OCCASION

This publication has been made available to the public on the occasion of the 50th anniversary of the United Nations Industrial Development Organisation.

DISCLAIMER

FAIR USE POLICY

CONTACT

Please contact publications@unido.org for further information concerning UNIDO publications.

For more information about UNIDO, please visit us at www.unido.org

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION
Vienna International Centre, P.O. Box 300, 1400 Vienna, Austria
Tel: (+43-1) 26026-0 · www.unido.org · unido@unido.org

18558

Distr.
RESTRICTED

IO/R.168
28 August 1990

UNITED NATIONS
INDUSTRIAL DEVELOPMENT ORGANIZATION

ORIGINAL: ENGLISH

INTRA-REGIONAL COOPERATION IN DEVELOPMENT OF PLANTATION-BASED FOREST INDUSTRIES

DU/RAF/87/117

Technical Report: Timber Pile Driving*

Prepared for the Preferential Trade Area for Eastern and Southern African States
(PTA)
by the United Nations Industrial Development Organization,
associated agency of the Food and Agriculture Organization
of the United Nations, which acted as executing agency for the
United Nations Development Programme

Based on the work of C.R. Francis
Timber Engineering Consultant*

Backstopping Officer: R.M. Hallett
Industrial Management and Rehabilitation Branch

United Nations Industrial Development Organization
Vienna

* This document has not been edited.

V.90-87347

CONTENTS

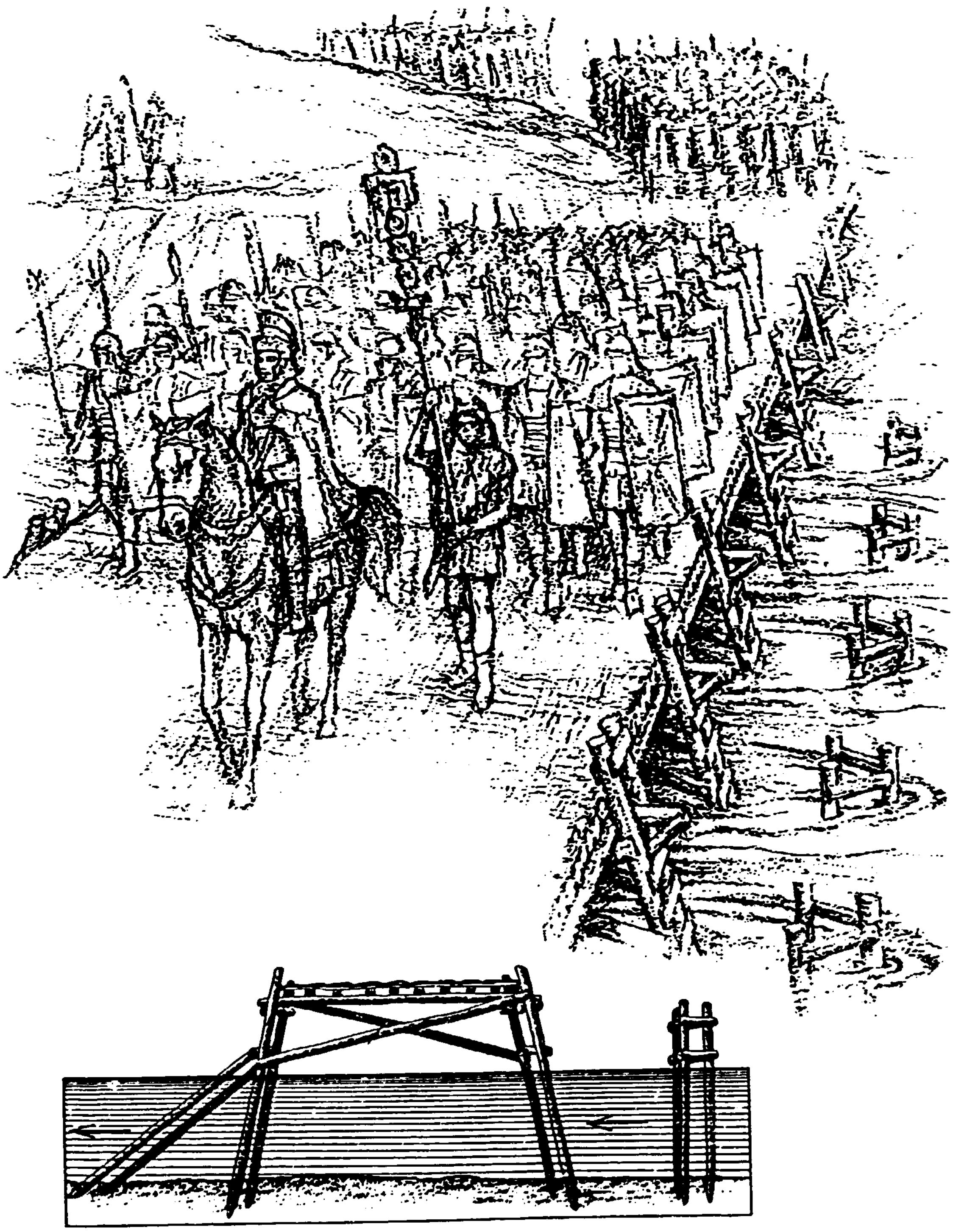


Figure 1

Julius Caesar's military bridge across the river Rhine. Reproduced from "The Battle for Gaul" by permission of Russel Sharp Ltd.

1. INTRODUCTION

Timber piles have been used for bridge foundations for thousands of years. Perhaps the best documented ancient piled bridge was Julius Caesar's military assault bridge across the river Rhine (55 BC). This bridge only had a life of a few weeks (he prudently demolished it at the end of his German campaign) but the speed with which it was built (about 10 days) demonstrates that the techniques of pile driving must have been very familiar to Roman military engineers. Fig 1. shows an artist's impression of this bridge.

The use of timber piles in industrialised countries in recent years has given way to concrete and steel piled structures. This has been due to a number of factors - the development of large diameter boring rigs, the much higher foundation loads imposed by modern long span bridges and perhaps not least, the lack of availability of suitable timber. Even where timber piles are used in industrialised countries, the piling rigs used, bear little resemblence to the equipment of only a few decades ago. Crane-held hanging leaders now hold diesel or pneumatic hammers giving over 100 blows per minute, and piles are driven with gangs of only four or five men. The technique of slowly winching up a block of cast iron and letting it fall on the pile head held by a timber derrick is all but forgotten.

Yet it should be remembered that the transportation networks, both road and rail, or such industrialised countries as the USA, Canada, Australia and New Zealand were largely connected by bridges founded on gravity-hammer driven timber piles.

The reasons for this are simple. These countries were short of development capital, they had suitable timber available, and the experience reaching back into a now forgotten European tradition - a tradition which eventually reaches back to Julius Caesar and beyond.

Many developing countries are now in the same situation as the above countries. The necessity for a road network for transportation is widely recognized and development capital is in short supply. Indigenous or treated timbers for piles are frequently widely available. (Pile timbers are not very good sawlogs). However, enquiries for pile driving equipment will result in offers for crawler cranes, subsidiary equipment and power hammers costing perhaps a million dollars - which these countries simply cannot afford.

This report does not claim to be a treatise on piled foundations. It contains some design data peculiar to timber piles, also advice on techniques. It presumes that, for the theoretical content at least, the reader is a professional civil engineer, experienced in bridge design and construction. Much of the theory and practice of driving concrete or steel piles is equally applicable to timber. Apart from mass, perhaps, a timber pile most closely resembles a precast prestressed tapered concrete pile. Much of the literature and experience with prestressed concrete is equally relevant to timber.

2. EQUIPMENT

There are three essential items of equipment for pile driving, plus several minor items which may be of use. Also, availability of general heavy hand tools - sledge hammers, shovels, jacks, spanners etc. is assumed.

Ropes, both steel and fibre are described by diameter. Steel wire rope is abbreviated to SWR.

2.1. Hammer The pile hammer is the basic item. It is simply a large piece of cast iron with suitable guides and a lifting eye, which is hoisted up and then let fall to hit the pile into the ground. (In some countries the hammer is commonly called a "monkey", presumably because it jumps up and down).

Details of hammers of 750 kg, 1000 kg and 1500 kg are shown in figures 2, 3 and 4. Within the limits of the handling equipment available, the heavier the hammer the better, although this should be tempered to the size of the piles to be driven. However, for general work a minimum weight of 1000 kg should be taken, and if gravel or boulders or deep depths are generally encountered, 1500 kg or 2000 kg should be used. The capacity of the avaiable winch must also be taken into account when deciding the hammer weight

It is only very large foundries which can undertake castings of this size. The advice of UNIDO may be sought if the casting is beyond the capacity of a local foundry. In this case, shipping costs must be allowed for.

A casting specification is given in Appendix A.

Besides the hammer proper, two ties and keys are also required. These are also heavy fabricated components, and if the hammer is being cast in another country, it may be advisable also to have these made there, where a heavy engineering industry probably exists. Then the fit of the whole assembly can be made the responsibility of the supplying foundry.

2.2 Winch The second essential item of equipment is a motorised two-drum winch. At least one, preferably both of these drums should have a direct line pull capability of the weight of the hammer. The drum to be used for driving should be capable of free running, that is it should run free when disconnected by its clutch from the driving gear. Thus worm or hydraulic driven drums are not preferred.

The writer has been in many developing countries which have a "machinery graveyard' full of wrecked and worn out trucks and machines. Inevitably there have been several tracked excavators in varying states of decrepitude.

The heart of a multi-purpose excavator (dragline, power shovel or simple crane) is a two-drum winch. The travel and slewing mechanisms are subsidiary, and it is generally these which give out. The winch mechanism has been observed to be generally in at least repairable condition, often with good condition SWR on the luffing drum.

A competent mechanical engineer should survey the derelict excavators available, and select one which has the capability to handle the loads involved in pile driving. The condition of the motor is not very relevant, in fact the excavator's original engine will probably be greatly oversized for its winch duties.

In practice, the theoretical horsepower required for driving is less than 5 - the excavator's engine will be sized to its travelling and slewing power requirements.

Just about any car or small truck motor could be used to power the winch, however, a diesel is preferable to a petrol engine for reasons of fuel economy under prolonged running, and the ability of a diesel engine to "lug" under suddenly applied torque at low revolutions.

No detailed advice can be be given, only a general requirement specification. Even this must be reconsidered in the light of the actual hammer weight and pile sizes and weights to be handled.

Line pull - both drums : 250 kN

Hoist speed, at least one drum : 100 - 120 mm/sec

Braking : Ratchet lock-on on both drums, disengagable

Fuel capacity : 8 hours

Base : Skid mounted, skidding and lifting eyes

Rope exit : just above skid base, horizontal, through live fairlead

Stability : With maximum line pull on both rope, safety factor against lifting when pivoted at level of top of skid base - 1.5

It may be necessary to provide a tail counterweight to the winch to provide this degree of stability.

The salvage, reconditioning and re-powering of the winch will be a fairly major operation, but whould not be impossible, given a competent entineer and a good mechanic with reasonab le workshop facilities.

2.3 Derrick The hammer needs guidance to ensure that it always lands square on top of the pile. This guidance is provided by a pair of leaders, spaced members between which the hammer slides. There are various ways in which the leaders may be fixed. In the equipment described here, they form part of a derrick, a tall pyramidal timber structure.

A typical 10 m high timber derrick is shown in figures 5, 6, 7 and 8.

The leaders in this derrick consist of 150 x 100 timber housed into 150 x 75 steel channel. The headers are rigidly connected together every 2.5 m by the platform members, with space behind them to let the hammer keepers travel the full height of the derrick.

The derrick should be accurately constructed of clear, straight grained strong tough timber of stress grade F 14 or better. Old growth Douglas fir has been the traditional choice, but other species of comparable mechanical properties may also be used. The upright members must be in one length. In case the length is beyond the capacity of any local sawmill to handle, then resort may be had to pit sawing or chain sawing, with due allowance for working back to true dimensions by adze or machine. These members may also be made of exterior quality gludlam.

The derrick is subject to major shock loads both in handling and during driving. For maximum strength all joints should be made with split ring connectors. Dimensions of commercial split rings are shown in Table 1.

Table 1

TYPICAL DIMENSIONS OF TIMBER CONNECTORS

SPLIT RINGS

Dimensions in Inches

	2½"	4"		2½"	4"
Split ring:			**Washers, standard:**		
Inside diameter at center when closed	2.500	4.000	Round, cast or malleable iron, diameter	2⅝	3
Thickness of metal at center	.163	.193	Round, wrought iron (minimum):		
Depth of metal (width of ring)	.750	1.000	Diameter	1¾	2
			Thickness	5/32	5/32
Groove:					
Inside diameter	2.56	4.08	Square plate:		
Width	.18	.21	Length of side	2	3
Depth	.375	.50	Thickness	⅛	3/16
Bolt hole:			**Projected area:**		
Diameter	9/16	13/16	Portion of one ring within member, sq. in.	1.10	2.25

If commercial split rings are not obtainable they can be cut from steel pipe. Suitable commercial pipes are 65 nominal bore API line pipe, schedule 40 (73.0 O.D. x 5.16 wall thickness) and 100 nominal bore heavy tube to B.S. 1387 or API line pipe, schedule 40. The pipe should be cut to the widths shown in Table 1 and the inside of each end tapered slightly in a lathe. The resulting ring is then cut through so that it will spring open to fit the groove.

Accurate construction is essential and it should be undertaken by skilled carpenters on an accurately level base. With all the angled making joints and allowing for disassembly after boring bolt holes to machine the split ring grooves, two or three weeks may be required for its construction.

The sheave details shown are typical only, but they do comply with B.S. requirements for lifting machines. However, if an excavator has been cannabalised for the winch the jib may well yield suitable sheaves. Whatever is used, care must be taken that the grooves are the correct size for the SWR used on the winch.

2.4. Tripper A tripper will almost certainly be required. This is essentially a release hook operated by handline from the ground. The details of a suitable tripper are shown in figures 8, 9 and 10.

Use of a tripper releases into the hammer all the energy which would otherwise be used in accelerating the winch drum when the hammer drops. It is essential if a geared or hydraulic winch is used, and is also essential if, in the absence of soils data, driving is done to a formula (See Section 3).

The inside and nose of the hook should be hard faced to minimize wear, then polished smooth to minimize friction and hence the effort which must be exerted to release the hammer

2.5. Auxillary Equipment Three 1500 kg capacity and one 3000 kg capacity "Tirfor" winches will be required. The smaller ones are for guying the derrick and the large one for raising the derrick and heavy handling work including skidding the derrick and the driving winch into position. The SWR normally supplied with Tirfor winches is normally too short for guying so additional SWR will be required. Also required will be slings, shackles, bulldog clips etc. Useful SWR attachments are often found on excavators and their buckets.

3. BEARING CAPACITY OF PILES

The ability of a pile to support a load may be limited by any of the following:

1. The capacity of the pile as a structural member

2. The transfer of load from the pile to the surrounding soil

3. The capacity of the soild to support the load transferred to it.

Condition 1 may be limited by one of four conditions:

(a) The capacity of the pile head to absorb the load applied to it. This is a problem of structural detailing. In all timber construction the limitations at the pile are likely to be not in the pile itself which has its grain parallel to the vertical force, but rather in the pile cap where side grain will be carrying the applied load. This will apply either in the case of a solid cap resting directly on top of the pile, or a double flitch cap fixed to the side of the pile as shown in Figure 12.

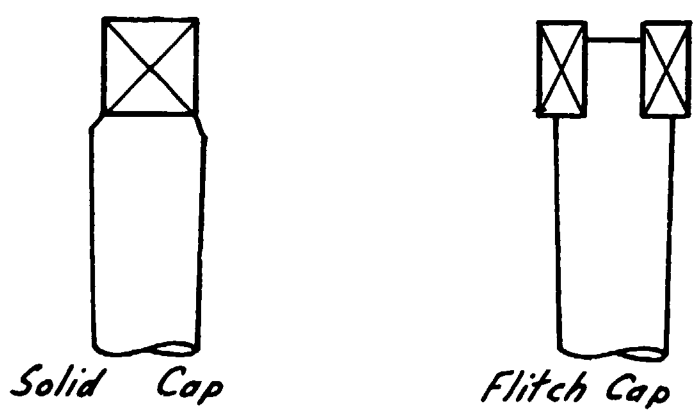


Figure 12 Pile Cap Types

It is possible for permissible stresses parallel to the grain to be exceededin the case of a concrete cap cast around the pile head.

(b) The capacity of the portion of the pile above ground as a column. This condition requires structural analysis . The effective diameter is generally taken as the diameter one third up the unsupported length. If the pile is one of several in a braced pier, the effective length may be taken from the cap down to firm ground or 1.2 m below ground level in soft ground. If the head of the pile is not adequately restrained then it should be treated as a cantilever column and the effective length will be twice the above.

(c) The capacity of the pile to resist lateral buckling in the ground. In the case of very soft ground it is conceivable that the active pressure of the surrounding soil is inadequate to restrain the pile from buckling. An analysis of this condition is given by Terzaghi (3) and he points out that the possibility is very remote. he further states that at the time that he wrote, there were no reported failures of piles through this mechanism.

(d) The capacity of the point of the pile to resist crushing. Piles are nearly always natural tree trunks and the point will be small in diameter than the butt. In the case of end bearing piles driven into rock or very dense soil the compression strength parallel to the grain at the pile tip should be checked

Condition 2. Transfer of load from the pile to the surrounding soil may occur either by friction of the soil against the vertical surface of the pile, or by direct pressure of a very hard stratum such as rock against the tip. In practice in the vast majority of cases both types of resistar are present although one or other will predominate. Depending on which is greater, a pile is referred to as a "friction" or an "end bearing" pile.

Terzaghi (3) has described the failure conditions of both friction and end bearing piles and this analysis forms the basis of the recommendations in Appendix K of Australian Standard 1720 (6 reproduced here as Appendix B. For the extreme case of end bearing piles founded on rock, Canadian Standard CAN3-56-M78(5) gives the following ultimate bearing capacities for various rocks.

Table 2

Maximum ultimate bearing capacity of Rock

Igneous and geneissic rocks in sound condition	29,000 kPa
Limestone bedrock - sound	19,000 kPa
Hard Shales,mudstones and soft sandstones	11,000 kPa

Condition 3. The vertical stress induced by a group of piles is greater than the stress imposed by a single pile because of the overlapping of the bulbs of pressure. Also, the settlement caused by a group of piles will begreater than that caused by one pile carrying the same load. In the types of piers constructed for short-span highway bridges, neither effect is likely to be important. Neverthe less, limitations are placed on minimum spacings. CSA CAN3-56-M78 requires a minimum spacing of 900 mm or 3 pile diameters center to centre, whichever is greater, for long friction piles in clay.

For cohesionless soils and end bearing piles the spacing is 750mm or 2½ piles diameters, whicnever is greater. The AASHO Standard Specifications for Highway Bridges (4) require a minimum spacing of 750 mm.

If the outer piles in a pier are raked to resist lateral loads then the tip spacing will greatly exceed these requirements.

Another situation where this condition could limit loads is where a thin hard stratum overlies deep soft soil The AASHTO specifications require borings to be taken below a doubtful hard stratum of a sufficient depth to determine the friction capacity of the underlying soil to support a friction pile.

In many cases a knowledge of the geology of the site and examination of banks upstream and down will aid judgement in such cases.

3.1 Determination of Allowable Bearing Capacity of Piles

The allowable bearing capacity of a pile is taken to be the ultimate bearing capacity divided by a factor of safety. The factor of safety to be used depends on the degree of certainty of determination of the ultimate bearing capacity.

The following table may be used as a guide:

Table 3

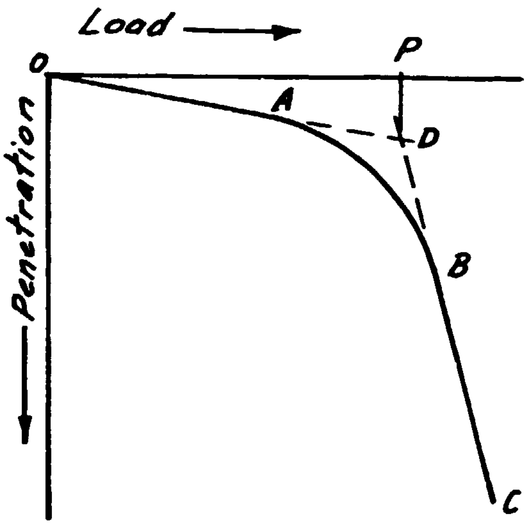
Pile Safety Factors

Method	Minimum Factor of Safety
Testpile loaded to ultimate, or twice design pile load	2
Soil mechanics investigation and calculation	2.5
Pile formula, end bearing pile	2.5
Pile formula, friction pile in cohesionless soil	4
Pile formula, friction pile in cohesive soil	6

Several methods of determining the ultimate bearing capacity of piles are in use.

Test Loading If an increasing series of loads are placed on a pile and its elevation observed, an elastic response will result for a while, then when soil failure occurs, a rapidly increasing penetration as shown by the line OAB in Fig 13. Part OA is the elstic response of both the pile and the surrounding soil. Soil failure starts at A and is complete at B. Beyond this, penetration increases rapidly with increasing load.

Figure 13



Penetration of Typical Test Pile

By convention, the ultimate load P is taken as that equivalent to point D, the intersection of the projections of the straight portions OA and CB. The permissible load is then P divided by 2 (or other factor of safety).

In practice it is a major operation to test load a pile. The ultimate load may be 50 to 100 tons. This requires a substantial platform, if steel billets are to be used as weights, or a huge box if gravel is to be used. Then all the loads must be weighed, and placed very carefully to maintain the whole weight system belanced on the small area of the pile head. further, it is really only possible if the pile is cut off at ground level. If piles are to project above ground to form a pier, then an additional pile must be driven for testing only, then abandoned.

An alternative approach is to use a pile on either side of the test pile to provide an upwards reaction. The reaction piles have a cap tied down to them and the test pile is jacked down against this. The load is observed from a load cell of some type, either mechanical or electronic. This method also has its drawbacks. The two reaction piles must be a metre or so higher than cut off level to contain the various bolt holes which should not be allowed in the finished pier. The jack must be manned continuously since any plastic deformation will result in loss of pressure. Also, it is not suitable for end bearing piles which penetrate soft overlying strata. In this situation the two reaction piles are likely to be lifted out of the ground by the test pile.

<u>Soil Investigations</u> The soil properties required for the formulae in Appendix B are the density, cohesion (for a cohesive soil) and angle of internal friction. In a cohesive soil free of large stones, holes can be drilled up to about 10 m deep using hand augers. As the hole is drilled, shear vane and penetrometer tests can be carried out in situ, and small undisturbed samples can be recovered with a thin walled sampling spoon. Drilling the hole is slow and arduous. Appropriate in situ testing equipment is necessary and preferably the services of a soil mechanics laboratory should be engaged.

Uncased holesin non cohesive soils (sands and gravels) will not stay open below the water table. In this situation a drilling rig and casings will be required to penetrate any significant depth. In these soils sufficient information can be gained from the "Standard Penetration Number' derived from driving a Raymond sampling spoon into the soil at the bottom of the casing. A graph relating standard penetration number and angle of internal friction is shown in Figure 14.

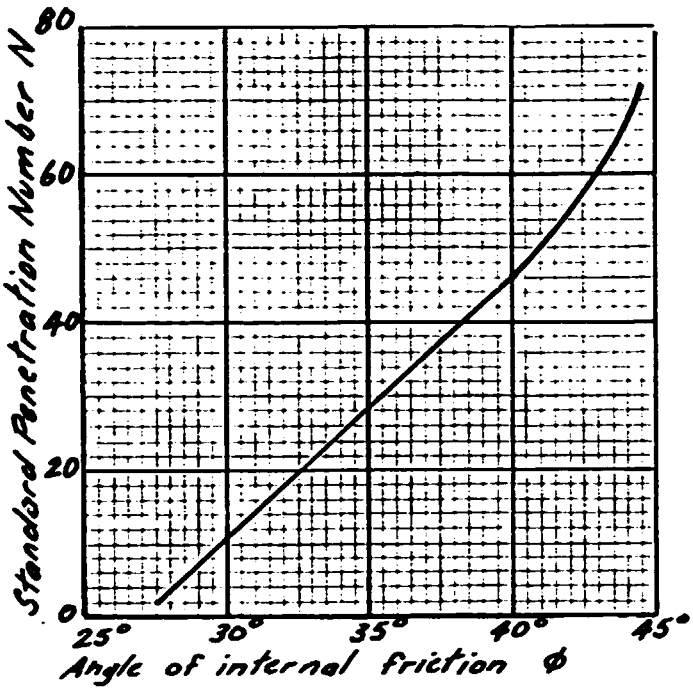


Figure 14. Standard Penetration Number corresponding to Angle of Internal Friction for Sands and Gravels

Pile Driving Formulae Many attempts have been made to relate the bearing capacity of piles to the energy imparted during driving and the resulting penetration. Two widely used formulae are the Engineering News Record formula and the Hiley formula. The Engineering News Record formula is preferred in the USA whil British Engineers prefer the Hiley formula. The Engineering New Record formula is commonly expressed as:

$$P = \frac{2WH}{S+1}$$ for a free falling drop hammer

where

P = Allowable bearing capacity

W = Weight of hammer (same umits as P)

H = Height of hammer fall (feet)

S = Final set (inches)

Built into this is a factor of safety of 6. If H and S are both expressed in inches, then the ultimate bearing capacity is Pult $= \frac{WH}{S+1}$

Or in millimetres

$$\text{Pult} = \frac{WH}{S+25.4}$$

The Hiley formula takes several energy losses into account, and is

$$\text{Pult} = \frac{WHe}{S+0.5c} + W$$

where Pult and W are as above

H and S are as above in millimetres

C = Sum of elastic rebounds, mm

e = efficiency of blow

$C = C_1 + C_2 + C_3$

C_1= elastic compression of packing (if any) and hammer

C_2= ground quake

C_3= elastic compression of pile

Tables are published for e and c for concrete piles e.g. by Reynolds (2) but little seems available for timber. e ranges from 0.69 down to 0.17 for the ratio weight of hammer to weight of pile ranging from 0.5 to 7, shown graphically in Fig 15.

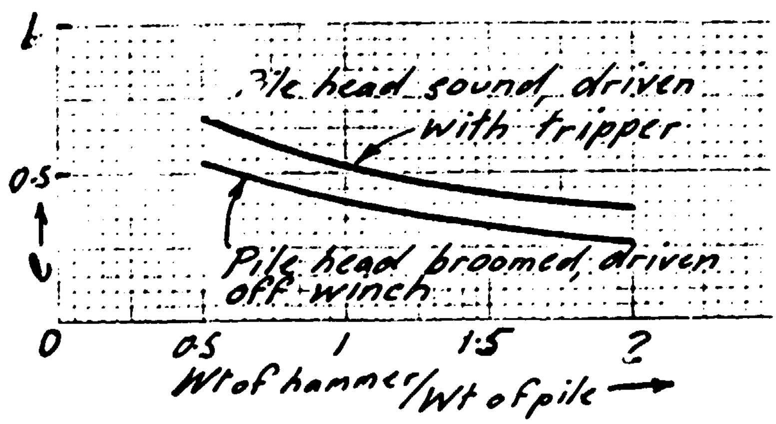


Figure 15. Efficiency factor, Hiley Formula

The major component of C is C_2 and this may be measured by drawing a pencil across a piece of paper fixed to the pile as the hammer hits. The pencil is guided by a long piece of board supported as far away from the pile as possible.

For ground quakes of about 10 mm to 20 mm, and efficiences of around 0.5, both formulae give rather similar results. The major differences are British recommendations of a factor of safety of 1.5 to 3, compared with teh American implicit factor of safety of 6. (Terzaghi discusses the shortcomings of pile formulae in detail). There is a major difference in the dynamic resistance developed by a pile driven into a cohesive and non cohesive soils.

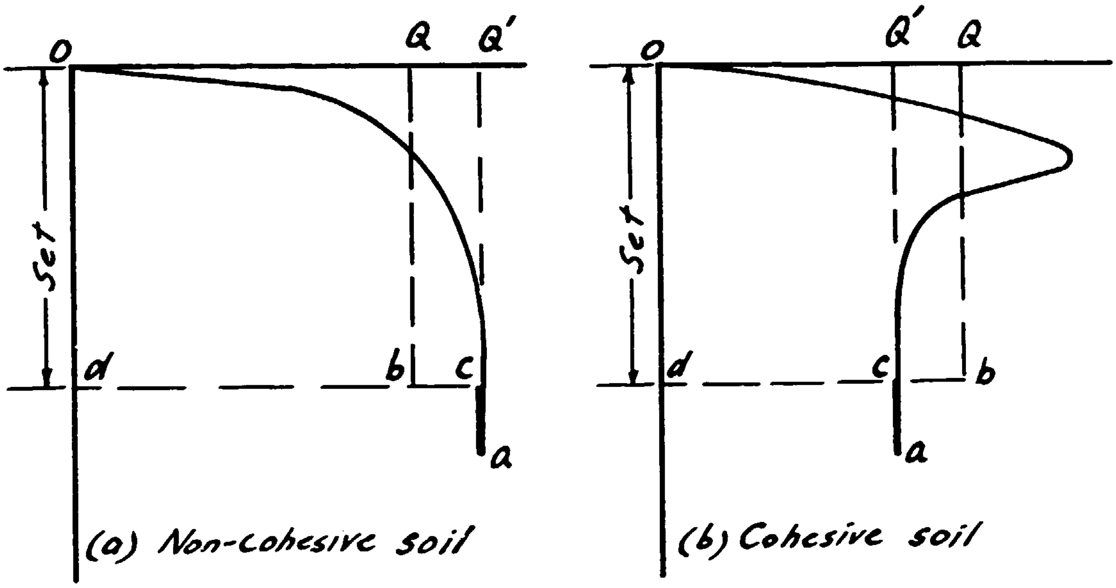


Figure 16

Dynamic Penetration of Piles in (a) Non-cohesive and (b) Cohesive Soils

Figure 16 (a) shows the increase of resistance as the point of the pile penetrates a non cohesive soil under a blow, line oa, which rebounds to 6. Energy driving formulae relate the area to OCd to the equivalen rectangular area, representing the energy OQbc. The pattern of dynamic resistance in a cohesive soil is very different quickly reaching a peak, then falling back as penetration proceeds as shown in Figure 16 (b). The equivalent area is OQbd, considerably greater than the actual final resistance. Both the Engineering News Record and the Hiley formulae attempt to rationalise these phenomend, but without taking these two differences in manner of penetration into account. Terzaghi completes his analysis of the short comings of pile formulae with the comment:

> "In spite of their obvious deficiencies and their unreliability, the pile formulas still enjoy a great popularity among practising engineers, because the use of these formulas reduces the design of pile foundations to a very simple procedure. The price one pays for this simplification is very high. in some cases the factor of safety of foundations designed on the basis of pile formulas is excessive and in other cases significant settlements have been experienced. The opinions regarding the conditions for the legitimate use of the formulas are still divided".

In the majority of practical cases, factors of safety are very much larger than those listed in Table 3. Piers for small bridges are usually to standard designs. Pile lengths and diameters are determined by what is in stock, from available trees. Pile depths are determined by minimum length specifications, scour depths or the desire to found on rock or a known hard stratum. Then the use of a pile formula becomes a monitoring exercise to ensure that other pre-conditions have been met, rather than a primary design calculation. In many of these cases the formula bearing capacities are not calculated at all, and only the final sets are compared to ensure that otherwise determined conditions are being met.

In practice, despite the prohibition given in clause K 3.3 of appendix B, the use of a pile driving formula is very frequently the only practical option open to the engineer. Even the use of a full scale loading test is frequently impractical although a jacking test may be possible without too much difficulty, and should certainly be resorted to in the case of friction piles in silt or soft clays.

4. PILE DRIVING

The positions of all piles to be driven should be carefully set out and offset, since the pegs will be destroyed by the piles. If raking piles are to be driven then due allowance must be made for variations in ground level. The area where the derrick and winch will be seated should be trimmed flat. Then small holes are dug to locate the tip of each pile. The derrick is moved in to position to locate the pile in the hole and plumbed using the Tirfor winches and guy wires fixed to suitable anchorages. With tapered piles, the face of the derrick will actually be leaning slightly backward, so that the axis of the pile lies plumb. The base of the derrick is packed solid and the winch attached at its rear. This is all heavy work, and the availability of a bulldozer or loader will ease and speed up operations.

The hammer is now shackled to the driving rope, lifted and fixed to the leaders, then hoisted to the top of the derrick. The tripper should NOT be used at this stage. It is not a particularly secure device - its very purpose is to allow the hammer to fall freely - and its use at this stage could easily result in a fatal accident.

The pile is now hoisted up against the leaders by the luffing drum using a choker sling near its butt end. When it is in position it is lashed to the headers at mid height of each platform with a single turn of 30mm fibre rope. A man is stationed on each platform. His job is to hit the rope up on the pile with a mallett. Unless this is done the lashing will bind and very soon break. This is very hard on the lashings. They must be regarded as expendable, and the cheapest availble rope should be used.

Driving may now commence, using light blows and driving from the winch. Verticality of the pile should be carefully checked at this stage, when it can be adjusted if necessary. This may be done with two theodolites, or with plumb bobs hanging from temporary frames created about 30 m from the pile, one square off to one side, the other on the line of the bridge.

Driving should start at the abutment piers. These carry the smallest live and dead loads, and will give, at least, size predictions of bearing capacity. The engineer can then decide whether he has adequate bearing or whether it may be wise to increase the number of piles in a pier. The marginal cost of a few more piles is still likely to be less than the cost and delay involved in conducting a soil investigation.

Once the pile has penetrated two or three metres and driving is becoming harder, the tripper may be fitted. This will increase the set per blow, and in any case it is essential if bearing measurements are to be taken. The procdure for taking these measurements has already been described.

It is important that too great a hammer drop is not used. Excessive hammer drop will result in a brooming of the pile head, reducing the set per blow and can also easily result in fracture of the pile itself, particularly in timbers with a tendency to brittleness. A heavy hammer and small drops will minimize brooming and the danger of fracturing the pile.

Driving should cease when the pile head is slightly above cut-off level, depending on the condition of the head and the amount of brooming. And a final caution - do not let an over enthusiastic gang drive a pile below cut-off level! This leads to all sorts of expensive remedial action having to be taken.

Observation of ground conditions and the use of pile formulae require the "set" or penetration per blow to be measured. This is most conveniently done with a board about 1.5m long with its lower end resting on a peg close to the pile. After each blow of the hammer, a pencil or scriber is drawn across the top end of the board against the pile. This is done for ten blows. The set is then one-tenth of the total distance marked.

The "Hiley" formula also requires the hammer rebound or "bounce" to be measured. Measuring directly is difficult and dangerous, although it can be done with one man holding a rule at the level of the pile head. As the hammer strikes, he applies upward pressure so that the rule follows the hammer up as it rebounds. An observer reads the height of the rebound as indicated by the rule at its maximum height. BE CAREFUL! Surer, though less accurate is to estimate the rebound from a safe position. In fact, because the rebouond has a secondary effect on the bearing capacity calculation, a small error in estimation does not have much effect on the result.

5. MATERIAL AND CONSTRUCTION SPECIFICATIONS

Piles must be manufactured from the trees available locally and specifications should be drawn up accordingly. Strength need not be a major consideration. Form, durability or treatability are as important. In the USA, cedar, a comparatively weak species is widely used for piling because of its durability. Any species which produces good quality utility poles may also be used for piling.

Table 4 may be used for minimum pile dimensions.

Table 4

Dimensions of Hardwood Timber Piles
(Adapted from Class B, ASTM D25-58)
dimensions converted to metric

Length m	1m from Butt				At Tip	
	Minimum		Maximum		Minimum	
	Circumference mm	Diameter mm (Approx)	Circumference mm	Diameter mm (Approx)	Circumference mm	Diameter mm (Approx)
Under 7.5	860	270	1450	460	640	200
7.5 - 9	970	310	1450	460	640	200
9 -12	1040	330	1600	510	560	180
Over 12	1040	330	1600	510	480	150

Unless species of proven durability are available piles should be treated. Pressure impregnation with creosote is preferred. These should be treated to refusal, and in any case to not less than 190 kg per m3. Cut off ends should also be liberally swabbed with two coats of hot creosote. Even better is to fit a temporary sheet metal collar round the pile head, sealing it to the pile with clay. This is kept filled with creosote for as long as possible. Bolt holes should also be treated by plugging one end and filling with creosote.

Penetration of piles should not be less than 3m in hard material nor less than one-third the length of the pile nor less than 6 m in soft material. Piles driven through soft material to a hard stratum should be driven into this a sufficient depth to rigidly fix the tips. A penetration of $2\frac{1}{2}$ tip diameters may be considered adequate for this. If the Standard Penetration Number of this stratum is greater than 40, pile tips will require reinforcing. This may take the form of a piece of steel tube, its length equal to the tip diameter. This is tightly fitted to the pile tip. Alternatively a cast and fabricated shoe may be fitted. See figure 16.

If a pile should shatter below ground or be more than 150mm out of position it should be withdrawn by jacking out of the ground or a fresh pile should be driven alongside. Piles generally will only shatter when a pocket of internal decay is present, and they are being driven to refusal in hard ground under long drops of a light hammer. Use of a heavy hammer and short drops will minimise the chances of shattering as well as causing a minimum of brooming of the pile head.

If a shattered pile is completely withdrawn then a similar but fatter pile may be re-driven in the same hole. If part of the shattered pile remains in the ground, then the new pile should be shod with a pointed shoe.

5.1 Preparation of Piles

Piles to be driven into rock or dense ground require a shoe to be fitted to the tip. A typical shoe is shown in Figure 17.. No shoe is required for driving into silt or clay but the tip should be sawn accurately square to the axis of the pile.

The head of the pile must be prevented from splitting by being enclosed in a steel ring of cross section 60 mm x 16 mm. The head is carefully shaped with an adze to receive the ring which is started with a heavy hammer. Final seating is done by the pile hammer itself. A dozen or more rings or varying sizes to suit the diameters of the piles supplied are required. As the edges of the rings become upset by blows from the hammer they must be returned to a blacksmith for re-shaping of the edges and removal of the upset.

Piles should be marked at 1 metre intervals from the tip with a shallow saw cut and each mark numbered so that the depth in the ground can be monitored as driving proceeds. When driving is complete one of these marks which will not be covered by fastenings or bracing should be enlarged to a groove about 15 mm deep and the distance to the point chiseled deeply into the wood. This provides a record for future maintenance inspections.

The Standard Penetration Number is an empirical test widely used in the USA and Canada because of its simplicity.

The test consists of observing the number of blows of a 140lb (63.6kg) weight falling 30 inches (760mm) to drive a standard spoon 12 inches (305mm) into the ground at the bottom of the bore hole. The spoon may be approximated by a 600 mm length of 50.8 mm diameter shaft, with its point chamfered down to 35 mm diameter over a length of 19 mm. The weight is usually cylindrical and slips over the drill rods to drive against a flange on the rods.

6. RECORDS

Records whould be kept on a standard form. A suitable form is shown in Appendix C.

Piles are numbered in accordance with a key plan which should include details of highway mileage directions of major towns, river flow direction, North point and details of the local elevation bench mark. Ideally, observations of set should be taken with all pile points at the same elevations or reduced levels (R.L.).

Appendix D shows the same form as it might be completed on a typical job. The sets have generally been observed at quite close to the same one-metre intervals. The near refusal at R.L. 355.0 and the comments made in the "Remarks" column are typical.
Even in the cases where a detailed soils investigation has been done, records of driving at 1 m intervals should be kept for all piles. This will indicate, at least on a relative basis, changes in soil properties over the bridge site, and also whether a pile has shattered under driving, which is indicated by a major increase in set. These records should be filed. They can be valuable future record for bridge up-grading or renewal in future years.

REFERENCES

1. Farraday, R.V. and F.G. Charlton. Hydraulic factors in Bridge Design. Hydraulics Research Station Ltd, Wallingford, Oxfordshire, England. 1983

2. Reynolds, C.E. Reinforced Concrete Designer's Handbook. Concrete Publications Ltd. 14 Dartmouth St. London SW1 4 Ed 1954.

3. Terzaghi, K. Theoretical Soil Mechanics. John Wiley and Sons, New York, 1943.

4. - Specifications for Highway Bridges, American Association of State Highway Officials, 341 National Press Building, Washington DD.C. 20004 U.S.A. 1973

5. - Design of High Bridges, CAN3-S6-M78 Canadian Standards Association, 178 Rexdale Boulevard, Rexdale, Ontario Canada M9W 1R3 1978

6. - Timber Engineering Code, AS 1720. Standards Association of Australia, 80 Arthur St. North Sydney, N.S.W.

Blackheart Malleable Iron Castings

1. Scope

This Japanese Industrial Standard specifies the blackheart malleable iron castings, hereinafter referred to as the "malleable castings".

2. Classification and Symbols

The classification and symbols of the malleable castings shall be in accordance with Table 1.

Table 1. Classification and Symbols

Classification	Symbol
Class 1	FCMB 28
Class 2	FCMB 32
Class 3	FCMB 35
Class 4	FCMB 37

3. Manufacturing Method

The malleable castings shall be processed with the heat treatment mainly intended for graphitization in order to give toughness to those after having been cast as the white pig iron castings.

4. Mechanical Properties

The tensile strength, proof stress and elongation shall conform to Table 2.

Table 2. Mechanical Properties

Classification	Symbol	Tension test		
		Tensile strength kgf/mm^2 (N/mm^2)	Proof stress kgf/mm^2 (N/mm^2)	Elongation %
Class 1	FCMB 28	28 min. (275) min.	17 min. (167) min.	5 min.
Class 2	FCMB 32	32 min. (314) min.	19 min. (186) min.	8 min.
Class 3	FCMB 35	35 min. (343) min.	21 min. (206) min.	10 min.
Class 4	FCMB 37	37 min. (363) min.	22 min. (216) min.	14 min.

Remarks 1. In determining the proof stress, the value of permanent elongation shall be taken as 0.2 %, however, the total elongation of 0.5 % under the load may also be used.

2. Units and numerical values given in parentheses are in accordance with the International System of Units (SI), and are given for reference. 1 N/mm^2 = 1 MPa

Reference: The hardness of the malleable casting is to be of 163 or less in Brinell hardness.

5. Appearance

The malleable castings shall be free from harmful flaws, blowholes, etc.

6. Shape and Dimensions

6.1 The shape and dimensions of the malleable castings shall conform to the drawings, and the dimensional tolerances of the mechanically unfinished parts shall conform to Table 3.

Furthermore, when the drawings or Table 3 is not applicable, it shall be as agreed upon between the purchaser and the manufacturer.

Table 3. Dimensional Tolerances

Unit: mm

Division of nominal size			10 and under	Over 10 to 18 incl.	Over 18 to 30 incl.	Over 30 to 50 incl.	Over 50 to 80 incl.	Over 80 to 180 incl.	Over 180 to 400 incl.	Over 400 to 800 incl.
Dimensional tolerances (±)	Wall thickness	Precision Grade	1.0	1.5	2.0	3.0	-	-	-	-
		Ordinary Grade	1.5	2.0	3.0	4.0	-	-	-	-
	Length	Precision Grade	0.8		1.0			2.0	3.0	4.0
		Ordinary Grade	1.0		1.5			2.5	3.5	5.0

6.2 The permissible value of draft angle shall conform to Table 4.

Table 4. Permissible Value of Draft Angle

Division of draft angle	Outside		Inside	
Grade	Precision Grade	Ordinary Grade	Precision Grade	Ordinary Grade
Permissible value	2/100	3/100	3/100	5/100

Remark: To the dimensional tolerances on length and wall thickness, the permissible value of draft angle may be added.

7. Tests

7.1 Tension Test

7.1.1 The tension test shall generally be carried out at the place of manufacture. In this case, upon the purchaser's request, the manufacturer shall admit the purchaser to be present at the test.

7.1.2 The test piece shall be cast in a sand mould on every one melt, and be heat-treated in the same furnace together with its representative malleable castings. The term "one melt" in a continuous melting, in the case of the same intended composition, shall be the tapping at every two hours.

7.1.3 The test piece shall be as cast, and the dimensions shall conform to Type A of Table 5. For Class 1, however, when the maximum thickness of wall does not exceed 8 mm, the dimension thereof may conform to Type B of Table 5.

Table 5. Dimensions for Test Piece

Unit: mm

Classification	Diameter	Gauge distance	Length of parallel portion	Radius of shoulder	Diameter of grip
Type A	14	50	60	15 min.	Approx. 20
Type B	12	42	50	15 min.	Approx. 18

7.1.4 The testing method shall be in accordance with JIS Z 2241. In measuring the strains, in order to obtain the total elongation under load, however, the divider may be used.

7.2 Hammering Test The malleable castings shall be investigated on its quality of heat treatment and existence of defects such as blowholes and fractures by hammering.

7.3 Destructive Test The malleable castings, other than those specified in respective standards, shall be investigated on their breaking conditions and fractures by breaking the test bodies which have been extracted from the groups of the same class and the same type. Above testing may, however, be carried out by the use of casting lugs on the main body.

8. Inspection

8.1 The inspection shall be carried out on the mechanical properties, appearance, shape and dimensions, and these shall conform to the requirements of 4., 5. and 6. The tension test piece shall be extracted one piece per one melt.

8.2 The hammering test and destructive test other than the inspection of 8.1 shall be as agreed upon between the purchaser and the manufacturer.

8.3 The malleable casting, which is regarded as not conforming to the requirements of 4. as it stands, due to unsuitable heat treatment, may be heat-treated again upon the purchaser's approval. The inspection after reheat treatment shall be carried out in accordance with 8.1 and 8.2.

8.4 Flaws and blowholes having slight effects in use may be repaired, upon the purchaser's approval, by welding and other suitable methods.

9. Retests

9.1 When any flaw or blowhole is found or the test piece which is judged as have influenced on the test results, these results shall be regarded as invalid, and, further, a spare test piece may be substituted for it.

9.2 When the test piece is broken, in a tension test, at a point beyond 1/4 of the gauge length from the centre between the gauge marks and the results have failed to conform to the requirements, this test shall be regarded as invalid, and, further, the spare test piece may be substituted for it.

9.3 When a part of the results of the tension test failed to conform to the requirements, retest may, generally, be carried out extracting further two pieces of test piece from the set from which the initial test pieces have been extracted. In this case, all the test results shall conform to the requirements of 4.

9.4 When the spare test pieces are not provided, the testing may be carried out on the actual product test pieces being cut out from the casting body itself. The mechanical properties, dimensions, extracting positions and testing methods of the actual product test pieces, however, shall be as agreed upon between the purchaser and the manufacturer.

DESIGN OF TIMBER PILES

INTRODUCTION. The purpose of this appendix is to provide guidance on the design of wooden piles pending the preparation of an Australian standard piling code to cover the design and use of piles generally. Until the latter code becomes available, the provisions of this appendix should not be interpreted as precluding the use of other well-tried and proven methods for designing timber piles.

K1 GENERAL. This appendix specifies the design procedures for single piles and pile groups subject to vertical and lateral loads. The procedures involve the use of soil parameters which must be determined by properly executed field and laboratory geotechnical investigations.

K2 DEFINITIONS. For the purpose of this appendix, a 'pile' can be considered to be a trunk of a tree which has been driven, without damage, into the soil to a depth where the combined resistance along the periphery and base of the pile is sufficient to satisfactorily resist the loads applied by the superimposed structure.

The term 'friction pile' implies that the dominant resistance is that due to side friction and adhesion.

The term 'end-bearing pile' implies that the dominant resistance is that due to the development of base resistance.

K3 LOAD CAPACITY OF PILES—VERTICAL LOADS.

K3.1 Load Limit. Although many codes and specifications limit the load applied to an individual pile to 250 kN, such a limitation is not necessary. Provided the geotechnical investigations establish that the load can be supported without unacceptable settlement, and that the pile can be satisfactorily driven without unacceptable damage, loads are limited by the allowable stresses in the pile.

K3.2 Allowable Stresses. The allowable stresses in the pile are specified in Section 6 of this Code.

K3.3 Bearing Capacity. The total load which can be transmitted to the soil, herein termed the allowable bearing capacity, shall be determined by formulas K1 and K2, or where relevant, by a full scale loading test.

The allowable bearing capacity shall not be determined by a pile-driving formula alone. Use of such formula shall be restricted to a situation where the soil conditions are uniform and driving procedures are not varied and a correlation can be developed between blow count, depth and settlement.

K3.4 Bearing Capacity in Clay. The allowable bearing capacity of a pile Q_A, driven into a saturated intact clay can be calculated from the formula—

$$Q_A = Q_s + Q_p = A\left[\frac{\left(9c_u + 4c_a\frac{D}{D_p}\right)}{F} + P_o\right] \quad .. \quad .. \quad \text{(K1)}$$

where

Q_s = load applied to the top of the pile by the structure

Q_p = total mass of the pile

A = area of the pile at the base or tip

c_u = immediate undrained shear strength obtained by laboratory or field measurements

c_a = adhesion between the soil and the pile obtained by suitable laboratory or field measurements, with a minimum value equal to the remoulded undrained shear strength

D = depth to base of pile

D_p = average diameter of pile

F = factor of safety, with a minimum value of 2 but the actual value depending on the variability of the soil and the confidence with which the soil properties can be specified

P_o = total overburden pressure defined by the expression —

$P_o = \gamma D$ where γ is the saturated density of the clay.

K3.5 Bearing Capacity in Sand. The allowable bearing capacity of a pile driven into sand can be calculated from the formula —

$$Q_A = A\left[\frac{\left(P'_o(N_q - 1) + \frac{\gamma_b D_p}{2}N_\gamma\right)}{F} + P_o + \frac{\gamma_b K_A D^2}{2F}\tan\phi_w\right] \quad .. \text{(K2)}$$

where

N_q and N_γ = factors which can be considered to be a function of the angle of friction ϕ_d as tabulated in Table K1.

TABLE K1

FACTORS

ϕ_d	25°	30°	35°	40°
N_γ	10	20	42	125
N_q	12	21	40	80

ϕ_d = angle of friction associated with a drained state and determined by a suitable laboratory technique

P'_o = effective overburden pressure at depth D and equal to $(P_o - \Upsilon_w d)$ where Υ_w is density of water and d is the depth of water above the base of the pile

Υ_b = buoyant density of soil and equal to $(\Upsilon - \Upsilon_w)$

K_A = coefficient of lateral earth pressure with a minimum value of $^1/_3$

ϕ_w = angle of friction between soil and pile and may be taken as $^2/_3\,\phi_d$.

K3.6 Bearing Capacity in Silt. The allowable bearing capacity of piles in silts, fissured clays and unsaturated soils require further special consideration.

K3.7 Buckling. Buckling of a pile in a soft soil need only be considered if the undrained shear strength of the soil is less than 24 kPa, and to establish whether buckling could occur when the piles are embedded in lower strength soils a specialist analysis will be necessary.

K3.8 Total Load Transmitted. Where end-bearing piles are embedded in a consolidated soil, the total load transmitted to the base is the sum of the load applied by the structure plus the down-drag load due to negative skin friction.

K3.9 Settlement. The immediate and total final settlement of a pile shall be calculated by established methods or measured by a full-scale loading test. For the purpose of calculation the soil may be regarded as a linear elastic medium provided the soil parameters for use in this theory are measured under test conditions which simulate the field stress state, or are determined from the results of a loading test on a typical pile.

K4 LOAD CAPACITY OF PILE GROUPS—VERTICAL LOADS.

K.4.1 Load. The total load, Q_T, which can be applied to a group of piles shall be taken as the smaller of Q_{T1} and Q_{T2}, where these values are expressed as—

$$Q_{T1} = mn\,Q_A \qquad \text{(K3)}$$

and for a saturated intact clay—

$$Q_{T2} = \frac{c_u}{F}\Big(B_1 L N_c + 2(B_1 + L_1)D\Big) \qquad \text{(K4)}$$

or for a cohesionless soil—

$$Q_{T2} = \frac{B_1 L_1}{F}\left(P'_o(N_q - 1) + \frac{\Upsilon_b B_1 N_\Upsilon}{2} + \Upsilon_b K_A D^2 \tan\phi_d\right) \qquad \text{(K5)}$$

where

$B_1 = mS_1D_p$

$L_1 = nS_2D_p$ (with $mS_1 < nS_2$)

m = total number of piles in the longitudinal direction

n = total number of piles in the lateral direction

S_1 = pile spacing in longitudinal direction

S_2 = pile spacing in lateral direction

N_c = bearing capacity factor from Table K2

TABLE K2

BEARING CAPACITY FACTOR

D/D_p	0	1	2	>3
N_c	6	7	8	9

K4.2 Settlement. The total final settlement of a pile group can be estimated from the settlement of a single pile by the use of suitable interaction factors. Reference can be made to the factors published by Morgan and Poulos (Ref. (1)).

K5 LATERAL LOAD CAPACITY OF SINGLE PILES AND GROUPS.

K5.1 Single Pile in Clay. As a guide to the ultimate lateral load P_u which could be applied to a pile at a height of e above the ground surface, Table K3 gives values of the term $\frac{P_u}{c_u D_p^2}$ for a range of ratios of e/D_p and D/D_p which apply to a single pile embedded in clay and unrestrained at the point of load application.

TABLE K3

VALUES FOR $P_u/c_uD_p^2$

D/D_p	e/D_p					
	0	1	2	4	8	16
4	4	3	2	1	1	1
8	16	14	12	10	8	4
13	30	28	25	21	16	10
16	47	42	40	32	26	15
20	60	56	51	45	36	26

To determine the allowable lateral load P_A, P_u is divided by a safety factor of not less than 3.

The design bending moment M in the pile corresponding to an axial load P can be calculated from the formula:

$$M = P\left(e + 1{\cdot}5\, D_p + \frac{P}{18 c_u D_p}\right) \quad \ldots \quad (K6)$$

K5.2 Single Pile in Sand. As a guide to the ultimate lateral load which could be applied to a pile embedded in sand the following formula may be used—

$$P_u = \frac{D_p D^3 \tan^2\left(45 + \frac{\phi_d}{2}\right)}{2(e + D)} \quad \ldots \quad (K7)$$

The allowable lateral load is obtained by dividing P_u by a factor of safety of not less than 3.

The design bending moment M in the pile corresponding to an axial load P can be calculated from the formula:

$$M = P\left[e + 0{\cdot}54\sqrt{\left(\frac{P}{D_p \gamma \tan^2\left(45 + \frac{\phi_d}{2}\right)}\right)}\right] \quad \ldots \quad (K8)$$

K5.3 Piles in Group. The lateral load capacity of a pile group requires special consideration.

K5.4 Deflection. The lateral deflection of a pile or pile group can be estimated by the use of the Winkler soil model or the linear elastic soil model.

Reference 1.

MORGAN, J.R., and POULOS, H.G. (1968) — 'Stability and Settlement of Deep Foundations in Soil Mechanics — Selected Topics' (Ed. I.K. Lee) Butterworth, London.

APPENDIX C

PILE DRIVING RECORD

Bridge No .... DATE: ....

Highway No .... Mileage (P K) ....River ....

Between .... and ....

Hammer weight ...... kg Winch machine No ....

Pile species .... Treatment ....

Pile No	Tip Dia.	Butt Dia.	Length	Hammer drop	Set	Rebound	Tip R.L.	Tripper used	Remarks

APPENDIX D

PILE DRIVING RECORD

Bridge No 69 DATE: 31-6-99

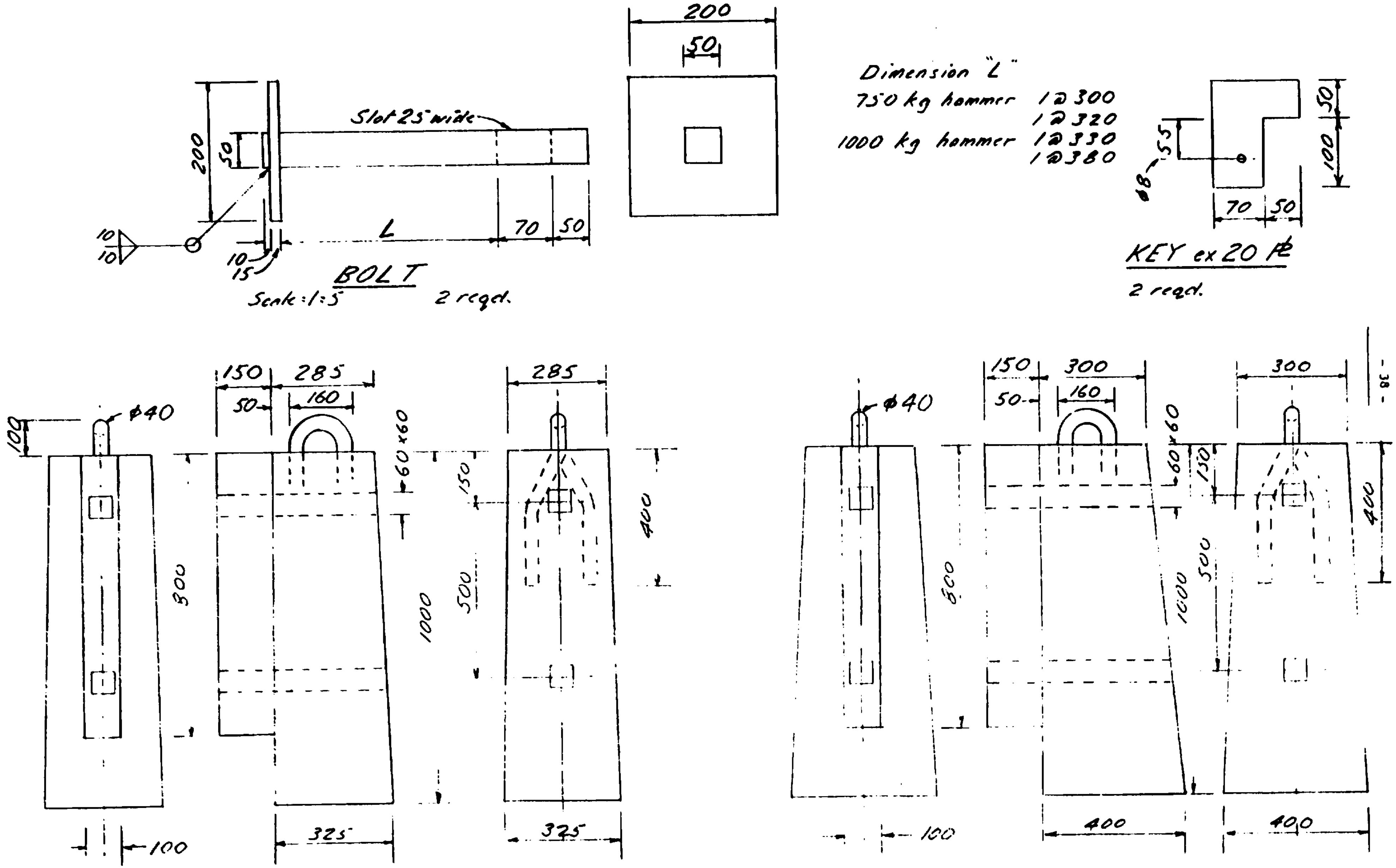
Highway No 101 Mileage (P.K.) 101·75 River Waipiro

Between Kaupati and Waikikamukau

Hammer weight 1500 kg Winch machine No 13

Pile species E. grandis Treatment Creosote (Pickle Ltd)

Pile No	Tip Dia	Butt Dia	Length	Hammer drop	Set	Rebound	Tip R.L.	Tripper used	Remarks
1	250	340	7-8	900	12·6	6	360·3	No	Soft
				900	14·0	4	359·3	No	Soft
				900	8·2	5	358·4	Yes	
				900	4·3	4	357·3	Yes	
				950	3·7	4	356·2	Yes	
				900	3·1	6	355·5	Yes	Hard driving
				900	1·7	10	355·1	Yes	Almost refusal Pile head brooming
2	240	320	7-6	900	18·0	5	360·3	No	Soft
				900	16·0	5	359·4	No	Soft
				900	8·9	5	358·3	No	Soft
				900	4·0	6	357·2	No	
				900	4·0	5	356·3	Yes	
				900	3·3	7	355·3	Yes	Pile running to East
				900	1·2	11	355·0	Yes	Near refusal. Pile head O.K.



750 kg PILE HAMMER FIG 2:

Scale: 1:10 Material: Cast Iron

1000 kg PILE HAMMER

C. R. FRANCIS,
Registered Civil Engineer
25-5-89

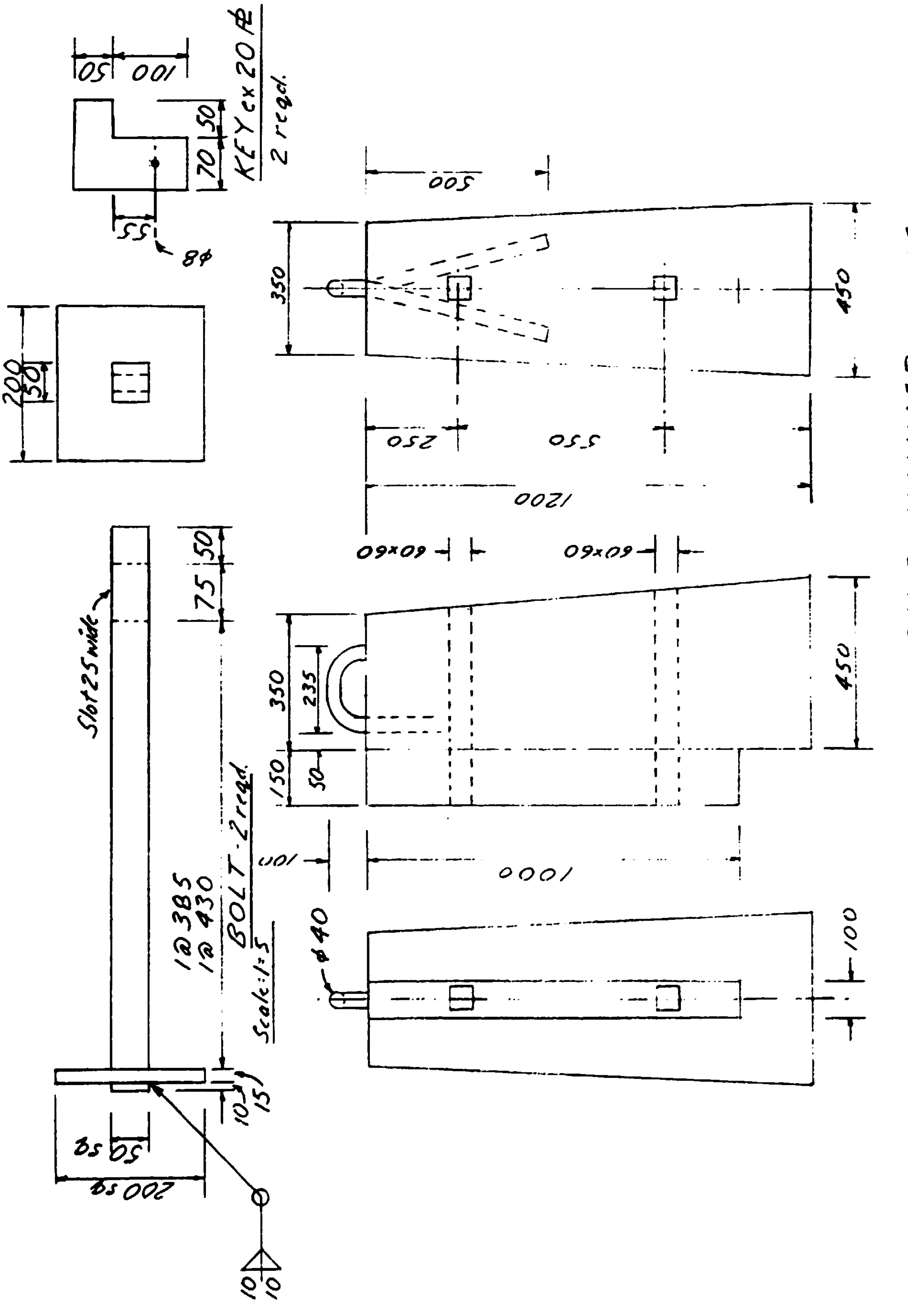


FIG 3: 1500 kg PILE HAMMER

Scale: 1:10 Material: Cast iron

C. R. FRANCIS,
Registered Civil Engineer
23-5-89

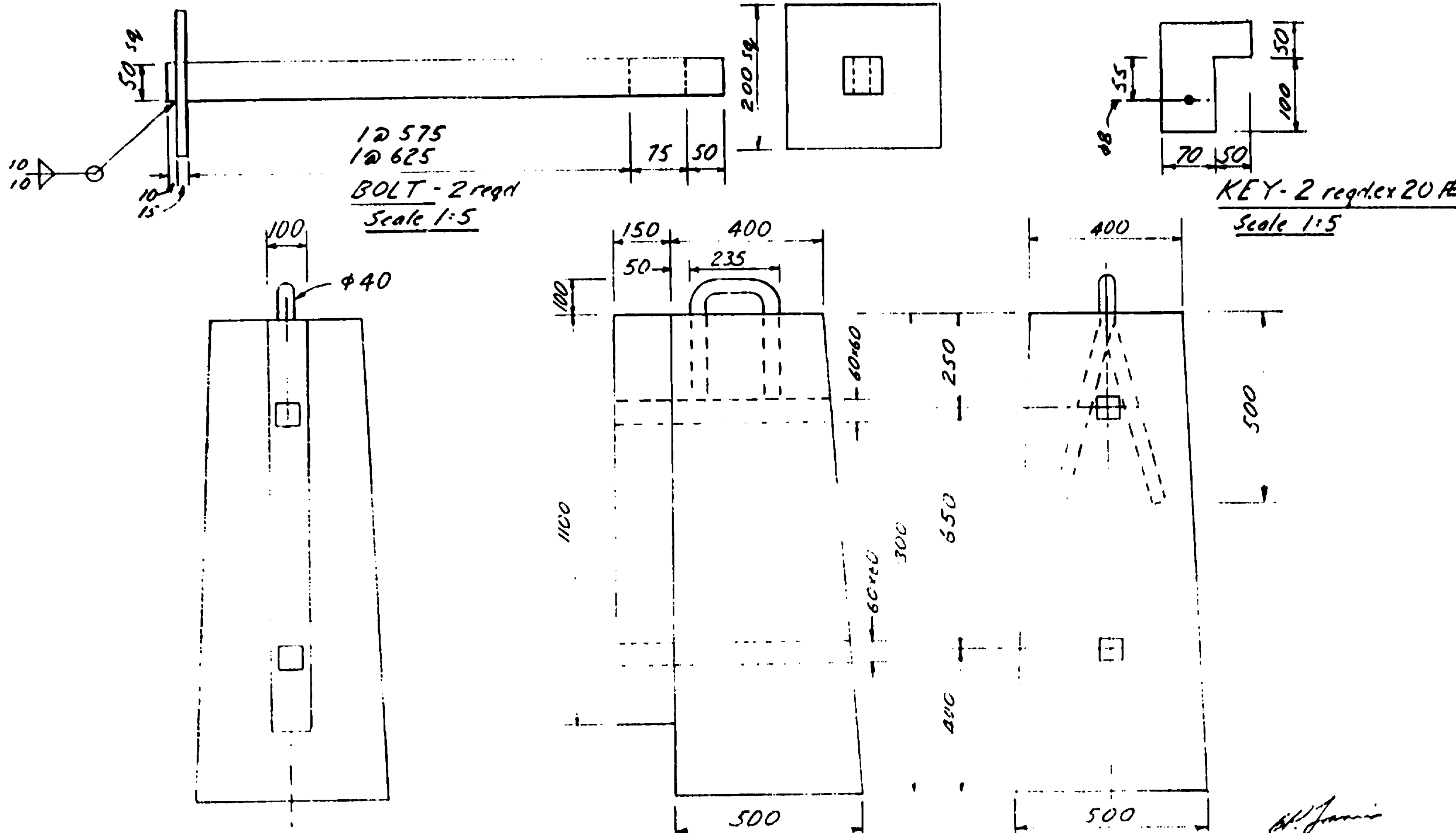


FIG 4: 2000 kg. PILE HAMMER

Scale 1:10 Material: Cast Iron

C. R. FRANCIS,
Registered Civil Engineer
7.9.7.89

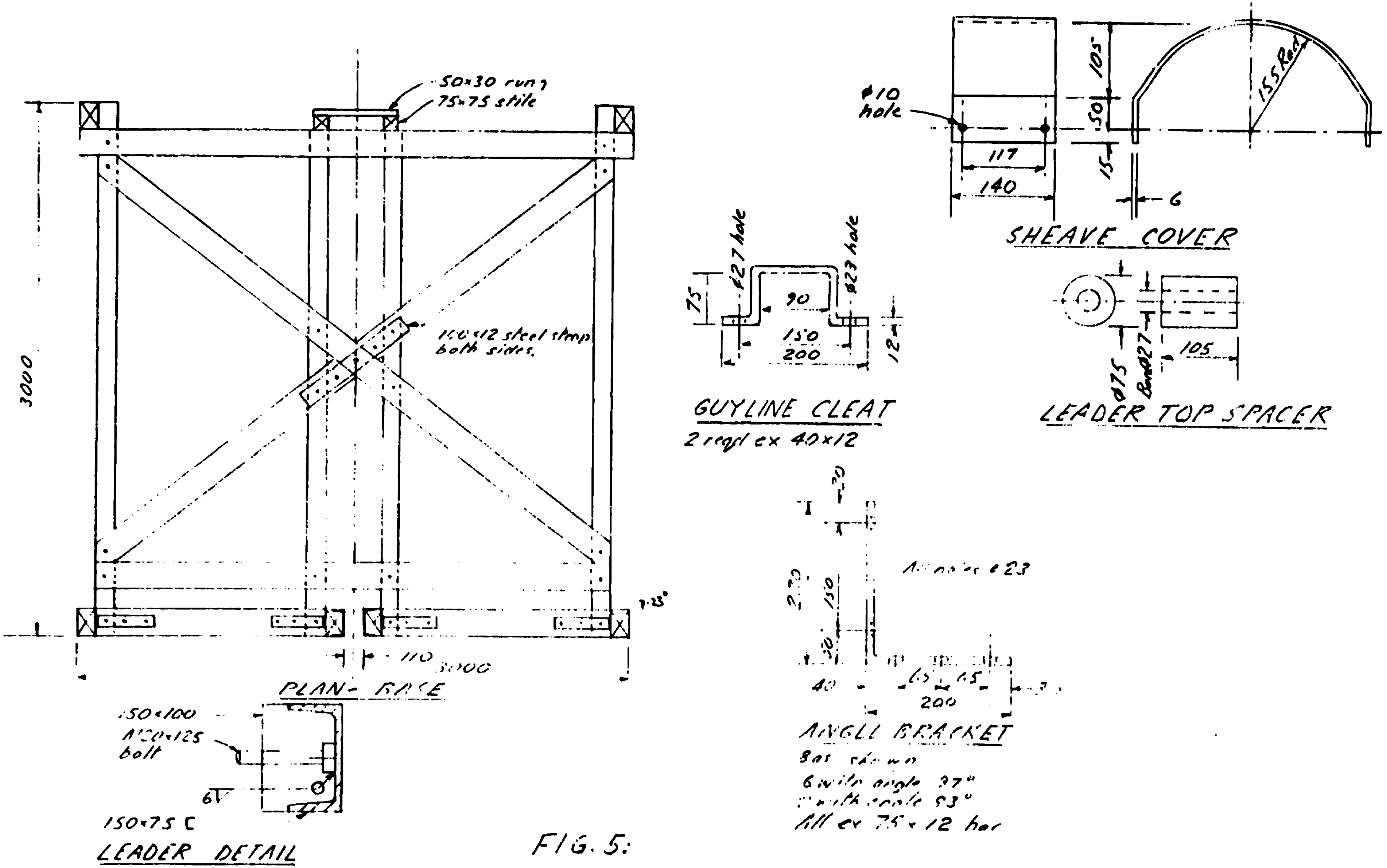


FIG. 5:

10 METRE PILE DRIVING DERRICK

BASE & STEELWORK DETAILS

C. R. FRANCIS,
Registered Civil Engineer
23·6·81

Note: All timber 150x100 exept as shown. Stress grade F14

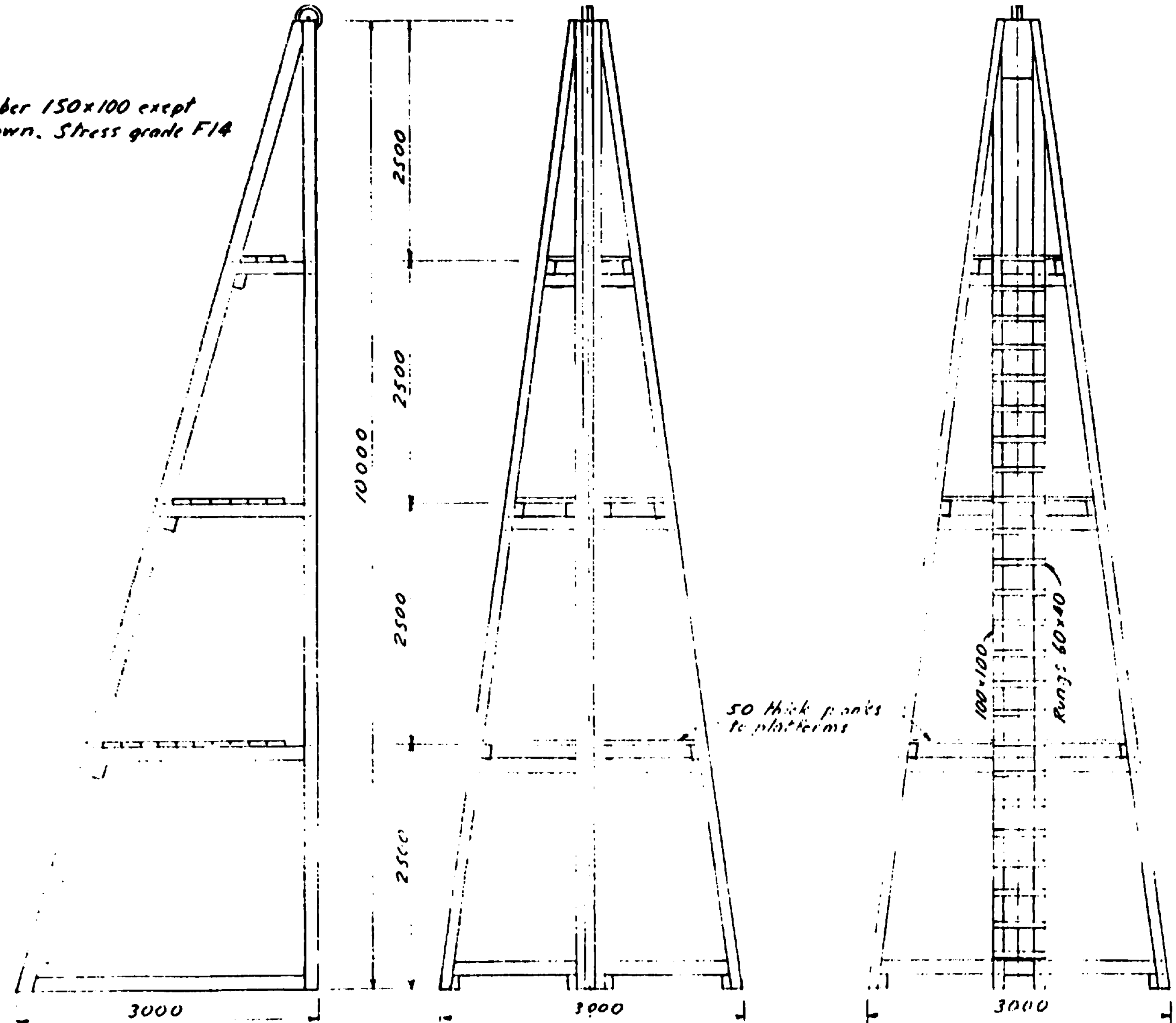


FIG. 6: 10 METRE PILE DRIVING DERRICK Scale: 1:40

C. R. FRANCIS,
Registered Civil Engineer
26.5.89

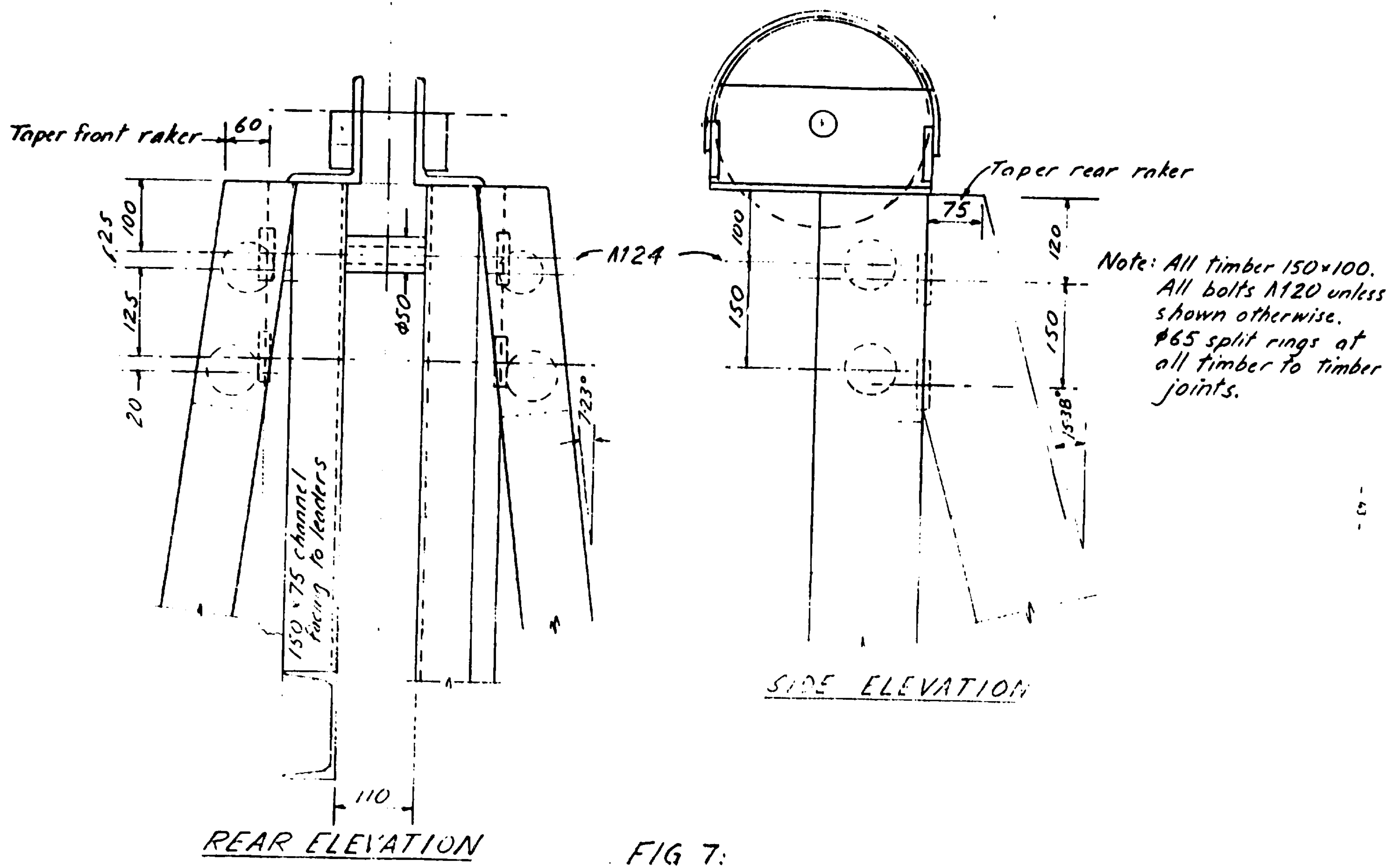


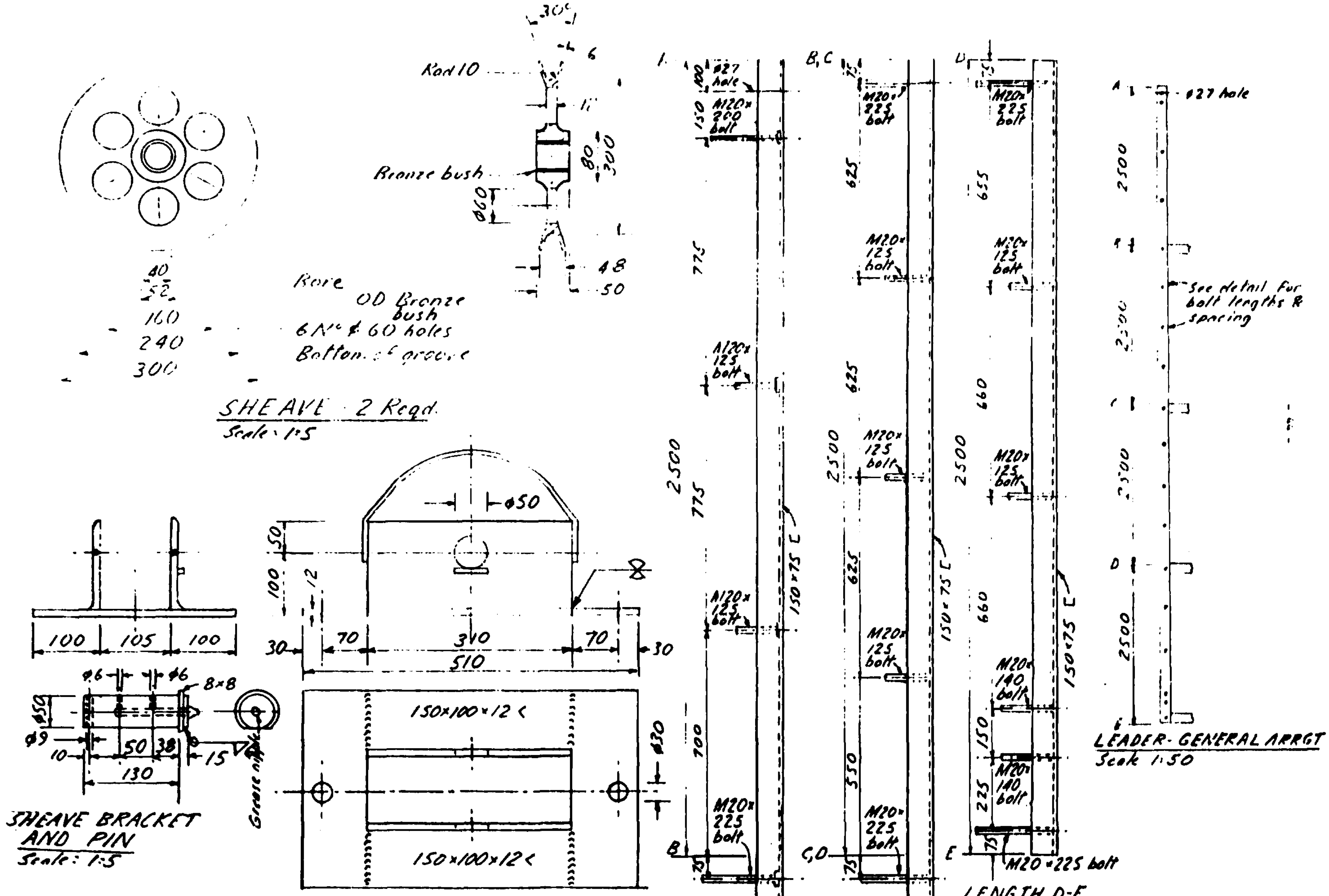
FIG 7:

10METRE PILEDRIVING DERRICK

HEAD DETAILS

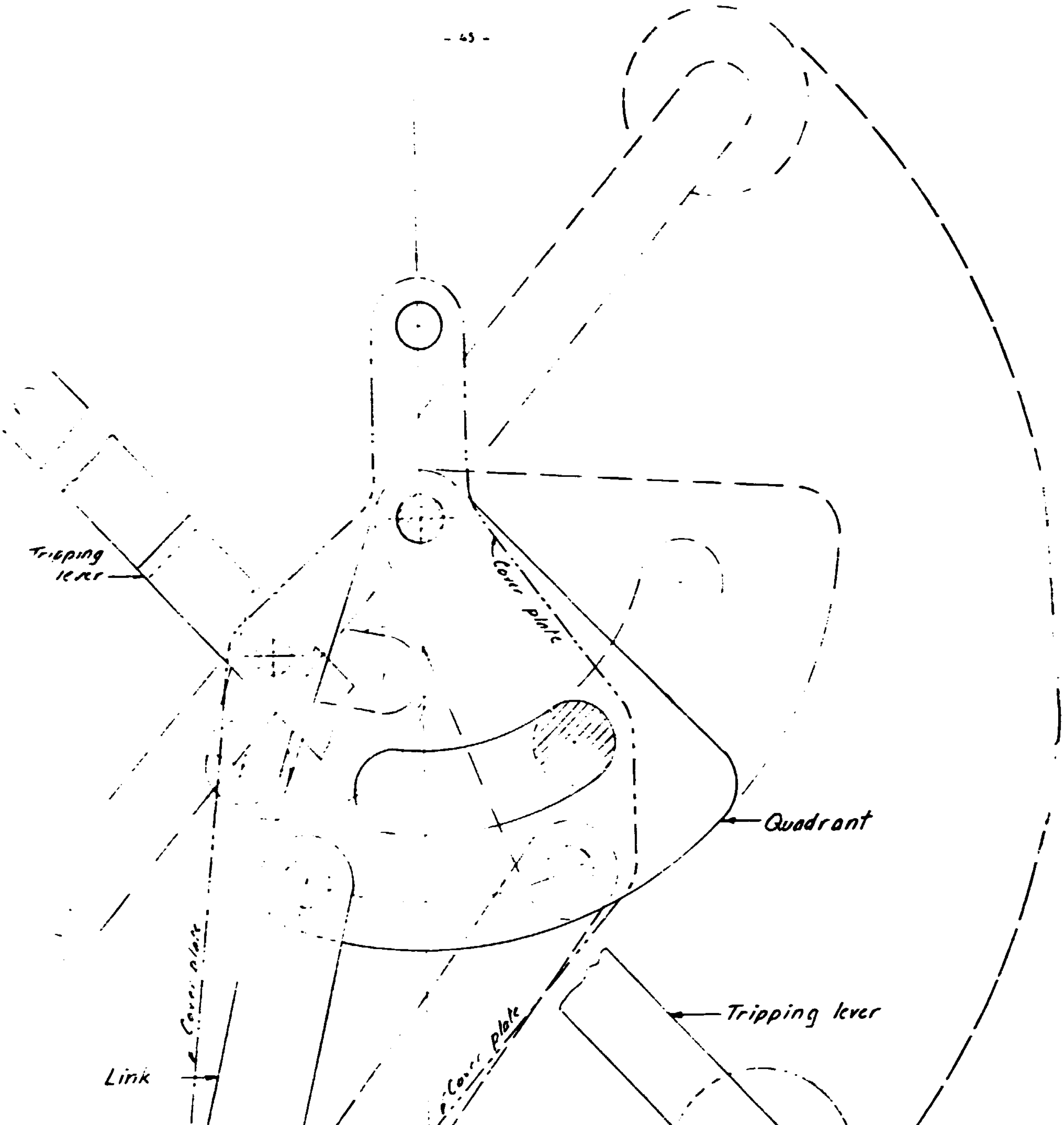
Scale 1:5

C. R. FRANCIS,
Registered Civil Engineer
12.1.81



SHEAVE 2 Reqd.
Scale: 1:5
Rod 10
Bronze bush
Bore
OD Bronze bush
6 No ϕ 60 holes
Bottom of groove
40
52
160
240
300
SHEAVE BRACKET AND PIN
Scale: 1:5
Grease nipple
150×100×12 <
150×100×12 <
510
340
LENGTH A-B
LENGTHS B-C C-D
LENGTH D-E
Scale 1:10
150×75 [
M20×225 bolt
M20×125 bolt
M20×140 bolt
M20×200 bolt
ϕ27 hole
2500
LEADER-GENERAL ARRGT
Scale 1:50
See detail for bolt lengths & spacing

FIG 8



C. R. FRANCIS,
Registered Civil Engineer
28-7-89

FIG 9: GENERAL ARRANGEMENT
TOGGLE RELEASE TRIPPER - 2 TONNE HAMMER

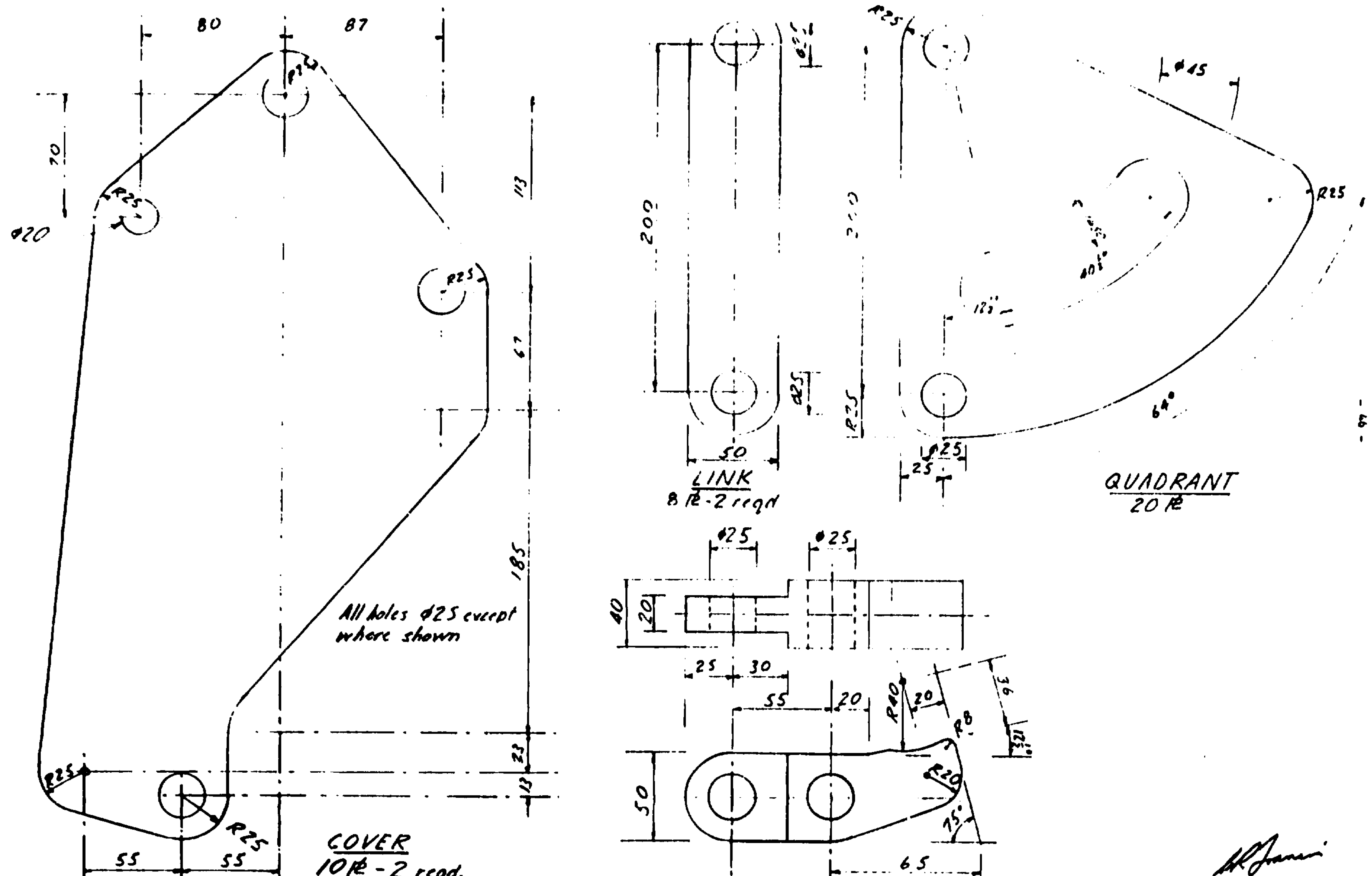


FIG.10: HOOK

TOGGLE RELEASE TRIPPER - 2 TONNE HAMMER

C. R. FRANCIS,
Registered Civil Engineer
27-7-89

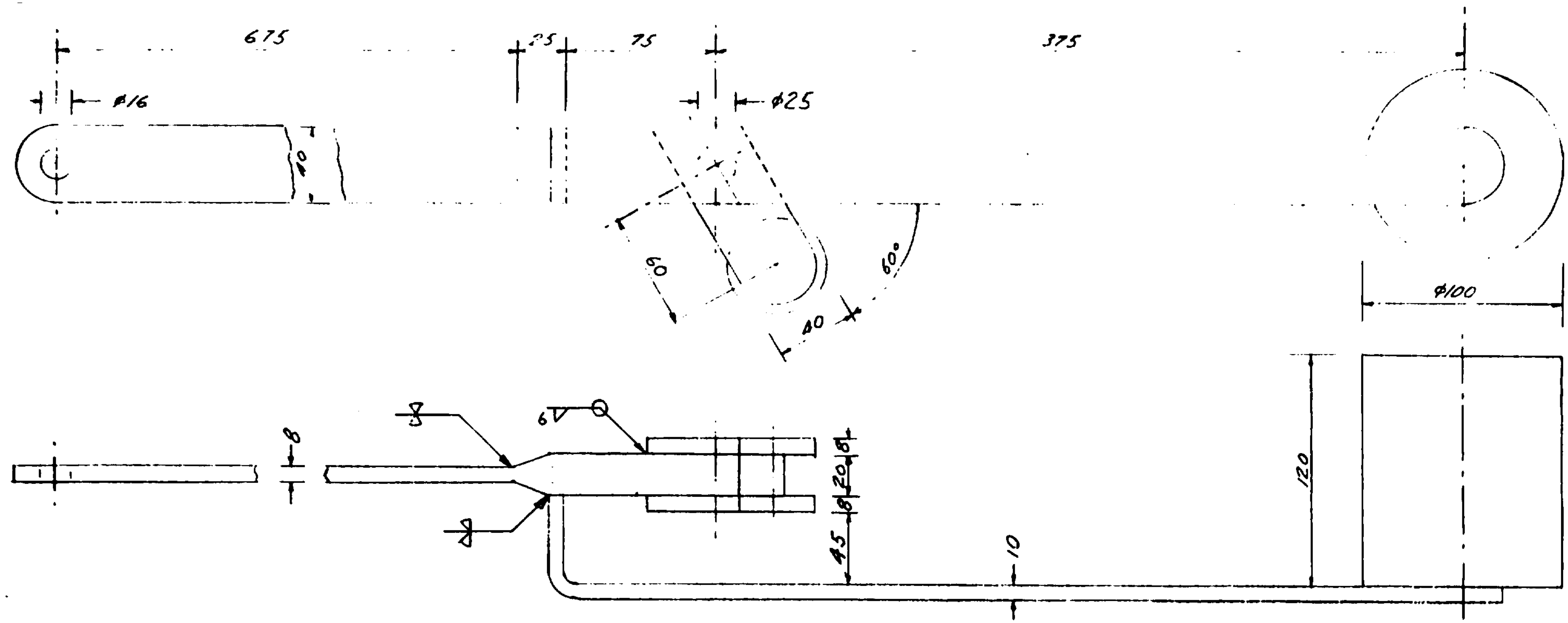
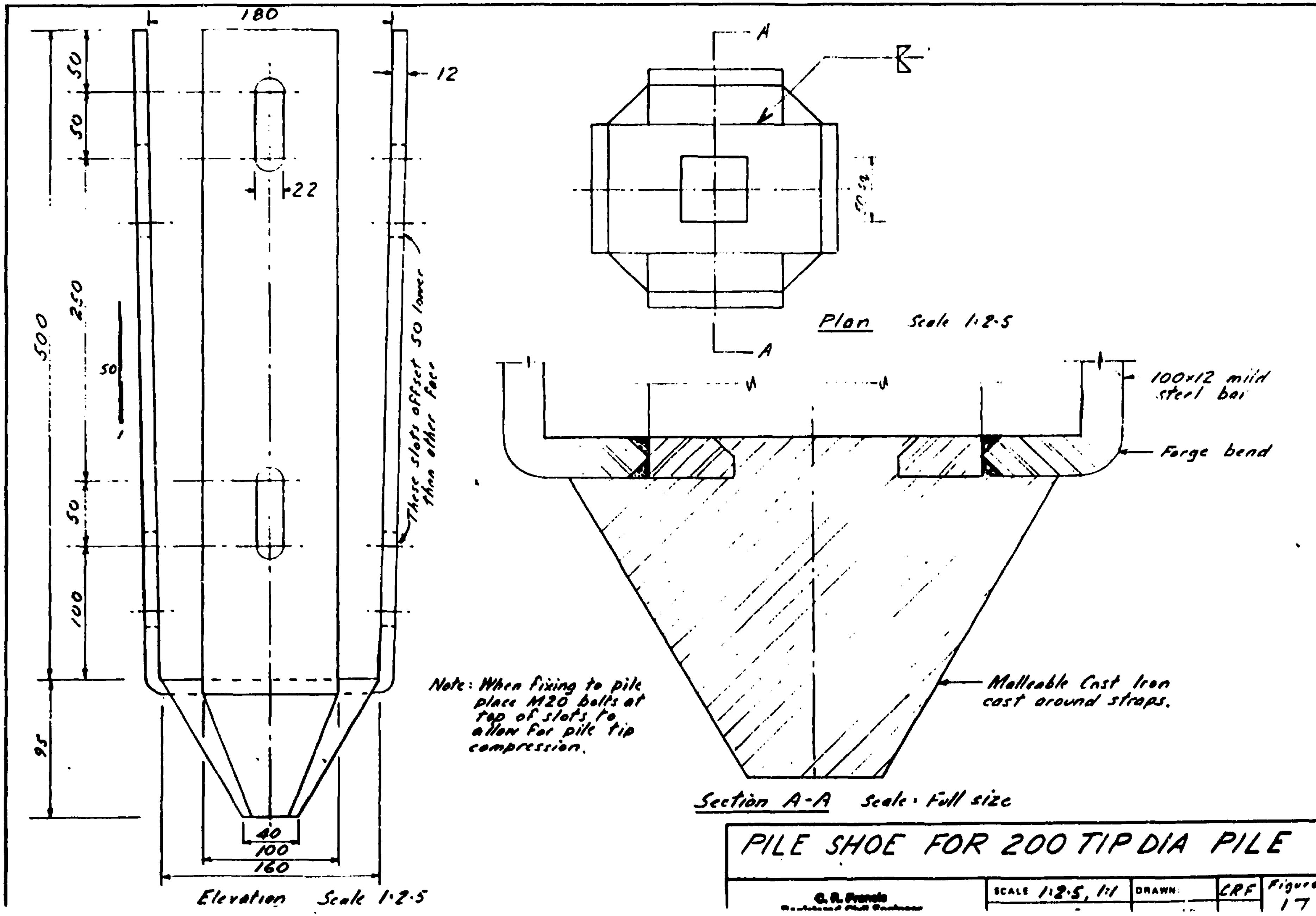


FIG 11: TRIPPING LEVER

TOGGLE RELEASE TRIPPER - 2 TONNE HAMMER

C. R. FRANCIS,
Registered Civil Engineer
27-7-89


180
12
50
50
22
250
500
50
50
100
95
40
100
160
These slots offset 50 lower than other face
Elevation Scale 1:2·5
A
A
Plan Scale 1:2·5
100×12 mild steel bar
Forge bend
Malleable Cast Iron cast around straps.
Note: When fixing to pile place M20 bolts at top of slots to allow for pile tip compression.
Section A-A Scale: Full size
PILE SHOE FOR 200 TIP DIA PILE
SCALE 1:2·5, 1:1
DRAWN: CRF
Figure 17
G. R. Francis