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Special Foundations - Part III

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Special Foundations – Part III

Ruben A. Gomez, P.E.

1.0 FOREWORD

Although I seldom write in the first person, I am going to do it this time because this part III of the series touches a certain experience of my career that I feel compelled conveying to the young engineers and even to those older engineers who have turned to earth sciences and geotechnical engineering as their new chosen careers.

In the early 1960's when I was a young in-training engineer, although I had the distinct privilege of having being taught by excellent and dedicated engineering professors all about the basics that one needed to know, I found very rapidly that such knowledge was not enough to compete with the more versed and experienced engineers. There were many instances where the jargon and daily common wisdom indicated new areas of knowledge that needed to be explored, and the more I read the more discovered subjects that I needed to know or rather deepen into.

The forty-eight or so pages that our otherwise excellent textbook *Soil Mechanics in Engineering Practice* by the notorious master engineer Karl Terzaghi had devoted to the art of pile foundation engineering was indeed a far cry from what a practicing engineer should and needed to know. Consequently, it took me hundreds of reading hours and learning from those "in the knowledge" and plenty of trial-and-error efforts on my own as well, to become deserving of attention, acceptance and consideration from my colleagues.

Furthermore and on that very line of thought, we should never ever be afraid of failure, for it has been said that we learn much more from our failures than we learn from our own successes, because we seldom know whether we have been too conservative or wasteful when all seems to go well.

With that in mind, this part of the series is a dedicated effort to cover all those areas that the inexperienced young engineer finds intimidating and is always "afraid to ask" in a subconscious and natural self-preserving act that some blatant charlatans in the profession unfairly and unfortunately have chosen to describe as the dubious practice of "fake it until you make it".

One of the contradictions I found in the learning process had to do with the principle that we all have heard one time or another: "the only bad question is the one that is never asked". That statement makes a lot of sense, unfortunately there is a counterpart to it that goes: "in the intellectual world you will be judged by your questions" which is most disturbingly contradictory to the learning process but one which unfortunately we may have to live with.

Now that my intention is clear and unequivocal, and further, having taken this trip to the world of common sense, the least common of all senses, I hope to have achieved here, at least to some degree, the above self-imposed goals and purpose consisting of bringing about a clear view of all what a novice engineer needs to know about pile foundations before going out and facing the world.

2.0 THE DEVELOPMENT OF PILE FOUNDATION ENGINEERING

Pile foundation is one of the oldest methods to carry the weight of houses, buildings, bridges and all other structures down to the underground. Since pre-historic times, in river valleys and flood prone areas where undependable alluvial soils were prevalent, pile dwellings were a preferred solution used since the beginnings of many cultural groups and tribes.

Construction records available from the times of the Babylonian King Hammurabi, as well as from the Roman Empire Era indicate that determination of the adequate length and size of piles and stilts were a particular concern. Unfortunately, much of the pile foundation knowledge was intuitive or based on individual experience, as it was never written down for the benefit of the ones following. As result of such practice, as the experienced engineers passed away, the younger engineers had to re-learn and re-invent the art of pile foundation.

Up until the 18th Century, raw wood pilings were bountiful as provided by nature and logs were available everywhere and usually close to most building areas. Because of the easy and abundant availability of timber piling, there was little or no economy concern amongst builders and engineers; in many instances there was an over-use of pilings triggered by its over-abundance.

During the 19th and 20th centuries, although timber piles were still used abundantly, new materials started to appear in the horizon. Those new piles had to be manufactured in factories far from the jobsite and shipped with the resulting higher cost. To justify the use of those new materials, pile capacity had to be increased proportionally.

For the thirty years from 1945 through 1975 there was more advance in pile foundation design than that achieved during the previous 300 years. In the rest of the 20th Century, as deforestation became a malady of the times, the use of concrete and steel piles became dominant in the construction industry and timber piles started to fall behind to almost extinction. We will see more on this matter as we cover the different kinds of piles.

3.0 NOMENCLATURE

In this section the reader will find all the conventional, unconventional symbols and notations used in this part of the series.

A (in^2 of ft^2) = area; pile cross-sectional area
C = constant; coefficient
c (psf) = cohesion
D (ft) = depth; depth of pile penetration
D′ (ft) = depth of pile embedded into bearing stratum
d (ft) = distance; depth of water table
f_c' = concrete ultimate strength
f_s' = steel ultimate strength
K = earth pressure coefficient
k = coefficient; slenderness coefficient
N = number of blows per foot
N_c ; N_q ; N_γ = bearing capacity factors
n = adhesion factor (between a cohesive soil and the pile)*
P (kips) = load; pile reaction; allowable load
p = average pile perimeter over the pile length
Q = ultimate failure load
q = bearing capacity of stratum
r (in. or ft.) = radius; pile radius
s (psf) = soil shear strength**
FS = factor of safety***
γ_f (pcf) = soil unit weight above water table
γ_b (pcf) = submerged soil unit weight
Φ = angle of internal friction

As we go along with the description of the course material, we may clarify and amplify on some of the terms and definitions so to avoid contradictions and ambiguities.

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*The adhesion factor for timber piles can be assumed as 1.0, however, in the case of concrete or steel piles the adhesion factor (n) shall be taken anywhere between 0.7 and 0.9 depending on the *asperity* of the pile surface.

**Soil shear strength is a function of three factors: the overburden pressure, the angle of internal friction and the average depth of the water table. For cohesive soils, such as clay, it is often assumed that the shear strength is taken as one-half of the unconfined compressive strength.

***We recommend a safety factor of 3.0, but in the last instance never less than 2.50.

4.0 PILE TYPES DEFINED

We will herein classify piles by two broad criteria:

a) according to load distribution behavior, and

b) by material, configuration and method of placing.

When it comes to the first group (a) as related to behavior of the pile in transferring the imposing loads into the ground, for years and scores, engineers have traditionally classified piles in two large categories, the first composed of those which transfer the superstructure loads into the ground by mere *skin friction*, and a second category that does the transfer by *tip resistance* only. Consistently, practice has shown the need to enlarge those categories to cover more meaningful variables that are closer to reality. Therefore, it seems reasonable that further classification was needed where lesser attention was paid to pile material and configuration, and larger emphasis placed on soil characteristics.

A practical and more realistic classification consists of four types:

I) End-bearing pile,

II) Friction pile in cohesive soil

III) Friction pile in cohesionless soil, and

IV) Combined friction and end-bearing pile.

The type I is a pile which is driven through loose soils with a very limited bearing capacity. In this case pile driving takes place all the way through those residual soils until they reach the bearing stratum or hardpan. Commonly, this is a case where the pile length can be anticipated and predicted based on the results of the soil borings previously made by the geotechnical engineer.

The ultimate failure load capacity of the pile is predicted by the following formula where the capacity is based either on the resistance of the bearing stratum, or on the ultimate stress of the pile material:

$$Q = k \,.\, q \,.\, A \leq k \,.\, f_c' \,.\, A$$

In above case, A is the effective area of the pile tip and k is a coefficient of slenderness which value can be determined depending on the length of the pile and/or the degree of lateral support expected from the soil. To the judgment of the foundation engineer, such coefficient could have values assigned varying from 0.33 to 1.0.

Type II comprises all piles driven into cohesive soils (such as clay) and intended as well

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as expected, to achieve their supportive capacity strictly by skin friction. In order to achieve such conclusion, accurate data must be available on the soil shear strength and the adhesion factor between the soil and the pile material. As already indicated above, in absence of more reliable information, the shear strength could be assumed to be one-half of the compressive strength:

$$s = \tfrac{1}{2} q$$

And, the ultimate pile capacity would be:

$$Q = (D)\ (p)\ (n)\ (s)$$

The allowable load to be used should then be:

$$P = Q/FS$$

Type III considers all piles driven through conhesionless soils (such as sand). In this case cohesion has zero value ($c = 0$), however, since density in the soil markedly increases with pile driving, the tip bearing capacity becomes a significant factor and should be taken into account. Therefore:

$$D_L = 20r$$

$$q = 1.3\ (N_c) + (\gamma)(D_L)(N_q) + 0.6(\gamma)(N_\gamma)$$

Then,

$$Q = (d)\ (p)\ (K)\ (\gamma_f)\ (\tan\Phi)\ (\tfrac{1}{2}\ d) + (D\text{-}d)\ (p)\ (K)\ [(\gamma_f)\ (d)\ (\gamma_b)\ \tfrac{1}{2}\ (D\text{-}d)]\ \tan\Phi\ (q)\ (A)$$

Again,

$$P = Q/FS$$

Type IV piles are a combination of the skin friction capacity of the pile embedded in the hard stratum and the end-bearing capacity. It must be said that in practical terms there is also going to be skin friction, it is matter of deciding how relevant that friction would be in the great scheme of load resistance. On the other hand, skin friction in the upper soil layers is usually ignored since some of it may slowly dissipate with time as the load gets transferred down to the lower denser layers of soil. Figure 4.1(a) graphically depicts the typical pile after having been driven in place and Figure 4.1(b) which represents the common “rib-cage” stress distribution encountered in this, by far the most realistic conception of pile behavior. The depressed areas of the soil profile around the pile are the natural result of soil consolidation taken place during the driving operation.

In this case, the so conceived pile capacity is estimated to be:

$$Q = (qA) + (D')\ (p)\ (n)\ (s)$$

Once more,

$$P = Q/FS$$

Going back to the beginning of this section, the second group (b) is identified and defined by the material, configuration and/or the method the pile is driven or placed into the ground. The most relevant examples of that group that deserve mentioning are:

Wood Piles

Timber piles were abundantly used and abused until the 1950's when other materials, namely concrete and steel, massively started to invade the market by metes and bounds and at the same time to demand for a more elaborate method of analysis and design.

One of the particular characteristics of wood piles is that they all come with a taper, as much as intended, since nature grows them with that configuration. Since the very beginning, pile drivers naturally and logically started to pound them into the ground tip first. Of course, said tapered shape had its good benefits and bad drawbacks as well. Amongst the many advantages, the following are worth mentioning: (a) easier driving due to a lesser soil resistance, (b) a larger butt to absorb better the driving energy, and (c) the tendency to densify the soil when driven through sandy stratums.

Some of the disadvantages encountered with the taper had to do with: (a) less cross-section at deeper depths where is more needed, (b) weaker tips, and (c) since they could not be produced to order, some difficulty has always existed in determining location of the critical section. The two figures that follow will pinpoint examples of those typical difficulties.

Man made and produced cylindrical piles with a constant diameter, have their critical section at the butt, however, when it comes to tapered piles, depending on their proportions, type, soil shear capacity, material and load combinations, the critical section may occur anywhere between the butt and tip. Therefore, the design engineer must be aware of those distinct possibilities and after his choices are made, he/she should check those intermediate sections in search for the critical section. In the same manner, the contractor must adhere to the pile proportions specified, and verify that those so dimensioned piles are the ones he has purchased and delivered to the jobsite.

In an effort to illustrate what has been stated above, in Figure 4.2 we have shown an example typifying an ideally well proportioned fifty-foot long pile with a diameter measuring thirteen inches at the butt and eight inches at the tip which has been driven through soft clay. The pile was assumed to work entirely on friction under a purported skin friction of 500 pounds per square foot (psf). On those terms and conditions, the maximum compression stresses found was 517 psi at the butt, decreasing down to the tip with no noticeable inconveniences. The ultimate load was 69 kips (in round figures) and by using a safety factor of 3.0, the recommended working load was 23 kips.

On the other hand, Figure 4.3 depicts a timber pile with slightly different proportions, the butt is still 13 inches, but the tip is now 7 inches and the intermediate dimensions are as shown in the figure. There is another fundamental difference, although such pile has been driven through soft clay with the same skin friction of 500 psf, but has landed on a

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stiff stratum with a resulting tip load of 1,500 psi to refusal. The referred figure provides the rest of the story, and whatever could be missing from it, will be up to you to fill in as part of the questions contained in the enclosed standard quiz.

Precast Concrete Piles

The early concrete piles were modeled after the timber piles that the engineers were so accustomed to handle. In most cases they were tapered after the timber piles, made of reinforced concrete, cast to the ordered length with a square section or other form of polygonal shape. They were regularly designed to resists the stresses caused by a variety of conditions created by handling and driving. Figure 4.4 depicts a common reinforced concrete pile and the typical handling positions.

However, casting of the concrete piles in such conditions was a slow and costly process. By trial and error the pile manufacturers found their way around and switched from "hard" into "soft" concrete by using pre-stressing methods. In order to reduce cost and expedite manufacturing, the concrete was deposited in long casting beds and cut to the desired length. Figure 4.5 shows some of the pertinent design and manufacturing details. The upper detail depicts a regular pile fabricated in a casting bed capable to produce ten 50 ft. piles out of the same pour and stressing operations. Mostly all of it translated into a gain but at the loss of the tapered configuration, which was indeed an unfortunate loss in the trade-off.

Drilled Piers

This is a generic term that covers a large variety of drilled and even excavated members of support as part of the foundation system of a building, bridge or any other free standing structure.

Generally, there are three main reasons that guide the foundation engineer to select the use of drilled piers; they are a) economy, b) noise control and c) vibration avoidance.

Given the right type of soil, the potential of economic advantage is a great incentive, since a hole in the ground can be drilled in matter of a few minutes and that same hole can be examined and inspected before any further steps are taken.

However, not all of them are advantages; there also are drawbacks that may render drilled piers inconvenient. They are affected by bad weather in a larger extent than driven piles would be. Two more inconveniences could be faced and solved in the same manner: high water table and cavernous limestone formations. They can both be taken care of, at least partially, by adding the proper encasements or shells.

In spite of the large number of systems readily available in the market, drilled piers can be grouped in four large categories, and they are:

1- Straight shaft end-bearing piers
They are generally drilled down to the hard pan or rock stratum and worked on the assumption that the surrounding soil around the pier would not make any contribution to the overall capacity.

2- Straight shaft friction piers
This type of pier derives its compressive load capacity from the friction developed between the surface of the pier and the cohesive soils around it.

3- Combination friction and end-bearing piers
This type of pier is merely a combination of the two pier types preceding it, however, such interaction is a closer representation of reality.

4- Belled piers
This is another form of end-bearing pier, except that the tip has been changed to an enlarged bell many times larger than the cross-sectional area of the shaft with the resulting increase in load capacity.

Pressure-grouted Piles

In addition to a concrete mix pump, the equipment necessary for the installation of this type of pile is similar to that shown in Figure 7.1, except for the drilling attachment which consists of a component called a "continuous flight hollow shaft auger". Such attachment is not only used to drill the hole in the ground to the required depth, but also and most important, while it remains in place the hollow core of the shaft is used as a conduit to deposit the concrete grout in place and down to the bottom of the hole.

As the concrete enters and fills the drilled hole, the auger is slowly withdrawn. The rate of concrete mix injection and auger withdrawal must be carefully synchronized, so as to maintain a positive head of grout and prevent the soil from around the hole to collapse over the fresh concrete.

If steel reinforcement is specified by the design engineer, the pre-assembled bar cage can be slid in place during the few minutes while the grout is still fresh and fluid. Evidently, for all those steps to take place promptly and in the proper sequence, field personnel must be well trained considering the negative consequences of any clumsy handling.

Drilling sequence and vehicular activity at the site must be kept under strict supervision while the concrete has not entered its curing process.

Some of the obvious benefits that come together with this method are:

a. no detrimental vibrations to affect existing structures or residential neighbors,

b. no undesirable noises and therefore, less complaints, and

c. easier and more accurate pile lay-out and location.

Pressure Injected Footings (PIF)

This type of pile is most unconventional and consists of taking a batch of zero-slump concrete (4 to 8 cubic feet) and ramming it into the soil (energy varying from 60,000 up to 200,000 ft-lbs.) to the required depth. Immediately after, a steel shaft is introduced into the hole and concrete is rammed into the shaft and as it progresses, the steel shaft is retrieved slowly out of the hole until the pile is complete.

Because of all the ramming, pounding and resulting vibrations, not two consecutive piles are allowed to be driven any closer than 9 ft. on centers, so to avoid disturbing the fresh concrete, nevertheless, on a second round those spaces in between may be filled in with more piles. Steel shaft diameters vary from 12 to 24 inches. Reported pile capacities ranged from 50 to a whopping 300 tons of maximum <u>working</u> load capacity. Standard concrete compressive strength at 28 days has commonly being used as 4,000 psi.

Stone Piles

There are cases within the ample spectrum of possible cases that may have to be confronted by the foundation engineer where solutions may be hard to come by, particularly when those selected construction sites are located within the confines of sanitary landfills, reclaimed lands, marginal wastelands, marshes or drained swamps where the soil is very poor and the hardpan is nowhere to be found.

When everything seems to fail, the engineer still has some idea resources at hand that could "save the day". In cases of one story lightweight construction, a building could be erected in the middle of a swamp just by removing the bad soil to the proper depth, replace it with selected fill and compacted to the proper density. Here applies the already proverbial case of the 500,000 square foot U.S. Post Office building in the Meadows, NJ. The site was a stretch of swamp land with basically 8 ft. of decomposed organic matter lying on an 80 ft. deep layer of varved clay. The foundation engineer made the decision to remove all the muck above the clay stratum and replace it with 16 ft. of well selected fill material adequately compacted. After successful completion of the post office facility, many other buildings follow suit and were erected nearby, on the same principle and with a minimum of problems.

Above example was brought up just to illustrate what has always been an open avenue to solve a problem of that nature, however, what is to be done when on a similar project site the specifications call for a "crack sensitive" walk-up style or even a medium-rise concrete building?

Although not a cure for all, a readily solution is the application of the stone piles. There are two versions of this solution depending on the position of the water table. The first one is by using the help of what is called *Dynamic Compaction,* which basically consists of repeatedly impacting the ground with a heavy mass dropped from the very top of a crane's boom. The depth and effect of the impact is related to three factors: the ballast weight, the height of the fall and the type of soil subject to it.

The ideal dropping weight should be about 4 ft. square and 3 ft. deep with a slight (say one foot) conical protrusion out of the center of the bottom side. It should be preferably made out of concrete and should have three well balanced lifting hooks. The edges do not have to be perfect and for the sake of economy, it could even be cast in a ground hole.

As the ground is repeatedly impacted by the weight the soil will get compacted and a hole in the ground will develop to the desired depth. The area of soil influenced by the impact may reach as much as 40 ft. deep. Since the determination of the falling weight and drop height are paramount, they should be suitable to the job application. They all can be deducted from the propagation depth formula:

$$D = K \sqrt{Wh}$$

Where:

D = depth of influence (ft)
K = an empirical coefficient
W = falling weight (in kips)
h = drop height (ft)

The empirical coefficient (K) must be selected by the type of soil material being compacted and gives consideration to the vibratory wave transmission. Use K as a value of 0.5 for sandy soils, 0.9 for clayey soils and 1.20 for organic matter and wet rubbish fills. For any other soil type in between those given parameters, follow your own judgment.

Once the hole is formed to the required depth and diameter, fill in the hole with a mix (by volume) of 80% crushed stone plus 20% caliche and compact it to the prescribed density.

Since Dynamic Compaction will not work well in waterlogged or saturated soils, its use should be limited to conditions where the water table is 6 ft. below grade or deeper. For saturated soils instead, use a standard excavator with the proper reach to dig the hole(s) in the ground and follow the same steps but use a mechanical tamper to compact the filling material. The resulting stone pile should be about 5 to 8 ft. in diameter and should be spaced in a grid pattern from 6 to 10 ft. on centers, each way. The so attained load capacity may vary from 20 to 60 kips each. Enclosed Figure 4.6 pictorially shows the main features of the procedure and Figure 4.7 depicts different stone pile patterns from a single to combinations of up to five piles, which could be used to carry isolated column spread footings as the design needs may demand.

In-situ Cast Concrete in Corrugated Steel Shells

As the titled implies this type of pile system consists of a corrugated steel shell which is driven into the ground to the required depth. Once passed the routine inspection for alignment and integrity, it is filled in with premixed concrete and reinforced if called for.

Although the system is adaptable to even higher load capacities, the 50 ton pile has been very popular since the early 1960's, especially amongst the many state highway departments throughout the American nation.

As a mimicker of the original timber pile, the 50 ton pile is more likely than not tapered from a nominal diameter of 8-½ in. at the tip up to 12 in. at the butt. The shell was (and still is) driven with hammers delivering from 24,000 to 32,000 foot-pounds of energy.

This type of pile is very predictable and dependable, its skin friction is one of the highest in the industry, and it increases with time as the surrounding soil gets a firm grip around the exterior corrugated surface of the pile.

Amongst the many benefits of this system it must be noted that before casting, the shell can be thoroughly *scope* inspected and if so desired or required, instrumentation can be dropped in it, which could be used for performance monitoring for years ahead after the pile has been put in service.

Steel Piles

Structural steel H section piles have been in use since the 1930's in cases where longer penetrations and/or larger capacities were needed, such as in the case of skyscrapers, bridges and dams. They also offer the advantage of easier and faster driving because on their way down, they displace lesser amount of material than their counterparts. Also, since they can be driven in tighter group patterns, the result is more economical pile cap designs.

Whether driven bared or encased in a steel shell and injected concrete, they can be used to carry exceptionally large loads. In the United States, there have been documented cases of developed pile capacities well in excess of 5,000 kips.

In closing this section it needs to be said that the selection of the most efficient pile to meet the particular conditions and requirements of a given job, may prove in some cases to be a difficult and elusive decision to make on part of the structural, geotechnical or even the foundation engineer.

When it comes to selecting a particular piling system the decision should not be taken lightly or made on a biased way and influenced by the convenience of, or the persistent selling pitch from any given supplier. When in doubt, the engineer should and have at hand his own check list or guiding specifications that he can use before embarking and locking into any particular solution. The following points and factors should play an important role in his decision:

#1- Length of pile. It bears on the stratification of the subsoil and the best penetration to help reach the ideal bearing stratum.

#2- Average depth of the seasonal water table.

#3- Access to the site, driving characteristics, best or most convenient method of pile installation.

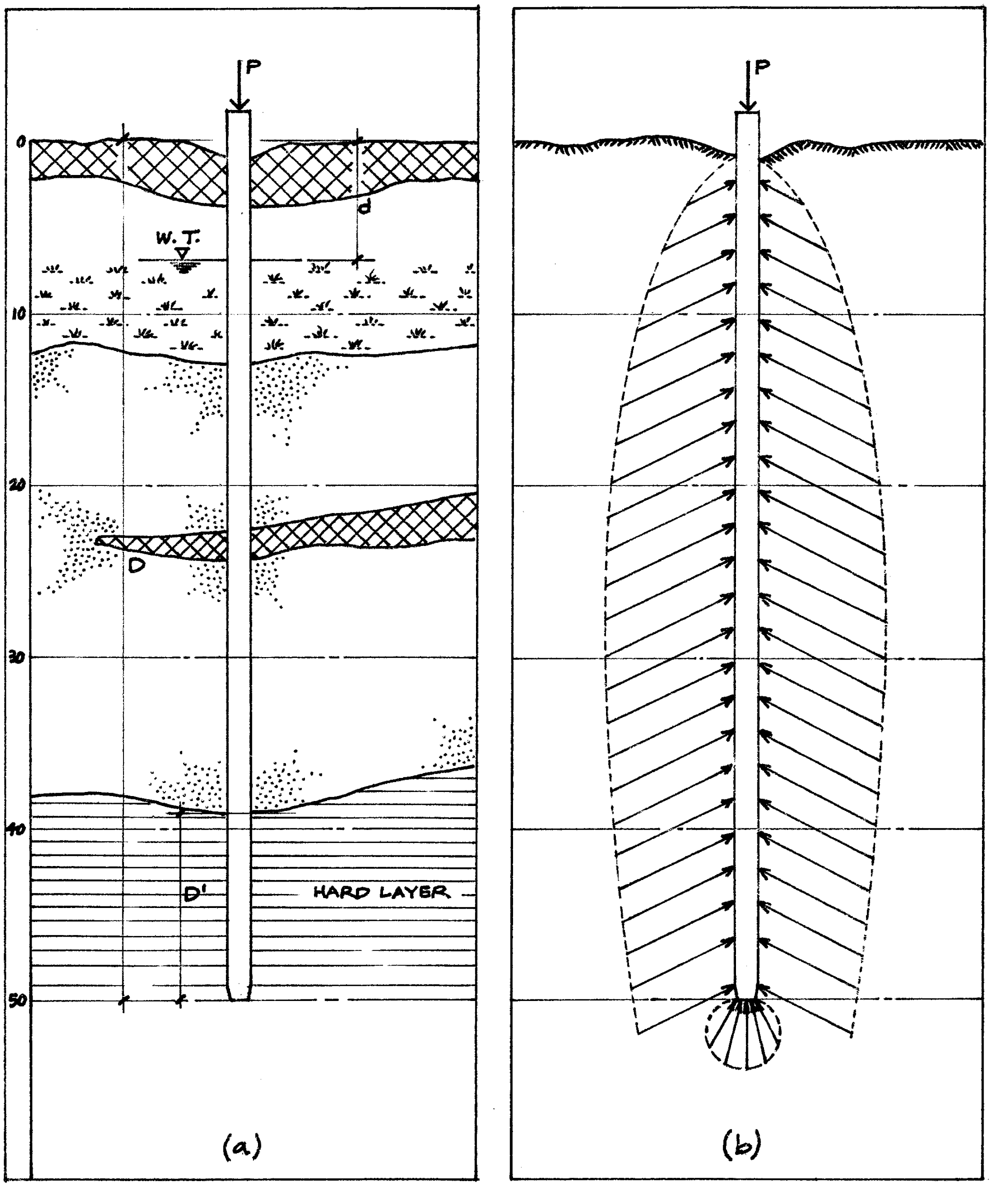
#4- Type of structure and its possible sensitivity to differential settlements.

#5- Type of loadings: total weight of the structure, ratio of dead loads to live loads. Type and nature of dynamic, repetitive and/or lateral loads.

#6- Presence of aggressive soils, hyper-acidity or contaminants.

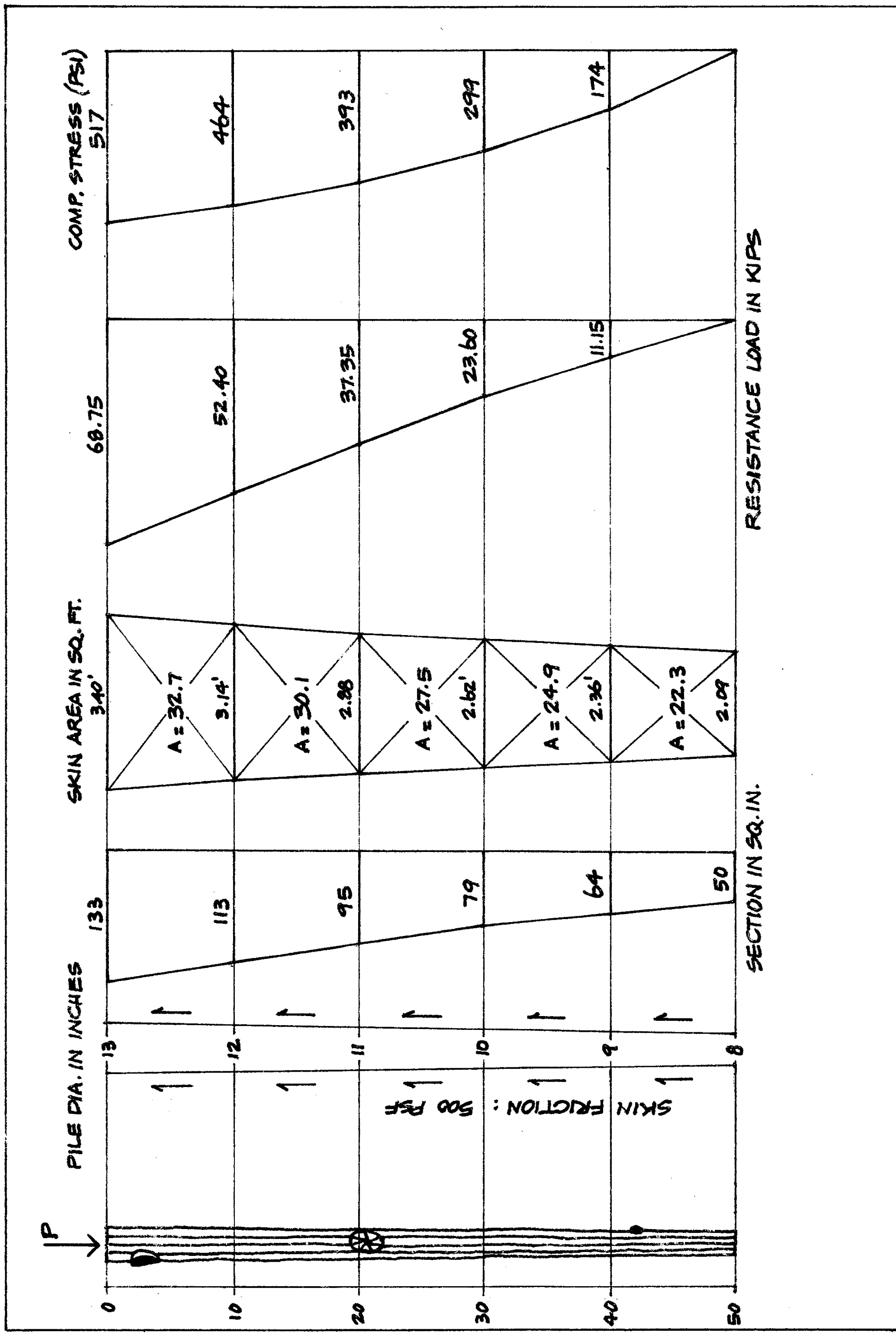
#7- Availability of equipment and skilled manpower.

#8- Other important economical considerations.



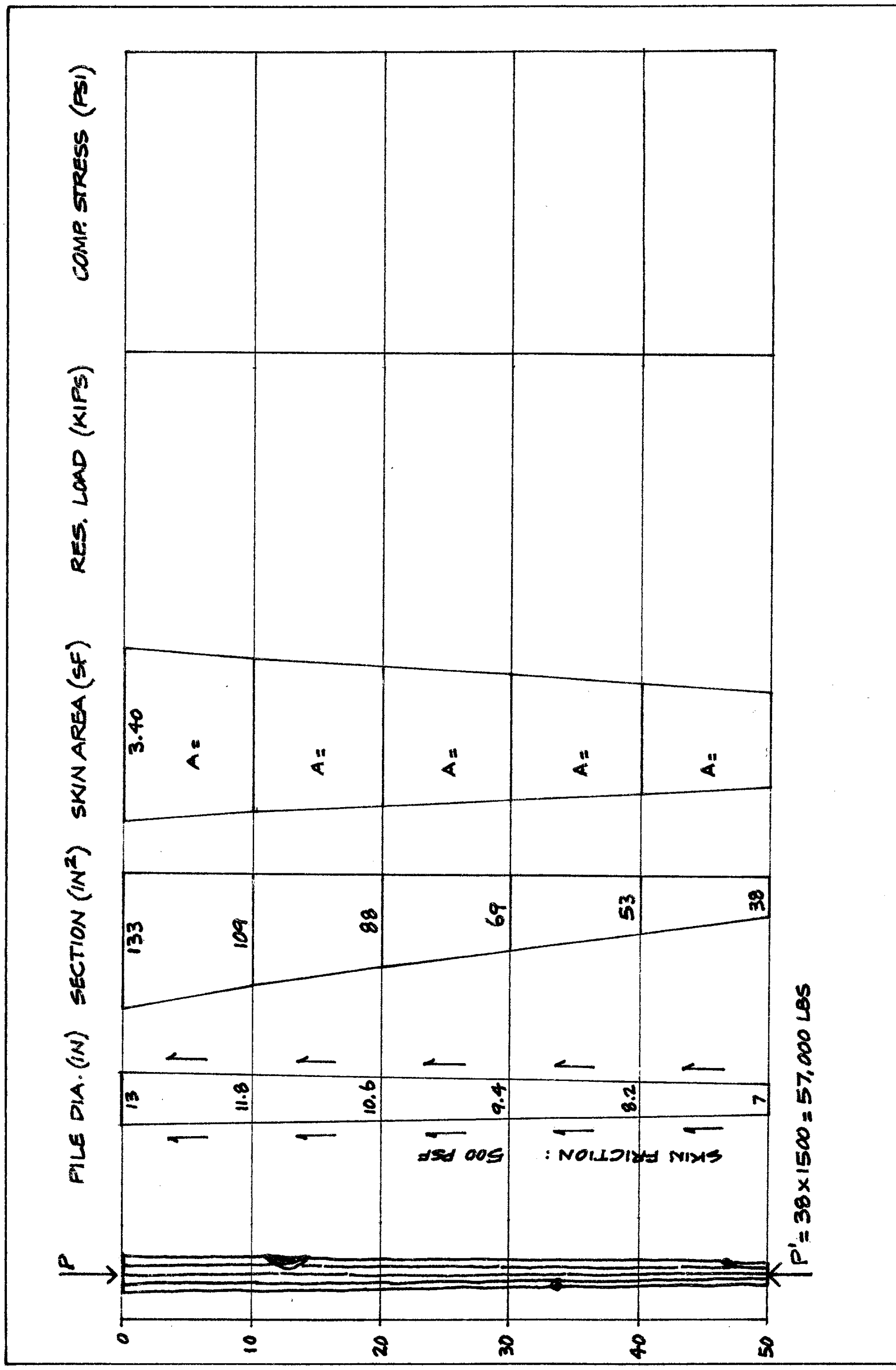
COMBINED FRICTION AND END-BEARING PILE

FIGURE 4.1



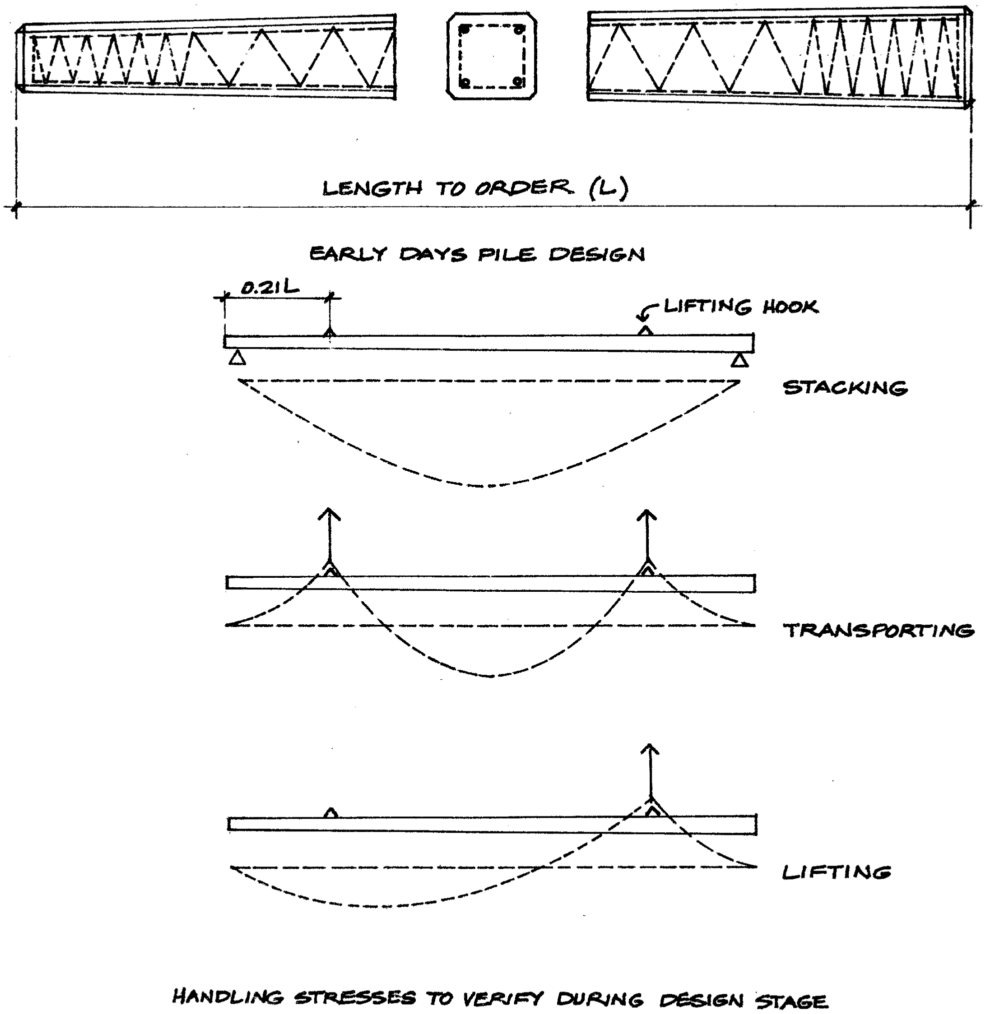
TIMBER PILE UNDER FRICTION LOAD CONDITIONS

FIGURE 4.2



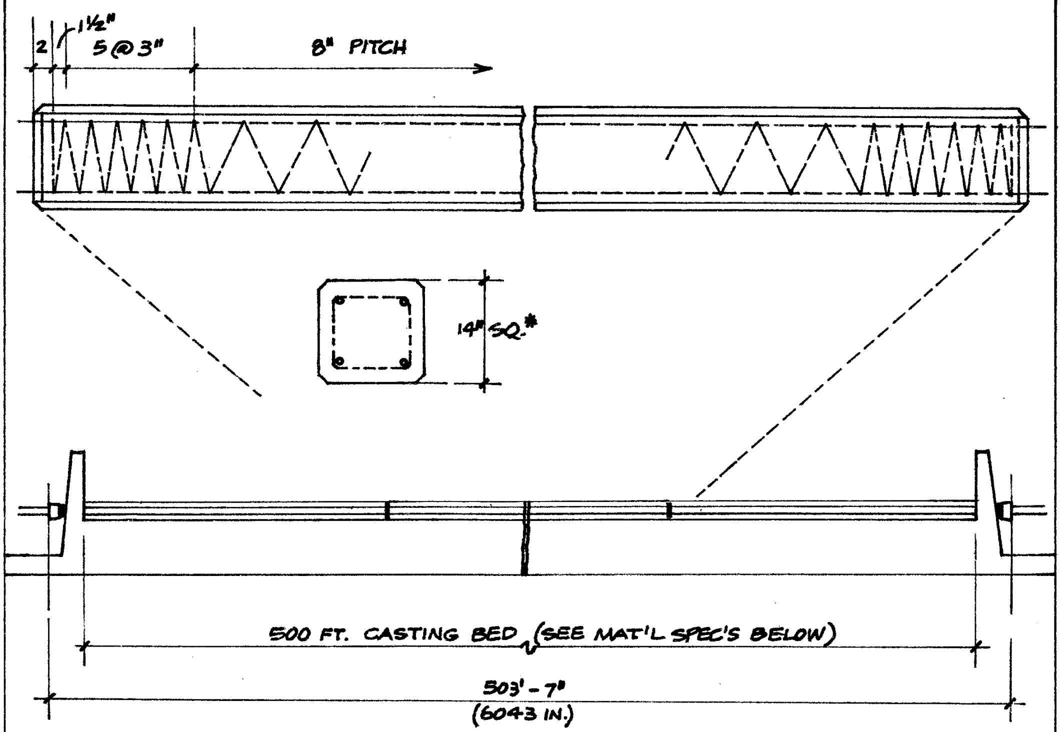
COMBINED FRICTION AND TIP BEARING TIMBER PILE

FIGURE 4.3



BEGINNINGS OF THE REINF. CONCRETE PILE

FIGURE 4.4



*GENERALLY ALSO AVAILABLE IN 10, 12 & 16" SIZES.

STRANDS: 7/16" - 7 WIRE, STRESS RELIEVED TYPE. $A = 0.120\ IN^2$, $T = 23.7$ KIPS.

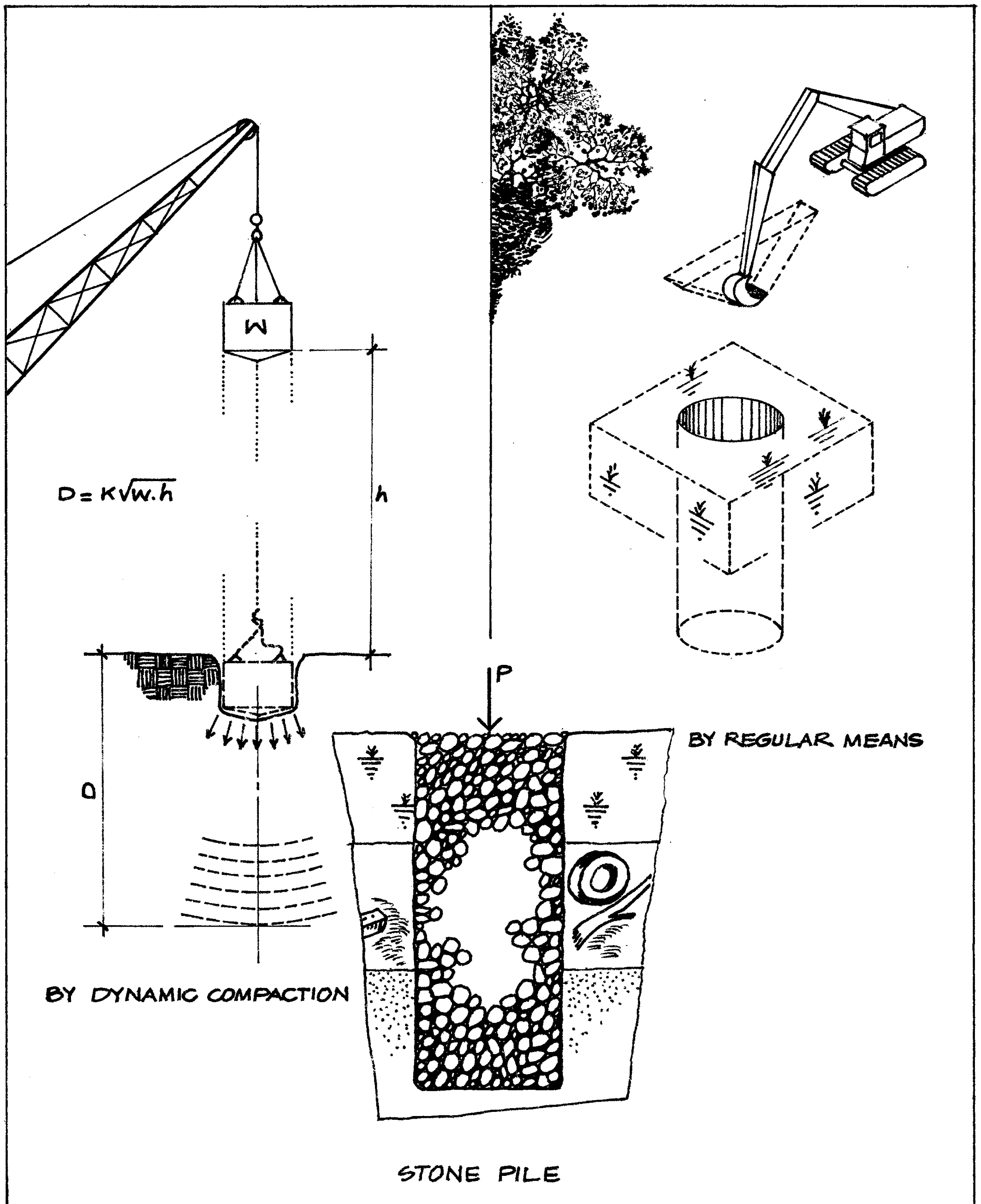
SPIRAL WIRE: #5 GA.

CONCRETE: COMPRESSIVE STRENGTH @ 28 DAYS : 6,000 PSI.
PRESTRESSING FORCE RELEASED @ 3,500 "

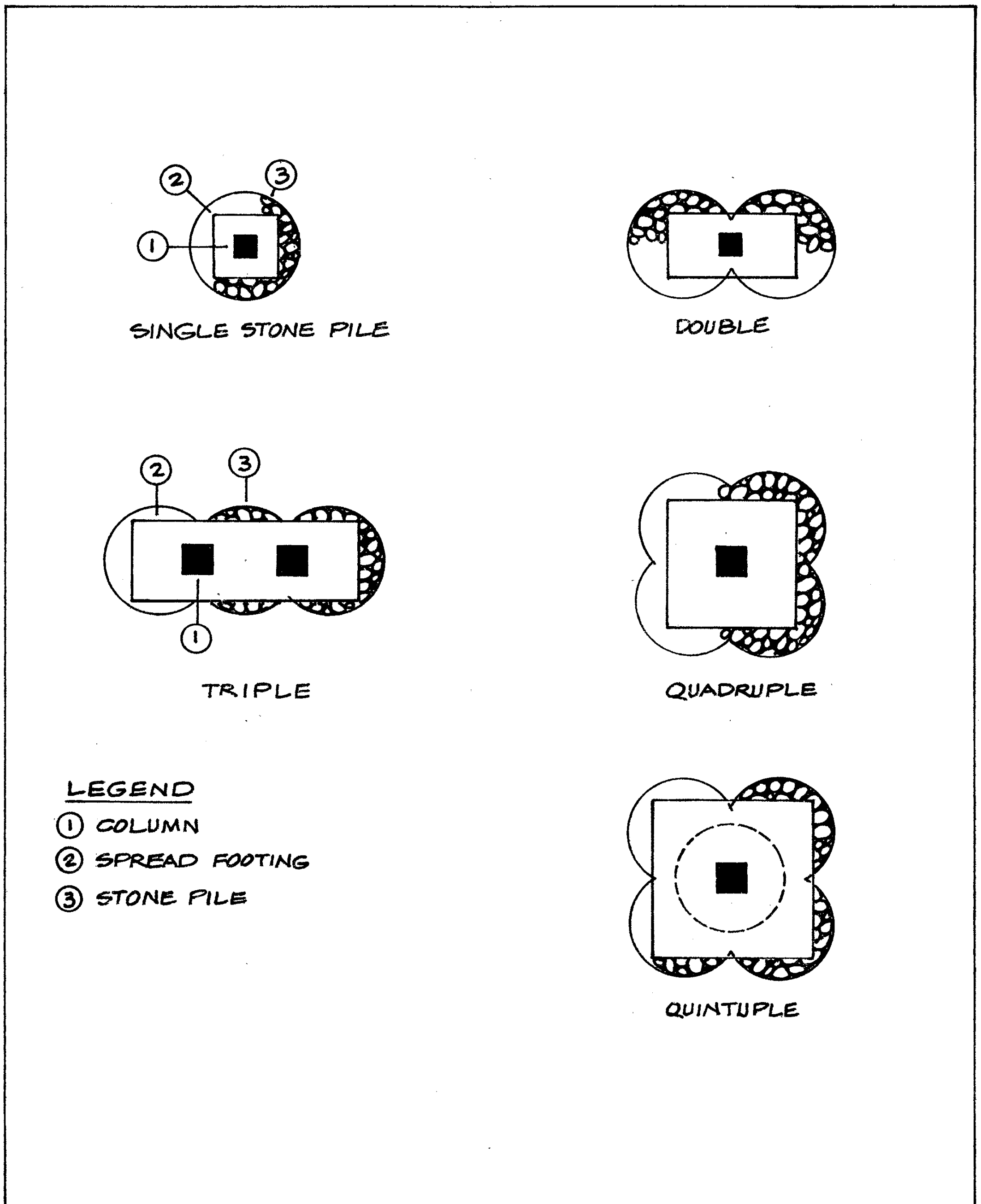
TOTAL STRAND ELONGATION: $\dfrac{23{,}700 \times 6{,}043}{0.120 \times 28{,}700{,}000} = 41.59$ IN.

MODERN PRESTRESSED PILE DESIGN

FIGURE 4.5



PRINCIPLE OF THE STONE PILE
FIGURE 4.6



STONE PILE COMBINATIONS
FIGURE 4.7

5.0 SINGLE PILE VS. GROUP PILES

As already indicated above, pile foundations have come a long way, not only through the thousands of years of human history, but also in the technological age of the last 80 years since the establishment of earth sciences. While the piles of the past were some 30 to 50 ft. long and about a foot in diameter, today we have seen giant super piles driven to great depths of 500 feet with diameters of 12 ft. and more. Most decidedly, since the first use of timber piles the art has grown and seen rapid changes and evolutionary advances.

So far, in this course we have dealt mainly with single piles and their interaction with the surrounding soil, however, in common practice the design engineer needs to have an understanding of piles interacting as a group. In fact, some building codes do not accept the use of single piles or even twin piles for that matter; therefore, we need to establish a bridge between the principles of behavior of a single pile as related to its work as part of a group.

The industry has had an extensive experience in the testing of single piles, while the information on pile group testing is rather scanty in comparison. Naturally, the reason of such difference is that loading tests of real-life size pile groups is very expensive and to some degree cumbersome. Fortunately, we have plenty of information on single pile tests from which we can extrapolate the results.

There are some limitations to those extrapolations though, from the tests performed it can be deducted that the load capacity of a group of piles driven through clay is *less* than the product of the load capacity of a single pile times the number of piles in the group. Some authors argue that such reduction is caused by the *overlapping of the bulbs of pressure* formed around each pile in the group. The argument made sense to many of the engineers before us and it inspired the so-called *efficiency formula.* The reason why we do not reproduce the formula here is because its use has become impractical and obsolete.

Terzaghi did not endorse the efficiency formula and used a different view on this matter. He conceived a group of piles with the soil in between them as a rigid and monolithic block that would transfer the imposed load upon them down to their tips and therefore to the stratum beneath them, and they would do that as a unit. Further, as the load increased, they would ultimately also fail as a unit.

Again, Terzaghi was closer to correctness than anybody before him and it is fair to say that he has remained so until today. However, in addition to his own, there are quite a few formulas around trying to translate his idea in a workable numerical form. We have found that amongst them, Ralph B. Peck's (1952) procedure is the most practical one when it came to its application to clay soils. Figure 5.1 provides some direction on the visual end and is needed for the total comprehension of Peck's procedure. It shows the case of a concrete column subject to a load P and bearing on a 7'- 0" x 11'- 0" concrete pile cap carried by 12 cylindrical concrete piles of 12 in. diameter each, which have been driven in place through a variety of soft clays.

If a column load is given, which is more likely than not the case, and having established the pile material, as well as the diameter (if cylindrical), section, length and spacing, we may start by determining the load capacity of a single pile. The pile surface perimeter is equal to 3.1416.d.D, "d" being the diameter and "D" the depth. In the same manner, the pile safe load is the surface perimeter multiplied by the shear resistance (s). By dividing the column load (plus an assumed pile cap weight) by the single pile safe load we can get the tentative minimum number of piles (n) needed in the cluster. The result will likely be a fractional number, if so, move up to the next round number. We are now able to determine on preliminary basis our pile cap size and configuration.

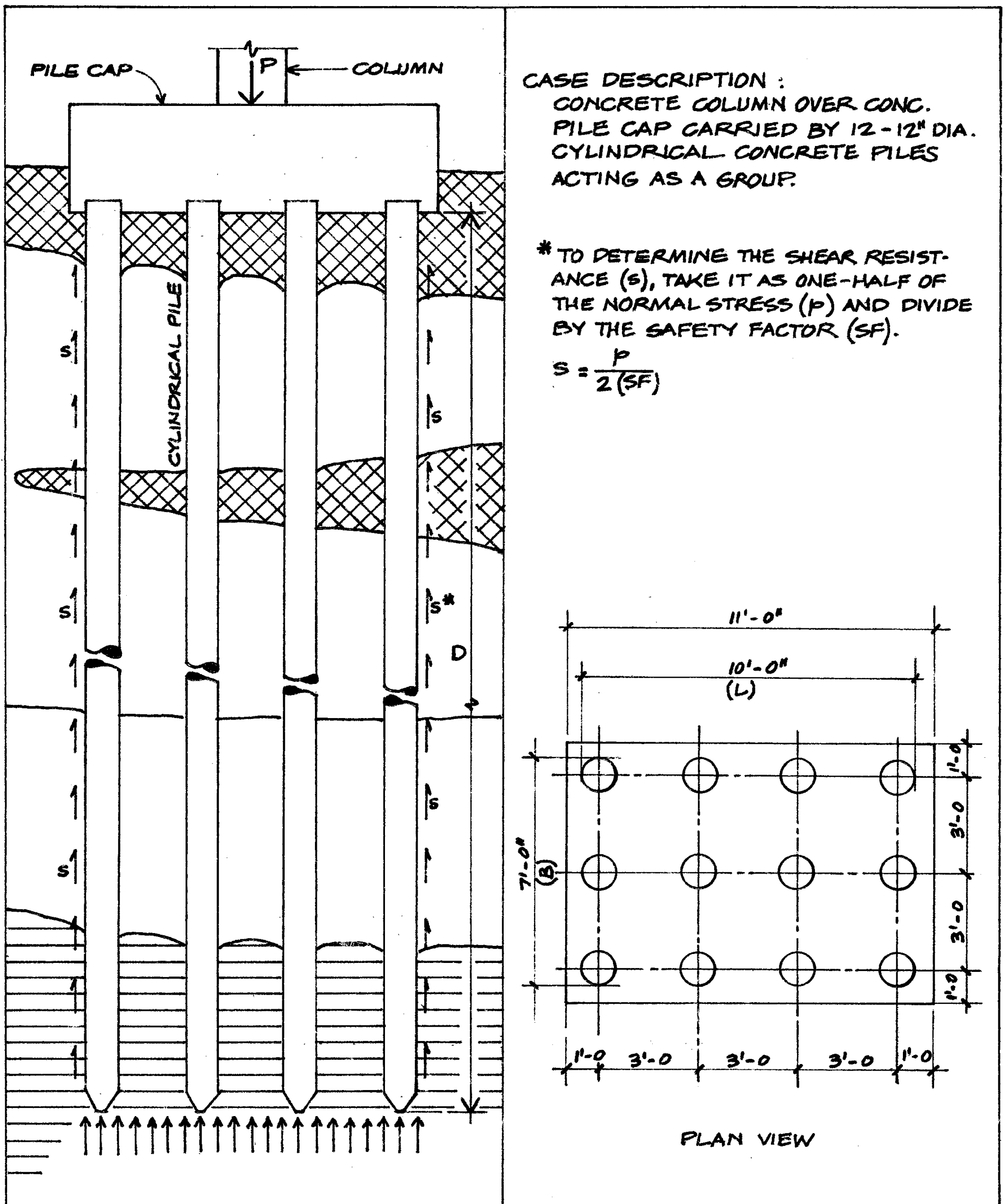
Next step is to verify the capacity of the pile group in a similar manner as done above. Calculate the surface perimeter of the pile group and multiply it by the shear resistance: (B).(L).(D).(s), the result will be the *total* shear resistance of the group.

For the next and final step, it is necessary to go back to the traditional *safe soil pressure equation:*

$$q_a = 0.95p\ (1+0.3B/L)$$

The total safe load of the pile group will be obtained by adding the value of q_a to the total shear resistance of the group from above. That load capacity must be equal or better than the given column load. If it is not, it may take one more trial cycle to merge the two figures.

A pile group driven in sand can be treated in a similar way, provided that the following considerations are taken into account: pile driving will increase the relative density and the friction angle of the sand stratum and therefore, the shaft resistance of the piles in place will also be increased accordingly.



COMMON PILE CLUSTER

FIGURE 5.1

6.0 PILE DRIVING EQUIPMENT

Piles are commonly driven into the ground by means of a *hammer* installed in a special device known as a *pile driver* supported by a crane. The pile driver consists of a pair of vertical guides known as *leads* within which the hammer is held. Those leads are carried on a frame in such a way that they can be supported in a vertical or inclined position. Such framework is attached to the crane boom on its upper end at a point known as the *boom-point connector* which in combination with the *spotter* at the lower end allows movement of the leads in six different directions. The total described assembly is mounted with the hoisting engine on a suitable base which is in turn installed on a truck, a set of crawlers or a railroad car for the operation on land, and on a barge to allow operation on water.

Figure 6.1 generally describes the different components comprised and integrated in a typical arrangement of pile driving equipment. Leads come with several sections of different lengths to allow accommodation for piles of a diversity of sizes. The *headblock* carries the crane lines over the top of the leads and is quite commonly available for a combination of two or three operational lines.

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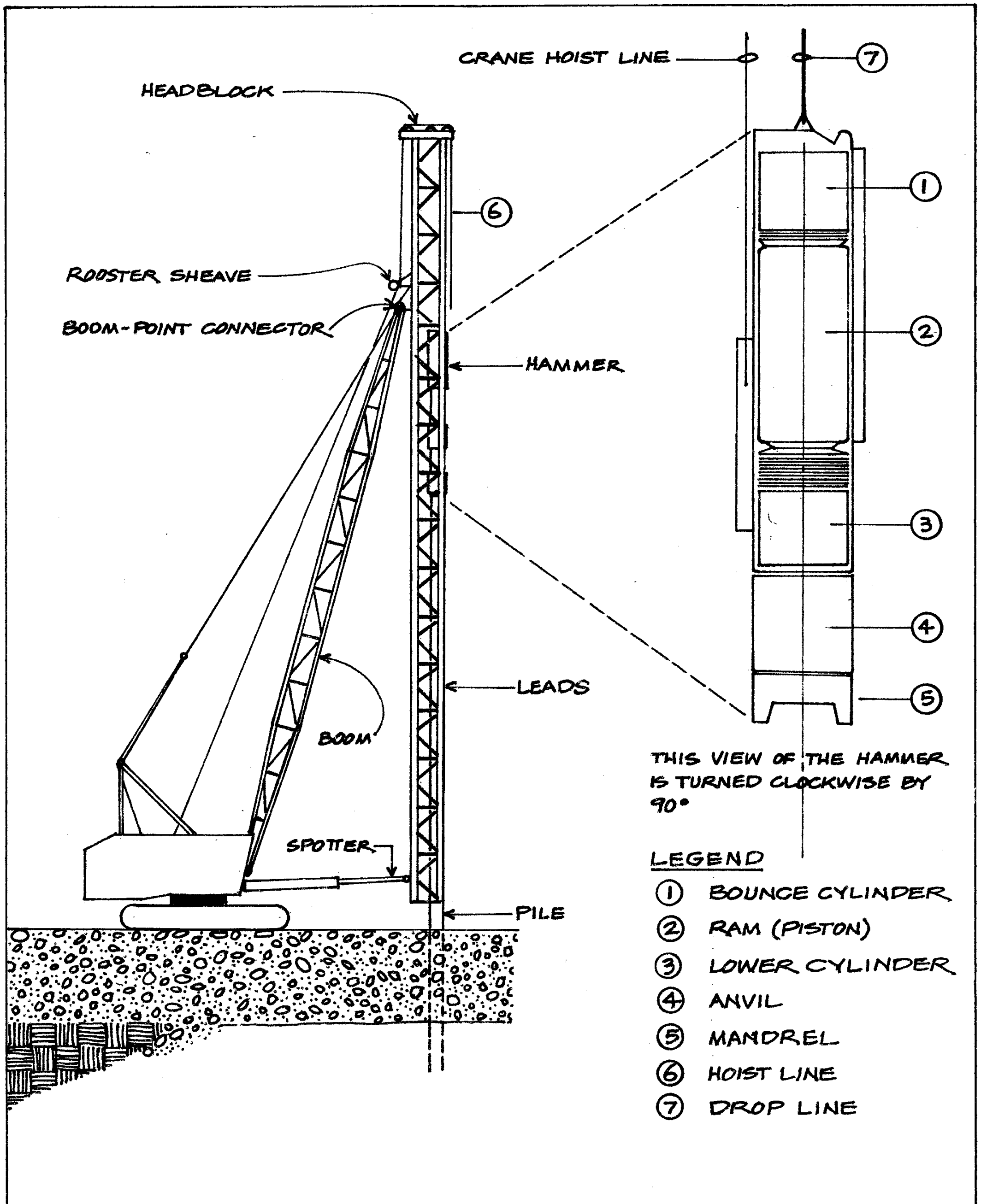


PILE DRIVER ON CRAWLERS
FIGURE 6.1

6.1 HAMMERS

Selection of the driving hammer is done by picking a ram weight and a rated energy. That selection is done in such a way that the hammer should be enough to be suitable for the job demands, but not in excess to risk damage to the piles. Some building codes also dictate particular hammers for specific type of piles and loads and therefore those rules must be complied with, unless exempted by the Building Official with the proper jurisdictional authority.

The following is a sequence showing the general performance characteristics of the hammers used in common construction practice:

DROP HAMMERS

This hammer is obsolete because its slow count makes it inefficient and uneconomical. Its description has been included in this count just for the purpose of completeness.

Drop hammers can perform within the range of one to four blows per minute while modern hammers can easily deliver 15 to 25 times more blows during the same period of time.

Its archaic mechanical system basically consists of a single line used to raise the ram to a great height from where it is released into a long free fall. The process is arduously and slowly repeated after each blow on the pile head.

SINGLE ACTING HAMMERS

These hammers are very popular for two reasons, their simplicity of design and their dependability in service. Although the impact energy is still derived from a free fall of the ram, the rising is accomplished by either steam or compressed air means.

They can achieve driving rates of 50 to 70 blows per minute and can be used for almost all soil conditions; however, they are most effective for penetrating medium to firm clays. Ram weights range between 3,000 to 6,500 pounds and energy ratings of 7,500 to 20,000 ft-lbs.

DOUBLE-ACTING OR DIFFERENTIAL HAMMERS

Double acting hammers, much as differential hammers can deliver from 90 to 150 blows per minute. Such a high blow-per-minute (BPM) pace speeds up pile penetration by overcoming soil friction and do well on either sandy or clayey soils.

Ram weights range between 1,000 to 5,500 lbs and hammer energy rates from 5,000 to 20,000 ft-lbs. There is one area where double-acting hammers have a clear advantage over differential hammers, since the formers have entirely enclosed rams they can be used to drive piles over wet or underwater conditions.

DIESEL HAMMERS

Basically they consist of a cylinder, ram, anvil block and fuel injection as a sealed self contained unit. The range of blows per minute is from 48 to 105. Ram weights run from 1,000 to 3,800 lbs and the striking energy is between 7,500 and 32,000 ft-lbs.

There are good reasons that have made this hammer very popular amongst pile drivers, they eliminate the need of auxiliary power, such as steam or compressed air, the unit is lightweight and under ideal conditions, very economical to operate.

In Figure 6.1 we have shown a basic self-contained diesel hammer designed on the two-cycle principle to deliver the driving energy down to the pile head in three different manners: *compression pre-load*, *direct impact* and *explosive force*.

SONIC HAMMERS

Sonic hammers consist of a series of eccentric weights oscillating in such a manner that they add their impulse energy in the vertical direction while cancelling their impulse in all other directions. Oscillation is induced by either single or multiple engines mounted on the pile driver's frame. The driver is connected to the pile by a hydraulic clamp and oscillates to its resonant frequency.

Where driving time is of the essence, this is probably the most effective hammer of them all and performs at its best in sandy soils.

VIBRATORY HAMMERS

This type of hammer rather than pounding on the pile vibrates by rising and lowering the pile itself at the rate of 16 to 24 cycles per second, which is close to the resonant frequency of sandy soils. Those vibrations overcome the friction and tend to fluidize the soil in contact with the pile on its way downward.

CLOSING REMARK

Having described the different type of hammers available, it is conclusively true that hammer selection must be done on a careful basis. As a last step in helping the choice of equipment, we have included below Table 6.1.1 as a pile load guide based on driving performance. This guide includes the four most common combinations of ram weight and hammer drop conducive to an efficient driving energy. This (or any other similar guide) is a must to be had when the engineer goes on inspection or in his supervisory role to the pile driving site, because it will give him a conclusive idea of the pile driving performance he would be about to witness and perhaps ultimately would have to judge.

TABLE 6.1.1

PILE LOAD REFERENCE GUIDE

PILE LOAD (TONS)	RAM WEIGHT: 1,500 LB DROP: 5 FT ENERGY: 7,500 FT-LB		RAM WEIGHT: 2,750 LB DROP: 6 FT ENERGY: 22,000 FT-LB		RAM WEIGHT: 4,000 LB DROP: 5 FT ENERGY: 18,200 FT-LB		RAM WEIGHT: 5,070 LB DROP: 5 FT ENERGY: 26,300 FT-LB	
	PENETR. PER BLOW	BLOWS PER FT.	PENETR. PER BLOW	BLOWS PER FT.	PENETR. PER BLOW	BLOWS PER FT.	PENETR. PER BLOW	BLOWS PER FT.
12	0.52"	23	1.26"	10	1.42"	9	2.09"	6
13	0.47"	26	1.16"	11	1.30"	10	1.92"	7
14	0.43	30	1.08"	12	1.20	10	1.78"	7
15	0.34	35	1.00	13	1.11	11	1.65"	8
17	0.27	45	0.87	14	0.97	13	1.44	9
20	0.24	51	0.72	17	0.81	15	1.22	10
25	0.20	61	0.56	22	0.63	20	0.95	13
30	0.15	82	0.45	27	0.51	24	0.78	16
35	0.11	110	0.37	33	0.42	29	0.65	19
40			0.31	39	0.36	34	0.56	22
45			0.26	47	0.30	40	0.48	25
50			0.23	53	0.26	47	0.43	29
55					0.23	53	0.38	33
60					0.20	59	0.34	36

SANDY SOILS

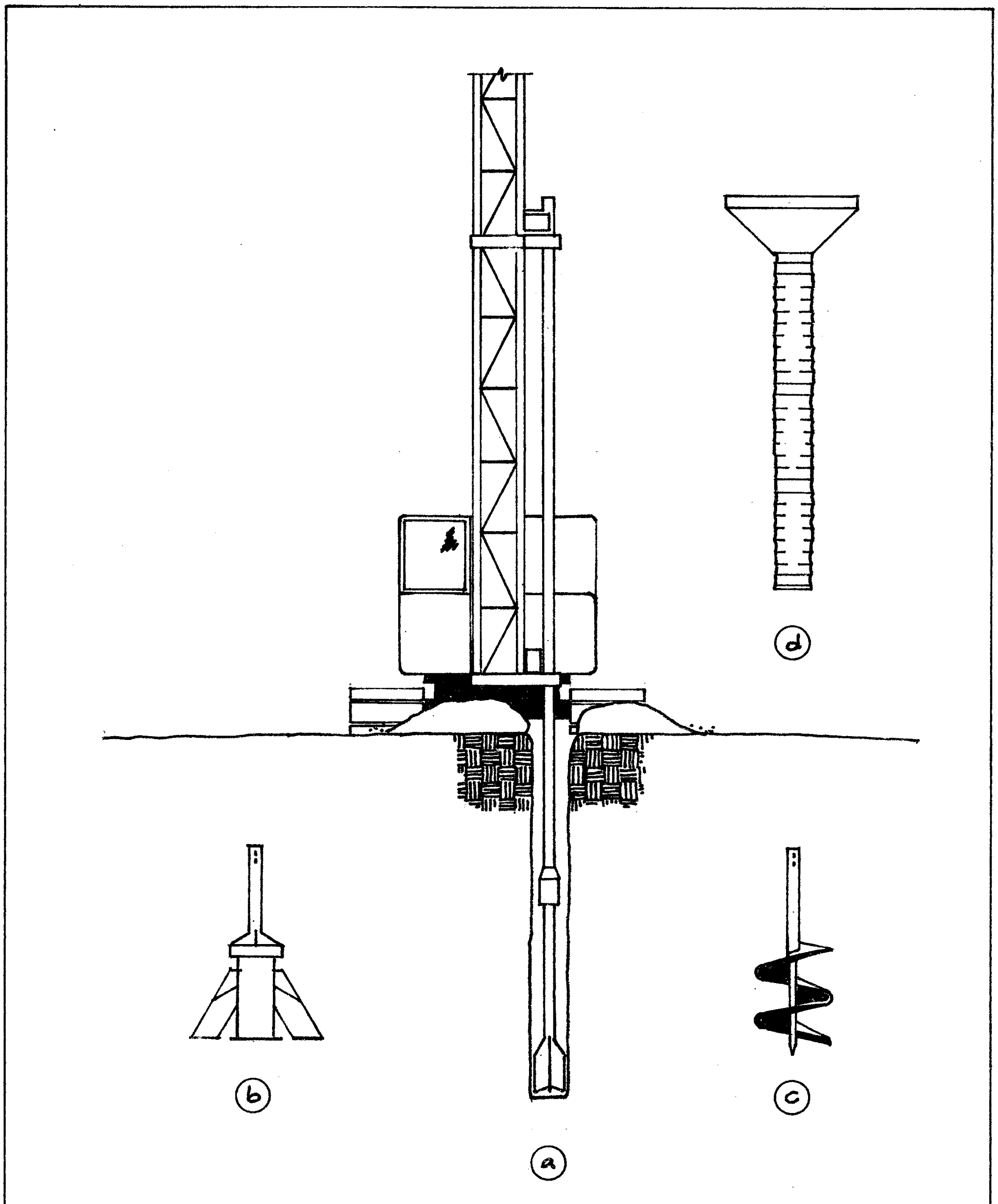
7.0 AUGERING EQUIPMENT

Generically speaking, augered piles are not much different from their counterparts which have been elsewhere in this course designated as drilled piers. There is such a multitude of similar systems proliferating in the industry that is has come to the point of them being difficult to classify.

Basically, an augered pile consists of a hole in the ground that is filled with concrete and steel, however, how it is done is where the big differences occur. In this section we will cover what it takes to make that hole in the ground.

The necessary equipment is similar to what is used for pile driving, except that the *driving unit* is exchanged by a *drilling unit* as shown in Figure 7.1(a) where the crane and its boom remain but with the addition of a drilling shaft and bit. The drilling unit depicted would be suitable for piles up to 12 inches in diameter and depths of 30 ft. in medium to soft soils. Larger diameters with deeper depths and through firmer soils would require the drilling unit to be installed on a drilling platform, which would provide for the higher torque requirements.

Figure 7.1 also graphically describes some other components which could become necessary depending on the design specifications. Graphic (b) shows a retractable under-reamer which can be used for the enlargement of the hole at its base, so to create a bell footing. Graphic (c) depicts a larger auger bit, and (d) an "elephant trunk" used as a tremie to facilitate pouring and at the same time avoid concrete mix spillage in the process.



PILE AUGERING EQUIPMENT
FIGURE 7.1

8.0 PILE TESTING METHODS

Building codes of the not too distant past commonly allowed pile testing procedures to be performed in a non-destructive protocol where the test pile was incrementally loaded to twice the design load. Such load was kept for a period of time while the settlements were closely monitored. During the second part of the program, the load was removed decrementally while the rebound (settlement recovery) was closely observed. If both, settlement and recovery remained within certain limits, the test was deemed as having passed and therefore approved by the Building Official.

Such described protocol was not only accepted in the industry, but it was used in thousand of locations through the nation. However, in the interim, important lessons were learned during those tests that tend to invalidate those widely accepted testing procedures of the past. Here is the most important lesson learned and transcribed verbatim from the proceedings of the American Wood Preservers Institute regarding testing on timber piles: "the total testing load tends to go out of the pile by *skin friction* very rapidly into the surrounding soil, and in normal conditions little of the load (if any) goes down to the tip. That observation is not true when piles are loaded to failure".

The above statement means that the pile mechanism of failure has a behavior which is time bound and by which the pile hangs on its friction capacity first and when all fails it uses its "last ditch defense": its tip. At that point the entire mechanism changes, making the capacity dependent entirely on the *tip resistance* all the way down to the point of failure. In order to cover those uncertainties, that is why and for as long as the calculations are based on empirical data, we recommend a factor of safety (called by some a "factor of ignorance") of 3.0, but never less than 2.50. However, when those calculations are based directly on observations and parameters obtained from the actual test of specimens which are representations of the real site conditions, that factor of safety may suffice as 2.0.

There are two important considerations contained into this learned lesson:

1. Since the actual capacity of a pile is not known until it is tested to failure, this mere statement invalidates the non-destructive pile tests of the past.

2. It comes to show what we have repeatedly indicated in prior courses, soil behavior is a time dependent phenomenon and therefore, there is a lack of synchronicity that tends to disallow application of the *law of superposition.*

Pile load testing has been well regimented by the ASTM Specification D1143-61T. Such document takes well into account and further recommends that testing piles be loaded incrementally to failure.

When it comes to the actual test set-up and instrumentation, we will show in the following pages some of the standard methods and their variations which have been successfully used in the Unites States and abroad.

The *cantilever testing method* of European extraction from the 1960's and consisting of a welded steel frame with either a single or double girder, the latter as shown in the following Figure 8.1. It should be noticed that depending of member size and loadings, the steel frame could develop large deflections which need to be carefully calculated. Although this method does not require any hydraulic jacks to generate the necessary compression load as the other methods do, however, it requires an adequate motorized mechanism to move and securely control the traveling of the counterweight.

The *Ebsary testing method* is greatly favored by many for its relative simplicity and its lower cost. Figures 8.2 and 8.3 cover the main features of such testing set-up which consists of a welded steel frame resting on a build-up of rough timber mat.

The steel frame is loaded with a stack of 10 ton precast concrete blocks bearing on each other until the required surcharge is achieved. The instrumentation is similar to those shown on the other methods herein described. With this set-up there is no need to have anchor piles, but just the test pile in the center of the lay-out. However, because of the irregularity of the timber mat, the system is difficult to level, plumb and/or stabilize and as such, the benefits are in an even trade-off with the drawbacks.

The *Standard testing method* is the most familiar and accepted pile testing procedure available and it takes at least five (5) piles to be performed as shown in Figure 8.4. Four of those piles serve as reaction piles to balance out the upward testing force applied by a hydraulic jack to the testing pile in the center.

Again, the redundant measuring system can be seen with more clarity in this case. Beyond redundancy, this design will also permit those readings to be confirmed one against the others.

INSTRUMENTATION UNIT

The instrumentation contained as part of the measuring unit may be as simple or as complicated as herein described in any of the following methods, for as long as the ASTM recommendations are observed. As a matter of general illustration, Figure 8.5 depicts a standard instrumentation unit used by several known foundation contractors on their larger jobs. The particular instrumentation covered by this detail applies very especially to any of the family of cast-in-place piles.

The basic data that needs to be recorded under the applied test loading procedure is the settlement of the test pile and the failure load. To assure reliable dependability and constant settlement readings, three separate and independent measuring systems are used. This guaranteed redundancy and confirmation is necessary should one of the systems fail. The primary system consists of two *dial extensometers* with a reading accuracy of 0.001 in. which are mounted on the opposite sides of the testing pile. Those gauges are intended to be supported and held by a frame work independent of the main frame system.

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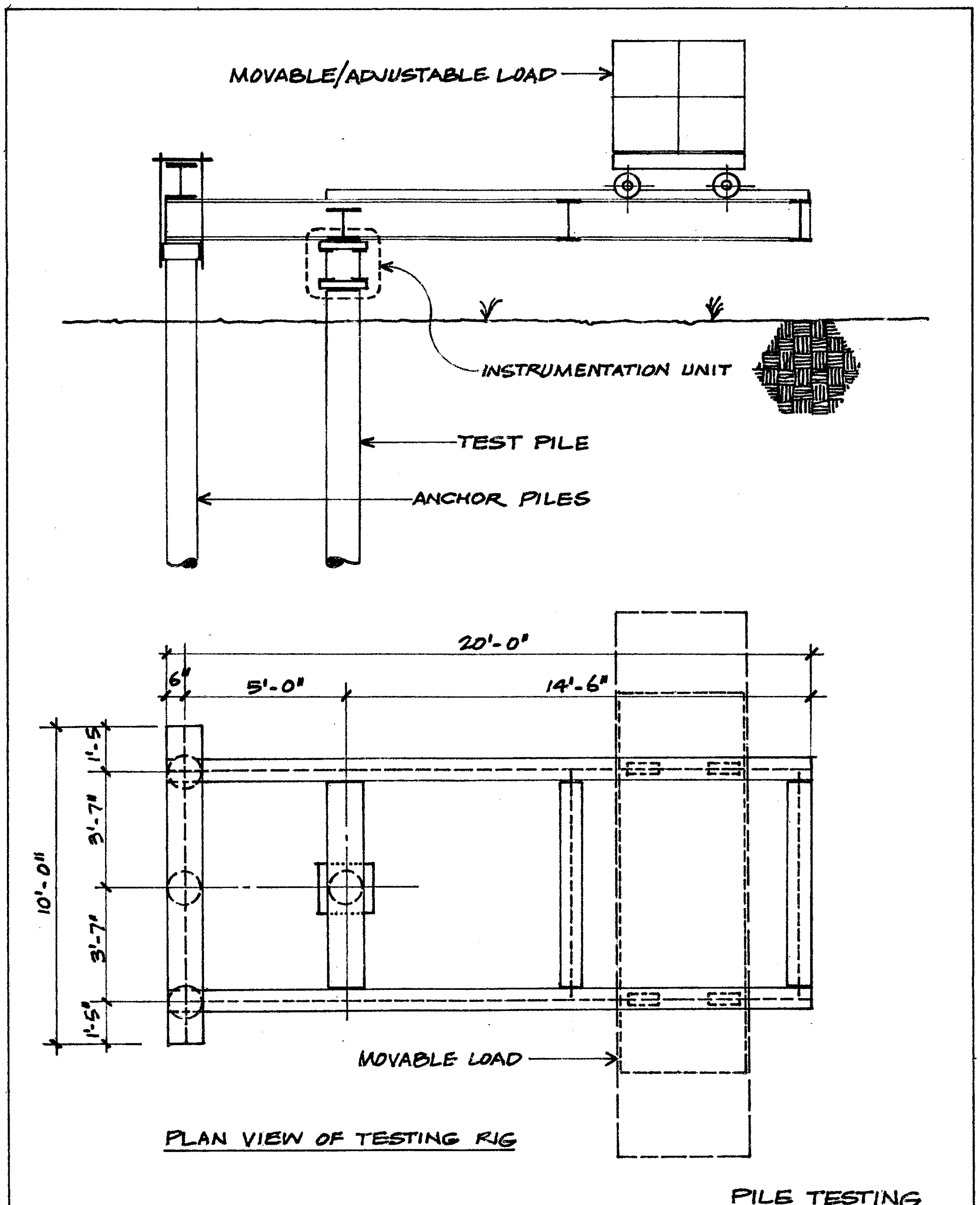
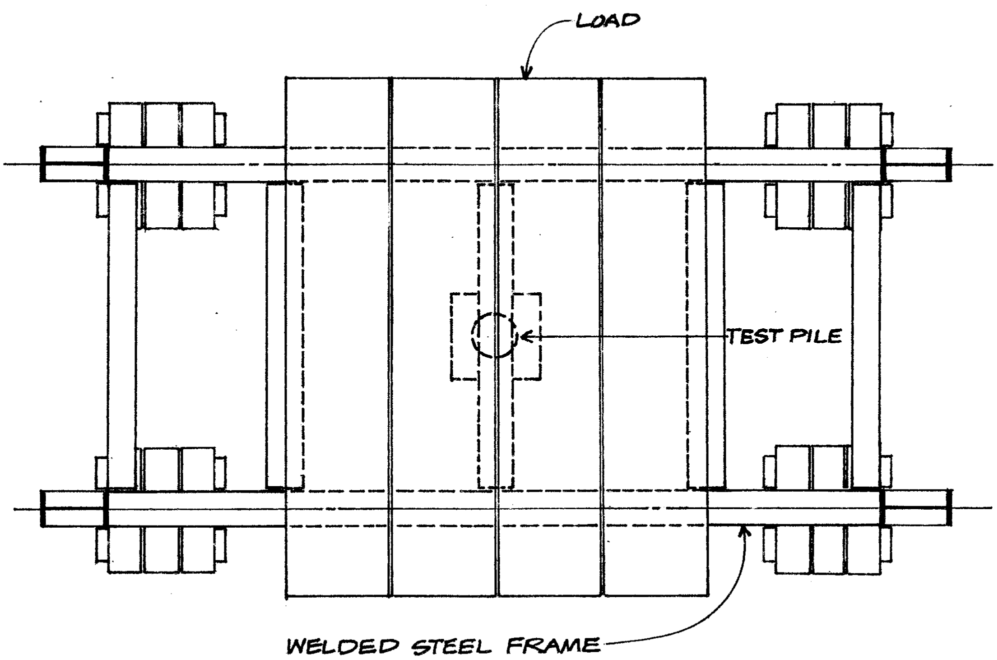
The secondary measuring system consists of a visible scale with graduations down to 0.02 inches to be read by using an engineer's level. The tertiary system consists of a high-strength wire under tension and mounted at 90 degrees in front of a mirror and another scale with graduations of 1/64 in. available for direct visual reading.

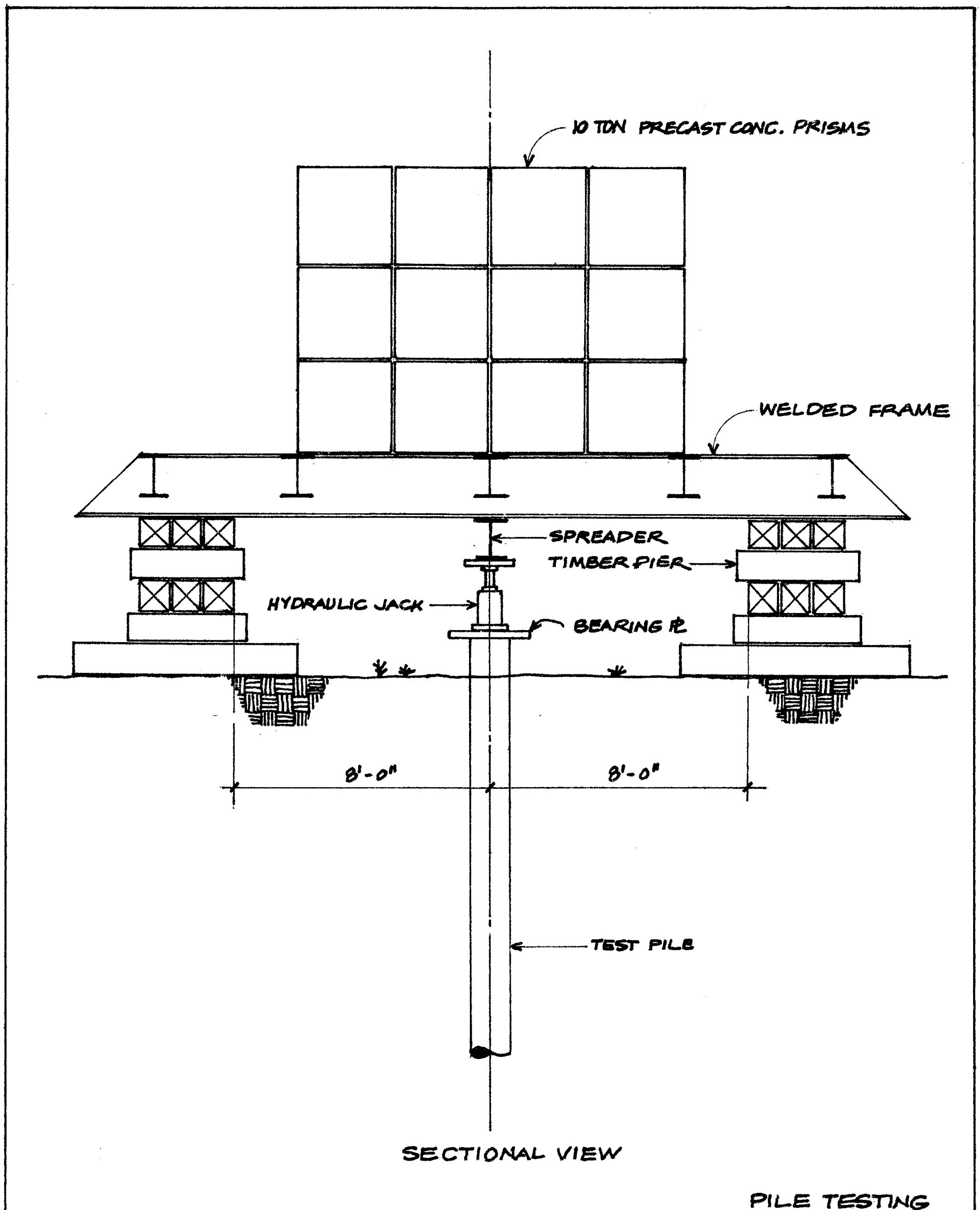


CANTILEVERED FRAME METHOD
FIGURE 8.1

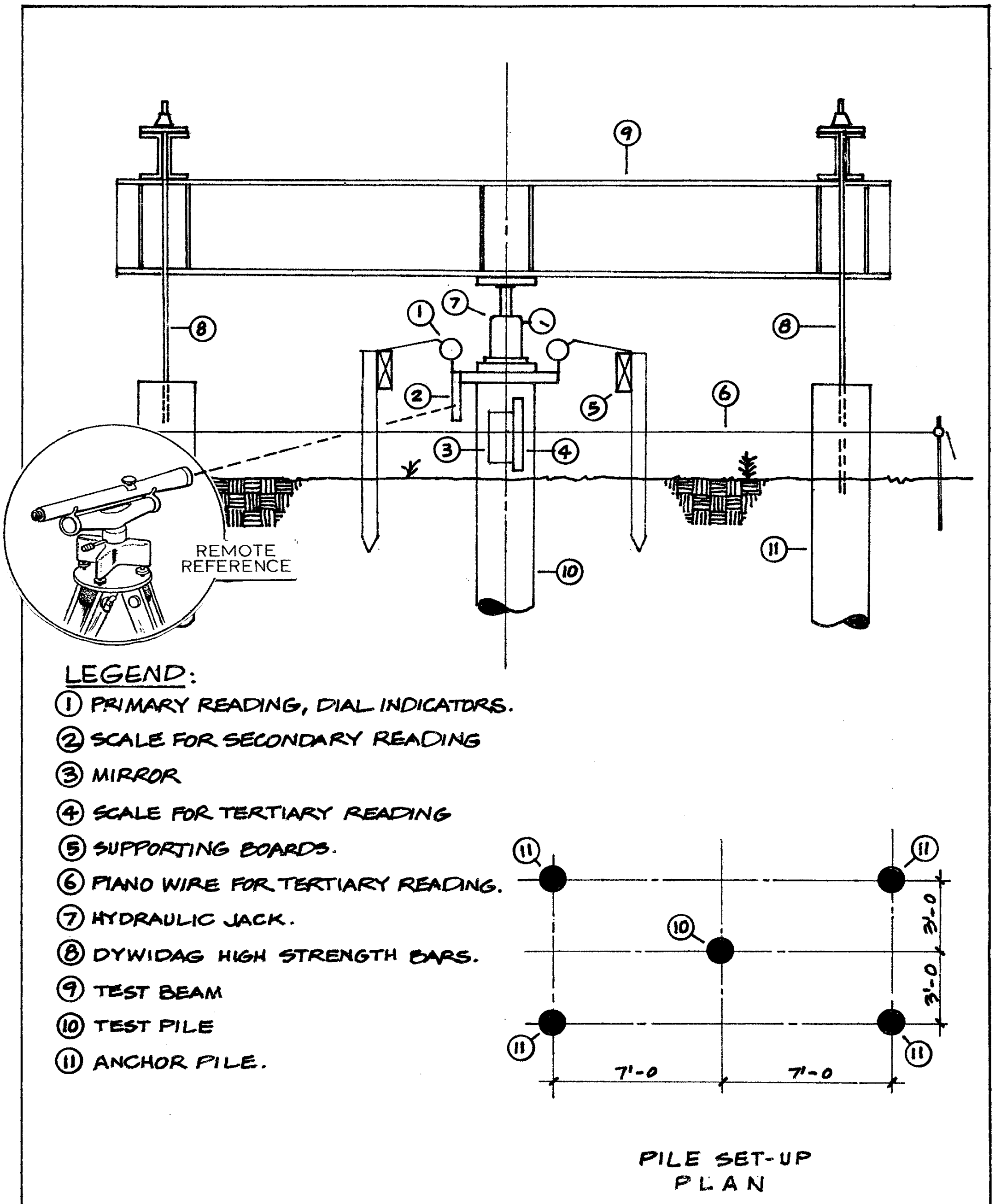


PILE TESTING SERIES

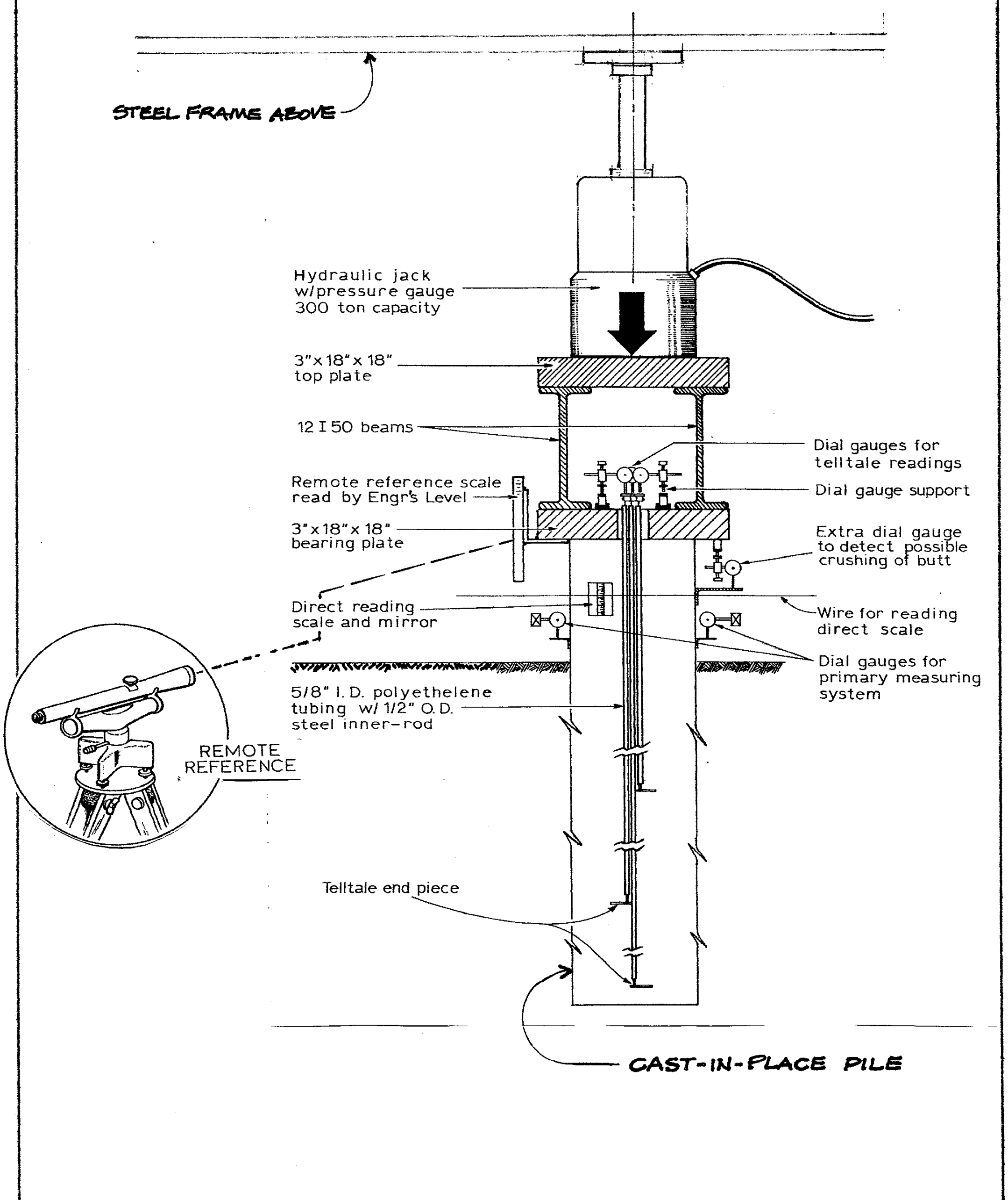
EBSARY TESTING METHOD
FIGURE 8.2



EBSARY TESTING METHOD
FIGURE 8.3



STANDARD TESTING METHOD
FIGURE 8.4



INSTRUMENTATION UNIT
FIGURE 8.5

9.0 CONCLUSION

Within the confines of this course's available space, we have made every possible effort to cover most of what you need to know about pile foundations, whatever we have missed you could likely get out of your own textbooks or the familiar reference books already in your own library.

As stated at the beginning, our intention and focus was centered on the needs of the junior engineer or those engineers not particularly versed in the art of pile foundations. Whether today or tomorrow is going to be your first day on the job, your first progress meeting, your first frightening "adversarial" encounter with the contractor(s) men, or your first job progress report; read this material before you are "thrown" into the arena and even further, take this material with you. Because once they discover you are new in this endeavor, the new kid in the block so-to-speak, those weathered and experienced men on the other side of the conference table are going to drop their weight around and try to intimidate you with all of their jargon, experience, viewpoints and questions. However, do not let them get the best of you, for you have the right company and the right tool just at the reach of your hand, and we are referring to the contents of this course. You are about to show them how much information you were able to assimilate in just your first reading.

As for the rest of the engineers who happen to read this material, perhaps this would be a refresher course where it all comes together from one single and concise source of information. Please feel assured that your comments will be welcomed, treated and answered on the basis of their own value and merits.

END

APPENDIX SECTION

We have reserved this section as a reference place containing the different field and recording forms that are useful to have at hand when the objective is to collect general data regarding pile inspections, pile handling, driving and testing. For that particular purpose and intent we have reproduced forms #072001rg06, DEB0012 and DEB0014 from our files and other related sources.

(FORM #072001rg06)

PILE DRIVING RECORDS

Project Name & Number:____

Weather Conditions: ____

Hammer Type:________________ Ram:____________ Fall (ft):______________

Pile #	Date Driven	Pile Size (in.)	Length (ft)	Blows Last Foot	Req'd Cap. (kips)	Actual Cap. (kips)	Remarks

Sheet #_____of_____

PILE TESTING LOG SHEET

PROJECT:

LOCATION:

DATE	TIME	LOAD INCREM. (TONS)	ACCUM. LOAD (TONS)	TIME LAPSE (HRS)	GAUGE (PSI.)	LOAD CELL (MICROS)	TEMP. (°F)	DIAL GAUGES (IN.) #1	DIAL GAUGES (IN.) #2	DIAL GAUGES (IN.) #3	DIAL GAUGES (IN.) #4	SCALE (IN.)	AVERAGE PILE HEAD (IN.)	REMARKS

TEST # ____________ FORM # DEB0012 SHEET # ______ OF ______

PILE DRIVING LOG SHEET

PROJECT NAME: ______________________ PROJECT NUMBER: ________

LOCATION: ______________________ DATE: ________

PILE DRIVER: ______________________

DEPTH	NUMBER OF BLOWS	REMARKS	DEPTH	NUMBER OF BLOWS	REMARKS
1			31		
2			32		
3			33		
4			34		
5			35		
6			36		
7			37		
8			38		
9			39		
10			40		
11			41		
12			42		
13			43		
14			44		
15			45		
16			46		
17			47		
18			48		
19			49		
20			50		
21			51		
22			52		
23			53		
24			54		
25			55		
26			56		
27			57		
28			58		
29			59		
30			60		

PILE # : ________

TYPE OF PILE: ________

TIME STARTED: ________

TIME FINISHED: ________

DRIVING TIME : ________ MIN.

CUT-OFF ELEV. : ________

DRIVING EQUIPMENT

a) MAKE: ________

b) HAMMER: ________

c) CHAMBER PRESS. ________ P.S.I.

d) HOSE LENGTH ________ FT.

e) EFFECT. ENERGY ________ FT.L

FORM # DEB0014

Since trips to the field can prove to be costly and time consuming, when the engineer goes out to a site inspection he/she would like to gather the maximum possible amount of information, both in quantity and quality, however, unexpected items brought up by the contractor or the emergence of unanticipated particular field conditions, may easily distract his/her attention and make him/her forget or overlook his/her main agenda. Consequently, we have also included herein a handy checklist of items to remember.

FIELD CHECK LIST

Date: Time: Weather:

Project Name____

Project Number____

Test Location____

Test Pile Data

Type of Pile (describe):____

Size:_______________________________ Length:_________________________

Material Specifications:____

Concrete Strength:____________________ Steel:___________________________

Splices (if any):____

Remarks:____

Pile Test Procedure

Loading Method:____

Step Load Termination Criterion:____

Rebound Termination Criterion:____

Ultimate Load:____

Instrumentation Type:____

Instrumentation Accuracy:____

Remarks:____
(Provide sketch on back, if necessary for clarity)

Observer's Name & Signature:____

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