

# PRESTRESSED CONCRETE PILES

by N. H. E. Weller\*

## Introduction

The design of precast piling is somewhat unique in structural engineering, as the pile is required to satisfy a number of only semi-related conditions, which are:—

1. *Total and nature of load to be carried.*
2. *Foundation conditions.*
3. *Handling of pile prior to driving.*
4. *Driving conditions of:*
  - (a) *Different strata to be driven through;*
  - (b) *Design load required;*
  - (c) *Driving equipment, tolerances, site control and head packing.*

The inherent variability of the soil, driving equipment and head packing, and the actual concrete strengths have made factual analysis of pile stresses and carrying capacities exceedingly difficult. Because of this, with the advent of a new type of pile, the tendency has been to develop empirical rules of design, of load capacity and of determining driving conditions. This has resulted in adoption, for different types of piles, of a range of pile sizes and designs for standard loadings. The actual structural design of these piles has largely been aimed at developing piles which do not give trouble in driving under the various driving conditions that may be en-

countered. Lack of the precise analysis of driving stresses has limited explanations of driving failure to general terms rather than to precise figures. Because of these factors, the following arbitrary basis for design of prestressed piles seems to have become adopted, mainly to suit driving conditions.

1. Apply an initial longitudinal prestress of 1000 p.s.i. to the pile.
2. Drive the pile with a hammer of weight approaching that of the pile. (This condition had been adopted to a limited extent for driving reinforced concrete piles prior to the advent of prestressed concrete piles).

The reasons for the economy and other advantages of prestressed concrete piles are as follows:—

## 1. Size of Pile and Equipment

The length of pile required is a function of foundation conditions. With deep foundations, the pile size and reinforcement were determined by the handling stresses. Since the modulus of a prestressed concrete pile is based on the full cross sectional area, while that of the reinforced concrete pile is based on the transformed area, a smaller cross section is possible with prestressed concrete piles to resist a given bending moment. This enables prestressed piles to be of smaller cross section, which also allows the use of lighter handling and driving equipment.

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## 2. Durability

The prestressed pile is composed of high strength dense concrete maintained permanently in compression. Its strength, under normal working conditions, is not dependent on the steel being close to the face of the concrete, as is the case with reinforced concrete. Consequently, the pile should be free of fine cracks and of the subsequent rusting of the longitudinal reinforcement and spalling of the concrete cover. For this reason, prestressed piles are very suitable for use in corrosive marine or atmospheric conditions, as for example in tropical harbours and industrial works.

## 3. Resilience

The prestressed pile, being smaller and more able to accommodate a relatively large concrete strain without cracking, is much more capable of absorbing impact forces. This is of assistance in marine structures.

Prior to the last war, the majority of the research work on piles had been aimed at the development of formulae to estimate the carrying capacity of the pile from the final driving conditions. This work had been based on calculating the resistance of driving " $Ru$ ", using a dynamic formula and taking some fraction of " $Ru$ " as the safe static bearing capacity of the pile. This work led to the development of a large number of empirical formulae, of which the Hiley Formula seems to have been the most widely adopted. To obtain comparable results from the various formulae, it was necessary to allow different factors of safety, and also to make different allowances for either build up of static resistance after driving,

or reduction of static resistance after driving due to later consolidation of the higher strata of soil. These formulae, combined with experience and test, have served, and still serve, as our basis for estimating the carrying capacity of the pile. They do not, however, enable accurate estimation of driving stresses.

D. V. Isaacs<sup>(1)</sup> proposed, in 1931, that it was possible to analyse pile stresses by applying the stress wave theory to the pile driving. An attempt was made to develop pile formulae using the stress wave theory in 1938. Latterly, further work has been done by E. A. Smith<sup>(2)(3)</sup>, D. H. Lee<sup>(4)</sup>, and others on pile formulae, using the stress wave theory as the basis. E. A. Smith has made some actual analyses of resistance to penetration based on the stress-wave theory, using a digital computer. Comparison of these results with existing formulae has been very informative, but no simple practical formula based on the stress-wave theory appears to be possible.

## Driving Stresses

The stress-wave theory has now been accepted as the modern basis for analysis of driving stresses. The theory briefly assumes that the hammer blow causes a compressive wave to travel along the pile at a velocity

$$V = \frac{\sqrt{E}}{\rho}$$

where  $E$  is the modulus of elasticity and  $\rho$  the mass per unit volume. For an  $E = 5 \times 10^6$ ,  $V = 9,500$  feet per second. The length and shape of this compressive wave is governed by the duration of impact of the hammer. The magnitude of the compressive stress rapidly builds up from zero at the instant of impact to a maximum figure, then tails off

again to zero at the cessation of the hammer blow.

Fig. No. 1 shows stress time graphs taken by Mr. Beresford of C.S.I.R.O. on behalf of John Holland & Co. The graphs were taken at a point 25 ft. from the head of 14 in. sq. pretensioned piles 70 ft. long. The piles were being driven with a four-ton steam hammer using a one-foot drop. The set per blow was about six inches. It will be noticed that at the commencement of the graphs a sharp peak in compressive stress occurs which represents the stress wave due to hammer blow.

For an infinitely long pile, this compressive wave would merely continue to run along the pole. With a completely free pile, the wave is reflected at the toe and returns up along the pile as a tensile stress of only slightly reduced intensity. In practice, the returning tensile wave is affected by the frictional resistance of the soil. In very soft ground, the conditions of the free pile is approached (as in the graphs of Fig.

No. 1), while in cases where the pile is driven to refusal, the wave returns up the pile as a compressive wave. In the intermediate conditions the intensity of the tensile wave is reduced.

In Fig. No. 1, the maximum compressive stress varied from 1200-1500 p.s.i. and the maximum tensile stress was about 500 p.s.i. The Young's Modulus of the actual pile tested was 6.4 million, giving a stresswave length of about 180 feet. As a result the returning tension wave would have been reduced in magnitude by the tail of the compressive wave. No cracking occurred in the test pile, but some had taken place in piles driven earlier.

This cracking may have occurred if piles having lower Young's Modulus were driven with harder head packing.

Alternatively, the tensile stresses may have been magnified by eccentricities in the manufacture or in the driving.

Consider the case where the time

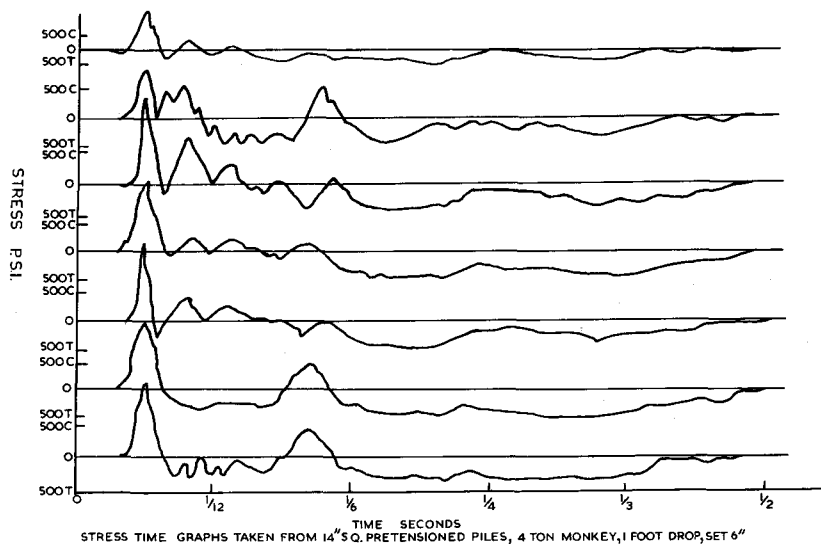


FIG.1 DRIVING=STRESS TIME GRAPH

Figure 1—Driving Stress Time Graph.

TABLE I

| Length | Distance driven | Av. no. blows per in. when driving stopped | Remarks                                      |
|--------|-----------------|--------------------------------------------|----------------------------------------------|
| 40 ft. | 38 ft.          | 5                                          | Horizontal crack at lifting point.           |
| 55 ft. | 5 ft.           | 4                                          | Horizontal crack 12 ft. from head.           |
| 55 ft. | 24 ft.          | 10                                         | Horizontal crack.                            |
| 55 ft. | 13 ft.          | 16                                         | Partial crack. Grew larger as set reduced.   |
| 40 ft. | 27 ft. 3 in.    | 13                                         | Cracks 8 ft. 6 in. & 11 ft. 6 in. from head. |

of the impact and the length of the wave (or blow) is very short and the pile is free. In this case, the magnitude of the tensile stress approaches the magnitude of the compressive stress. The maximum compressive stress from the hammer blow encountered in most pile-driving lies between 1000 p.s.i. and 2,500. Therefore, if tensions are allowed to develop approaching this figure, tension cracking will develop in concrete piles.

In practice, however, the normal time of impact of the hammer blow probably varies from about .005 seconds to .08 seconds (0.016 seconds Fig. No. 1), giving  $E = 5 \times 10^6$  a length of wave varying from 50 feet to 760 feet. For lower values of  $E$ , the minimum length could be even shorter.

Consider a 50 ft. pile driven freely with a 50 ft. length of wave. Under these conditions, the front of the returning wave will have reached the mid point of the pile just as the compressive tail passes, and it is evident that induced tensile stresses adequate to cause tensile cracks can occur. If the wave length had been say 150 ft., the returning tensile stress would have been partly compensated for by the later compressive part of the wave, and tensile stresses would have been reduced to about half of those obtained with the 50 ft. wave length. That is to say, the potential tensile stress depends on the relationship of length

of pile to length of wave; consequently, danger of "tensile" cracking is greater with longer piles.

This problem was encountered by the author when driving a number of 14 in. square post-tensioned piles of length varying from 40 feet to 65 feet. These piles were to be completely driven. They were designed as 65 ton piles and were to be driven through clay to a sand clay strata. The piles were designed for driving with a 32 ft. kip blow (4 ton hammer 4 ft. drop) and were post-tensioned with a theoretical  $P/A$  after losses of 625-740 p.s.i. Because of limitations of equipment available, driving commenced with a steam hammer having a 2.5 ton ram and 4 ft. drop using 3 in. Oregon head packing. While some piles were driven satisfactorily, tension cracks developed in others.

These cracks probably occurred as hair cracks in the early stages, and then with harder driving opened further and fretted as ground resistance increased to a point where driving was discontinued.

At this stage, the small hammer was discarded. Driving was later recommended with a 4 ton hammer (4 ft. maximum drop) and a steam hammer (13.5 ft. tons). With the new equipment, development of tension cracks ceased, and partly driven piles were driven to depth. It was also possible to complete the driving of piles previously cracked. However, rebound of the hammer

TABLE II

| Case                    | Wt. of hammer<br>wt. of pile | P/A after<br>losses | Max. set per<br>blow of test<br>drop | Max. set per<br>blow of $\frac{1}{2}$<br>drop |
|-------------------------|------------------------------|---------------------|--------------------------------------|-----------------------------------------------|
| General Case .....      | 1.0                          | 800                 | 1"                                   | 2½"                                           |
| Under 45'-0" Long ..... | 1.0                          | 650                 | 1"                                   | 2½"                                           |
| Under 45'-0" Long ..... | 1.0                          | 500                 | 0.75"                                | 2"                                            |
| Over 45'-0" Long .....  | 0.75                         | 800                 | 0.75"                                | 1½"                                           |
| Over 45'-0" Long .....  | 0.67                         | 800                 | 0.5"                                 | 1"                                            |
| Over 45'-0" Long .....  | 0.75                         | 600                 | 0.4"                                 | 0.75"                                         |

was still noticeable with the heavier 4 ton hammer. Fig. No. 2 shows a group of these piles on which driving had ceased prior to change of driving equipment.

It has been shown previously that the magnitude of the potential tensile stress is a function of:—

1. Pile length.
2. Foot resistance.
3. Side friction.
4. The maximum compressive stress from the hammer blow.
5. Length of compressive wave.

To obtain economy in piles where the tensile stresses required in handling and in the final structure are low, we must control Items 4 and 5.

The total compressive stress in the pile varies directly with the energy of the blow.

The length of the compressive wave increases with increase in  $E$  and with duration of the hammer blow.

The duration of the hammer blow  
(a) reduces with ratio of weight of hammer, over weight of pile;

(b) Reduces with increased stiffness of the head packing.

It is interesting to note here that piles with a "lower" prestress ought to be manufactured, if possible with a greater value of  $E$ , to maintain margins of safety in driving. That is to say, if the prestress is reduced, the concrete strength ought probably to be increased.

Table II is put forward as a guide

which may be used as a rough check for particular sites and conditions.

It will be noted that Table II quotes a maximum set per blow. This maximum refers to the initial stages of driving, and, to abide by it, it is necessary to be able to vary the hammer drop in the initial stages. If this is not practicable, as for example when using a steam hammer, a head packing which is soft and resilient at commencement of driving should be used, both to lessen the maximum compressive stress in the pile and to increase the wave length. Such head packing should have a  $K/A$  figure (see Table II) of not more than 6000 lbs/sq. inch/inch.

Table II assumes that the piles are being driven to normal safe load capacities of the order of:—

12 in. square — 35 tons.

14 in. square — 45 tons.

15 in. square — 55 tons.

Fig. No. 2 shows a 16 in. octagonal 50 ft. long pile ( $P/A$  after losses of 780 p.s.i.) being driven at Homebush with a 4 ton monkey to a set of 5 blows per inch for a drop of four feet. In the initial stages, the drop used was 1 ft. and then 2 ft-0 in. to limit the set per blow to 2 inches.

**Field Control of Driving:** Because prestressed concrete piles are in the nature of a high-quality product, they are designed for high stresses,



Figure 2-16 in octagonal pile-driver with 4-ton monkey.

and consequently require good quality driving practice.

#### (a) Variation of Drop Hammer

This has been accepted as good practice in driving reinforced concrete piles for a long time, and unless special provision is made on the lines indicated it should be adhered to.

#### (b) Eccentricity of Blow

The anticipated eccentricity of blow is a function of the frame and hammer tolerances. Prior to driving, the hammer position should be checked to see that it will theoretically give a central blow. The fall of the hammer should then be observed over the course of the driving to maintain a central blow. This eccentricity of blow normally results from wear in the leaders, or the hammer guides, or in the deterioration of the frame and/or pitching the pile out of position.

On one project inspected by the author, some 180-12 in. sq. pre-tensioned piles had been driven with

only four failures, and then suddenly six failures occurred in 20 piles. These piles were driven with a 2 ton monkey falling 6 feet. The piles in the group which failed generally ran from RL-13 to -17 at 10 blows per 1¾ in.-2 in., and then from RL-17 to -25 at 10 blows per 1 in. The total penetration was 32-37 feet. Just prior to inspection, the hammer drop had been decreased from 6 feet to 4 feet except for the final test, and the trouble had been largely overcome.

A check of the pile frame showed the leaders to be 5¾ in. wide at the top and 6½ in. wide at the bottom. The throat of the monkey was 5¼ in. wide. The frame had been built with two head pulleys, both off center, one pulley for pitching and one for driving. Because of this, the monkey developed a definite thrust in one direction, and this most probably had contributed to the deformation of the leaders. Observations of the driving indicated that the hammer appeared to drive, at times, with an eccentricity of up to 1½ in. to 2 in. on the top of the pile. A 2 in. eccentricity on a 12 in. pile would double the magnitude of the compressive head stresses. A number of these piles had sheared at points 2 ft.-10 ft. below the head and at an angle somewhat less than 45°. This could have been a compressive failure which occurred at the weakest section of concrete. It did occur in some cases through the jet pipe inlet and the top fixing hole.

#### (c) Holes in Piles

The forming of holes through piles for lifting holes or jetpipe inlets should be avoided if possible. They form a definite stress-raiser, and when associated with local under-compaction are a source of trouble.

#### (d) Control of Pile Head

A cast fabricated M.S. helmet about  $\frac{1}{2}$  in. oversize, fabricated with a tongue for attaching to the frame, is always worth-while. The use of a through bolt and plate for keeping the pile in position is undesirable, as it may cause unnecessary secondary stresses.

### Types of Piles

#### (a) Pretensioned

Prestressed concrete piles in most frequent use are pretensioned. They are normally manufactured as solid square or octagonal piles. The octagonal pile has a slight dimensional advantage for driving conditions, and is more suitable for the use of continuous spiral secondary reinforcement.

These piles, with their fully bonded steel, give a good crack pattern under overload conditions, and can be cut off and bonded into headstocks as can normal reinforced concrete piles. Since it is impractical to extend them as prestressed piles, they are normally extended as reinforced concrete piles or by splicing, using a steel sleeve similar to that used for composite timber and concrete piles.

#### (b) Post-tensioned Piles

Because of the end anchorage cost associated with post-tensioned piles, they are normally only competitive with pretensioned piles when they can be made on the site or are over 40 feet long. Simple square or rectangular cross-sections are normally used except for long lengths or large diameters when they are usually made hollow and stressed together in sections. The general design of post-tensioned piles is the same as for pretensioned piles apart from the detail of the end anchorages, where helical or other binding is

used to strengthen the concrete locally. With a greater force per tendon the steel cover is normally greater and this combined with the added protection obtained from the duct forming tube contributes to a pile of maximum durability.

### Piles Post-tensioned With Wire Cables

Piles post-tensioned with wire cables may be cut off in the normal way without loss of prestress providing the grouting of the cables is adequate and is well controlled. They are normally extended in reinforced concrete, sometimes by exposing mild steel bars cast in the head and sometimes by casing the head and extending the pile using a larger cross section.

### Piles Post-tensioned With Macalloy Bars

These piles may be grouted with special control to avoid loss of prestress when cut off. Such bond, however, from the grout is not usually sufficient in cases where the pile is subsequently driven again. However, the prestress may be made definitely effective again by forming a new anchorage and prestressing the pile using Macalloy wedge anchorage grips.

Piles may be extended by coupling on an additional length of bar, and either casting an extension length of pile in place or, more usually, jointing on a further precast section. This property can be used to advantage in long piles, the cross-section of which is controlled by handling stresses. It makes possible the use of piles of smaller cross section. Joining on an extension length permits the use of smaller pile frames, and can be economical for long piles both on land and in water. Any of the methods listed in

TABLE III

| <i>Jointing Material</i>      | <i>Time before Re-driving</i> |
|-------------------------------|-------------------------------|
| Ciment Fondu                  | 24 hours                      |
| High Strength Portland Cement | 3 days                        |
| Polyester Resin               | 1 hour                        |

Table III may be used for jointing.

In the case of piles driven at Wansworth, London, the initial section was driven about 18 feet, the resin placed on the joint, the next section placed in position, and the bars coupled and stressed. The driving was continued shortly after  $\frac{1}{2}$  hour, to a set of  $\frac{1}{4}$  in. using a four ton hammer. No extrusion or distress appeared at the joint. The top section was later pulled off, failure occurring in the concrete just above the jointing material.

The resin used was "Artrite 13-10"<sup>(5)</sup> which has a compressive strength of about 12,000 psi and a tensile strength of about 1500 psi. The material is supplied as a resinous liquid and a powder which are mixed together on the site and which takes about 15 minutes to set.

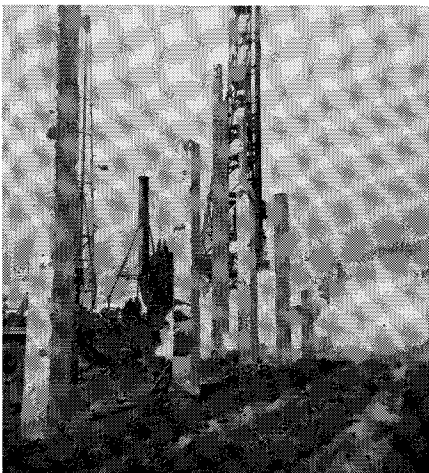


Figure 3—Partially driven piles.

Fig. No. 9 shows a number of 18 in. and 14 in. square piles, post-tensioned using Macalloy bars, used in the new 1000 ft. long Riverside Quay at Hull, England. These were friction piles seventy feet long, and were precast in three sections, with the center section hollow for lightness. The top framing and bracing were also precast, the whole being made continuous using an insitu concrete deck.

Fig. No. 5 shows the new Thames Jetty at Erith, which was recently constructed on 6 ft. diameter cylinders having a  $6\frac{1}{4}$  in. thick wall. These cylinders were precast in approximate five-foot lengths, jointed, stressed longitudinally with Macalloy bars, floated vertically into location, and sunk to a chalk stratum below the river bed.

### Post-tensioned hollow tube piles

Post-tensioned hollow tube piles have been used quite extensively in America by the Raymond Pile Co. These piles are of the order of 4 to 6 feet diameter with 5 inch walls. The piles are cast in sections with cored holes; the sections are then aligned horizontally, the longitudinal tendons threaded through, and the whole post-tensioned together. The piles are later driven from the top, in the manner of standard piles. This type with its large modulus and relatively light section, has become favoured in both America and the United Kingdom for deep foundations in water.

TABLE IV

| Weight of Hammer and Helmet<br>Weight of one foot of pile |        | 25     |        | 35     |  |
|-----------------------------------------------------------|--------|--------|--------|--------|--|
| Head packing stiffness .....                              | 10,000 | 20,000 | 10,000 | 20,000 |  |
| Maximum allowable free fall ...                           | 4'-3"  | 3'-0"  | 3'-6"  | 2'-6"  |  |
| Minimum set. Blows/inch .....                             | 7      | 10     | 5      | 9      |  |

## Pile Loads

I do not propose to discuss in any detail the various piling formulae. However, Mr. Donovan Lee<sup>(5)</sup> has put forward a much simplified approach based on the Stress Wave Theory. In it, he develops a relationship between the ground resistance at the time of driving and the peak pile stress as described later.

The driving conditions listed in Table IV are such as to cause a maximum anticipated compressive stress in the pile of 2,500 p.s.i.

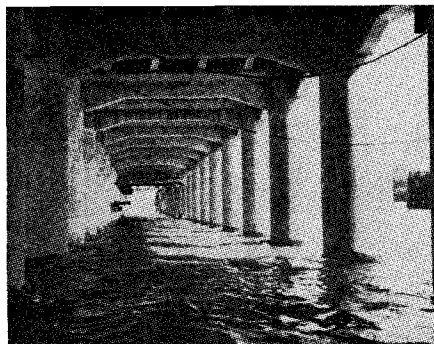


Figure 4—Post-tensioned pile hull.

A head packing stiffness of 10,000 lbs./sq. inch/inch at a head stress of 2,500 lbs./sq. in. is obtained with the following approximate thickness of different materials, measured, when compressed under the static weight of the hammer, after 25 blows.

2½ in. sacking or felt;  
or 3 in. rope;  
or 1½ in. rubber.

This thickness when halved gives

a head packing stiffness of 20,000 lbs./sq. inch/inch. The stiffness ( $k/A$ ) of the head packing is also proportional to the stress.

It is not possible to give figures for equivalent thickness of soft timber packings, as the stiffness is variable and also increases considerably with repeated impact.

When the pile is at the end of driving, i.e. at its minimum set, with the peak foot stress at the maximum allowable value, then the peak applied force at the foot  $F$  is known. This force  $F$  is balanced by a resistance to penetration  $R$  plus an inertia force  $mf$ , where  $f$  is the toe acceleration and  $m$  is a representative mass. Calculations show that  $R$  may be about 70% of the peak value of  $F$ ; hence, provided the pile is driven to the limiting conditions, the resistance to penetration is given by  $R = 0.7 \times \text{area} \times \text{maximum allowable compressive stress} \dots (A)$ .

In the case of a typical pile, the resistance to penetration is composed of two parts—the side fric-



Figure 5—Looking downstream beneath the jetty head.

tion  $R_f$  and the toe resistance  $R_t$ . As a consequence, despite driving to the minimum set, the peak foot stress will generally be less than the allowable value. The engineer will not usually know the relative proportions of toe and frictional resistance but the total resistance to penetration will not be less than that given by the expression (A).

For a maximum pile compressive stress of 2,500 p.s.i., the formula reduces to:

$$R = 0.78 \times \text{area Pile in sq inches (TONS).}$$

This figure of ground resistance should be modified, depending on the nature of the foundation, to ob-

tain the ultimate static bearing capacity for a permissible settlement. In general, the resistance to penetration will considerably exceed the ultimate bearing capacity as determined above, except for essentially granular soils of reasonably high permeability.

## References

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