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PRESSURE INJECTED FOOTINGS

by

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A THESIS

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled PRESSURE INJECTED FOOTINGS submitted by IAN F. MORRISON in partial fulfilment of the requirements for the degree of MASTER OF ENGINEERING.

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1 SYNOPSIS

This report is a design and installation guide for Franki type piles (pressure injected footings). Design methods currently used in practice are presented and an example illustrating the use of the design methods is given. Problems often encountered during the construction of this type of pile are described along with commonly used preventative or remedial measures. Construction and inspection practices are outlined.

2 INTRODUCTION

It should be emphasized from the beginning that any design techniques used to estimate the bearing capacity of pressure injected footings, uses assumptions whose accuracy are a function of the variance between the conditions assumed in the design and the insitu conditions. Simplification in the model used to formulate the design may also cause significant errors. If possible, load tests should be carried out at each site to verify the design. Experience with pressure injected footings in similiar stratigraphy is essential for safe, efficient design. Bearing capacity design calculations should be used to envelope probable bearing capacity values and to facilitate economic evaluation of the load carrying capacity of the individual pressure injected footing.

During construction, problems of pile heave and excessive ground vibration may occur. Techniques to analyse and solve the heave problem are outlined. A test procedure to evaluate vibrations due to piling, and guidelines on the allowable levels of vibration are also presented.

The serviceability of pressure injected footings is highly dependent on the technique used to install them. Because each pile is constructed within the ground, the inspector must rely on remote inspection techniques. The quality of the construction materials and the quantity of energy used to construct the piles must be highly standardized to produce piles with consistent performance capabilities.

3 History and Current Practice

3.1 History

The terms pressure injected footing, bulb pile, franki pile, and compacto pile are all terms that have been used to describe a type of foundation pile. This paper will use the term pressure injected footing (PIF) because it is considered that this term most accurately describes the installation and the soil to footing interaction.

The PIF was introduced to the engineering/construction world over 70 years ago by Edgar Frankignoul in Belgium. He speculated that a pipe could be driven into the ground by the impact of a drop hammer on a plug of compacted granular material located inside the bottom of the pipe. His theory was that when the plug was repeatedly struck by the hammer, large friction forces would be developed between the pipe and the plug and the pipe would subsequently be pulled into the ground. His idea proved correct and this method of driving a pile has since been used throughout the world.

Frankignoul considered the plug to be merely a device for advancing the the pile into the ground. When the appropriate depth was reached the plug was expelled by the drop hammer. Zero slump concrete was then rammed out of the end of the pipe as it was slowly withdrawn thus forming the shaft. No special attempt was made to form an enlarged concrete base at the bottom of the pile. This early PIF was essentially a friction pile. Today emphasis has shifted to the base of the PIF, and it is now regarded essentially as an end bearing pile. Consequently, the question of how to construct and evaluate the performance of the base is of paramount importance to the designer.

3.2 Current Practice

The sequence of PIF construction is illustrated in Figure 1. As can be seen, the construction sequence can be broken down into three operations: driving down, forming the base and forming the shaft.

3.2.1 Driving Down

The pipe, or drive tube, can be driven into the ground by either of two methods. The first is the same as Frankignoul's method previously described. After reaching the desired depth the drive tube is withdrawn slightly to reduce resistance at the tip, and the plug is expelled by repeated blows of the drop hammer. Some of the plug is left in the drive tube to prevent entry of foreign material. The height of the seal varies from 0 to 300 mm, depending on the soil conditions. The plug usually consists of zero slump concrete although any granular material will do. This method of driving the tube is termed bottom-driving. In the second method, mechanical pile driving hammers are used to drive the drive tube as though it were a pipe pile. This method is called top driving. An expandable steel closure plate or boot is fitted on the bottom of the drive tube to prevent entry of foreign material. After reaching the desired depth, zero slump concrete is placed in the drive tube and the steel boot is driven off with the drop hammer. As in bottom driving, an appropriate seal is left in the drive tube.

3.2.2 Forming The Base

After expelling the plug (bottom driving) or driving off the boot (top driving), the base is formed by ramming zero slump concrete out the end of the drive tube with the drop hammer. It is possible to do this as the seal offers only slight resistance to the expulsion of the concrete. The appropriate seal is maintained during this process by putting a clearly visible mark on the cable attached to the drop hammer so that the distance from the mark to the bottom of the drop hammer equals the length of the drive tube. By observing the position of the mark with respect to the top of the drive tube as the drop hammer impacts on the seal, a measure of the seal thickness is obtained. While insuring that the seal is always adequate, the ramming of concrete into the soil is continued until the energy required to ram a unit volume of concrete into the soil satisfies the design criteria. Details of the design criteria will be discussed in subsequent sections.

3.2.3 Forming The Shaft

The traditional method which is still popular today, is to withdraw the drive tube in a series of short steps while alternately ramming concrete out of the end of the drive tube, thus, leaving in the ground a shaft of compacted zero slump concrete. As with the base forming procedure, an appropriate seal is always maintained. The process continues until the desired cutoff elevation is reached. It is important to note that very large pressures are developed between the shaft and the surrounding soil during the formation of the shaft.

Another method of forming the shaft is a major departure from Frakignoul's original concept. After the base is formed, a steel casing is placed inside the drive tube to bear on the concrete base. A small charge of zero slump concrete is placed inside the casing and the drop hammer is then employed to compact the concrete which at the same time ensures that the casing is firmly seated in the base. The drive tube is then withdrawn, leaving the casing inplace, the casing is later filled with normal slump concrete. This method developes very small pressures between the shaft and the surrounding soil.

3.3 Advantages Of Using PIF

A diagram of a completed PIF is shown in figure 2. Several advantages of using PIF are discussed in the ensuing paragraphs.

By pulling a heavy walled steel tube into the soil with the driving action of the drop hammer, very little danger of contamination of the finished caisson exists. The steel tube with the plug at the base prevents any seepage of water or entry of soil into the caisson during driving down or forming the base. If there is entry of foreign material into the tube the operator of the piling rig is immediately aware of this and can take corrective action.

The application of extremely high energy blows in forming the expanded base has two advantages. It provides a very dense concrete bulb through which the column loads are distributed. Secondly expanding the base improves the bearing stratum by densification created by the high impact energy of the hammer during installation and the related expansion of the base.

Additional carrying capacity is developed during construction of the shaft. Maximum skin friction is developed by ramming the compacted concrete out against the soil, that has already been compressed by the driving of the tube, with blows of between 25,000 and 50,000 N-m of energy.

The effects due to local variations in the soil type and density are minimized by the general upgrading of the soil properties at each pile location as a result of the densification of the bearing stratum.

3.4 Terminology

The following terms are often used when discussing the construction of pressure injected footings (PIF):

Lead- Upright or tower of the piling rig.

Lead man- The key man on the crew. This man directs the rest of the crew and controls the amount of concrete used for each blow or number of blows used to form the base and shaft.

Cathead- Rotating drum used to raise and lower concrete, tools etc. by means of a cable wrapped around the drum.

Rigger- Operates cathead and assists lead man on the lead.

Hammer drum- Drum around which the hammer line is wrapped.

Operator- Operates hammer drum and motorized equipment on the rig.

Hammerline- Cable extending from the hammer, to the top of the lead and down to the hammer drum.

Wraps- The wraps of a cable around a rotating drum. The friction created by the tension of the cable on the rotating drum is the driving force used to raise objects.

Labourers- Usually two on a rig. Labourers work under the direction of the lead man. They work on the ground filling concrete buckets and guiding the concrete buckets up to the lead man, changing worn cables and doing any manual jobs required during the construction process.

False blows- Blows of the hammer, while making the base or the shaft, where too little or too much concrete is in the driving tube. The energy delivered by a false blow is not standard and cannot be used in any data that is used in calculations. False blows should still be recorded but labelled as such in the construction inspection record of the pile.

4 DESIGN

4.1 Static Design Method

4.1.1 Meyerhof: Static Method

Pressure injected footing capacity may be estimated by the analysis of the effective stresses surrounding the enlarged base bulb as indicated by the early work of Meyerhof (1959). The author relates an empirical, transient pressure/density relationship to a Terzaghi/Prandlt type of elastic-plastic failure surface. An iterative method balancing density change with (ϕ) angles is used to locate the slip surface about the pile tip.

This method is still commonly used in practice today. A brief explanation of this method will be outlined, followed by rules for its use and improvements that have taken place subsequent to the original paper.

Meyerhof (1959) proposed that the volume of plastic deformation of cohesionless materials under dynamic loading could be derived in compaction tests by equating the energy per blow to the work done in deforming the sample. Assuming an approximately parabolic pressure density relationship, the peak pressure, which would determine the plastic deformation and subsequent unit weight, was derived as follows:

(1)

$$P_p = \frac{3 \cdot W \cdot h}{A \cdot S} \times \frac{W}{W + P/2}$$

where:

P_p =peak pressure(kPa)

W =weight of hammer(kN)

P =weight of sample(kN)

A =area of sample(m^2)

h =height of fall of hammer(m)

S =permanent compression of sample (m)

As indicated by Whiffin (1953), the degree of soil compaction depends mainly on the intensity and duration of the pressure pulse. Field observations with various types of compaction equipment showed that for a given soil type and water content, the limiting value of the dry density by compaction is almost exclusively governed by the peak transient pressure induced in the soil. In cohesionless soils, the degree of compaction was found to be practically independent of the pressure pulse duration, but was enhanced by continuous vibration roughly in proportion to the additional local pressures resulting from the oscillator.

Figure 3 indicates that for dry Ottawa sand the dry density of the soil increases with the peak pressure at a decreasing rate and generally only a small increase was found beyond a peak pressure of about 3.5 kg/cm² (50 lb/in²) Confined compression tests show that impact pressures produce greater densities than corresponding static pressures.

Meyerhof (1959) proposed the following density-pressure relationship based on experimental compaction results:

$$D_r = D_2 - \frac{D_2 - D_1}{1 + 2.3\left(P_p/P_c\right)^c} \tag{2}$$

Where:

D_r =relative density of soil after application of pressure

D_2 =maximum relative density

D_1 =initial relative density

P_p =applied effective pressure

P_c =pressure constant derived from experimental results

c =compaction index derived from experimental results

Meyerhof also noted that for fully saturated sands the pressure needed for a given degree of compaction was greater than for a dry material due to the dynamic pore water pressures induced during compaction. For coarse cohesionless soils the pressure-density relationships for dry materials can probably be used in conjunction with the effective peak pressures. In addition the physical characteristics of the sand were found to affect the degree of compaction greatly.

The stress induced in the soil by the drop hammer is taken as the pressure producing compaction in the soil. Meyerhof (1959) assumed that the stress transfer to the soil, while the base is being expanded, will be similiar to the case of a deep circular pier being loaded to failure (Meyerhof, 1951). The major principle stress in the plastic zone is given by:

(3)

$$P_1 = \lambda \cdot K \cdot \gamma \cdot D \cdot e^{2\theta \tan\phi} \cdot \tan^2(45 + \phi/2)$$

Where:

γ =unit weight of soil

θ =angle between vertical and direction of stress

λ =shape factor

ϕ =reduced angle of internal friction allowing for compressibility of the soil

D =depth of base

K =coefficient of earth pressure

In the elastic zone the major principal stress is determined from the Boussinesq-Mindlin equations using the principal stresses calculated from Equation 3 and applied along the failure surface; this stress can be expressed by:

(4)

$$P_1 = \frac{3q \cdot B^2}{16 \cdot r^2} \cdot \cos\theta$$

Where:

B =width of base

r =distance from footing centre

The theoretical limits of the zone of soil compaction are at the points where the major principal stress ratio is equal to the coeficient of passive earth pressure.

Meyerhof (1959) states that in the plastic zones the major principal stress follows the path of the bisector of the angle between the radial and tangential slip lines, while in the elastic zone this stress acts radially from the centre of the base (Figure 4). Using Eqs. (3) and (4) the major principal stress can be computed at various points in the soil, and its value p1 is substituted for the pressure p in equation (2) to give the relative density Dr of the material at a particular point. The angle of internal friction (ϕ) corresponding to this computed value of Dr can be ascertained from laboratory drained shear tests on representative soil samples obtained from the site. If such results are not available, approximate values of (ϕ) and Dr can be deduced from the results of standard or static penetration tests, where it has been found (Meyerhof, 1959) that for sands approximately:

$$\phi = 28^\circ + 15\,D_r \tag{5}$$

Using the new values of (ϕ), the major principal stresses are recalculated to give a new set of relative densities and revised values of (ϕ). This process is repeated until the final stresses p1 correspond to the final (ϕ) angles.

For intermediate plastic zones the angle (ϕ) is taken to vary linearly with distance, from the top of the expanded base where the impact is applied, to the limit of the plastic zone. Meyerhof uses this assumption to derive modified bearing capacity factors ($N\gamma'$) and (Nq') as indicated in Figure 5. The equation to find the total bearing capacity of a PIF can be stated:

$$Q = q \cdot A + f \cdot S \tag{6}$$

where:

$$q = \frac{\gamma \cdot B \cdot N_\gamma'}{2} + K_b \cdot \gamma \cdot D \cdot N_q' \tag{7}$$

$$f = \frac{K_s \cdot \gamma}{2} D \cdot \tan \delta \tag{8}$$

q =unit base resistance (kPa)

f =average unit shaft friction (kPa)

A =area of base (m^2)

S =surface area of shaft (m^2)

γ =average affected unit mass of the compacted soil (kg/m^3)

δ =angle of skin friction with the compacted soil

B =diameter of base (m)

D =depth of base below ground level (m)

K_b =earth pressure coefficient on the shaft within the failure zone near the base. Varies between 0.5 for loose sand (ϕ about 30°) to about 1.0 for very dense sand (ϕ about 45°)

K_s=average earth pressure coefficient on shaft. Varies between 0.4 to 0.7 (refer to Figure 5 for values).

$N\gamma'$ and Nq'=bearing capacity factors for deep piers (D/B 10) depending mainly on the angle of internal friction of the soil in the failure zone near the base.

For a foundation depth D/B less than 10:

$$q = \frac{D}{10 \cdot B}\left(\frac{\gamma \cdot B \cdot N\gamma'}{2} + K_b \cdot \gamma \cdot D \cdot Nq'\right) \tag{9}$$

Note: In Equation 9, an embedment of D' into a cohesionless stratum underlying cohesive soil, the depth D' should be used instead of D in the factor outside the brackets but the total depth D remains within the brackets. This is to compensate for the decreased shearing resistance of the cohesive material within the failure zone.

Meyerhof suggests that in uniform cohesionless soils of various relative densities and not underlain by more compressible material at greater depth, for shallow bases up to about 6 m deep, bearing capacity frequently controls the allowable load. Settlement usually governs the design for deeper bases.

The denser the original cohesionless material, the less effect expanding the base of a PIF will have on the bearing capacity of a pile. Meyerhof's theory indicates that a PIF in loose sand would have a bearing capacity of approximately six times that of a pile with no compaction of the material around it or three times that of a driven pile.

Meyerhof's analysis and field observations indicate that for single piles formed in loose cohesionless soils the horizontal extent of the compacted zone along the shaft has an overall width of about six times the shaft diameter increasing to about seven or eight times the base diameter at a short distance below the base. The compacted zone extends below the base to a depth of about five times the base diameter as illustrated in Figure 6.

In a later paper, Meyerhof (1960) produced a design chart to simplify design estimates. The chart indicates the safe bearing capacity of the base of a single PIF in uniform cohesionless soils of various original relative densities and not underlain by softer soils at greater depths (Fig.7).

In estimating the safe bearing capacity of the base it has been assumed that the water table is at the surface. If the limit of the seasonal variation of the water table is at or below base level, the safe bearing capacity would be twice the values given in Figure 7. A linear variation of the bearing capacity between these limits can be used for other positions of the water table. The factor of safety against soil failure was taken as 5 in Figure 7, this is greater than the customary value of 3, however, at the time the Figure was constructed no tests to failure had been conducted.

The original relative densities of the soil is given in the upper part of figure 7 in terms of the average angle of internal friction, (ϕ), and the corresponding approximate limits of the standard penetration resistance (N blows per foot or blows per meter of penetration). The latter is usually more readily determined.

For very fine or silty sands below the water table, only one half of the number of blows of the standard penetration resistance above 15 blows per foot (50 blows per meter) is to be used when employing the design chart. In a later paper Meyerhof (1962) indicates that the standard penetration resistance of silty sands can be taken halfway between that of clean sand and silt. Meyerhof considers that a silty sand is defined as a sand of which less then 30 percent passes 0.06mm (approximately No. 200 USS sieve) and nothing passes 0.002mm. (clay size), silt is a soil of which more than 50 per cent passes 0.06mm and nothing passes 0.002mm. Materials with an intermediate grain size distribution can be interpolated between these limits, while very fine sands can be taken between clean and silty sands. More over, only silts of the rock flour type, having little or no plasticity (liquid limit less than 30 percent and plasticity index less than 5 percent) are included in the present estimates. The safe load on non-plastic silts can be determined by the rules for very fine and silty sands, while plastic silts should be treated by the methods used for clays considering that no compaction of the surrounding soil takes place as indicated by Terzaghi and Peck (1948).

The angle of internal friction of a silty sand and non-plastic silt is about 5 degrees less than that of a clean sand of the same relative density. To adjust for this in Figure 7, the values for the safe load of PIF bases in loose and compact sands should be used for compact and dense silty soils respectively. For loose non-plastic silts, where the angle of internal friction is less than 25 degrees, no values appear on the chart and additional estimates of the safe load are required. These can be calculated by the same methods used to calculate the bearing compacity in sands as outlined earlier in this paper. Figure 7 shows that the safe load on a PIF in silt should not exceed about one half of the safe load in sands of the same relative density unless greater loads are confirmed by load testing.

Meyerhof (1960) notes that a concrete batch before compaction has an approximate volume 25% greater than the concrete batch after compaction and this reduced volume of concrete should be used in calculating the size of an expanded base of a PIF. Nordlund (1982) indicates that the reduction in volume of zero slump concrete during compaction is closer to 10%. This reduced volume should be used in determining the size of base *insitu*.

After determining the appropriate angle of friction and the required base depth, the safe load on the base is then given in the lower part of Figure 7 for various base diameters.

As an alternative to use of SPT results, the base resistance can be estimated from the results of static cone penetration tests. The average penetration resistance of the soil in the failure zone near the base of PIF was found in field tests to be about twice that of cone penetration tests conducted beside driven piles. This can be attributed to the greater degree of soil compaction and corresponds well to the estimates of relative bearing capacity of driven versus expanded base piles. Since the point resistance of a driven pile is close to the average static penetration resistance in the failure zone, it follows that the unit base resistance of caisson piles is approximately:

$$q_p = 2\, q_c \tag{10}$$

where:

q_c = average original static penetration resistance.

The static cone penetration resistance (qc tons/ft^2 or kg/cm^2) of the soil is given on the middle line in figure 7. A factor of 4 has been used to determine the safe bearing capacity of the base.

If the relative density of the soil varies with depth, the average value in the theoretical failure zone should be used. The theoretical failure zone extends from a distance of about 4 times the base diameter above the base, to 1 base diameter below the base. Moreover, if purely cohesive soils overlie cohesionless soils, the total bearing capacity is given by the sum of the bearing capacity of the cohesionless soil (ignoring the cohesive overburden), as given in the upper part of Figure 7, and the corresponding bearing capacity of the cohesive overburden in proportion to its thickness. In this way the relative depths of the soil deposits in the theoretical failure zone are used to weight their contribution to the bearing capacity of the PIF.

The safe bearing capacity of a caisson is the safe load on the base or the structural load on the shaft whichever is smaller. The safe load on an uncased concrete shaft can be taken as one quarter of the cylinder failure stress multiplied by the cross sectional area of the shaft. For a cased shaft, the safe load on the steel shell (after some reduction for possible corrosion) can be added to that of the concrete.

In order to drive a pile or expel concrete from the base of a PIF and produce compaction of the soil, the energy per blow of the hammer must be great enough to overcome the ultimate bearing capacity of the soil near the base. The energy per blow of the hammer required for expanding the base and the number of blows per ft^3 or per m^3 of expelled base concrete, required for final soil compaction in order to obtain the above mentioned bearing capacity (Figure 7) can be estimated from Meyerhof's theory of the bearing capacity of deep circular footings. A design chart giving the final number of blows required is shown in Figure 8. In this diagram it has been assumed that the ground water level is half way between the surface and the base level and that after compaction the material is very dense at the base (final angle of friction (ϕ)=45°). If the water table is at base level the energy per blow of the hammer shown should be increased by 25 percent. If the water table is at the ground surface the indicated energy per blow of the hammer can be reduced by 25 percent. A linear variation between these limits of energy per blow can be used for other positions of the water table.

4.1.2 Application

The following are notes on the application of Meyerhof's theory on the design of PIFs.

1. SOIL: The theory is only applicable in soils that can be densified by transient energy. This includes clean sand, silt of the rock flour type (non-plastic) and combinations of these two soil types.

   Meyerhof gives a limit of applicability in silts by specifying that the liquid limit must be less than 30 percent and the plastic limit less than 5 percent. Whenever PIF are designed for soils outside these limits, the bearing capacity must be determined by standard methods of load distribution on the undisturbed soil at the base and the skin friction on the shaft.

2. WATERTABLE: The bearing capacity table designed by

Meyerhof assumes that the ground water table is at the soil surface. This is an extremely important aspect of the use of the tables and unless corrections are made the obtained values will be unduely conservative.

3. DEPTH OF BASE: The relation of the width of the base to the depth is very important especially at base depths 6 m or less. This must always be checked and corrections made as outlined in Section 4.1.1.

4. OVERLYING COHESIVE SOIL: If cohesive soil or loose fill overlies the cohesionless soils in which the base is to be formed, the total pressure under the base must be assumed to be the sum of the pressure due to the cohesionless soil plus an increment due to the overburden in direct proportion to each layers thickness.

5. UNDERLYING COHESIVE SOIL: Where cohesive soil underlies the cohesionless soil in which the base is formed, the safe capacity of the base is dictated by the load distributon on the surface of the cohesive soil. The load is assumed to be distributed on a circular area on the surface of the cohesive soil. This area being the base of a truncated cone whose diameter at the top is equal to the width of the base and whose sides are at 60 degrees from the horizontal.

Unless all the preceeding factors are known and fully considered in applying Meyerhof's theory, the results obtained could be in error and their use for estimating would be misleading with regards to the factor of safety of the base of the PIF.

4.2 Dynamic Design Methods

4.2.1 Meyerhof: Dynamic Method

Meyerhof (1962) also suggested that an approximate estimate of the safe load on the base of a PIF could be obtained from a formula which had been derived from the Hiley driving formula using a factor of safety of 5. The safe bearing capacity in tons is approximately:

$$Q_d = \frac{Wh}{\left(\frac{6}{n \cdot A_p}\right) + 0.1} \text{ (tons)} \tag{11}$$

Where:

Wh = energy per blow of hammer (ft-tons or m-tons)

n = number of blows per ft^3 or m^3 of concrete expelled from base

A_p = area of expanded base (ft^2 or m^2)

It may be noted that in the denominator of this equation the the first term represents the set of a spherical incompressible base of which the top remains at constant level and the second term is approximately one half of the temporary elastic compression (c) of the base and soil in inches (c=0.2 in. or 0.5 cm.) approximately, for hard driving. Actual field test values should be used where available. It should be noted that this formula is quite crude and should be used with caution and only when information is limited and precludes the use of Meyerhof's static or Nordlund's dynamic theories.

4.2.2 Nordlund: Dynamic Theory

Nordlund (1982) proposed the following derivation for the base resistance of a PIF. Nordlunds derivation relates the amount of energy required to expand the base of a PIF to the volume change in the surrounding soil. This energy relationship gives the stress on the surface of the base bulb. Nordlund then used this surface stress to predict the bearing capacity. The surface stress is modified by a dimensionless constant "K", which includes a factor of safety and a multiplier based on the type of soil the pile is based in and the type of pile shaft constructed.

To introduce energy into the system, the potential energy relation (force times distance) can be used. In terms of an expanding base sphere this is:(Fig. 9)

$$dE = F \times dr \tag{12}$$

Where:

E = energy

F = total normal force acting on the surface area of the base

r = radius of the base

Assuming that the base approximates a sphere, then:

$$F = P_u \times 4\pi r^2 \tag{13}$$

.

.

Where:

P_u = the pressure acting normal to the surface of the spherical base

Therefore:

$$dE = P_u \times 4\pi r^2 dr \tag{14}$$

.

.

Which implies:

$$\int_0^E dE = \int_{r_1}^{r_2} P_u \times 4\pi r^2 dr \tag{15}$$

.

.

Therefore:

$$E = P_u \left(\tfrac{4}{3}\pi r_2^3 - \tfrac{4}{3}\pi r_1^3\right) \tag{16}$$

.

.

.

$$E = P_u \left(V_2' - V_1'\right) \tag{17}$$

.

.

Where:

V_1' = volume of the base before enlargement

V_2' = volume of base after enlargement

To determine the input energy "E", the potential energy relationship is available:

(18)

$$E = e \times W \times H \times n$$

Where:

e =ratio of energy delivered to that theoretically available

W = weight of the drop hammer

H = drop height

n = number of blows

Combining and rearranging Equation 17 and 18 yields:

(19)

$$P_u = e \times W \times H \times \frac{n}{V_2' - V_1'}$$

Defining B' as the number of blows of "W" times "H" energy that is needed to ram a unit volume of compacted concrete into the soil, the above formula becomes:

(20)

$$P_u = e \times W \times H \times B'$$

Referring to Figure 9, the bearing capacity of the base may be estimated as:

(21)

$$L_u = P_u \times \pi r^2$$

Where:

L_u = the ultimate bearing capacity of the base

Combining Equations (20) and (21) gives:

$$L_u = e \times W \times H \times B' \times \pi r^2 \tag{22}$$

since:

$$\pi r^2 = \sqrt[3]{\frac{9\pi}{16}} \times (V')^{2/3} \tag{23}$$

Then:

$$L_u = e \times W \times H \times B' \times \sqrt[3]{\frac{9\pi}{16}} \times (V')^{2/3} \tag{24}$$

Since concrete is measured in the field by its bulk volume, it is desirable to use bulk volume rather than compacted volume in the formula. The relation between the bulk and compacted volume of concrete is:

$$V' = 0.9\,V \tag{25}$$

Where:

V' = volume of concrete after compaction

V = bulk volume of concrete

Similarly, in the field, the number of blows of the drop hammer required to ram out the concrete should be based on the bulk volume of the concrete. Therefore:

$$B' = B/0.9 \tag{26}$$

Where:

B = number of blows of "W" times "H" energy needed to ram a unit volume of bulk concrete into the base

NOTE: As it is not possible, in the field, to measure unit volumes of concrete, "B" is often taken as the average number of blows per cubic foot required to ram out the last batch of concrete (1 batch being 5 cubic feet or 0.14 cubic meters).

To produce an allowable working load, the ultimate working load is divided by a factor of safety:

$$L_w = \frac{L_u}{FS} \tag{27}$$

Where:

L_w = allowable working load

FS = factor of safety

Combining all of the above equations produces the expression for the allowable working load for the base of a PIF:

$$L_w = \frac{e \times (0.9)^{2/3} \times \frac{1}{0.9} \times \sqrt[3]{\frac{9\pi}{16}}}{FS} \times \left(W \times H \times B \times V^{2/3}\right) \tag{28}$$

Or:

$$L_w = \frac{W \times H \times B \times V^{2/3}}{k} \tag{29}$$

.

.

Where: K= a dimensionless constant (30)

$$K = \frac{FS}{e \times (0.9)^{2/3} \times \frac{1}{0.9} \times \sqrt[3]{\frac{9\pi}{16}}}$$

Recommended values of "K" for different soil and shaft types are given in Figure 10. If a load test is conducted at the site a value of "K" can be back-calculated from the results. This value of "K" can then be used in subsequent calculations to determine the required number of blows of the last batch, for various base sizes.

Nordlund (1982) indicates that load tests have shown that the "K" values given in Figure 10 will produce factors of safety of 3 to 4 for a cased shaft in residual soils and 2 to 3 for all other conditions. This method has been used extensively in the United States and has been established in the building codes of the U.S. Navy, Army, Federal General Service Administration and in North Carolina.

4.3 Design Example

4.3.1 Background

As with all design techniques, care must be taken with PIF design to ensure that the boundries within which the technique is applicable are adhered to. In the following example the soil composition precludes the proper use of Meyerhof's static and dynamic design formulations as descibed in Meyerhof 1959 and 1960 respectively. Numerous design examples of Meyerhof's static and Nordlund's dynamic design methods are given in the papers Meyerhof 1959 and Nordlund 1960 respectively.

The data used in the following design example is taken from a pile load test conducted at Keephills Alberta for the foundation of a thermal power plant. The pile load test was conducted on December 2, 1977 by Hardy Associates 1978 Ltd.. Additional soils data was taken from a site investigation conducted by the same company in 1976, (Hardy Assoc. 1976). The following data is required in the design:

1. Depth to base =12.8 m
2. Depth to watertable =6.09 m
3. ϕ of basing material =35 degrees
4. Soil gradation:
   - 42% sand
   - 42% silt
   - 16% clay
5. The average standard penetration test blow count for the material in the failure zone was 49.2 blows/meter (15 blows/foot).
6. For the soil around the shaft of the pile, which was constructed by the compaction method, the average effected unit weight of the soil can be taken as being equal to 1762 kg/m^3, Ks for this material can be taken as 0.5 and δ as equal to 20°.

7. A hammer with a mass of 3265.9 kg and a drop of 6.1 m was used to expell the concrete to form the base.

8. The base was formed by extruding a granular driving plug of 0.2 m^3 volume and 5 buckets of zero slump concrete of 0.14 m^3 each. The blows per bucket were:

   1st bucket: 14 blows

   2nd bucket: 13 blows

   3rd bucket: 14 blows

   4th bucket: 16 blows

   5th bucket: 21 blows

9. The test pile failed at a load of 408,200 kg (900 kips).

4.3.2 Computation

4.3.2.1 Meyerhof Static

The following points illustrate the steps that must be followed to calculate the allowable bearing capacity of a PIF using Meyerhof's static method.

1.

As indicated in Point 8 of the previous section, "Background", the bulk volume of the material used to form the base of the pile was 0.90 m^3. Assuming a subsequent compaction of 25%, the insitu base volume would be 0.675 m^3. This would imply a spherical base of 1.00 m^2 diameter.

2.

Applying Figure 7, which is a graphical presentation of Meyerhof's (1959) design method, to the preceeding data indicates an allowable base load of 208,700 Kg (230 tons).

3.

Applying modification 2 as set out in section 4.1.2: "Application"; as the water-table is half way between the base and the surface, we multiply the value obtained from Figure 7 by 1.5. This gives a total base resistance of 313,000 Kg.

4.

To determine the shaft resistance according to Meyerhof's static formulation we apply Equation 8. Using the previously stated data we arrive at an ultimate shaft resistance of 39,000 Kg. Using a factor of safety of 5 (comenserate with the value of the factor of safety used in the design chart for the allowable base resistance, Figure 7). An allowable shaft resistance of 7,800 Kg is obtained.

5.

Adding together the allowable base resistance and shaft resistance derived from Figure 7 and Equation 8 respectively. An allowable pile load of 320,000 Kg is calculated.

4.3.2.2 Meyerhof Dynamic

1.

Applying Formula 11 to the background information gives an allowable pile load of 318,000 Kg is obtained.

4.3.2.3 Nordlund Dynamic

1.

To use Nordlund's design technique, a value of "K" must be decided on. Looking at Figure 10; till with granular matrix, compacted concrete shaft, a suitable value of "K" is be found to be equal to 20.

2.

Applying Equation 29 to the above stated variables and and using a recommended "K" value of 20, an allowable working load of 137,400 Kg is calculated.

4.3.3 Discussion of Results

TABLE 1

DESIGN METHOD	ALLOWABLE LOAD (Kg)	FACTOR OF SAFETY
Meyerhof Static	320,000	1.27
Meyerhof Dynamic	318,000	1.28
Nordlund Dynamic	137,000	2.97

Comparing the calculated allowable loads of the three design methods in question to the actual failure load of the test pile, it is found that Meyerhof's static method yielded an actual factor of safety of 1.27 (ie. 408,200/320,000). Meyerhof's dynamic formula yielded a similar actual factor of safety of 1.28 (ie 408,200/318,300). Nordlund's dynamic method yielded an actual factor of safety of 2.97 (ie. 408,200/137,400). Clearly Nordlund's method proves to be quite accurate in this case, while both of Meyerhof's methods proved to grossly over estimate the allowable load of the test pile.

This particular example was chosen to illustrate the effect of a clay content above the level recommended by Meyerhof as the boundary to the applicability of his design procedure. The soil that the base of the test pile was formed in had an excessive clay content. As indicated in numerous design examples in Meyerhof (1959), Meyerhof's static design method can provide sufficiently accurate design estimates if the boundaries to the techniques applicability are followed. No test results indicating the accuracy of Meyerhof's dynamic design method were found and it is therefore suggested that this method only be used when no other methods can be used due to the limited availability of soil information.

As indicated in the above example, Nordlund's dynamic design method can provide useful design data. Nordlund's technique has been used extensively (Nordlund 1982) over a wide range of soil types. Several soil types and two shaft configurations along with corresponding empirically derived "K" values are given in Figure 10.

To conclude this section I must emphasize the prudence of conducting pile load tests. The large influence that the insitu soil conditions have on the bearing capacity of PIF dictate that where ever economcally feasible, on site pile load tests should be conducted.

5 CONSTRUCTION PROBLEMS

5.1 Heave

Pressure injected footings are usually installed in soil profiles that are dominantly sands or gravels. However in Western Canada this pile type has often been used in cohesive soils where the shaft is driven through lacustrine clays and silts and the base is formed in sand layers or possibly in dense clayey till. As outlined by Hagerty and Peck (1971) when piles are driven into clean granular soils, heave is likely to be small. The soil displacement caused by the intrusion of the pile into the soil being taken up by volume change in the soil. However, significant vertical or horizontal heave can occur when piles are driven into fine grained soil deposits. The analysis carried out by Hagerty and Peck (1971) indicates that the major factors influencing the degree of heave were the clay content as it relates to the permeability of the soil, the driving sequence of the piles and the geometry of the pile layout pattern.

The vertical differential heave of the soil, when installing PIF, can cause the shaft of the pile to separate from the base of the pile. The junction of the shaft and the base being the region in the pile were the reinforcing in the shaft does not provide sufficient tensile resistance. Hence the structural integrity of the pile is destroyed and the bearing capacity of the pile is reduced to that of the skin friction on the shaft of the pile. High soil or water pressures or differential vertical heave can also destroy the competence of fresh concrete within the shaft.

In investigating several case histories , Clark, Harris and Townsend (1979) have shown that most of the damaging differential heave takes place when the tube of an adjacent pile is driven into the ground. Damaging heave of adjacent piles while driving out the plug or forming the base of a PIF has proven not to seriously affect pile capacity as it tends to heave the entire adjacent pile and does not exert substantial differential heaving forces. The differential heave of surrounding piles associated with forming the shaft of a PIF has not been found to be significant.

The potential for shaft uplift and separation from the base can be assessed using the concept of effective stress. During the driving of the casing there is an upward displacement of the soil which tends to pull the shafts of adjacent piles upward. The data and theory indicate that this effect, which depends on the pile spacing and soil properties, decreases with increasing depth of driving. At some depth a condition of plane radial strain exists and driving the casing below this depth produces no significant upward displacement of adjacent pile shafts. For the 500mm to 600mm shaft diameters commonly used in Western Canada, this depth has been determined, by observation to be in the range of 8 to 12 pile diameters and corresponds to the critical depth described by Meyerhof (1976) for the maximum pile shaft resistance. Thus if the uplift on the shaft of the pile above the critical depth exceeds the ultimate shaft capacity below this depth plus the structural strength of the pile shaft in tension at its connection to the base, the shaft will separate from the base.

If such calculations indicate that shaft uplift could occur, then positive measures to reduce uplift are required. Such measures could include preboring or the formation of a tension base which consists of increasing the shaft diameter in the area where the shaft connects to the base. To avoid damage to piles in which the concrete has not had time to attain structural competency it is a general rule that adjacent piles within 9 diameters should not be driven within the same 24 hour period.

As indicated by Hagerty and Peck (1971), who also proposed a method of estimating pile heave, piles should be driven in a sequence from the center of a foundation area outwards or from oneside of a foundation to the other. A driving sequence that starts at the perimeter of a foundation area and works toward the center tends to confine the soil and increase vertical pile heave.

In areas where it is critical that the piles be structurally sound Fellenius (1981) indicated that it may be more prudent to install driven piles rather than a PIF type of foundation as driven piles are less prone to construction difficulties and may perform in a more consistent manner.

To facilitate analysis of the possible damage done to adjacent piles while installing a PIF, accurate survey records should be maintained on the heave of adjacent piles. Elevations of adjacent piles should be taken before driving the tube, after forming the base and after forming the shaft. Generally piles that heave less than 25mm when the tube of an adjacent pile is driven may be considered to be sound. Piles that heave more than 75mm can be considered to be probably damaged.

To determine if separation in a pile has occurred, a sonic test, as outlined by Clark (1979) may be performed. If damage has occurred to piles as indicated by the sonic tests, load tests will have to be conducted on representive piles and appropriate rehabilitative measures taken. To enhance the feasibility of any rehabilitation program it is essential that accurate reccrds of the construction and inspection be maintained.

5.2 Vibrations

A problem with the installation of any driven pile is the vibrations set up in the soil and the related damage to buildings that can occur. Because of the large energy used in constructing pressure injected footings, significant ground vibrations can be set up. The problem of vibrations due to pile driving has been studied extensively. Several tests procedures and correlations are available.

As reported by DeVos (1973) building damage has been successfully correlated to ground vibration by using the methods outlined by Crandell (1964). Crandell suggests that the vibrations in the ground can be measured in terms of the displacement, velocity or acceleration. These components of vibration can be measured easily using a three component seismograph. Empirical evidence suggests that the best correlation appears to be between observed ground velocity and structural damage.

Correlatons of building damage to ground vibration are made in terms of the energy ratio (ER).
Crandell defines the energy ratio to be:

$$ER = \frac{a^2}{F^2} \tag{31}$$

Where:

a =acceleration in ft/sec²

F =cycles/sec

$\frac{a}{F}$ =the peak velocity of the surface particle

The frequency used corresponds to the major component of the amplitude. The resultant velocity is the vectorial combination of the maximum transverse, vertical and longitudinal component velocities having regard to a limited time or phase interval. This method will yield an apparent "worst case" energy ratio and will therefore give a conservative result.

Crandell in his study of damage to buildings from blasting vibrations, determined that below an energy ratio of 3.0, damage will not occur to soundly constructed and properly maintained buildings. Several building jurisdictions indicate that for construction vibrations to be acceptable the energy ratio in the vicinity of structures must be less than or equal to 1.0. Studies conducted by Hardy Assoc. (1978) Ltd. indicate that an energy ratio of 0.07 corresponds to the vibration condition which will dislodge or crack extremely poor and already cracked plaster. This level can be taken as a threshold below which new architectural damage will not occur.

6 INSPECTION

6.1 Construction Inspection

6.1.1 Driving Procedure

To commence the driving process, the tube is lowered to the ground surface. A chalk mark is placed on the hammer line even with the top of the tube. This enables the lead man to know where the hammer is with respect to the bottom of the tube at all times during pile construction. The hammer is raised and a bucket containing gravel or zero slump concrete is used to drop sufficient material in the tube to form the bottom plug. The hammer tamps the material with increasing amounts of energy until the plug is firm and starts to pull the tube into the ground. By watching the hammer line mark the operator and/or lead man can determine if the plug is holding and how much of the plug remains in the tube. The lead man adds material during driving if required. An experienced lead man will gauge the expulsion of the plug material so as to maintain a minimum of plug material in the drive tube at base elevation. The lead man should not allow the plug to be completely driven out of the tube. The amount of plug left in the tube is determined by the soil and ground water conditions. If the tube is allowed to sit after it has been driven to base elevation water may seep into the tube due to permeability of the plug material. If this happens new plug material is added and the tube is driven to a new base elevation. This may be a problem if the zone of material that is acceptable to produce the base in is thin.

6.1.2 Driving Inspection

Before start of the work, the tube and hammer lengths are measured. The tube should be marked in one quarter meter increments, from the bottom to the top, with each full meter numbered. A reference point should be made on the lead 0.25 meters above ground level. All measurements should be taken from this reference mark. Locating the reference point above the bottom of the lead eliminates measuring problems created by ground movement and spilled concrete around the tube at ground level.

The inclination of the drive tube to the vertical or the desired angle of incidence, should be checked before driving commences. Often the lead man may tilt the tube backward slightly, as the motion of the hammer inside the tube will straighten the tube to vertical as driving proceeds. Each rig is different and only experience with a particular rig produces the desired results.

By having the tube marked in quarter meter intervals before driving commences, the inspector can count and record the blows per quarter meter intervals required to drive the tube to the base elevation. This information can be plotted and compared to SPT or static cone information, obtained during drilling, for correlation of penetration resistance and soil types. Driving information can also be used as a criteria to determine the depth required to form a competent base. The driving data is used to provide soil information between testholes.

It is necessary to watch the hammerline mark to determine if the plug is slipping too fast for an accurate blow count to be obtained. If the material the pile is being driven into is very hard and driving is slow (40 or more blows per quarter meter). The inspector can suggest predrilling with an auger diameter equal to the tube diameter. If the tube bounces during driving (this usually only occurs for the first meter of driving) tie downs or pull downs can be suggested. The major problem with the tube bouncing at the start of driving is that the tube may jump out of alignment, interfering with clearances, etc. in latter phases of construction.

When the base elevation is reached, the tube may be pulled up a few centimeters by means of two block and tackel devices attached to plates at the top of the drive tube which are often referred to as "ears". The plug is then partially driven out while the tube is held at a constant elevation. If the plug is tight the tube may be raised and lowered a few centimeters, several times, to keep the hole open under the tube and to maintain the base elevation.

The hammerline mark must be observed when the block and tackle devices are being attached so the length of plug can be estimated. The tube should not be lifted more than the length of the plug while driving it out. If the plug is driven completely out of the tube, foreign material may slough into the tube producing a dramatic softening in the sound of the hammer striking the plug material. Water spraying out the top of the tube also indicates problems. If this happens, driving must stop immediately and approximately a meter (as measured in the tube) of gravel or zero slump concrete should be poured into the top of the tube. This material should then be struck several times to plug up the base of the tube. The base zone has now been softened to an unexceptable degree. This zone must be driven through and a new base zone located.

6.1.3 Base Construction

After the plug is driven out of the tube the base is formed. The tube is held rigid and a small charge of zero slump concrete is dropped into the tube. The hammer is dropped a set distance that corresponds to a predetermined energy level, dependent on the weight of the hammer. This drop height generally stays the same throughout base construction unless stage construction of the base is employed in which case the drop height varies being smaller at the start of base construction and larger at the end. The use of this stage construction procedure is to avoid what Meyerhof (1963) termed overcompaction of the soil. This overcompaction of the soil refers to the soil in the failure zone around the base going to residual strength rather than remaining at peak strength. Considering the large soil strains during base expansion, it is questionable if this concept has any validity.

The input energy per blow is the drop distance times the hammer weight. The total input energy is the number of blows times the input energy. The lead man determines when to add more concrete by watching the hammer line mark. It is very important that the lead man uses the minimum possible plug while forming the base. If the amount of plug used is too great, hammer energy will be lost by wedging the material against the side of the tube rather than against the base soil. This should be considered in the design formulations and extra blows of the hammer should be added to the design blow count. When the base is partially constructed the need for a plug should be eliminated as the zero slump concrete used to form the base material will seal out all foreign material.

The concrete can be considered expelled when the hammer line mark indicates that the base of the hammer is striking just below the base of the tube. Specified energy inputs may be in terms of energy per blow or total energy, for a given volume of concrete, usually $0.14 m^3$. After completion the base will have a small pocket in the top due to the shape of the bottom of the hammer.

6.1.4 Base Inspection

If the tube jumps or penetrates further into the ground while the base is being formed, the concrete is not being expelled from the tube in an acceptable manner and the blows should not be counted as contributing to base energy input. If the tube starts to rise during base construction, the cause may be one of the following; the block and tackel lifting device, used to hold the tube stationary while the plug was being removed, may be under tension, or the soil surrounding the base has such a large resistance to displacement that the only way an increase in volume can be allowed is by forcing the drive tube up. The former is not desirable but the latter may be. In a case where the energy exerted on the concrete is lifting the tube, the depth, pore pressure build up/permeability of the soil and the insitu density of the base soil as it relates to the required basing energy must be revaluated to ensure that the required properties of foundation will be insured.

The maximum energy input for base construction should not exceed a certain value or pulverization of the zero slump concrete will result. A good rule of thumb maximum is 6,100,000 N-m per 0.14m^3 of concrete. For example, forty blows of a 6.0 m drop with a 2500 kg hammer gives a total energy input of 6,000,000 N-m. Rather than exceed 40 blows, it is better to lift the tube slightly after 30 blows and continue base construction. This is acceptable as the capacity of the base of the pile is near its maximum and an attempt to increase the compaction of the soil around the base or increase the size of the base is futile.

If the hammer line mark rises with respect to the top of the tube while concrete is not being added, the soil at the base is pumping back into the tube. If this condition continues past the first few blows, the soil conditions are not adequate for basing , or a condition of high water pressure exists at the bottom of the tube. A new plug should be placed in the tube and the tube driven to a greater depth where competent soil or favorable water pressure conditions are present. If the tube is too short to be driven to a deeper depth an alternative solution may be to leave sufficient plug in the tube to prevent water intrusion and shut down for 15 to 25 minutes. By doing this the water pressure at the base level may dissipate to a point where normal base forming procedures can be continued.

If the specified input energy can not be obtained within 5 base buckets of 0.14 m^3 the soil at the tube base is not competent for a base and the tube should be driven deeper to more competent soil.

The hammer line mark, while forming the base must be observed to ensure that the hammer is being used to form the base properly and is not punching a hole below the tube base. If a hole is being punched, the blows are known as false blows and are not part of the base construction input energy. The base formed using false blows has a lower volume of concrete per specified energy level. In other words an operator may add a few more blows to the last bucket of a base to bring it up to the specified energy level for that particular size of base. Thus not having to use another bucket of concrete to form the base but producing a base that will not be up to the specified size versus energy level criteria.

It is very important that the blow count increase for each additional 0.14 m^3 bucket used for construction. If the blow counts do not rise this may indicate that no volume change in the soil around the base is occurring. This generally indicates high pore pressures are developing around the base. However, another explanation is that the asymptotic expansion pressure of the base as described by Ladanyi and Johnston (1974) has been reached. Without piezometric data it would be impossible to tell which one of these conditions is occurring, therefore, if this happens the tube should be driven to a deeper elevation where the soil is more competent and the pile should be rebased.

Periodically the base of the hammer should be checked for wear. The piling process tends to wear the end of the hammer into a conical shape. If the end becomes too pointed the energy vector of the hammer will be diagonal rather than vertical. This will create a situation where the material inside the tube wedges against the sides of the tube rather than the soil at the bottom of the drive tube. The end of the hammer can be built back up to a cylindrical shape by welding material onto the hammer or by cutting the point off. Both of these procedures are slow and generally require that the piling rig be shut down for a considerable length of time. For this reason it is best to check the bases of the hammers at regular time intervals that correspond to breaks in piling, so that the hammers can be repaired, if necessary, during this break. If the amount cut off the hammer is substantial, the required blow count to base size relationship will have to be modified to take into account the lighter hammer weight.

6.1.5 Shaft Construction

After the base is constructed, the hammer is lifted out of the tube and the reinforcing steel cage is lowered into the tube. The base of the cage should be made up with the bars bent inwards and welded together (NOTE: The rebar must be of weldable quality). The hammer is put back in the tube and dropped in the cage from a meter or two to seat the cage base in the base concrete. The hammer is then lifted and a small charge of concrete is dropped in the tube. The hammer is then lowered to rest on the concrete. The tube is raised a few centimeters and the hammer dropped to expell the concrete and pack it outwards through the cage and against the soil. The energy per unit length of the shaft may be determined and specified, however the accuracy of any bearing capacity calculations based on the energy used to make the shaft is questionable due to the vagueness of the theories relating soil shear strengths with soil pressures.

The actual soil pressure on the shaft would be a function of the ground reaction at a given depth. A drop of approximately 1.5 m to 2.0 m is normally used. It is good practice to use more blows for shaft construction next to the base than for the upper part of the shaft.

Tension or uplift piles, used to resist heaving pressures, are constructed with the same procedure for the base and a gradual change from base to shaft construction while placing the first 0.28 m^3 of concrete above the base. To accomplish this the tube is raised in gradually increasing increments while the number of blows per increment of withdrawal is decreased. The purpose of this exercise is to create a conical section between the expanded base and the smaller diameter shaft. In this way the reinforcing steel is well seated in the lower and larger diameter section of the shaft ensuring that the shaft will not separate from the base if it is subjected to tension.

With a tension or regular based pile, the shaft is then continued to the ground surface or the required cut off elevation below the ground surface. The top is given one or two extra blows to provide a hard and relatively smooth top that can be trimmed by the removal of the circular edges extending around the hammer indentation.

6.1.6 Shaft Inspection

The cage should go into the tube as soon as the base is completed and be well seated in the base pocket. Forming a false shaft by building the shaft up before the cage is installed is not desirable. The input energy used in shaft construction should be used to pack and expand the shaft against the soil not against the tube. If too large a plug is used while making the shaft the cage may become wedged inside the tube and when the tube is lifted the cage may come up with the tube, separating it from the base. The amount of plug used while making the shaft can be determined by watching the level of the hammerline mark above the tube. A contractor may want to use a larger amount of concrete per blow than is desirable when making the shaft as this increases the construction rate.

If the piling is carried out near an excavation, there should be sufficient soil on all sides to prevent "blow out" of one side. If the rebar cage is to small or sticks on the hammer, it will either come up with the hammer or be crushed. If it has been crushed the results will be obvious at completion of the pile. The rebar cage will be totally missing or one or more of the reinforcing bars will be missing. Often the lead man can hear the cage being crushed or can feel the hammer being caught in the crushed cage. If the lead man acts before the the cage is too badly damaged the cage may be saved and another cage put in the pile and piling continued. If the cage has been totally crushed inside the pile it may be necessary to abandon the pile.

6.2 Inspection Of Zero Slump Concrete

Zero slump concrete can have very high strength with a relatively lean mix. One of the main factors in the quality of the concrete is the quality of the aggregate and sand used. A good design mix consists of a well graded aggregate and sand with a maximum size not exceeding 25mm diameter, not less than 6 bags of cement per cubic meter and 14 to 18 liters of water per bag of cement.

Zero slump concrete, unlike slump concrete, can be visually examined for quality with a fair degree of accuracy. The aggregate can be visually examined for gradation. Rubbing the surface of the larger pebbles will indicate cement content. If the larger pebbles are completely coated with 1mm to 2mm of cement and fine sand the cement content will be acceptable. Forming a patty of the zero slump concrete by compacting it by three or four blows of a cupped hand can indicate the moisture content of the concrete. At the proper moisture content the patty of concrete will stay together when tossed once or twice approximately 10cm in the air. The surface of the patty should remain dull with no water appearing on the surface.

Segregation of coarse and fine materials in the ready mix truck must be watched for. If the materials are separating in the truck they will be even more separated when the concrete is dumped into the pile. Excess fine or coarse material will reduce the strength of the concrete. A dirty drum or a build up of fines on the blades in the drum will result in segregation. Trucks should be inspected for cleanliness each day. Blades should be replaced when they start to show signs of wear and the dried on concrete should be removed from the inside of the trucks by a hammer and chisel at the end of every day. This is more work than is generally required by concrete truck operators and the entire procedure should be explained to the operators before the first batch of concrete is delivered. A dirty drum may also cause "balling" of the concrete, this also produces a lower strength concrete as the balls tend to have a higher moisture content than the unballed concrete.

Concrete that is too dry will puff or powder under compactive energy and will not be pliable enough to form around the reinforcing cage leaving areas of the cage exposed and prone to deterioration from corrosion. Concrete that is too wet will not pack well because of excessive pliability and incompressibility due to its high water content. A form of modified Proctor test can be conducted on the zero slump concrete to determine its optimum water content. Concrete that is too wet will stick to the tube and the hammer. The moisture content of zero slump concrete is generally around 5 to 8 percent, the moisture content must be over 3 percent to hydrate the cement in the mix. Some evidence is available that suggests that concrete with less than 3 percent moisture content will hydrate from drawing the necessary moisture from the surrounding soil.

The sound of the hammer striking the concrete will be different when striking wet, dry or good concrete. The maximum time zero slump concrete should be used after batching is dependent on temperature. If the temperature is above 5°C, the maximum time that the concrete is usable after batch is three hours. If the concrete temperature is below 5°C the concrete should not be used. This condition will occur if the air temperature is very low. If the zero slump concrete is stored in buckets or piles the temperature should be observed before using. In cold conditions aggregate temperature can be increased by using hot water while batching.

7 CONCLUSION

Pressure injected footings provide a high capacity foundation and have been used in Canada for over 40 years (Clark, Harris, Townsend, 1979). During this time, design and construction experience have produced effective techniques that deal with Canadian soil conditions and compliment construction practices.

The design example in section 4 illustrates that the bearing capacity of a single PIF can be estimated with acceptable accuracy. Standardized and formalized construction practices and inspection procedures, as specified in Section 6, will increase the quality and consistency of performance of a PIF foundation. A section recommending inspection recording procedures is included in the Appendix. Construction problems that occur during the installation of PIF must be delt with at the time of their discovery. Problems that are left unattended exaggerate in importance and the cost of rehabilitation. Some common problems and their solutions have been investigated in Section 5.

Diagramatic Illustration of Steps to Install

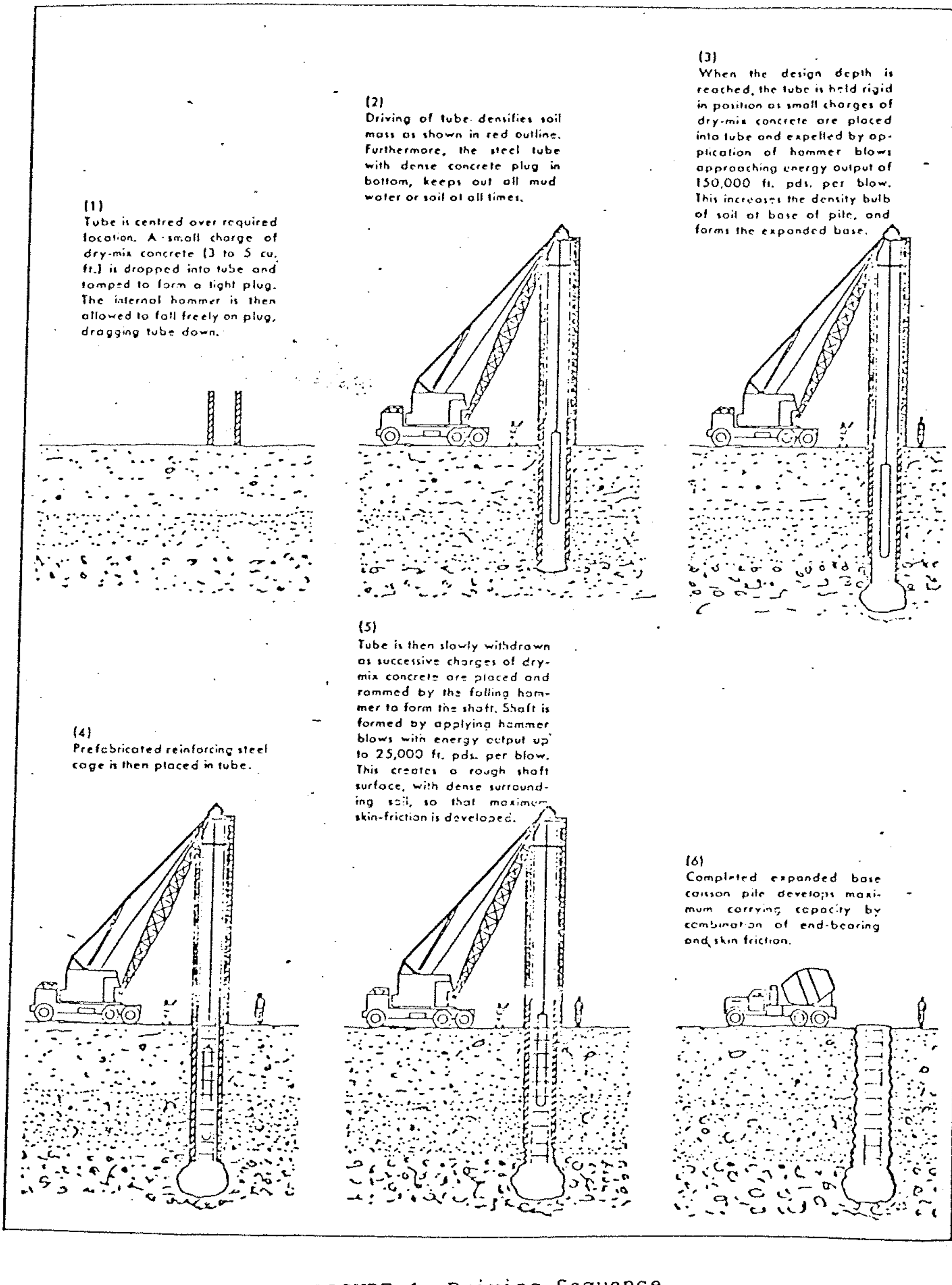

FIGURE 1: Driving Sequence

Monenco Ltd.

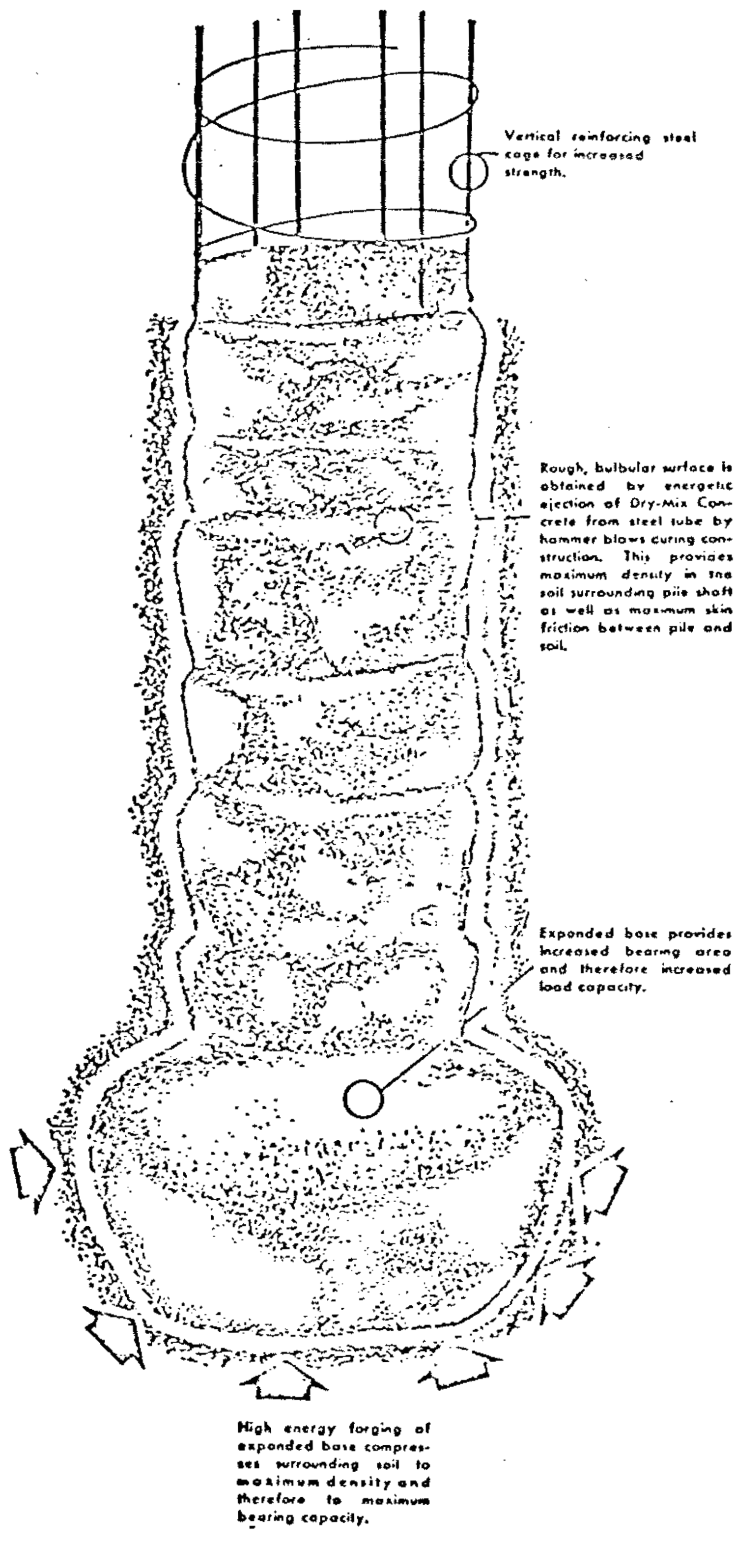

FIGURE 2: Cutaway View Of A PIF

Monenco Ltd.

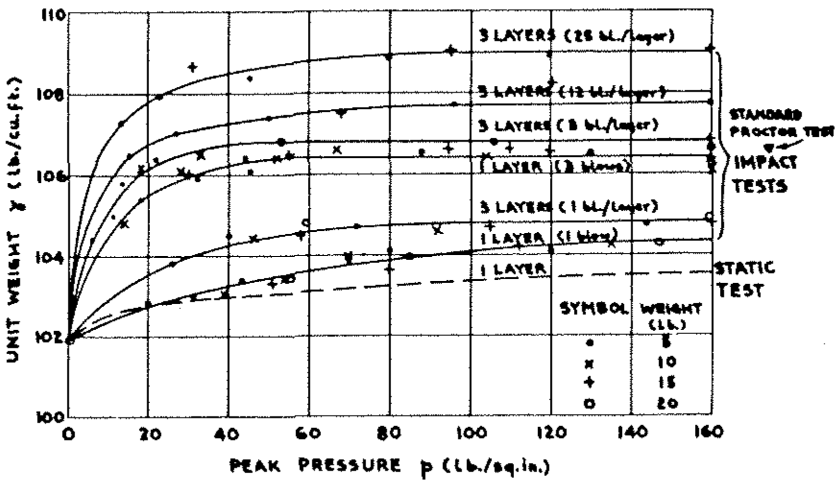

FIG. RESULTS OF COMPACTION TESTS ON DRY OTTAWA SAND

FIGURE 3: Results Of Compaction Tests

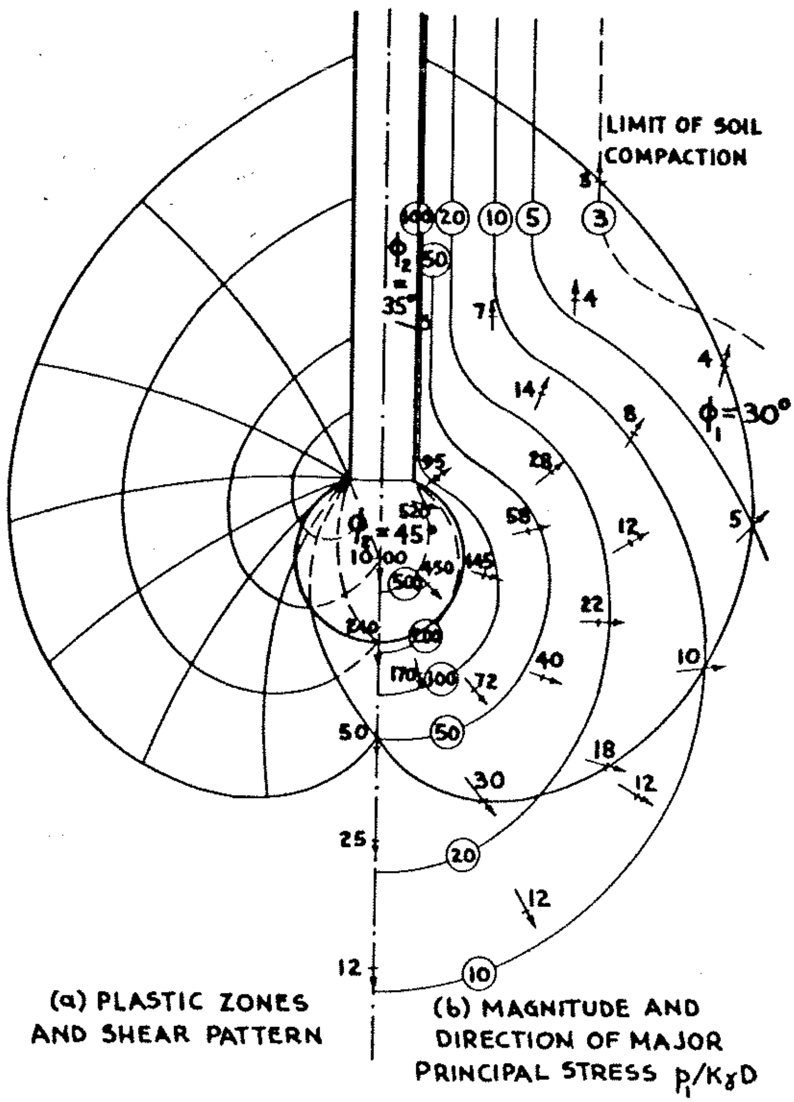

FIG. 4. PLASTIC ZONES AND MAJOR PRINCIPAL STRESS AFTER EXPANDING BASE OF CAISSON

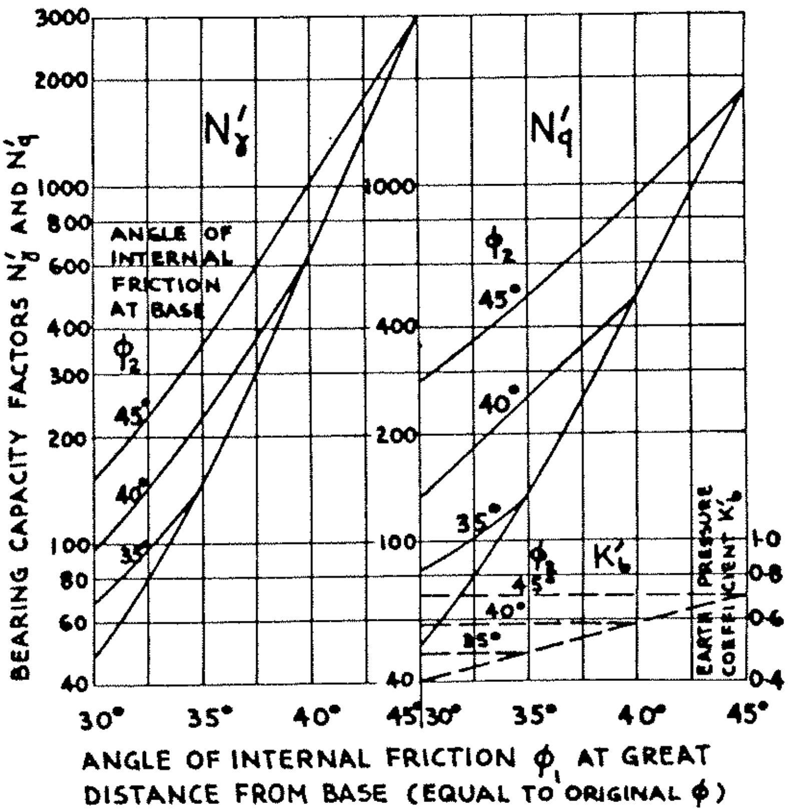

FIGURE 5: Bearing Capacity Factors For PIFs In Cohesionless Soil

Meyerhof 1959

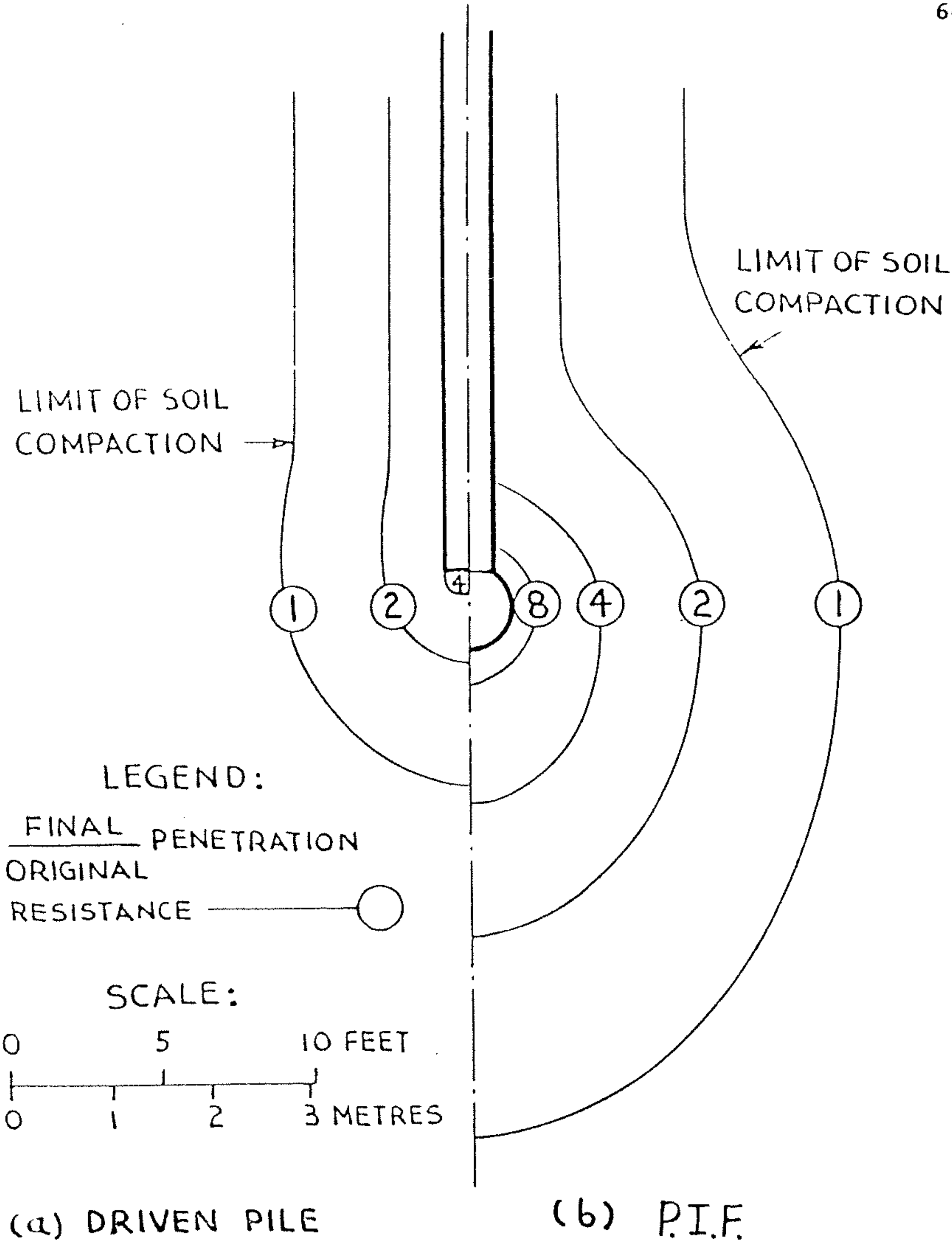

FIGURE 6: Compaction Of Loose Sand Near A PIF

Meyerhof 1964

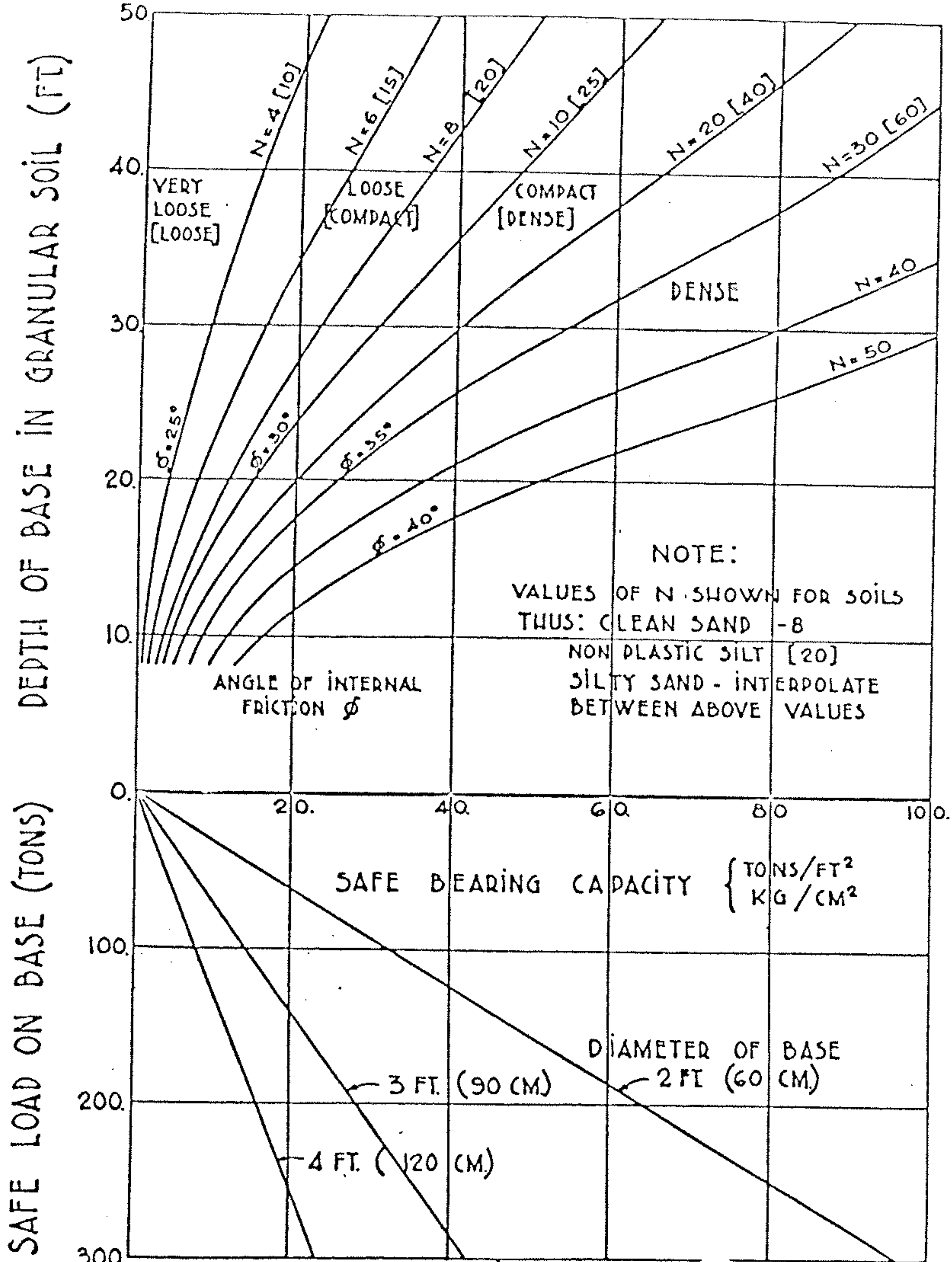

FIG. 7

SAFE LOAD ON SINGLE CAISSON PILE IN COHESIONLESS SOILS

NOTE: THIS FIGURE TO BE READ IN CONJUNCTION WITH FIG. 8

Meyerhof 1964

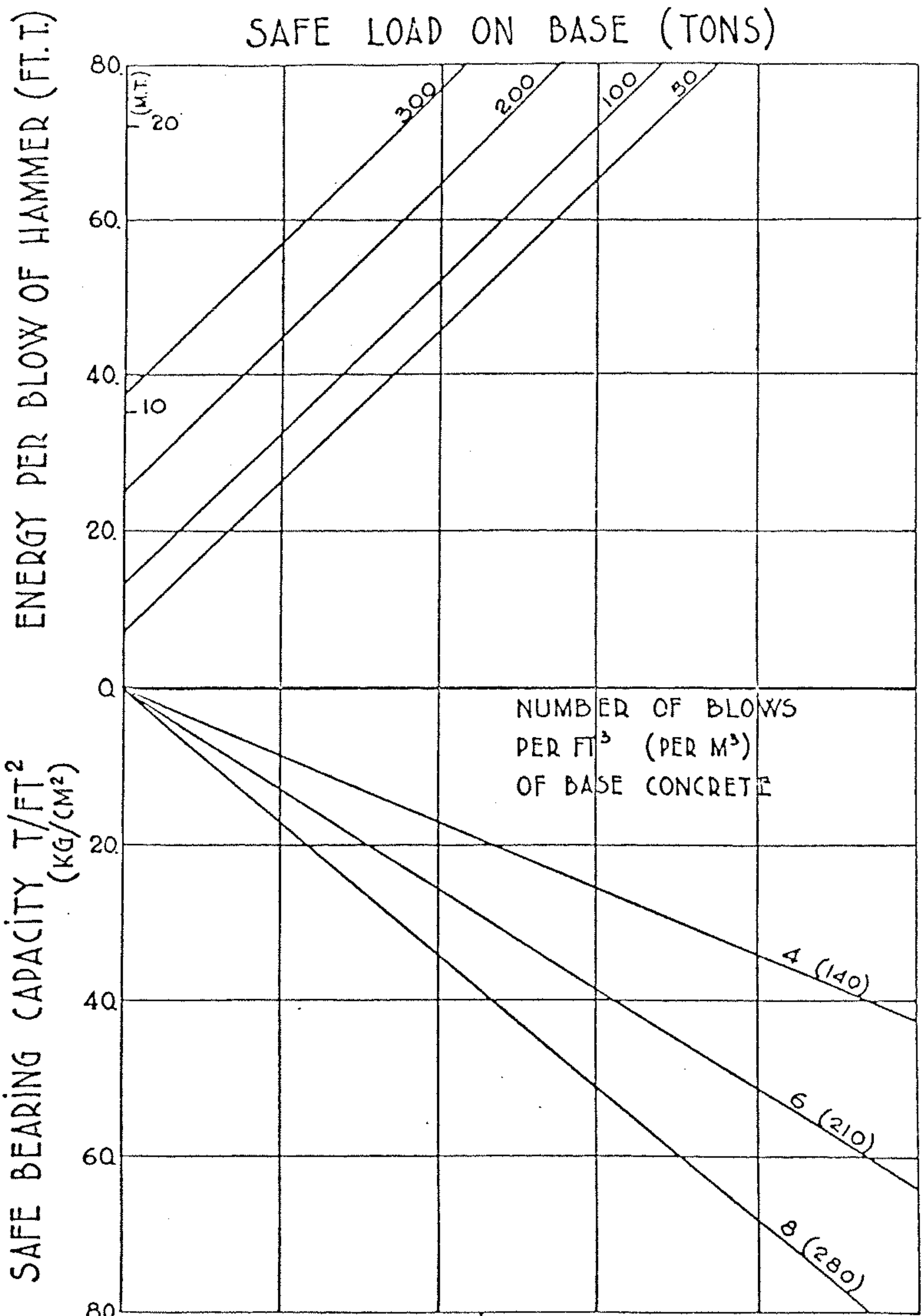

FIG. 8

ENERGY AND BLOWS ON BASE OF CAISSON PILE IN COHESIONLESS SOILS

NOTE: THIS FIG. TO BE READ IN CONJUNCTION WITH FIG. 7

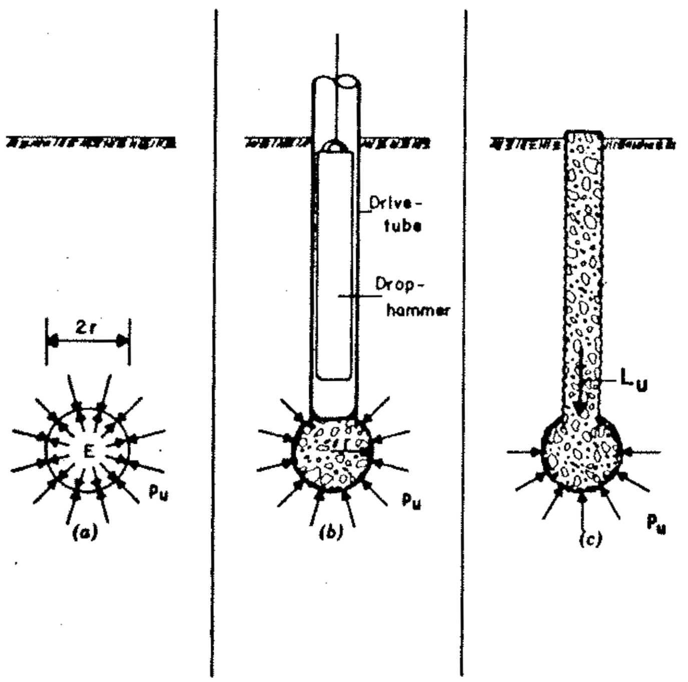

FIG. —Stress Conditions around Base: (a) Theoretical Dynamic; (b) Actual Dynamic; (c) Actual Static

FIGURE 9: Stress Conditions Around The Base Of A PIF

Nordlund 1982

Recommended Values of *K*

Soil type (1)	*K* for a compacted concrete shaft (2)	*K* for a cased concrete shaft (3)
Gravel	9	12
Medium to coarse sand	11	14
Fine to medium sand	14	18
Coarse sand	18	23
Medium sand	22	28
Fine sand	27	35
Very fine sand	32	40
Silty medium to coarse sand	14	18
Silty fine to medium sand	17	22
Silty fine sand	24	30
Residual	$600 + N$ (but $K \nless 18$)	$1{,}800 + N$ (but $K \nless 50$)
Fine sand with limerock fragments or shells, or both	18	25
Till with granular matrix	20	27
Till with clayey matrix	30	40

Note: N = number of blows from Standard Penetration Test.

FIGURE 10: Recommended Values of "K"

Nordlund 1982

8 BIBLIOGRAPHY

Clark, J.I.1979. Failure and Subsequent Rehabilitation of a Dynamically Cast-In-Place Concrete Foundation. A.S.T.M., STD 670

Clark J.I., Harris, Townsend, 1979. Heave of Compacted Expanded Base Piles.

Crandell, F.J. 1964. Transmission Coefficient for Ground Vibrations Due To Blasting. Jour. Bos. Soc. Civ. Eng., p153-168

DeVos, B.J. 1979. Franki Canada Ltd. Vibration Study. Hardy Assoc. (1978) Ltd.

Fellenus, B. 1980. Review of Pile Construction at Keephills Thermal Plant. Private correspondence to MontrealEngineering Ltd.

Garbrecht, D. 1977. Pile Testing by Impact. Proc DMSR77

Hagerty, Peck, 1971. Heave and Lateral Movement Due to Pile Driving. A.S.C.E. November

Hardy Assoc. (1978) Ltd.. 1976. Site Investigation Keephills Thermal Electric Power Station.

Ladanyi, Johnston,. 1974. Behavior of Circular Footings and Plate Anchors Embedded in Permafrost. Can. Geotech. Jrnl.

Meyrehof, G.G. 1951. The Ultimate Bearing Capacity of Foundations. Geotechnique, vol.2, p103

Meyerhof, G.G. 1959. Compaction of Sands and Bearing Capacity of Piles. A.S.C.E., December

Meyerhof, G.G. 1960. Design of Franki Displacement Caissons in Nonplastic Silty Soils. Proc. Symp. Pile Found. Sixth Int. Cong. Bridge and Struct. Eng. p105

Meyerhof, G.G. 1964. Design of Franki Pressure Injected Footings in Cohesionless soil. University of Nova Scotia, Halifax

Meyerhof, G.G. 1976. Bearing Capacity and Settlement of Pile Foundations. A.S.C.E. 1976

Nordlund, R.L. 1982. Dynamic Formula For Pressure Injected Footings. A.S.C.E., March, vol.108, No.GT3

Whiffin, A.C. 1953. The Pressure Generated by Soil Compaction Equipment. A.S.T.M., Tech. Pub. No.156, p186

Terzaghi, Peck, 1948. Soil Mechanics in Engineering Practice. Wiley, New York

9 APPENDIX: INSPECTION RECORDS

The following records should be kept when inspecting the construction of pressure injected footings:

1.0 Before piling commences record:

- -Pile location (construction grid reference)
- -Erection time
- -Spacing in relation to surrounding piles
- -Prebore depth

2.0 While "going down" record:

- -Blows vs. penetration
- -Hammer drop
- -Plug slippage
- -Material added to the plug (measure this by observing the new plug height in the drive tube)
- -Sand elevation as determined by the change in driving resistance
- -Depth at the end of driving (measure from the tube markings)

3.0 Check to see if the casing lifted as the plug was being expelled. If it did rise, record this amount.

4.0 Check concrete quality and temperature.

5.0 While basing record:

- -Number of blows per bucket
- -Number of buckets and the amount of concrete per bucket
- -Hammer drop

-Number of "false blows" per bucket

6.0 Determine the depth of base for each pile by referencing the top of the drive tube to a known survey elevation, that is, measure the length of tube above the H.I. (height of instrument).

7.0 Record the depth to base using the tube markings. This is used to double check the above measurement.

8.0 Determine the amount the casing lifted while forming the base.

9.0 Measure and check:

-Longitudinal rebar length

-Diameter of longitudinal and spiral reinforcing

-Grade of the rebar (stamped on the rebar at 3ft. intervals).

-Tack distance of the spiral

10.0 For tension piles measure the length of any additional reinforcement

11.0 When the hammer is inserted, after the rebar cage has been placed inside the drive tube, check to see if the hammer line mark is approximately flush with the top of the tube.

12.0 For tension piles record:

-The number of buckets used

-Blows per bucket

-Hammer drop

-The amount the casing was lifted for each bucket

13.0 For regular and tension pile shafts, record:

-the length of shaft built

-The number of hammer blows per bucket of concrete

-The amount the casing was lifted for each bucket

14.0 Record the hammer drop while building the shaft.

15.0 Obtain the concrete ticket number from the ready-mix truck driver and the volume of water added to the concrete.

16.0 Record the number of bars visible at completion and the length of rebar cutoff.

17.0 survey the elevation to the top of a rebar and mark the surveyed rebar with orange paint. This will be the reference point used when determining heave.

18.0 Survey the elevation to the top of the shaft. (Use the lowest point on the top of the shaft)

19.0 Measure the distance between the lowest point on the top surface of the shaft and the top of the previously marked rebar.

20.0 Record the time of completion.

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