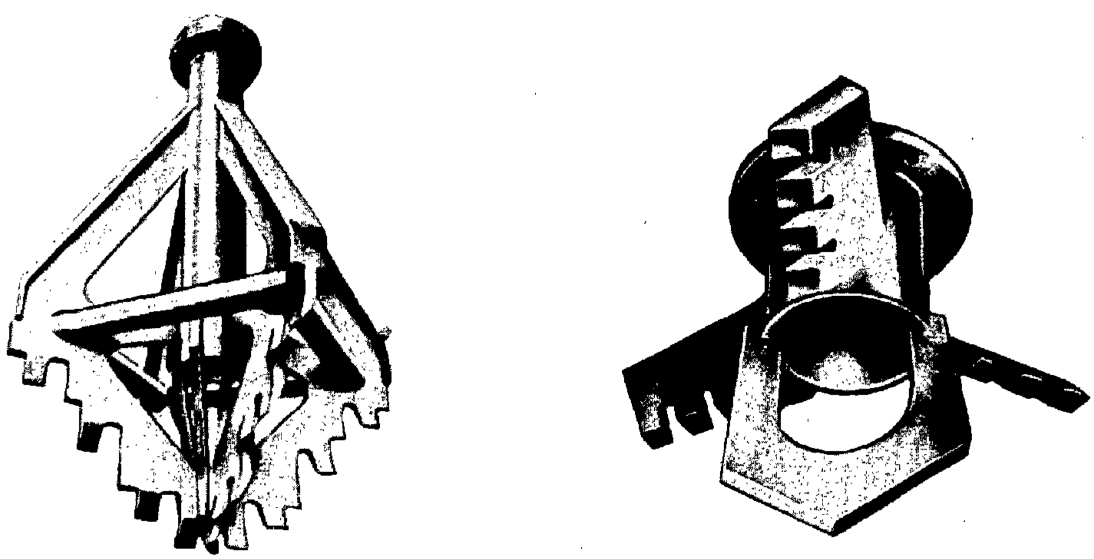


Fig. 8.15a. -- Rotary drills with cutters for loose soils (Document from Salzgitter).

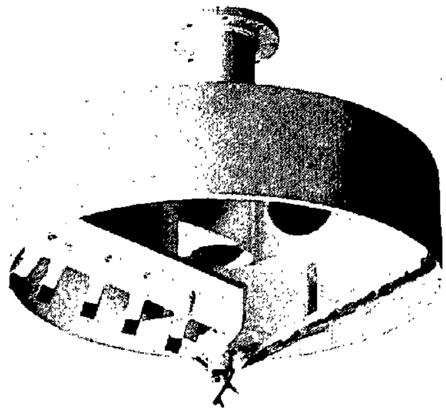


Fig. 8.15b. -- Another type of drill with cutter and crown guide, for loose soil (Document from Caldweld).

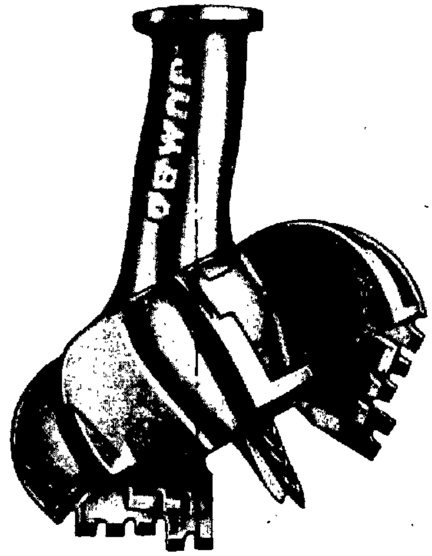


Fig. 8.16. -- Special drill with jumbo cutters (Document from Salzgitter).

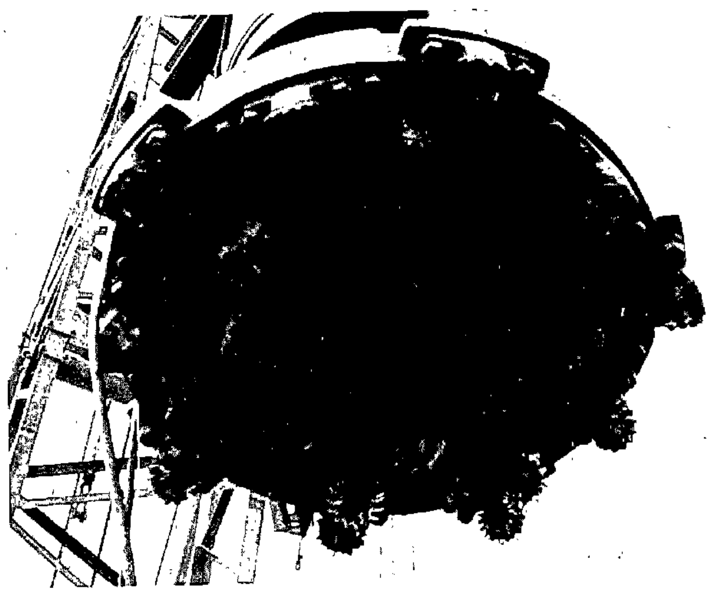


Fig. 8.17a. -- Single-table drill used on Hydrofond machine (Document from Soletanche).

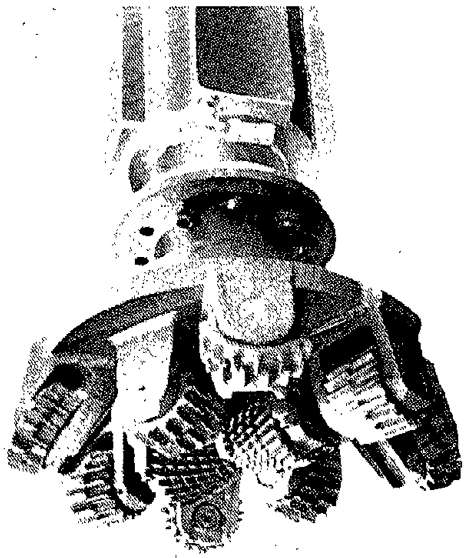
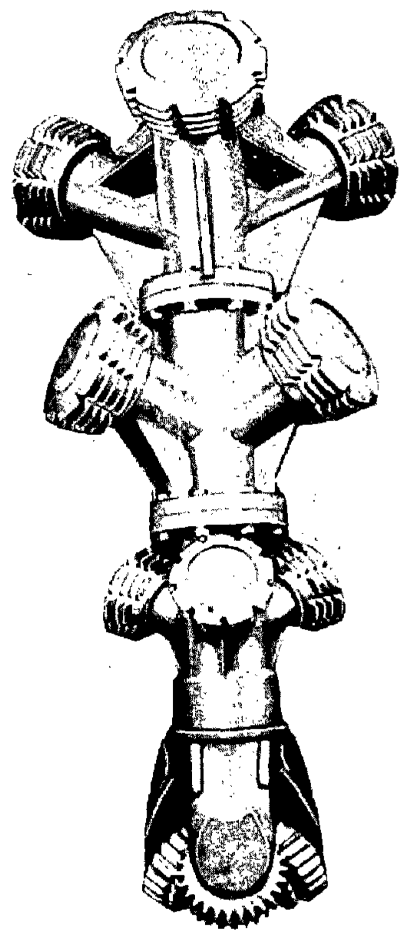


Fig. 8.17b. -- Another example of single-table drill (Document from Calweld).

Fig. 8.18. -- Rotary drill with tiered levels.



DRILLING SLURRY STATION

This Technical Note gives the classic composition of a slurry station (Fig. 9.1). Several makes and models of equipment are presented along with the main characteristics. Also presented are the various procedures and equipment used to test the slurry characteristics.

1. WHAT IS A SLURRY STATION?

A station always includes a manufacturing unit and a treatment unit (Fig. 9.2).

a) Manufacturing unit

Storage of powdered bentonite a is most often done in 50-kg (110.25-lb) sacks located under protection from bad weather (shed or tarpaulin) or even in silos.

The manufacture of the drilling slurry which consists of water and bentonite generally proportioned at 50 kg per cubic meter (3.12 lbs per ft^3) of water (5%) is achieved by one of the two following procedures:

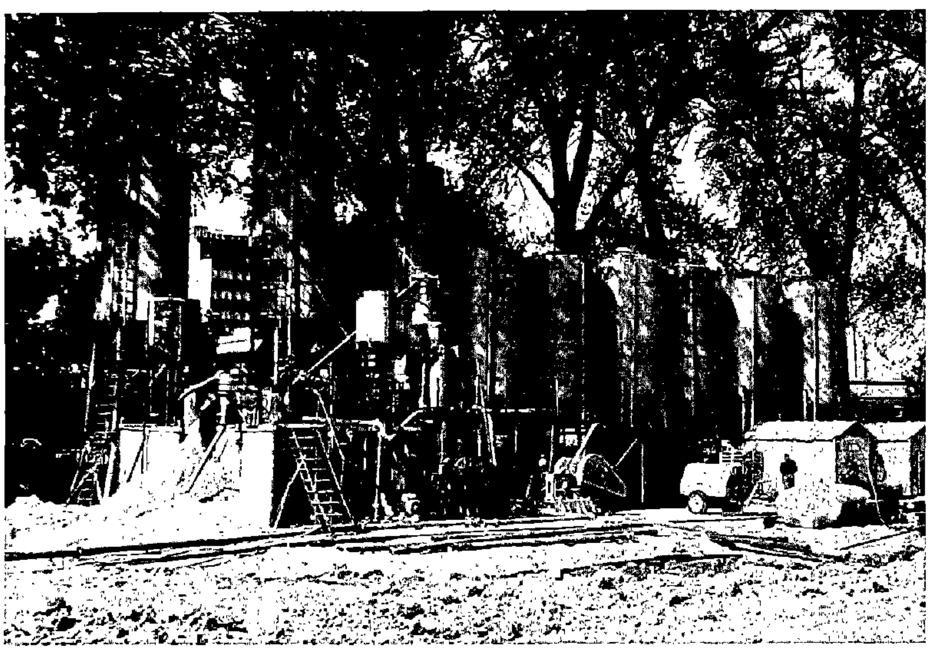


Fig. 9.1. -- General view of slurry station.

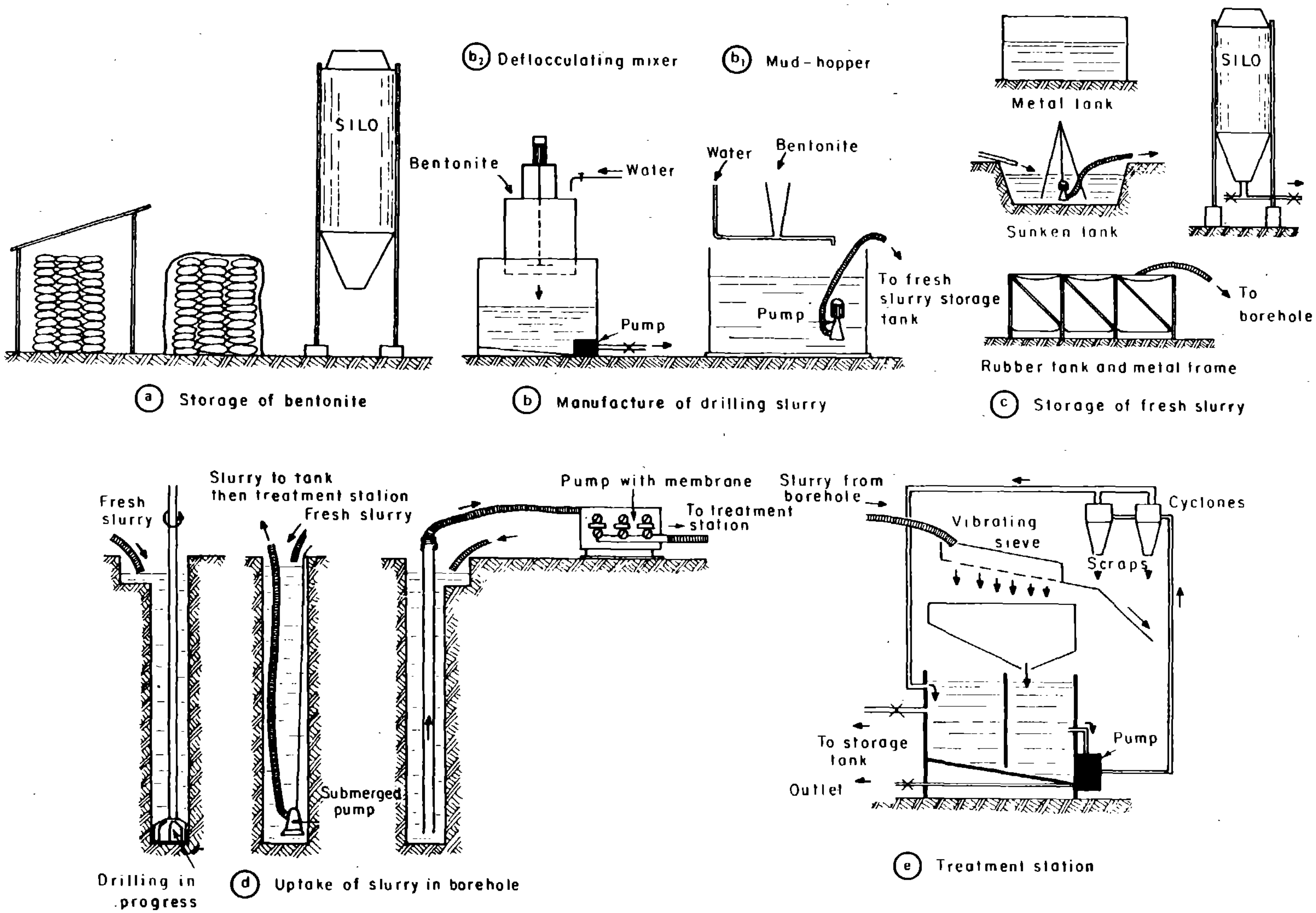


Fig. 9.2. -- Diagram of unit for fabrication and treatment of bentonitic slurry.

MUD HOPPER SYSTEM (b1)

The powdered bentonite is emptied into a funnel, through the bottom of which water is foced to flow at right angles to the bentonite flow. The water is concentrated into a jet that becomes loaded with bentonite and the mixture is carried into a trough where a return pump completes the slurry manufacturing.

MIXER-TYPE BLENDING SYSTEM (b2)

The mixer, also called an "Auto-Deflocculating Mixer", is a vertical mixer: an electric motor, with or without speed variation, drives a vertical shaft that has a turbine with paddles attached to it. A high circumferential speed (5 m/s to 80 m/s - 16.40 ft/s to 262 ft/s on some models) permits perfect homogenization of the drilling slurry.

Storage of the new slurry (c) is either in a metal-tank silo or in metal-framed, flexible vats, or in simple ponds created by excavation (with or without wood lining). In order to maintain the viscosity of the slurry, mixing pumps or air hoses may be used.

b) **Treatment unit**

The stored mud is carried to the borehole by gravity or by pumping. As the excavation is advanced, the slurry becomes loaded with sediments and should be sent to the treatment station while maintaining the level of the slurry in the borehole by addition of new or regenerated slurry (d) .

The treatment unit (e) includes:

-- *One or several vibrating screens*

• The slurry passes through the screens and falls into a primary tank and is then taken up by a pump and sent to a hydrocyclone.

• Residue or refuse from the screens is evacuated to a discharge.

-- *One or several centrifugal desanders (cyclones)*

• The cyclone eliminates fines larger than 0.1 mm (3.9×10^{-3} in.) by centrifugal action (silt and clay sizes can remain in suspension). The refined slurry comes out of the cyclones and is either sent directly to the storage tank or is sent to a secondary tank from which the desanded slurry can then be sent to the tank of refined slurry. At the lower portion of the cyclone unit there is a line from the primary tank so as to be able to condition the treated slurry if desirable.

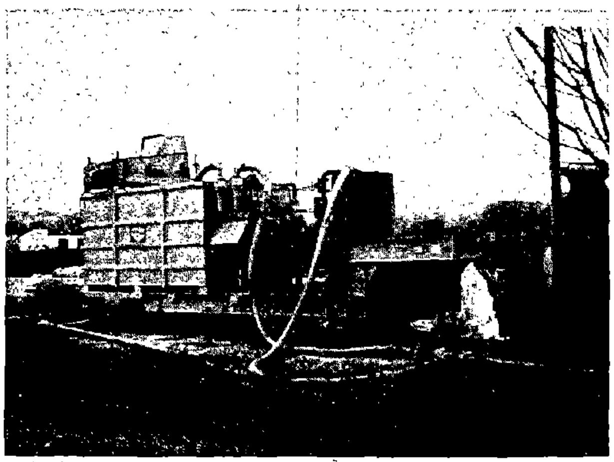


Fig. 9.3. -- Mobile treatment unit.

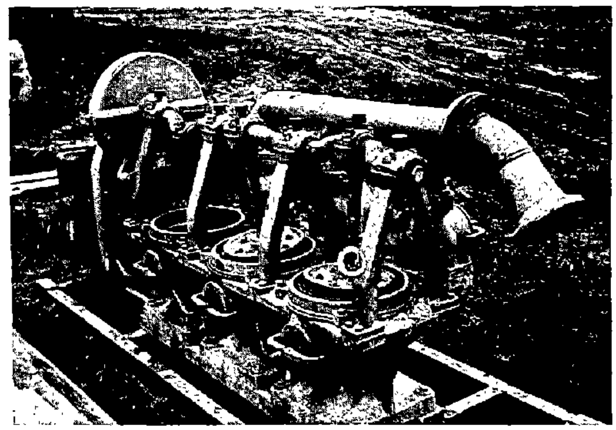


Fig. 9.4. -- Example of slurry pump.

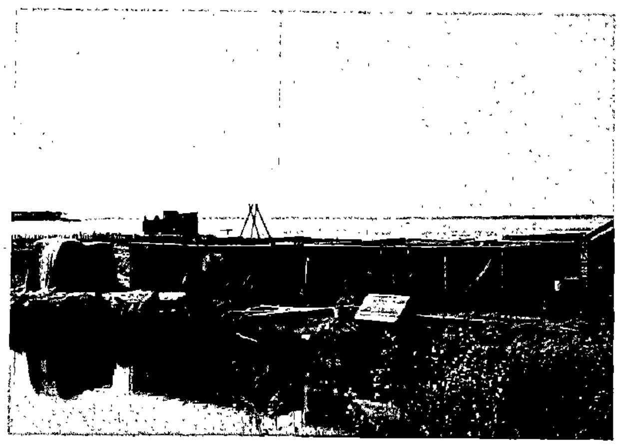


Fig. 9.5. -- Storage of fresh slurry in flexible tanks.

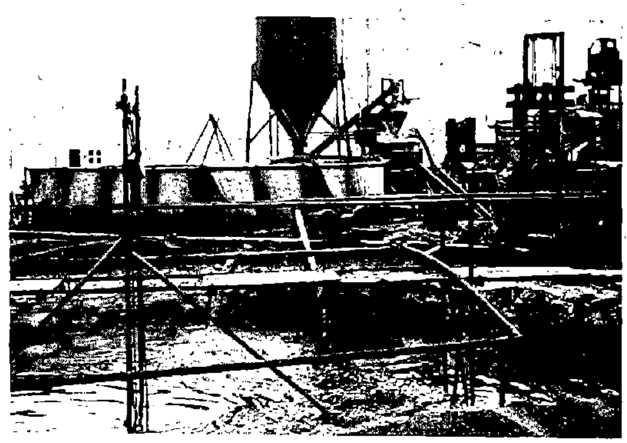


Fig. 9.6. -- Return tank made by excavation.

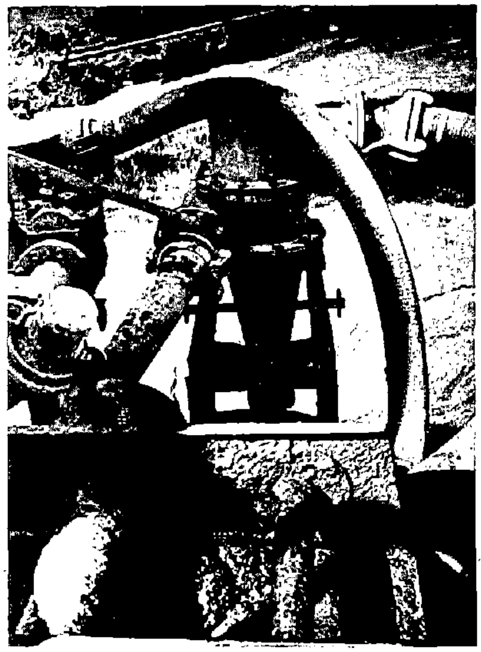


Fig. 9.7. -- Centrifugal desander with vibrating sieve in foreground

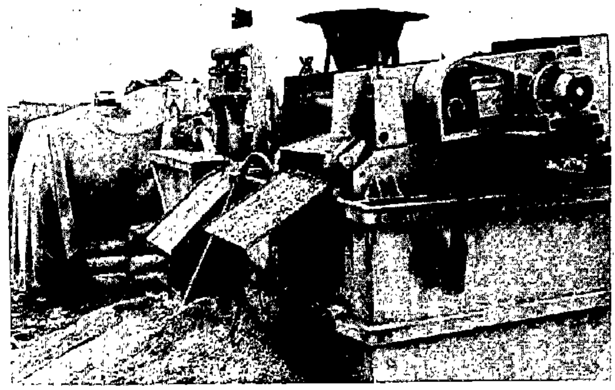


Fig. 9.8. -- Vibrating screen in mud-hopper with mixing and treatment unit.

• Current treatment units using a 1 m^2 (10.76 ft^2) vibrating screen (2 mm - 0.08 in.) mesh and a hydrocyclone permit treatment of slurry at a rate of from 30 to 50 m^3/h (1059.3 to 1765.5 ft^3/h).

2. SOME BRANDS AND MODELS OF EQUIPMENT

a) Auto-deflocculating mixers

-- Dosapro distributed by SEM.
Types EPM 0 to EPM VII 3 HP to 50 HP. Current model EPM I with patented Raynero turbine of 8 HP, Ø turbine 18 to 23 cm (7.0 to 9.0 in.).

-- Rayneri, Montreuil
Dynabloc type to 1,500 rpm, of 0.5 to 23 HP according to the model.

-- SMA of Val-Notre-Dame.

b) Pumps (recovery and distribution pumps)

1. *Centrifugal Pumps*

• GASOLINE OR DIESEL MOTORS

	discharge in m^3/hr (ft^3/hr)
-- Bernard	9 to 70 (318 to 2,472)
-- Major Crampton	7 to 100 (247 to 3,351)
-- Deloule	11 to 60 (388 to 2,119)
-- DIA, distributed by CPI, series SZ	42 to 240 (1,483 to 8,474)
-- Geho, distrubed by MIM, series VP	30 to 320 (1,059 to 11,299)
-- Rensome, series D	12 to 306 (424 to 10,805)
-- Richier, series P	16 to 306 (565 to 10,805)

	discharge in m³/hr (ft³/hr)
• ELECTRIC MOTORS	
-- Major Crampton	3 to 92 (106 to 3,249)
-- Jeumont Schneider	2 to 20 (71 to 706)
-- Pelger, distributed by Crampton	36 to 216 (1,271 to 7,627)
-- Ransome	22 to 306 (777 to 10,805)
-- Richier	10 to 306 (353 to 10,805)
-- Sihi	6 to 130 (212 to 4,590)
-- Salmson, distributed by LMT	40 to 50 (1,412 to 1,766)

2. *Diaphragm pumps (with diesel motor)*

-- DIA (CPI), current models:	
• SH 100.1	35 (1,236)
• SH 100.2	60 (2,119)
• SH 120.3	80 (2825)
-- Domine (Atlas Copco), model PDM 3	70 (2,472)

3. *Electric, submersible pumps (single or triple phase)*

	discharge in m³/hr (ft³/hr)
-- ABS, distributed by SIHI, series AFP	58 to 700 (2,048 to 24,717)
-- Deloule, series Aqueval	15 to 1,100 (530 to 38,841)
-- DIA, distributed by CPI, series T	14 to 84 (494 to 2,966)
-- Flygt-France	18 to 1,800 (636 to 63,558)
-- Guinard, series EVI	22 to 160 (777 to 5,560)
-- Grindex	40 to 360 (1,412 to 12,712)
-- Richier	10 to 228 (353 to 8,051)
-- Robot, distributed by Sobatelec	30 to 216 (1,059 to 7,627)
-- Weda, distributed by Ingersoll-Rand, series L	42 to 330 (1,483 to 11,652)

4. *Piston pumps*

-- Bonne Esperance: pumps with one or two double-acting pistons with motors from 4 HP to 140 HP that may attain outputs of 1,500 l/min (396.3 gal/min) under a pressure of 30 bars (435 psi) (output of 347 l/min - 91.7 gal/min with p = 130 bars - 1,885 psi). With the series BE 3 in. to 7 in. (cylinder diameter), the weight of the pump varies from 360 kg to 3,500 kg (793.8 lbs to 7717.5 lbs).

-- Domine Johnson, distributed by Atlas Copco: Trido pumps with 3, single-acting pistons with 8 HP or 16 HP motors, with service pressures of 30 bars (435 psi) and 74 l/min or 130 l/min outputs- 19.6 gal/min or 34.3 gal/min (weight of the pumps 250 kg or 520 kg - 551.25 lbs or 1146.6 lbs).

c) **Vibrating screens or sieves**

-- Babbitless:

• Open, vibrating screen models.

C 19 A, 0.43 m x 1.28 m (1.41 ft x 4.20 ft)
C 27 A, 0.66 m x 1.42 m (2.16 ft x 4.66 ft)
C 32 A, 0.80 m x 1.82 m (2.62 ft x 5.97 ft)

-- Chauvin Industries:

• Model ROL HC 9 with a rectangular screen 0.60 m x 1.50 m (1.97 ft x 4.92 ft), with meshes of 0.6 x 2.5 mm (0.02 x 0.1 in.) and output of 20 to 25 m^3/h (706 to 883 ft^3/hr).

• Model POL HC 18, 1.20 m x 1.50 m (3.94 ft x 4.92 ft), output of 40 to 50 m^3/h (1,412 to 1,766 ft^3/hr) for meshes of 0.6 x 2.5 mm (0.02 x 0.1 in.).

-- Strasbourg Forgers - Comessa:

• Vibrating, monoplanar sieve models with variable inclination:

0.50 m x 1.50 m (1.64 ft x 4.92 ft)
0.75 m x 1.50 m (2.46 ft x 4.92 ft)
1.00 m x 1.50 m (3.28 ft x 4.92 ft)
1.25 m x 1.50 m (4.10 ft x 4.92 ft)

d) **"Cyclone" or centrifugal desander**

-- Alsthom - Sogreah:

• Model TT 280 TDF.TST permitting treatment of boring output up to 50 m^3/h (1,766 ft^3/hr).

• Other models with outputs > 50 m^3/h (1,766 ft^3/hr).

-- Linatex:

• Model SE 12 for treatment station with maximum output of 50 m^3/h (1,766 ft^3/hr).

• Other models that can treat up to 500 m^3/h (17,655 ft^3/hr).

It should be noted that these two companies manufacture a complete purification group (or treatment unit) (maximum output 50 m^3/h - 1,766 ft^3/hr including: vibrating sieve, tank, return pump and cyclone.

3. DETERMINATION OF DRILLING SLURRY CHARACTERISTICS

a) Measure of density

The tool used is the "Baroid" balance (Fig. 9.9), a true Roman balance including a cylindrical cup with constant volume and a beam directly graduated for density. After filling the cup with mud, the beam is balanced by moving the slide and the density is directly read.

b) Viscosity of the mud

The Marsh funnel (Fig. 9.10) allows expression of the viscosity by the time required for refilling of a receptacle of a given capacity, that is to say 946 cm^3 (1/4 of U.S. gallon).

This funnel is composed of a sieve of 2 mm - 0.08 in. (No. 10 ASTM) into which the mud is poured up to a mark (around 1.5 liters - 0.40 gal), the accurately calibrated nozzle of 6/32 in., (about 4.75 mm) being preliminarily sealed. A note is made of the time for filling of the receptacle below. *The number of seconds elapsed expresses the Marsh viscosity.*

c) Sand content

All of the material that does not pass through a sieve of 80 microns (No. 200 ASTM) is called "sand." Sand content is expressed by volume in relation to overall volume of the mud sample.

The classic job site tool is the "elutrimeter" (barette with graduated, conical base). For example, samples of 100 cm^3 (0.03 gal) of mud are taken and passed through a sieve of 80 microns. The sand retained on th sieve is recovered, placed in the burette and washed by a light water flow until the water contained in the burette becomes perfectly clear (Fig. 9.11).

The volume of sand remaining at the bottom is measured in cubic centimeters by direct reading and from this the sand content in percent is obtained.

Attention - Some burettes are directly graduated in percent. There is also a line mark giving the volume of mud to be put in the burette.

d) Filtrate (free water) and cake thickness

The apparatus universally used is the Baroid filter-press (Fig. 9.12). This is composed of a mud reservoir (a) installed in a frame (b) , a filration device (c) , a system for collection and measurement of the quantity of free water and a pressure source (d) . A graduated beaker (e) recovers the filtrate.

Fig. 9.9. -- Measure of density with Baroid balance.

Fig. 9.10. -- Measure of viscosity with Marsh funnel.

Fig. 9.11. -- Measure of sand content with elutriometer.

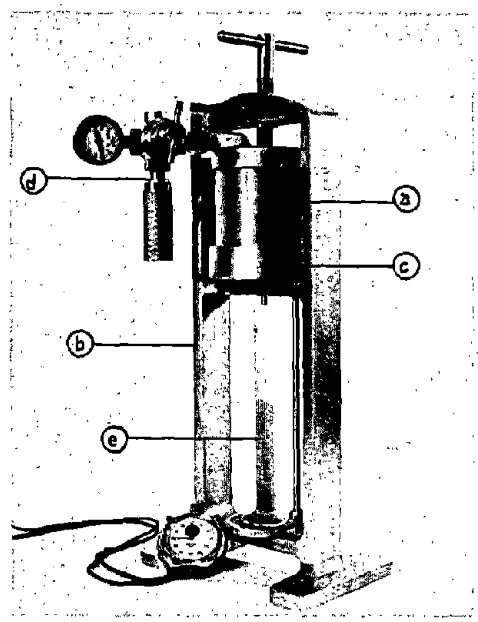


Fig. 9.12. -- Baroid filter-press. Measure of free water.

The test is performed during 30 min under a contant CO_2 pressure of 7 bars (100 psi).

The free water is recorded in cubic centimeters. This is the quantity of filtrate recovered in 30 min. The test may be limited to 7.5 min, in which case the quantity of free water recovered is considered as the halfway mark of that measured at 30 min.

The thickness of the cake is measured with a scale to the nearest millimeter, after dismantling of the device (Fig. 9.13) and by elimination of the superficial gel by washing under a water jet.

e) **Measure of pH**

Color-coded papers, impregnated with chemical solutions, are dipped in the mud to be tested. The color obtained is then compared with the colors of a reference table graduated in pH units.

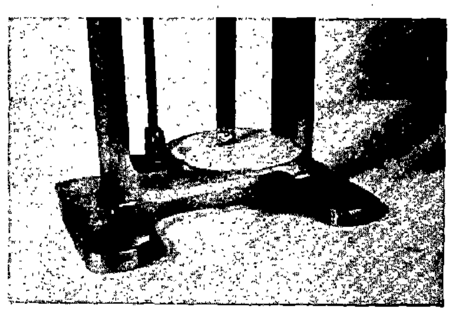


Fig. 9.13. -- Measure of thickness of filter cake.

DRILLING TOOLS FOR BARRETTE PILES

The drilling methods used for construction of diaphragm walls have permitted shapes other than cylindrical when bored piles are involved. The construction methods generally make use of grab buckets of rectangular or oblong form and occasionally require the use of special tools such as the hydrodrill by the Soletanche Company.

The grab buckets consist of two scoops that when closed form a pocket. The cutting edges of the buckets or scoops, either reinforced or not, are of rectangular or semi-circular shape. The scoops are equipped with interchangeable teeth (Fig. 10.1) or teeth that are carved into the cutting edges (Fig. 10.2) and these teeth are distributed in such a way so that they fit into each other to form a closure (Fig. 10.3).

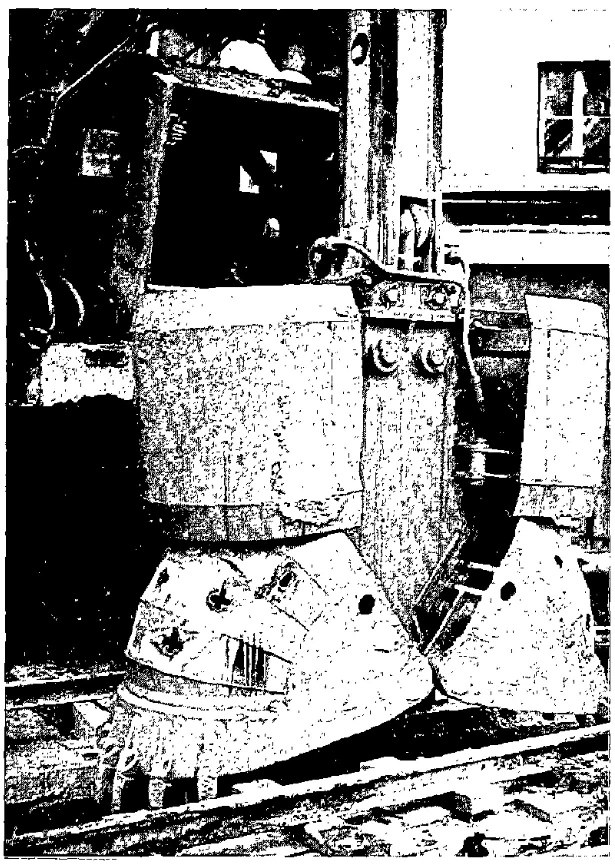


Fig. 10.1. -- Grab bucket (Document from Galinet).

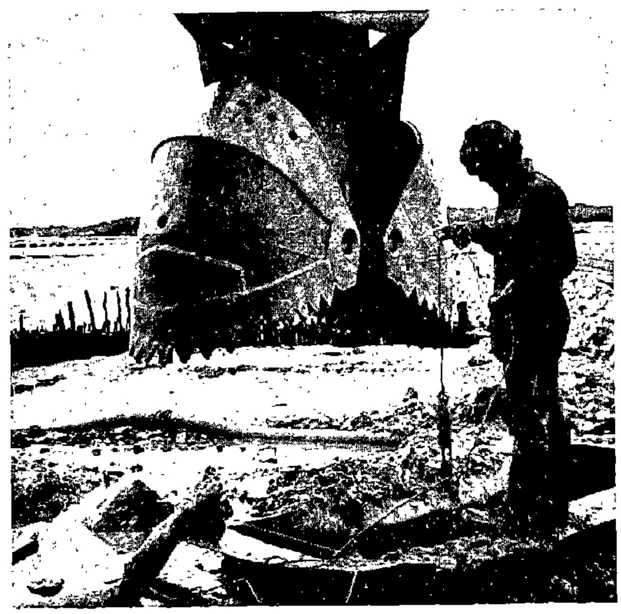
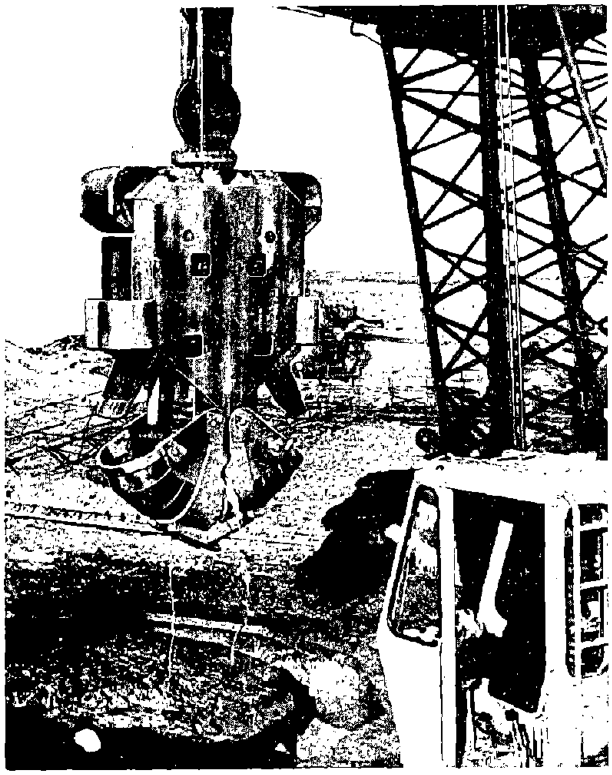


Fig. 10.2. -- Grab bucket with teeth carved into cutting edge.



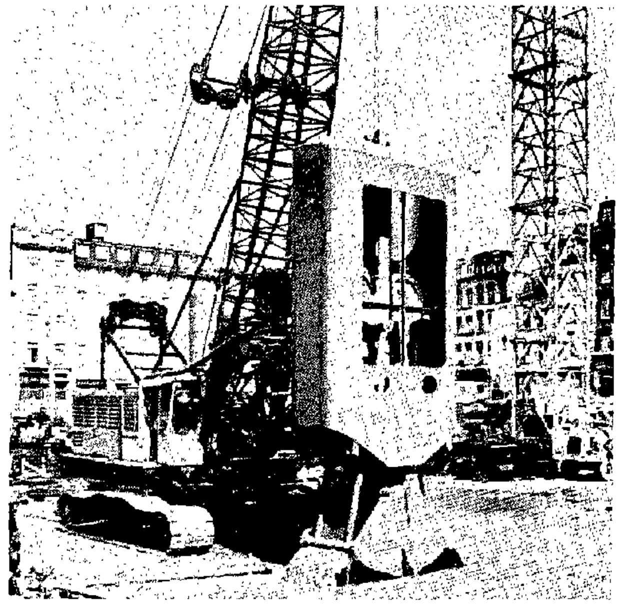


Fig. 10.4. -- Bucket suspended by crane-derrick cable (Document from SIF-BACHY).

Fig. 10.3. -- Another type of grab bucket. Note overlapping of scoop teeth in closed position (Document from Gallia).

Openings are sometimes put in the tool to allow drilling mud to flow rapidly from the bucket without much loss of soil particles. There is a large variety of buckets as to weight (1 to 17 t) and as to structural characteristics.

These permit construction of barrettes from 0.30 m (1.0 ft) to 1.50 m (5.0 ft) wide and from 1.20 m (4.0 ft) to 3 m (10.0 ft) long.

The classification of buckets presented in the table is according to the guidance system rather than the closure system as this classification also corresponds to types of machines that may be encountered on projects.

1. BUCKETS SUSPENDED BY A CABLE

The drilling bucket is suspended by the boom of a conventional crane (Fig. 10.4) or by a hydraulic shovel (Fig. 10.5), using a cable. These are the same as used in percussion drilling.

In order to ensure a vertical excavation that completely avoids the tendency of rotation with depth, the grab buckets all include a skirt or guide with dimensions that precisely correspond to those of the opened bucket. This skirt adds weight to the bucket and ensures the verticality of the excavation. Hence, the name sometimes used for these tools is "self-guided buckets" (Fig. 10.6).

The method of closure for the scoops is most often by cable, and hence the name "bicabled buckets." Through the use of pulley-blocks (up to 6 parts) in the interior of the bucket, the second cable controls the opening and closing of the scoops (Fig. 10.7).

The method of closure may also be by use of hydraulic rams with a ram for each side of the bucket.

When the hydraulic fluid is sent from the surface by hoses, this is a *single-cable, hydraulic* bucket (Fig. 10.8).

When the hydraulic pressure is controlled using electric cables, the bucket has, under watertight casing, a hydraulic unit (electric motor + pump). These are *hydro-electric* buckets (Fig. 10.9a and b).

Fig. 10.5. -- Bucket suspended by derrick of hydraulic shovel (Document from Poclain).

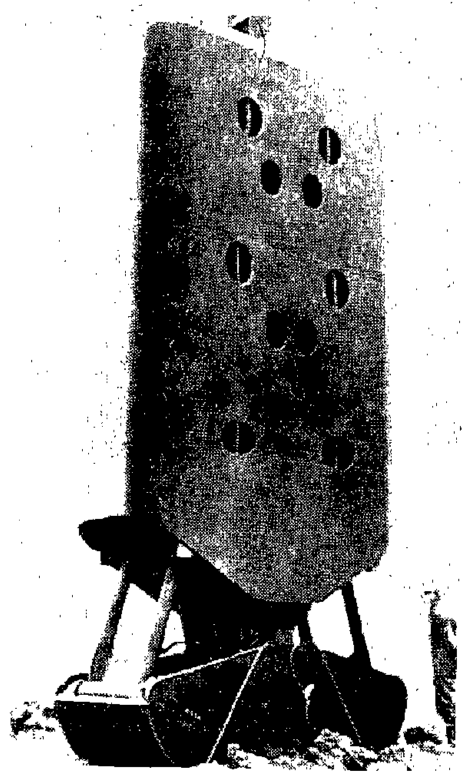


Fig. 10.6. -- Self-guided, weighted bucket (Document from SIF-BACHY).

Bucket and machine model	Closing principle	Various brands
Buckets suspended by a cable used with a hydraulic shovel or crane with derrick	— closing by cable	Benoto, Gallia, Domine-Bachy, Keller, Tranchesol, Solétanche, Intrafor-Cofor
	— closing by hydraulic jacks with lines	Benoto, Gallia-Cella, Tranchesol, Solétanche
	— hydro-electric closing	Benoto, Gallia-Cella, Felhman
Kelly buckets sliding in a guide that is attached to the carrying hydraulic shovel or crane with derrick	— closing by cable	
	— closing by hydraulic jacks with lines	BSP, Gallia, Soil-Mec, Solétanche, Galinet
	— hydro-electric closing	Gallia-Cella
Hydraulic bucket with one or two jacks using an extension arm adapted to the derrick of a hydraulic shovel		Poclain

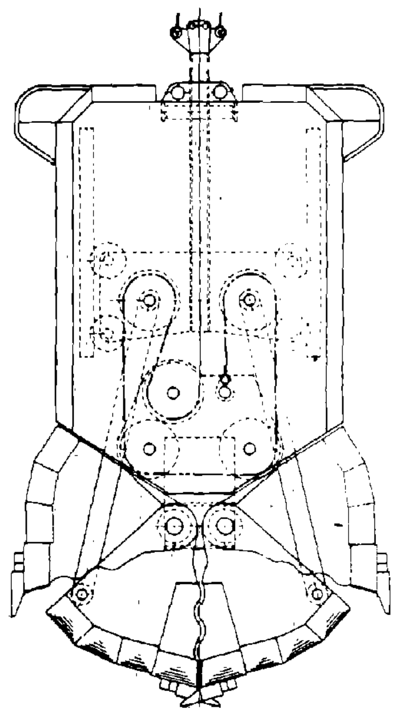


Fig. 10.7. -- Two-cabled bucket (Document from Gallia).

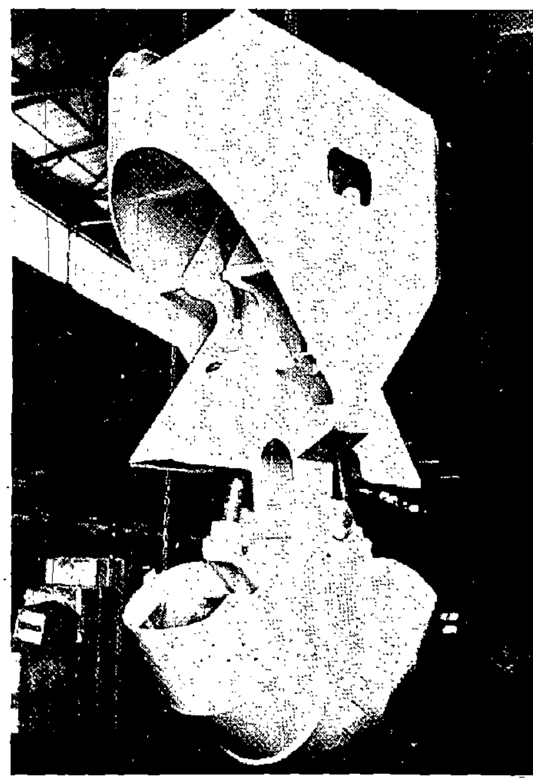


Fig. 10.8. -- Single-cabled hydraulic bucket (Document from Benoto).

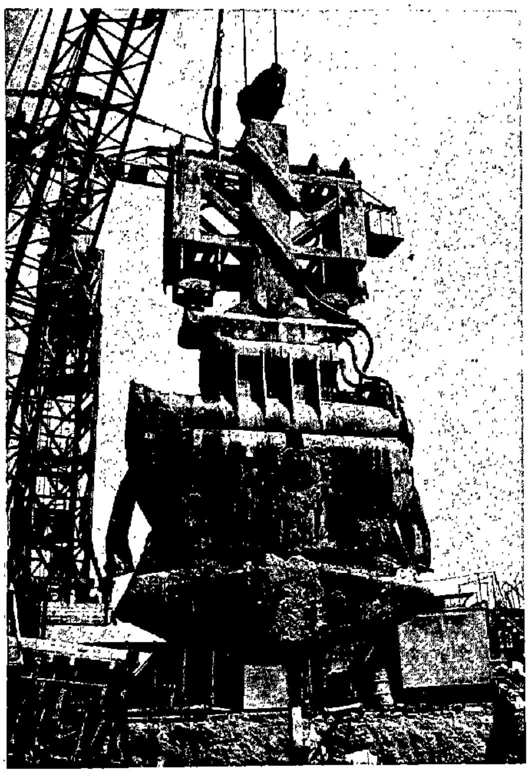


Fig. 10.9a. -- Example of hydro-
electric bucket
(Document from Benoto).

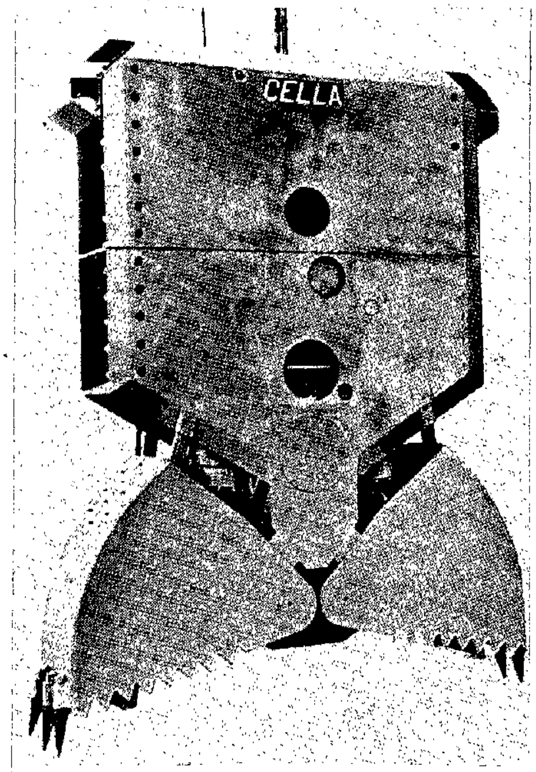


Fig. 10.9b. -- Another type of hydro-
electric bucket
(Document from Gallia).

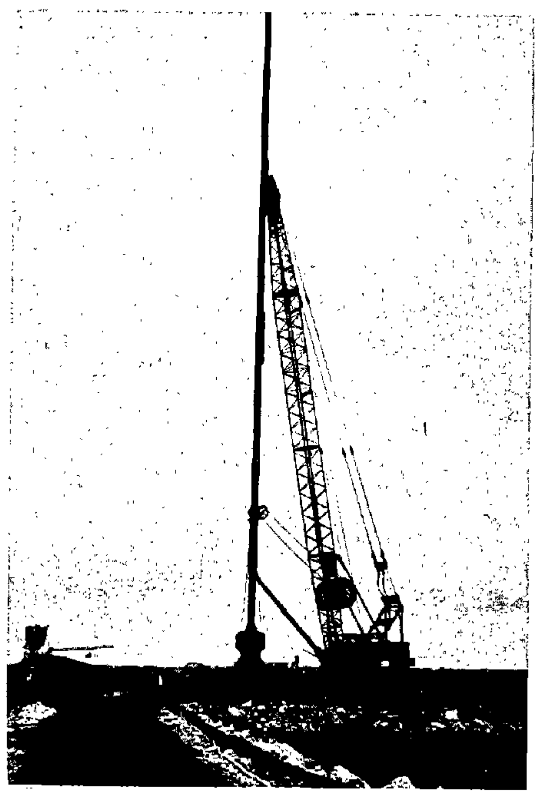


Fig. 10.10. -- Kelly bucket maneuvered by crane cable.

2. KELLY BUCKETS

These buckets are fixed to a sliding kelly within a guidance system that is fastened to the frame and derrick of the crane (Fig. 10.10).

Kellys may be tubular, square, or simply constructed with wide-flanged beams. After closing of the bucket, the kelly is lifted by one or more cables activated by a winch that is located on the crane.

Kelly buckets have, as do suspended buckets, cable or hydraulic closures, and the scoops are activated in this case by one or two jacks. In the case of buckets using a closing cable, this cable passes through the inside of the kelly while in the case of hydraulic buckets, hydraulic fluid is supplied to the rams by a rigid conduit that is fixed to the kelly or by hoses. In case hoses are used, drums are employed to wind and unwind the hydraulic hose.

Figures 10.11 and 10.12 show, respectively, a type of winding drum used and where the hydraulic lines are looped from a point near the mid-height of the kelly, commonly referred to as the "elevator" connection.

It is noted that kelly guidance is assuredly excellent without being completely rigid. As the machine is operating, some vibrations may be observed because of the flexibility in the kelly guide and in the machine mounting. The two systems previously described each have their proponents but tests have shown that the guidance, by either of the procedures, may be good or bad depending on the skill of the operator and the conditions imposed by the soil.

3. BUCKETS FIXED TO A HANGING MOUNT

As in the construction of cylindrical piles, the hydraulic bucket is fixed to the end of a rigid arm composed of several elements. This arm is fitted to the outer arm of the shovel by a hinge as in the procedure developed by Poclain (Fig. 10.13) and by Benoto (Fig. 10.14). Drilling is performed by the combined movements of the outer arm and the other arms of the shovel. The present, maximum depth that may be attained with the GC 150 shovel is 16 m (52.5 ft).

Fig. 10.11. -- Detail of winder for lines of hydraulic bucket.

Fig. 10.12. -- Hydraulic kelly bucket with "elevator" pulley system for lines (Document from Galinet).

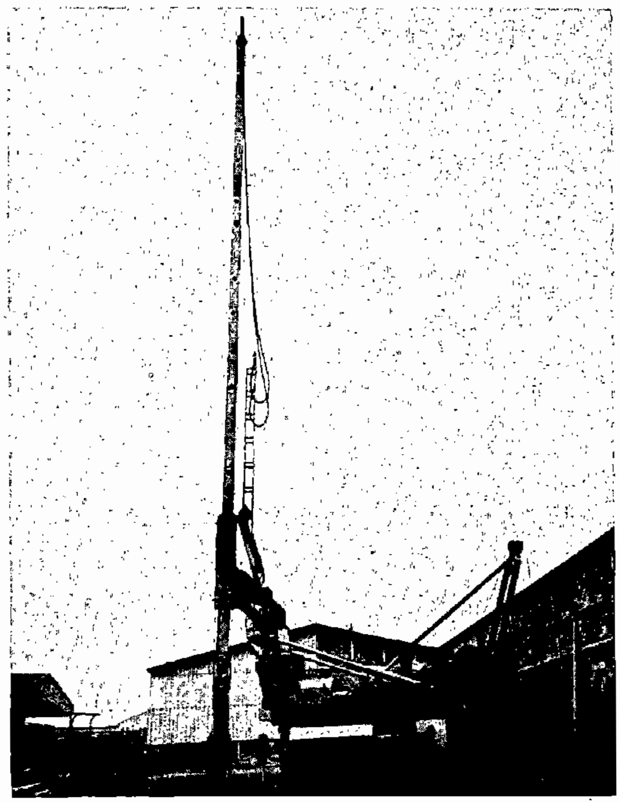


Fig. 10.13. -- Hydraulic bucket attached to end of extension arm.

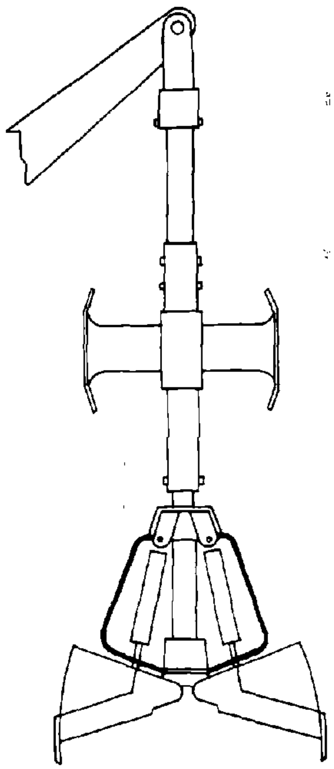


Fig. 10.14. -- Hydroelectric bucket mounted on Benoto arm on boring equipment.

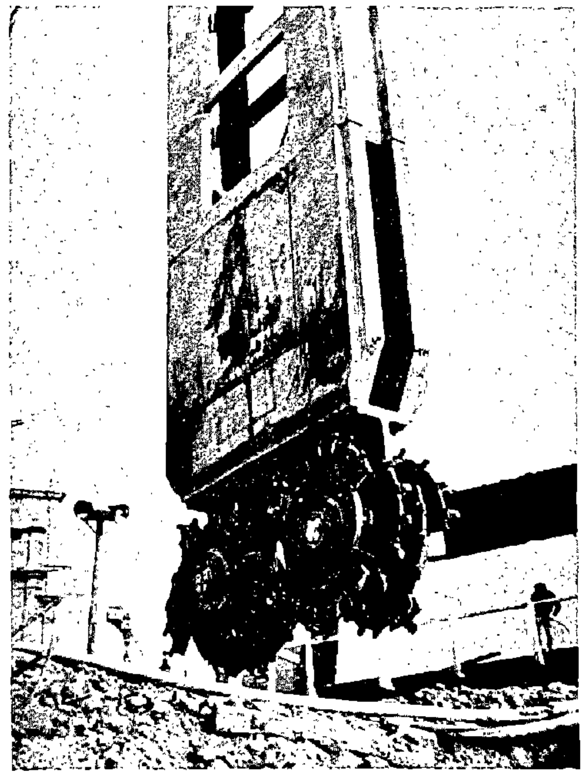


Fig. 10.15. -- Head of hydro-drill (Document from Soletanche).

4. THE HYDRO-DRILL

This tool, adapted to the excavation of hard soils, is itself a machine. Suspended by a crane, it includes (Fig. 10.15) two bits with toothed wheels activated by a hydraulic motor which is protected by a casing that is mounted inside the guidance skirt. The sediments are raised by reverse circulation by a pumping system located on the surface.

This tool is available in only one size: 2.70 x 0.65 m (9.0 x 2.1 ft).

DRILLING MACHINES WITH REVERSE CIRCULATION

Contrary to the classic boring method, the products of excavation that are mixed with the drilling mud are evacuated by the velocity inside of the sections of drill pipe. A high ascending velocity is brought about either by a suction outfit (Saltzgitter procedure with centrifugal pump and vacuum generator) or by an "airlift-ejector" located above the tool and supplied with compressed air by a compressor (Honigmann method).

1. REVERSE CIRCULATION BY SUCTION (Fig. 11.1)

The machines described in the following table employ a suction group that may or may not be a part of the machine. This includes:

a) Vacuum generator

The vacuum generator has the primary function, before the beginning of excavation or after each addition of pipe, to suction the air from the centrifugal pump as well as from the lengths of pipe and thus to fill the system with fluid and ensure raising the water or the mud.

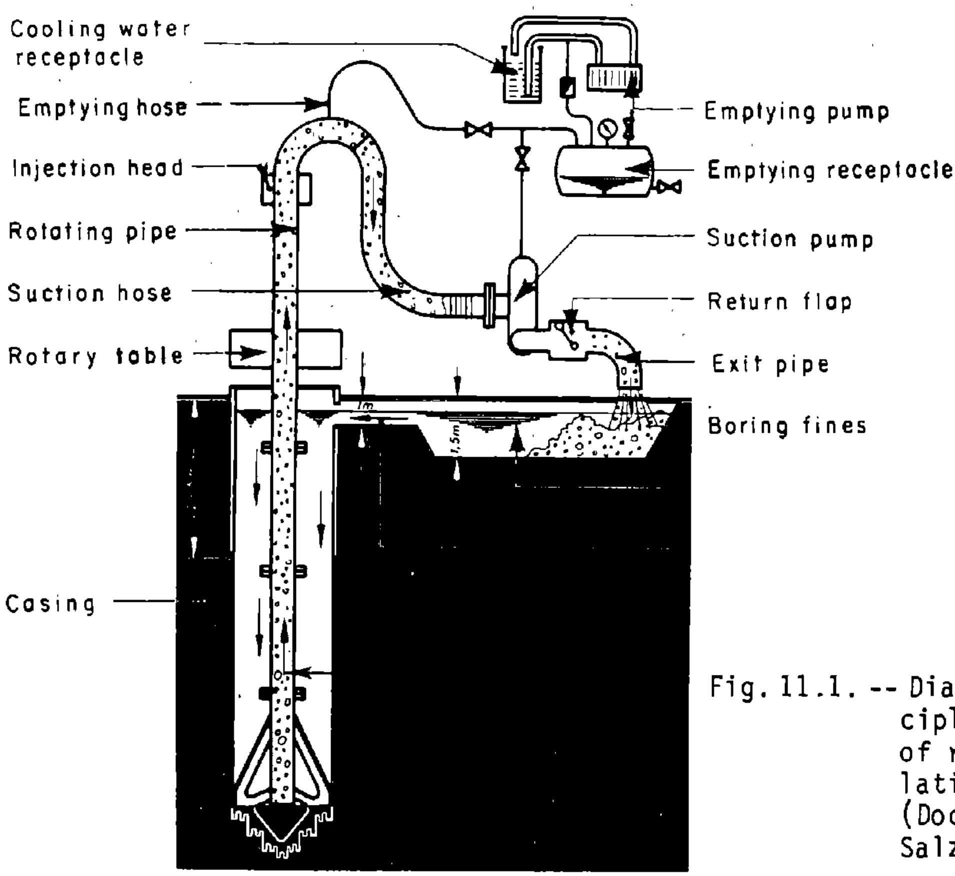


Fig. 11.1. -- Diagram of principle for method of reverse circulation by suction (Document from Salzgitter).

TABLE TO DESCRIBE BOREHOLE MACHINES WITH REVERSE CIRCULATION

Name of Manufacturer	Machine model	Weight (kg)	Motor power	Max. torque with rotary table (kgm)	Speed of rotation (rpm)	Pump output (l/mn)	Collector pipe diameter	Borehole diameter (m)	Max. depth of borehole (m)	Observations
SALZGITTER 52, rue de Londres 75008 Paris Tél. : 292 26 97	RC 6	12,200	diesel Deutz F 4 L 514, 60 HP to 1,800 rpm	1,000	0 to 40	4,000	15 cm (6")	0.40 to 1.50	200	Machines BB 6, BB 8, BB 10 (see TN No. 7) can also operate with reverse circulation.
	S 150 H	rotary table 1,920 complete group 3,300	diesel Deutz F 6 L 912, 68 HP to 1,800 rpm	1,100 (table of 300 mm) 1,800 (table of 800 mm)		H H_{BF} 14.50 m H H_V 8.50 m				
	RC 8	14,000	diesel Deutz F 6 L 514, 86 HP to 1,800 rpm	1,700 (table of 800 mm)	0 to 23	8,000	20 cm (8")	0.60 to 2.50	300	
	S 200 H	rotary table 2,900 complete group 4,200				H_{BF} 13 m H_V 8.50 m				For equipment with ejector-emulsifier, the pump is replaced by a compressor of 5 m^3/mn (Ø 15 cm) to ≃ 20 m^3/mn (Ø 30 cm).
	S 300 H	related to table type 2,900 Ø 800 6,500 Ø 1,200 complete group 7,800	diesel Deutz F 8 L 413, 140 HP to 1,800 rpm	3,600 (table of 800 mm) 16,000 (table of 1,200 mm)		16,000	30 cm (12")	0.60 to 2.50	300 to 500	
SOLÉTANCHE ENTREPRISE 7, rue de Logelbach BP 309 75822 Paris Cedex 17 Tél. : 227 65 73 622 25 00	CIS (Soletanche reverse circulation)	14,000 (driving) 20,000 (driving + rotation)	160 HP 210 HP		24	8,000	20 cm (8")	0.60 to 1.50	Practically no limit	This machine also is used to construct barrette piles or diaphragm walls.
	Hydrofond	?	6 hydraulic, immersible motors distributed like a star driven by hydraulic group of 380 HP	10,000	0 to 22	7,000	20 cm (8")	1.30 to 3.00	250	Machine in two versions: T (work with soil), M (work at aquatic site). Force on tool increased with ballast and may attain 80 t.
CALWELD Smith International (Galinet, Paris)	Information unavailable							up to 2.10	250	

H = Lifting height; H_{BF} = $H_{BACKFLOW}$; H_V = H_{VACUUM}

(3.8 l/mn = 1 gal/mn; 0.028 m^3/mn = 1 ft^3/mn; .139 kgm = 1 ft-lb; 0.453 kg = 1 lb; 0.0254 m = 1 in.; 0.91 t = 1 ton)

b) **Suction pump**

This is a centrifugal pump of high output (240 m^3/h - 8,474 ft^3/hr through a series of pipe lengths of Ø 15 cm - 6.0 in., and about 1,000 m^3/h - 35,310 ft^3/hr for Ø 30 cm - 12.0 in.). Because it is essential that the level of water or mud be maintained constant with respect to the level of the soil or surface casing, a sufficient supply of water or mud is necessary.

The drilling mud and the recovered products are emptied into tanks so as to be able to screen the cuttings, and the mud may re-enter the borehole by gravity or by pumping.

2. REVERSE CIRCULATION (HONIGMANN METHOD) (Fig. 11.2)

The equipment is arranged so as to be able to use compressed air at the bottom of the drill pipe and above the tool using the principle of a Mammouth pump (emulsifier-ejector), in order to ensure the raising of the cuttings.

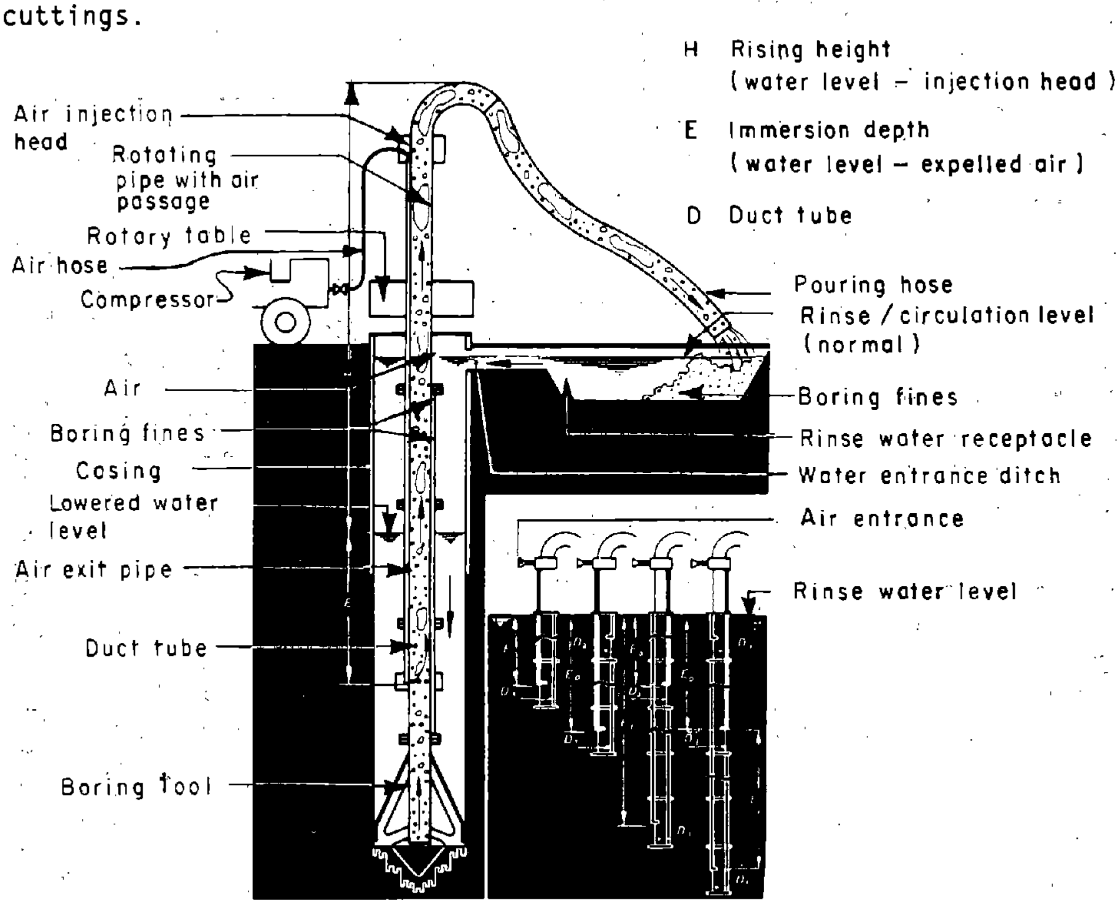


Fig. 11.2. -- Diagram of principle for method of reverse circulation by blowing of compressed air (Document from Salzgitter).

The characteristics of the compressor and its output of air depend on the diameter of the opening in the drill pipe. The volume of air from the compressor should achieve an upward fluid velocity of about 5 m/s - 16.4 ft/s (6 to 10 m^3/min - 212 to 353 ft^3/min for a Ø of 20 cm - 8.0 in.). The air pressure should be at least 6 bars (87 psi), corresponding with a maximum depth of excavation (immersion) of about 50 m (164 ft). This depth, E_1, is related to the pressure of the compressor minus 1 atmosphere, but clearly greater depths may be attained with placement of air inflows at various depths along the pipe lengths at distances of $E_1/2$. However, the maximum borehole depth depends on the nominal opening of the pipes: this is 400 m (1,312 ft) for a diameter of 15 cm (6.0 in.) and near 750 m (2,460 ft) for a diameter of 30 cm (12.0 in.). From a practical point of view, raising of the cuttings is not a limiting factor in this operation.

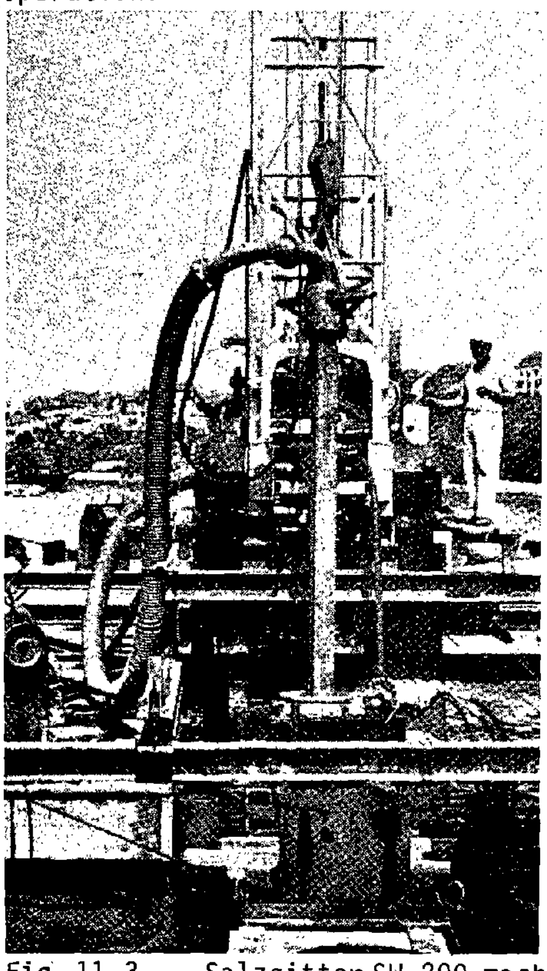


Fig. 11.3. -- Salzgitter SW 200 machine (Drilling by suction Ø 700, Document from Salzgitter).

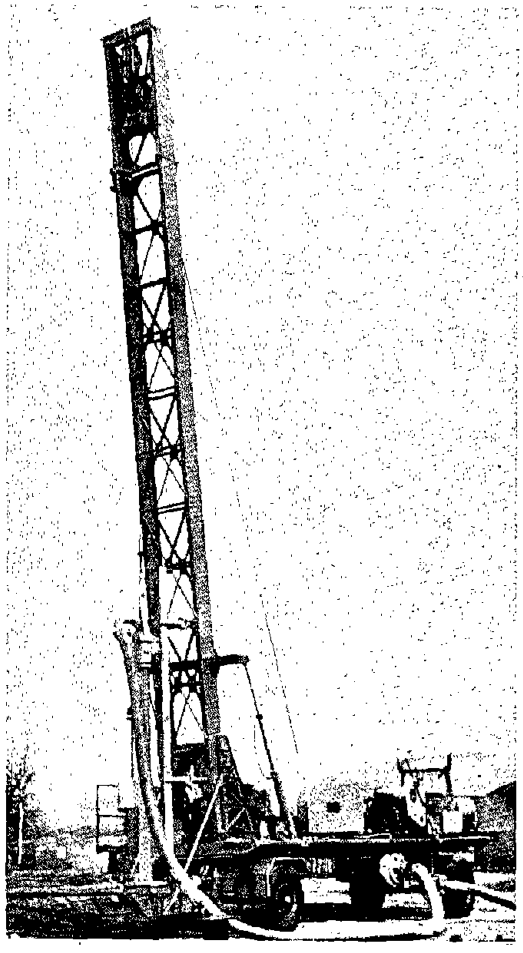


Fig. 11.4. -- Arrangement for drilling with reverse circulation, Salzgitter Model RC (from Salz.).

3. CHARACTERISTICS OF MACHINES

The following table summarizes the principal characteristics of reverse-circulation machines used in France. Although Salzgitter machines work in rotation, only the CIS machine by Soletanche (Fig. 11.5) can combine percussion drilling and rotation for excavating formations that are particularly resistant.

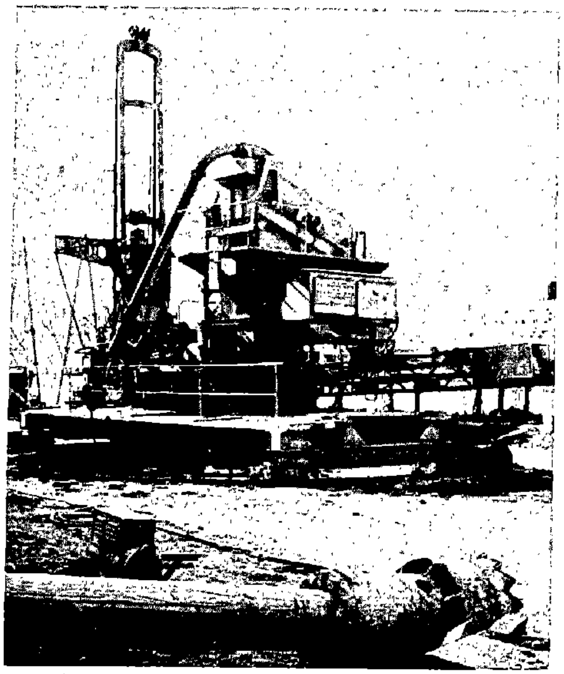


Fig. 11.5. -- Soletanche arrangement for drilling with reverse circulation, (CIS) (Document from Soletanche).

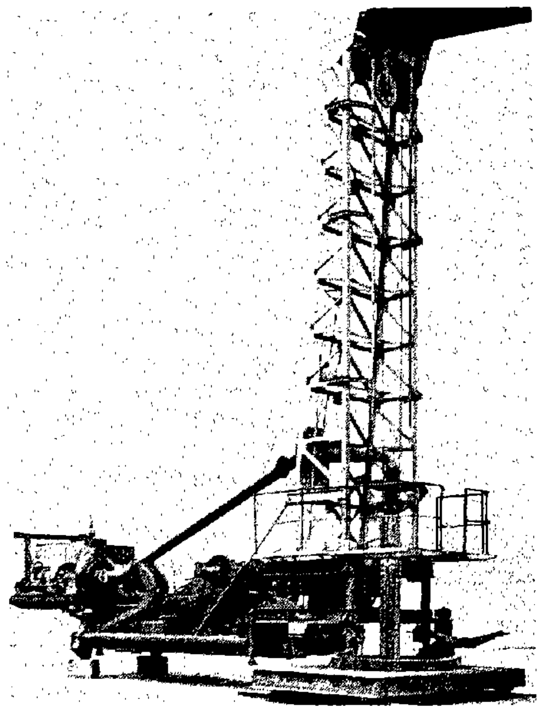


Fig. 11.6. -- CIS 60 R machine for drilling with reverse circulation (Document from Soletanche).

The Hydrofond (Figs. 11.7 and 11.8) is a machine that is especially powerful and unique because it can descend into the borehole as excavation progresses. This machine was developed by the Soletanche Company and has, directly above the tool:

-- six hydraulic motors distributed in a star formation around a hollow shaft of 8 in. (20 cm) by which removal of the detritus is achieved;

-- a hydraulic driver also submersible that furnishes a torque of 10,000 kgm (72,400 ft-lbs) with a variable rotation speed of 0 to 22 rpm;

-- an electric submersible pump;

-- a superimposed weight from heavy pipe (ballasted with cast iron) that produces a force on the tool of 50 t (55 tons) that may be brought to 80 t (88 tons);

-- with the Marine version, the apparatus is suspended by cables and supplied by hoses. The torque is taken up by plates or runners that push against the walls of a surface casing;

-- with the Terre or land version, the torque is resisted at the head of the borehole by a series of squared bars, wedged against a guide in the frame of the machine.

-- the normal boring diameters are the following: 1.30 m (4.3 ft) - 1.50 m (5.0 ft) - 1.80 m (6.0 ft) - 2.10 m (7.0 ft) - 2.40 m (8.0 ft) - 2.75 m (9.0 ft) - 3 m (10.0 ft).

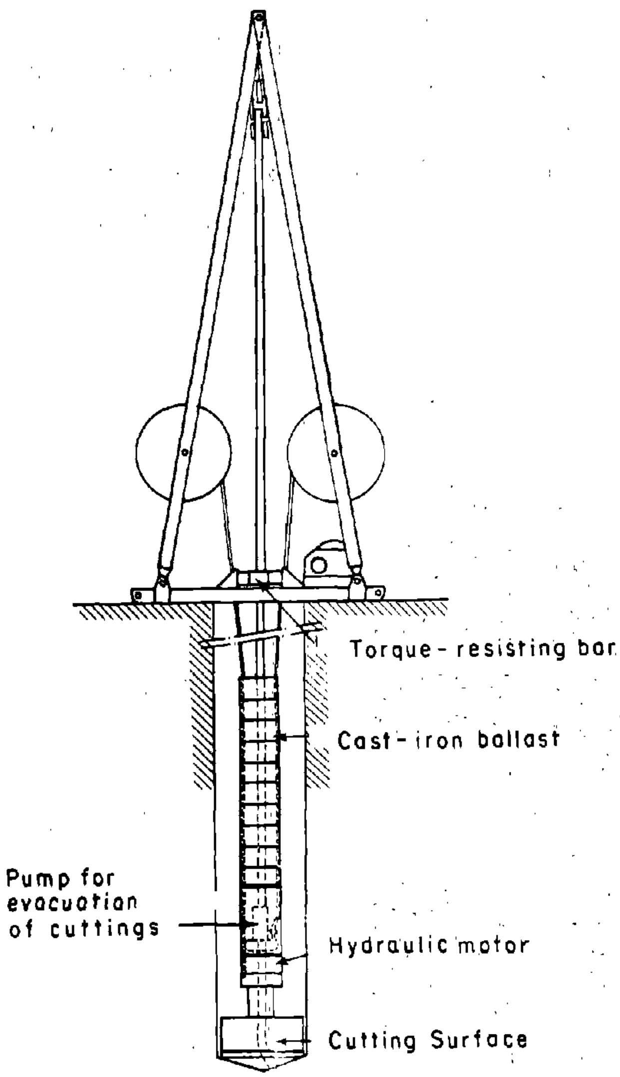


Fig. 11.7. -- Diagram of principle for method of Hydrofond drilling (Document from Soletanche).

Fig. 11.8. -- Arrangement for Hydrofond drilling (Document from Soletanche).

DRILLING BY USE OF AIRLIFT

This equipment is used mainly:

-- to perform recovery or extraction of soft materials (silt, sands and pebbles) contained in the enclosures of sheet-pile walls forming cofferdams or in driven or vibrated temporary casings for piles.

-- in boreholes with water after placement of a suction basket. This involves elimination of fines and thus cleaning of the borehole wall in order to obtain a maximum output of water without risk of plugging.

-- in pipes for the investigation of piles by geophysical methods where the bottom of the pile has been cored or perforated, either to increase the clarity of the water before passage of the television camera or to clean out a pocket of sediments or mud before injection of grout into the cavity.

1. FUNCTIONING PRINCIPLE (Fig. 12.1)

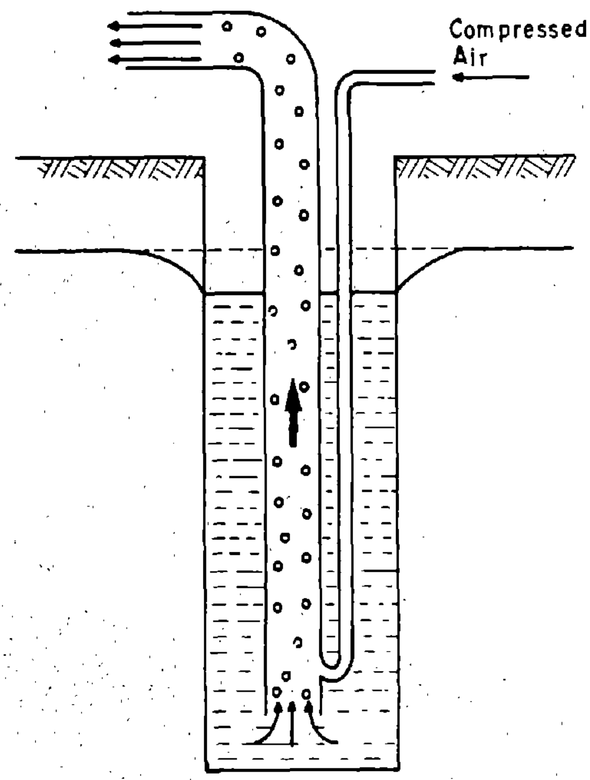


Fig. 12.1. -- Principle of functioning of emulsifier.

The introduction of compressed air, about 30 cm (12 in.) above the base of a vertical pipe filled with liquid, produces a profusion of air bubbles that rise and expand as the pressure decreases. The mixture (water + sediment + air) thus emulsified, which has a lower density than that of the ambient liquid, rises with increasing velocity and creates a suction at the base of the pipe that will lift a liquid that is loaded with sediments.

It is on this principle that Mammoth pumps are constructed to raise muddy waters that are loaded with sand and gravel. An airlift is also used in the reverse-circulation drilling method (see TN No. 11, Honigmann method).

In practice, the apparatus is of simple construction. It is composed of a metal column of between 150 to 300 mm (6 to 12 in.) in diameter, at the bottom of which are connected one or more compressed air conduits of 20 to 60 mm (0.8 to 2.4 in.) in diameter. This piping is suspended from the boom of a crane. The critical part of the operation consists of maintaining the base of the pipe at 10 to 20 cm (4.0 to 8.0 in.) above the materials to be extracted. This presents some problems when eliminating loose soil from a casing that has been installed by driving.

2. OPERATING CONDITIONS

The conditions related to proper functioning of the equipment involve:

-- geometric characteristics of the airlift,

-- priming and service pressures, and

-- quantities of air to be injected.

a) Geometric characteristics

The coefficient of submergence is the ratio between the submerged height h_i and the total height h (Fig. 12.2).

In practice, the submergence coefficient should be from 0.5 to 0.66, 0.5 for an airlift of 50 m (164 ft) total height and 0.66 for a height h of about 10 m (32.8 ft).

The diameter D should be selected according to the output desired: it is thus related to the volume of air to be injected (see §c).

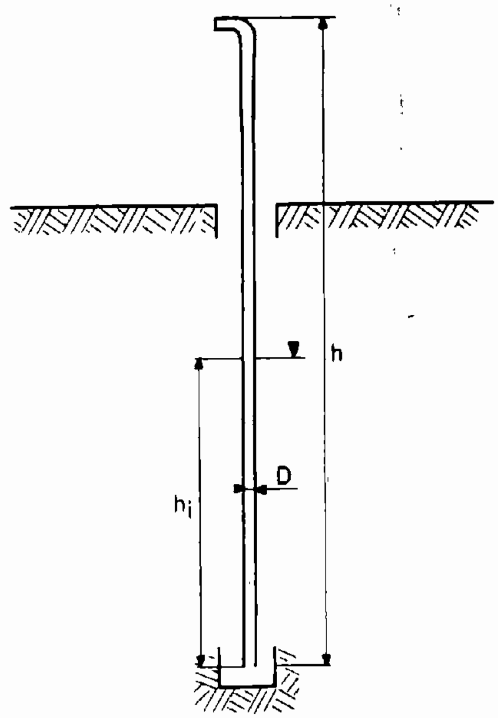


Fig. 12.2. -- Diagram showing geometric characteristics of emulsifier.

b) **Priming pressure and service pressure**

The air injected by the compressor should be sufficient to drive the column of water whose initial height corresponds to the submerged height at rest. The priming pressure is thus slightly above that of this column of water.

In the case of continuous pumping there is a lowering of the level of the water table as pumping continues. The service pressure is thus less and corresponds to the height of the water in the borehole after pumping is ceased.

c) **Quantity of air injected**

The volume of air injected is on the order of two or three times that of the water to be pumped. A sufficient output of air is also required to cause the mixing of the soil particles with the water and the raising of the "emulsified" fluid. On the other hand, if the amount of air is too high, there is risk of formation of a continuous stream of air through the fluid and a halt to pumping.

It must also be noted that the water evacuated can contain a maximum of 10 to 20% sand and gravel by volume.

3. USE OF AIRLIFT IN THE BORED PILE TECHNIQUE

The airlift is used [2]:

a) During boring

The construction of piles with percussion-driven or vibration-driven casing may be performed using an airlift, for which the role is then to eliminate rapidly the loose soil inside the casing. This method permits the resumption of placement of a temporary casing when there is premature refusal to driving; for example, in a formation of compact gravel or one of great thickness.

b) At the end of boring for the cleaning of the slurry in the borehole

Here the airlift serves as a pump for recycling of the drilling mud. As was noted above, it is important that not only a sufficient supply of air is required but also that water or bentonitic slurry is supplied to the borehole so as to maintain an essentially constant level of fluid in the borehole.

This technique, which has the advantage of simplicity, permits attainment of significant depths. The low bulk of the apparatus and its tubular form (that of the tremie) make it well-adapted to the cleaning of the bottom of the borehole even after placement of reinforcing.

To augment the information given earlier concerning the relation between water, air output, and the percentage of materials raised, a graph has been prepared (Fig. 12.3) that gives the diameter of the airlift for ascending speeds from 4 m/s to 6 m/s (13.23 ft/s to 19.68 ft/s). Considering the complexity of the hydrodynamic phenomenon (multi-phase discharge of a quantity Q of water plus air plus solids) and considering the small number of experimental studies that have been actually performed, the graph in Fig. 12.3 gives only approximate values on the ouput of the air compressor and on the output of water required for functioning of the equipment.

c) For cleaning of the concrete-soil contact

The apparatus here has a diameter that is less than that of the usual airlift. The airlift-pipe must be lowered into the pipes that were installed in the pile for the geophysical investigation.

Its arrangement (Fig. 12.4) is also slightly different because the air pipe and the pipe for lifting the water are concentric. They may be adjusted in a manner so as to regulate the height at which the air enters the lift pipe.

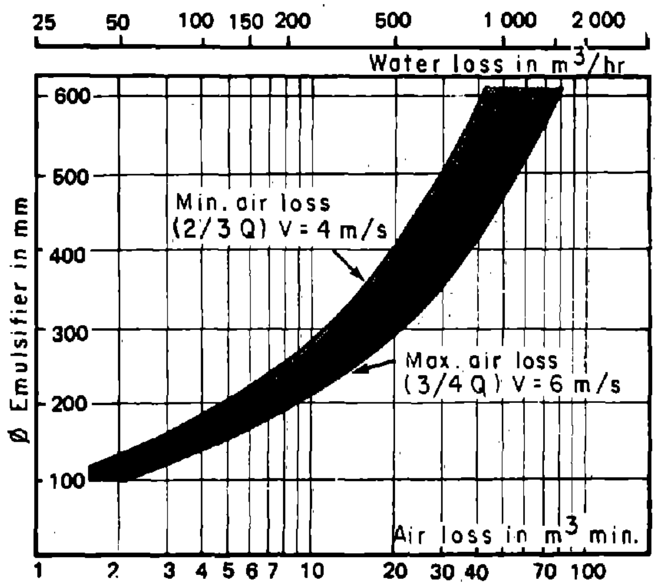


Fig. 12.3

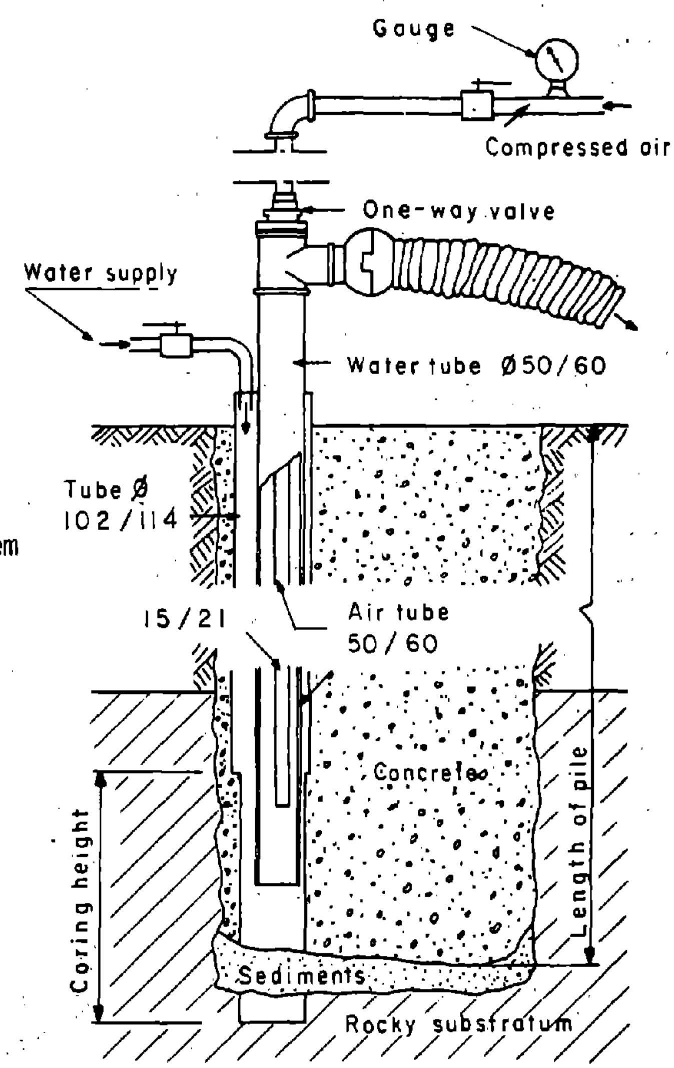


Fig. 12.4. -- Diagram of system used to clean concrete-soil contact of pile.

The procedure generally has two phases:

-- *mixing* of sediments in the water. The air pipe is lowered 25-30 cm (10.0-12.0 in.) beyond the base of the water pipe;

-- *suctioning*, the air pipe is raised to its initial position 30 cm (12.0 in.) above the base of the water pipe.

If the output of water injected at the top is insufficient, the air valve must be closed and the level of water must rise before resuming pumping.

The air pipe alone may also be used for cleaning of the pipes of 50/60 mm (2 to 2.4 in.) in diameter required for geophysical investigation by the acoustic method. However,the operation then consists of several phases of successive blowing and refilling.

In conclusion, the apparatus described should permit:

-- cleaning of the acoustic-investigation tubes of all diameters that are obstructed by the mud or by various debris. The cleaning will free the pipe for the passage of the devices used in wave transmittal for the investigation;

-- cleaning of the acoustic investigation tubes of 102/114 mm (4 to 4.5 in.) in diameter as well as the cored hole in the concrete at the tip of the pile to increase the clarity of the water and improve the view for the camera;

-- cleaning defects in piles, prior to grouting, that have been detected by geophysical investigation.

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APPENDIX A - Bored-Pile Schedule

APPENDIX B - Pile-Concreting Report

Note from translator: The following forms are translated and are shown here for interest. They are distributed in France at various offices of the Ministry of Transportation.

Contractor	:		Contract No.	: ....
Construction Supervisor	:		General Contractor	: ....
District	:		Foundation Contractor	: ....
Subdivision	:		Project Superintendant	: ....
Project	:		Geotechnical Office	: ....
Structure	:		Engineering Office	: ....
Foundation Element	:		Quality Control	: ....

CONSTRUCTION SCHEDULE FOR BORED PILE NO. ....

FOUNDATION PLAN NO........

THEORETICAL DIAMETER (or barrette dimension): .... THEORETICAL INCLINATION: ....

A. Drilling Begun .... at .... a.m./p.m. Completed .... at .... a.m./p.m.

			Type and main characteristics of drilling and lifting machines Access conditions
GROUND ELEVATION AT TIME OF DRILLING		.... ft Site Datum	
Drilling elevation		.... ft SD	
Level of bottom of borehole	anticipated	.... ft SD	
	actual	.... ft SD	
Total length of borehole		.... ft	
Upper level of casing or collar		.... ft SD	
Lower level of casing or collar		.... ft SD	
Length of casing or collar		.... ft	

Date	Time	Depth	SOIL DESCRIPTION		TOOL				OBSERVATIONS -(setbacks, cave-ins, cavities, waterflow, slurry loss, recycling of slurry, etc.)
					Type	Diameter	Weight	Freefall height	

CAUSES AND CONSEQUENCES OF CAVE-INS DURING DRILLING

ORIGIN AND DURATION OF DOWNTIMES (breakdowns, weather conditions, etc.)

Verifications	Verticality or inclination		Diameter	
	Test method	Results	Test method	Results
______ at ______ a.m. p.m.				
______ at ______ a.m. p.m.				
______ at ______ a.m. p.m.				

Cleaning of Bottom of Borehole	Method used	Results	
		Borehole bottom level	Height of sediments
______ at ______ a.m. p.m.			
______ at ______ a.m. p.m.			
______ at ______ a.m. p.m.			

Water level in borehole before concreting .... ft SD ....at......... a.m. p.m.

Retrievable temp. casing :
Outside diameter : ______
Thickness : ______
Weight per ft : ______
Component lengths : ______
Type of joint : ______
Extraction method : ______

Drilling Slurry

Composition	Brand	Type	Proportion
Bentonite			
Additives			
Water source			

Bentonite storage { in sacks ☐ / in silos ☐
Mixing { "Mud Hopper" pulverizer ☐ / agitator deflocculator ☐ / centrifugal pump ☐
Slurry troughs { type number ______ / unit capacity ______

TREATMENT OF RECYCLING UNIT

Vibrating screen	Cyclones	Circulation pump
Model: ______	Type : ______	Type : ______
Mesh : ______	Number : ______	Output : ______

PROPERTY TESTS

Date and time of sampling		__at__ am pm	__at__ am pm	__at__ am pm	__at__ am pm	__at__ am pm	__at__ am pm	__at__ am pm
Measure	Initial	at mixing	During construction at various levels at______ ft	at______ ft	at______ ft	at______ ft	At bottom of borehole before recycling	before concreting
Density								
Viscosity								
Sand content								
Cake / Filtrate								
pH								

B - Casing - Liner

Refer to Plan No.

Upper level : ______ ft SD	Nature of casing or liner : ______
Lower level : ______ ft SD	Type and method of possible coating : ______
Total length : ______ ft	Component lengths { ______ x ______ ft / ______ x ______ ft — Joints : ______
Outside diameter : ______ ft	Bracing system : ______
Thickness : ______ in.	Observations - Stockpiling - Placement difficulties:
Placement { before ☐ / with ☐ } reinforcing	

C - Reinforcing

Upper level : ______ ft SD	Method of fastening of hoops to longitudinal bars { by welding ☐ / by tying ☐ / combination ☐
Lower level : ______ ft SD	Method of fastening of cage components to each other { by welding ☐ / by connectors ☐ / combination ☐
Total length : ______ ft	Component lengths { ______ x ______ m / ______ x ______ m — Length of overlaps } ______
Outside diameter : ______ ft	Centering guides { Type : ______ / Number : ______ every ______
Base of cage { ☐ without basket / ☐ with (I.D. ______ in.)	Storage conditions : ______
Cage { suspended at top ☐ / placed on bottom ☐	Coating system : ______
Observations - Placement difficulties:	

D - Acoustic investigation access tubes

Number	Diameter in.	Upper level ft SD	Lower level ft SD	Total length ft	Types of plugs	Method of attachment to reinforcing
	2/2.4					
	4/4.5					

E - Concreting

Begun ______ at ______ a.m. p.m. Completed ______ at ______ a.m. p.m.

Source of design of mix	Concrete components	Category	Class	Origin	Proportion
Standard design ☐ Special design ☐ Job-site design ☐ Other ☐ – : ______ – : ______ – : ______	Cement				
	Aggregates				
Supplying by cement mixers 1st ______ yd^3 2nd ______ yd^3 3rd ______ yd^3 4th ______ yd^3 5th ______ yd^3 6th ______ yd^3 7th ______ yd^3 Vt = ______ yd^3	Water				
	Additives				

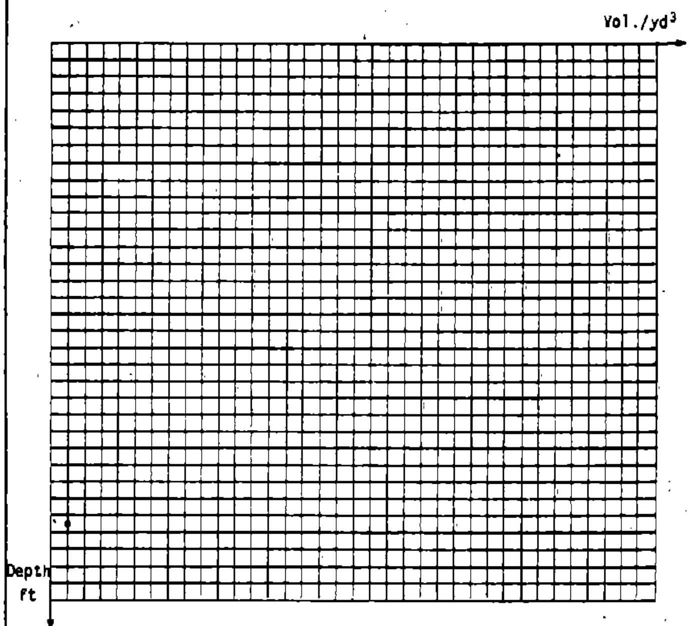
Placement techniques		Type of priming plug
Tremie without couplings ☐ with couplings ☐	O.D.: ______ in. I.D.: ______ in. upper level (tremie base) : ______ ft SD base level : ______ ft SD distance under tremie : ______ ft	– concreting directly from mixer ☐ – concreting using bucket ______ yd^3 ☐ – concreting by pump ☐
Bucket	Model ______ Capacity ______ yd^3	with pistons ☐ other ☐ output ______ yd^3/hr

Placement control	: ______________________
Maximum concreting level	: ______________________ ft SD
Height of fresh, purged concrete:	______________________ ft
Upper pile level before trimming:	______________________ ft SD
Pile length before trimming	: ______________________ ft
Corresponding theor. vol.:	V_t = ______ yd^3/ft × ______ ft = ______ yd^3
Volume of excess in last truck:	V_e = ______ yd^3
Volume used in overflow and purging:	V_p = ______ yd^3
Actual, total consumption (V_p included):	$V = V_l - V_e$ = ______ yd^3
Actual pile volume before trimming:	$V_r = V - V_p$ = ______ yd^3
Overpour	$V_s = V_r - V_t$ = ______ yd^3 $\frac{V_r - V_t}{V_t} \times 100$ = ______ %

CONCRETE QUALITY TEST

Cement mixer or No.	Addition of water gal/yd³	Slump, in.	Number of tests comp.	Number of tests bending

CONCRETING CURVE



Trimmed height ________ ft – Planned elevation ____________ ft SD

OBSERVATIONS: Operation of central mixing plant, supply irregularities (concrete delays), possible setbacks (loss of priming in the tremie, rising or lowering of reinforcing, difficulties with extraction of temporary casing, caving, etc.)

F - Diagram of location of finished pile in relation to theoretical location

(report and mark position of access tubes)

Finished pile length :	______________ ft
Final soil level :	______________ ft SD
Total form of pile :	______________ ft
Height above ground :	______________ ft
SUPERVISOR	

Note - For all operations, taking of photographs is recommended, especially when there are setbacks or difficulties.

Test of finished pile

Method of acoustic investigation	Date	Operator	Report No.
wave transmission using reflection meth.			
lateral transmission of waves			
radioactive transmission of waves			

Coring	Date	Operator	Report No.

Corer: Brand : ______ Type : ______

Coring tool { Type : ______ Length : ______ ; O.D. : ______ I.D. : ______

Crown type : ______

DEFECTS OBSERVED	
Position	Assumed nature

View with T.V. camera	Date	Operator	Report No.

Depth	Level		Description	% core	Force lb	Speed rpm	Adv. rate in/mn

Repairs by grouting

Preliminary test for water flow { Pressure : ______ psi ; Output : ______ gal/mn

Cleaning { with pressurized water ☐ ; with airlift ☐

Number and type of conduits	Access tubes	Test boreholes	Specific boreholes	Seals Type	Seals Levels
grout					
saturation (blowholes)					

Tests	1st phase		2nd phase	
Type	stable ☐ unstable ☐		stable ☐ unstable ☐	
Grout: Composition	Type	Proportion	Type	Proportion
Cement				
Sand				
Clay				
Water				
Additives				
Viscosity				
Injection: Maximum pressure	______ psi		______ psi	
Volume	______ gal		______ gal	
Duration	______ mn		______ mn	

Mixer	Pump
Type : ______	Type : ______
Volume : ______	Output : ______
Speed : ______	Max. pres. : ______

OBSERVATIONS - DIFFICULTIES ENCOUNTERED:

Contractor : ....
Construction supervisor: ....
Structure : ....
Foundation element : ....

General contractor : ....
Foundation contractor : ....
Geotechnical office : ....
Quality control : ....

REPORT ON CONCRETING OF BORED PILE NO.........

FOUNDATION PLAN NO.........

DIAMETER OR DIMENSIONS : .... INCLINATION : ....

1. TEST OF CONCRETE COMPONENTS

Names Categories Classes	Origins	Samples		OBSERVATIONS
		Taken with	Date	Tests performed in place
1.1 - Cements				
1.2 - Aggregates				
1.3 - Water				
1.4 - Additives				Name of technician

2. TEST OF FRESH CONCRETE - TRANSPORT

Concrete type ________ Reference ________ Mixing Plant ________

Batch or truck no.	Moisture content of sands	Mixing times	Drum revolutions	Water content	Water additions	Slump		Workability or fluidometer	Start of pour	Gradation (sieves)	
						before	after			☐	☐

Observations on fresh concrete and operation of center

Batch or truck no.	No. of passes of trucks	HOURS				Volume of concrete (yd^3)	Temperature of concrete	Tests made per truck	
		Depart batch plant	Arrive at concrete site	Start of emptying	Finish of emptying				

Observations of transport

Sample preparation and test program

Types of tests	________	________	________	________
Number	________	________	________	________
Destinations	________	________	________	________
Tests to be performed (types - number - ages)	________	________	________	________

3. PLACEMENT

3.1 - Weather

Hours	Tempera-tures	Humidity	Precipitation		Atmos-pheric condi-tions	Wind velocity	Sky condi-tions	Sunshine	OBSERVATIONS
			Type	Rate and/or duration					

3.2 - Drilling - cleaning

Method of drilling -- method -- brief description Methods used -- equipment -- personnel State of borehole before concreting -- dry, water, slurry -- water level -- caving -- conditions at bottom Cleaning -- nature of excavation soils -- methods of cleaning -- cleaning efficiency -- examination of excavation bottom -- condition after cleaning -- after lowering of reinforcing -- cleaning setbacks	

3.3 - Reinforcing	
Composition -- sectional-assembly cage -- steel overlap -- welded connections -- stirrup-hoops -- centering devices -- hang-up risk -- basket at bottom of cage -- acoustic investigation devices Handling - storing -- handling methods -- stiffeners, lifting beams -- lifting and lowering -- conditions of storage area -- soiling of rebar -- misalignment Behavior during concreting -- raising of steel -- lowering of steel	

3.4 - Placement	Note beside each observation the number of the batch involved
Technique used -- name -- brief description -- equipment -- special devices Tremie priming -- technique -- difficulties -- air purge -- setbacks Concrete feed -- method -- rate -- setbacks	

Placement of concrete	The number of the batch involved and concrete levels attained
-- ease of flow	
-- raising of concrete	
-- flushing of sediments	
-- overly rapid set	
-- operating mistakes	
-- loss of priming	
-- raising of water or silting	
-- aspect of the flushed concrete	
-- bottom sediments flushed	
-- other current problems	
Serious setbacks	
-- description	
-- causes	
-- equipment used	
-- results	
-- extraction of concrete	
Extraction of temporary casing	
-- methods	
-- setbacks	
Overpour of concrete (data for computation)	
-- theoretical volume of borehole from actual dimensions	
-- volume of poured concrete	
-- volume remaining in last truck (approximate)	
-- volume possibly refused	
-- volume ejected by purge	
Trimming	
-- method	
-- description	
-- setbacks	
	Date Writer

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