PROCEEDINGS OF THE SRI LANKAN GEOTECHNICAL SOCIETY SEMINAR ON

EXPERIENCE IN DEEP FOUNDATIONS

Venue : ICTAD Auditorium - "Sawsiripaya"

Date: 13th December, 1994 from 9 am - 5 pm

S AS Kulathilaka

FOREWORD

The Sri Lankan Geotechnical Society has as one of its objectives the dissemination of knowledge to engineers in the field of Geotechnical Engineering by holding regular seminars. Past seminars have been held on

(i) Shallow Foundations for Buildings
(ii) Design and Construction of Deep foundations
(iii) Earth Retaining Structures
(iv) Ground Development Techniques
(v) Computer Applications in Geotechnical Engineering
(vi) Geotechnical Practices in Difficult Ground Conditions
(vii) Geotechnical Engineering in River Basin Development

In all these seminars the focus has been to record for the benefit of our engineers the experience that is available in the country.

The past few years have seen several projects being done in which deep foundations have been used. We are now moving into a phase where both economy and safety require a greater understanding of the behaviour of sub-surface structures. The society, therefore, felt it opportune to record the Experiences of Deep foundations, which is the subject for this seminar. We thank M/s Bauer Ceylon(Pvt.) Ltd. for having come forward to sponsor part of this seminar.

This is the first seminar that is being held by our society after the death of our revered President Prof. A. Thurairajah. Therefore, we also publish in this volume, an Appreciation that was sent by our society for his funeral ceremonies at Jaffna.

Prof. B. L. Tennekoon
Vice President, Sri Lanka Geotechnical Society,
25th October 1994

An Appreciation
of
Professor A. Thurairajah

Prof.Thurairajah's contribution to Sri Lanka are manifold. At a time when many Sri Lankan professionals decided to emmigrate and work abroad, Prof.Thurai decided to stay back and provide his intellect, energy and talent for the development of his chosen profession. He can rightly be considered to be the Father of Geotechnical Engineering in Sri Lanka. He has been largely responsible for growth of Geotechnical Engineering in this country. His numerous students will always remember him as a leader who was always available to them, at any time of day or night, to advise them on their problems at induvidual levels. He guided to setup the Sri Lankan Geotechnical Society in 1986 and was unanimously chosen as its founder president and the position he held until his death. He was responsible for introducing soil mechanics as a subject into the Engineering degree curriculum in this country, in 1960's and was responsible for setting up a world class soil mechanics laboratory at the University of Peradeniya in 1965.

It is appropriate to recount Prof.Thurairajah's contribution to soil mechanics in the international arena. His pioneering work with professors Ken Roscoe, Andrew Schofield and Peter Wroth at the University of Cambridge, on modelling the behaviour of soft clays, is now a standard reading material in most postgraduate courses on soil mechanics all over the world. Many of his students now hold professorship in Geotechnical engineering all over the world. Despite the large amount of administrative work he undertook, he remained a first rate academic contributor right until the end. His latest contributions were to the XIIIth International conference on soil mechanics and foundation engineering in January 1994 and to the National Symposium on Landslides held in March 1994.

With the death of Prof.Thurairajah on June 11th 1994 the Sri Lankan Geotechnical Society has lost a true leader, a teacher, a friend and a guide. At this moment of grief, the society extends to Mrs.Thurairajah and to the rest of his family heartfelt condolences and sympathies.

Prof.Thurairajah may be no more, but his fragrance will remain with us for the rest of our lives.

SEMINAR ON

EXPERIENCES IN DEEP FOUNDATIONS

VENUE : Auditorium of the Institute for Construction Training and Development (ICTAD)

DATE : 25th October 1994

A G E N D A

8.30 - 9.00	Registration
9.00 - 9.05	Welcome Address - Vice President SLGS
9.05 - 9.30	Keynote Address - Dr.A.N.S. Kulasinghe

SESSION I

Chairman - Dr. Sunil de Silva

9.30 - 9.50	An early example of top ~~dam~~ - down construction - Dr.A.C.Visvalingam
9.50 - 10.10	Construction Experience in bored and cast in-situ pile Foundations - Mr.T.M. Fernando
10.10 - 10.40	Tea
10.40 - 11.00	Experience in Well Foundation - Mr.D.S.Dantanarayana
11.00 - 11.20	Skin friction in piles - Prof.B.L.Tennakoon
11.20 - 11.40	Identification of type of pile failure from results of Load-settlement curves - Prof.B.L.Tennakoon
11.40 - 12.45	Discussion
12.45 - 13.30	Lunch

SESSION II

Chairman - Mr.D.P.Mallawaratchi

13.30 - 13.50	Evaluation of load carrying capacity of driven pre-cast concrete piles - Prof.B.L.Tennakoon
13.50 - 15.00	Video Presentation of Deep Foundation Techniques by Messers Baur's
15.00 - 15.30	Discussion
15.30 - 15.40	Summing up - Dr.S.A. Kulathilake
15.40 - 15.45	Vote of Thanks - Dr.H.G.P.A. Ratnaweera
15.45 - 16.15	Tea

CONTENTS

AN EARLY EXAMPLE OF TOP-DOWN CONSTRUCTION

A.C.Visvalingam **Engineering Consultant**

INTRODUCTION

One of the most expensive and potentially dangerous items of work in building construction is digging down for basements in such a manner as to prevent the collapse of the vertical faces of the resulting excavations. In this connection, a term which is heard more and more frequently now is "top-down construction". This expression is employed to describe a construction procedure where parts of the permanent structure are built commencing at ground level and are, thereafter, progressively continued downwards in such a manner that the completed parts, in conjunction with previously-installed continuous sheet piling or diaphragm walling, help to retain the sides of the deepening excavation.

Once the substructure work has been carried out economically and safely, superstructure work follows.

The impression given by some recent articles in the technical press is that this technology is new, but the example I shall talk about today is one which was designed and constructed over thirty years ago. As far as I am aware, the term "top-down construction" had not even been coined by then.

In "top-down construction", just as much as parts of the permanent works are utilized as components of the temporary works, parts of the temporary works may also be incorporated into the permanent structure. There have also been some recent modifications to this technique, where upward construction, commencing from ground level, is also progressed at the same time as the substructure work is extended downwards. This is certainly an advance on the old method but the added complexity may be justified only in exceptional cases where commercial considerations call for a particularly intensified speed of construction. I do not propose to go into these two aspects on this occasion but shall confine myself solely to describing the one specific example of "top-down construction" I alluded to a moment ago.

PARK LANE HILTON

In 1961, the structural design of the then tallest reinforced concrete building in the United Kingdom was entrusted to W.V.Zinn & Associates, Consulting Engineers, who were my employers at the time. The building was to house a 30-storey hotel, the Park Lane Hilton, in London. I may mention that, at the last moment, after construction had commenced, the height of the building was limited to 27 storeys by the authorities because there was, I believe, a line of sight problem involving Buckingham Palace.

By the time the detailed design of the temporary works and the main elements of the permanent structure were entrusted to me, the basics of the "top-down construction" concept had been formulated in outline by the Chief Engineer of my firm. It is quite possible, however, that this concept had been adopted elsewhere by others even before 1961, although this particular name may not have been given to it then.

The Client for this project was the multi-millionaire Charles Clore who also owned the contracting firm, Token Construction, which was to carry out the work. As the Managing Director of Token Construction had been a reinforced concrete designer himself for over 25 years, this assignment was all the more challenging in that the structural designs was expeted to be subject to careful scrutiny not only by the London County Council's own checking engineer, from the point of view of safety, but also from the Client-Contractor, who would look for low cost and ease of construction.

DESCRIPTION OF SITE, SOIL, LAYOUT AND FOUNDATIONS

The site is located in one the most prestigious and extremely busy locations in London, facing Park Lane (Fig 1). Three narrow streets form the boundaries of the site on the two sides, and at the rear. The building was designed to cover the entirety of the site. Precautions had to be taken to keep the surrounding streets clear for public use at all times, and to safeguard the neighbouring buildings.

I cannot remember what the soil strata were in the top few metres but, below them, it was London blue gault clay with a permissible bearing pressure of 10 tons.ft^{-2} or 1100 kN.m^{-2} at a depth of 10-11m. There was no problem of groundwater infiltration.

The building itself was to consist of 3-4 basements, a 4-storey Podium and a Y-shaped Tower extending 26 storeys above the Podium (Fig 2). The Tower protruded through the Podium over one half of the site, whereas the other half of the

Podium contained the two spiral car ramps leading to the basement car parks.

No load-bearing piles were used to take the permanent loads. Instead, the Podium was founded on a raft at the basement 3 level, and the Tower on an independent raft at the basement 4 level. That is, the depth of formation of the Tower raft exceeded that of the Podium raft by the height of one basement plus the difference between the thicknesses of the two rafts (Fig 2).

The settlement computations had indicated that the Tower and the Tower basements would settle by about 75mm, whereas the Podium and the Podium basements would rise up by about 25mm during construction, and thereafter (Fig 3). Consequently, the Tower and Tower basement structural levels were initially made up to about 100mm higher than those of the Podium and Podium basements so that, at the end of construction, the levels of the slabs in the Podium and the Tower would have gone a long way towards becoming equal.

Hinged slabs and beams were provided in the gaps which were left between the lower floors of the Tower and the Tower basements, on the one hand, and the Podium and the Podium basements, on the other (Fig 4).

CONSTRUCTION

Continuous bored sheet piles of 450mm diameter were first constructed right round the perimeter of the site to a depth of some metres below the formation level of the rafts (Fig 5). In addition, a number of temporary load-bearing bored friction cum end-bearing piles (G to R) were constructed about 2.5m inside the boundary at intervals of about 13m along the stretch ABCD.

Excavation was carried out inside the area ADEF so that berms sloped at 45 degrees were left intact to support the perimetral bored sheet piles. This was necessary because the bored sheet piles were not capable of acting as free vertical cantilevers to support the required depth of excavation, of up to 11.0m or so. That is, initially, the excavation went to the full depth only over the area A'D'E'F' and was sloped up to basement 1 level at ADEF. The sheetpiling acted as a cantilevered retaining wall from a little below ground level down to basement 1 level.

The exposed surfaces of the sloped earth berms were gunited so as to prevent deterioration in wet weather.

The Podium raft was constructed over the area A'D'E'F', and whatever parts of the structure which could be constructed up to basement 1 level over the area ABCD were completed. A U-shaped strip of the slab at basement 1 level was then constructed over the other half of the site so that, in combination with the slab over the car park area, what was called a "ring beam", to the plan shown in Fig 6, was formed.

The horizontal ring beam ABCD was reinforced to resist the horizontal reactions at basement 1 level from the perimetral bored sheet piling to which it acted as a prop. It was also designed to resist the vertical loads to which it would be subjected at intermediate stages of the construction, and when functioning as part of the permanent structure. Its weight was taken on the temporary load-bearing piles G to R.

Now, a small excavator was lowered through one of the ramp openings in order to carry out the excavation of the earth from under the basement 1 level slab. This excavation was carried down to the basement 3 level and the 1.0m thick raft constructed to a somewhat similar ring beam shape to that shown in Fig 6.

With the propping provided by the basement 1 and basement 3 ring beams, it was a simple matter to excavate down to the formation level of the Tower raft without any shoring as the stiff clay at this depth was capable of standing up in a vertical face of over 4m.

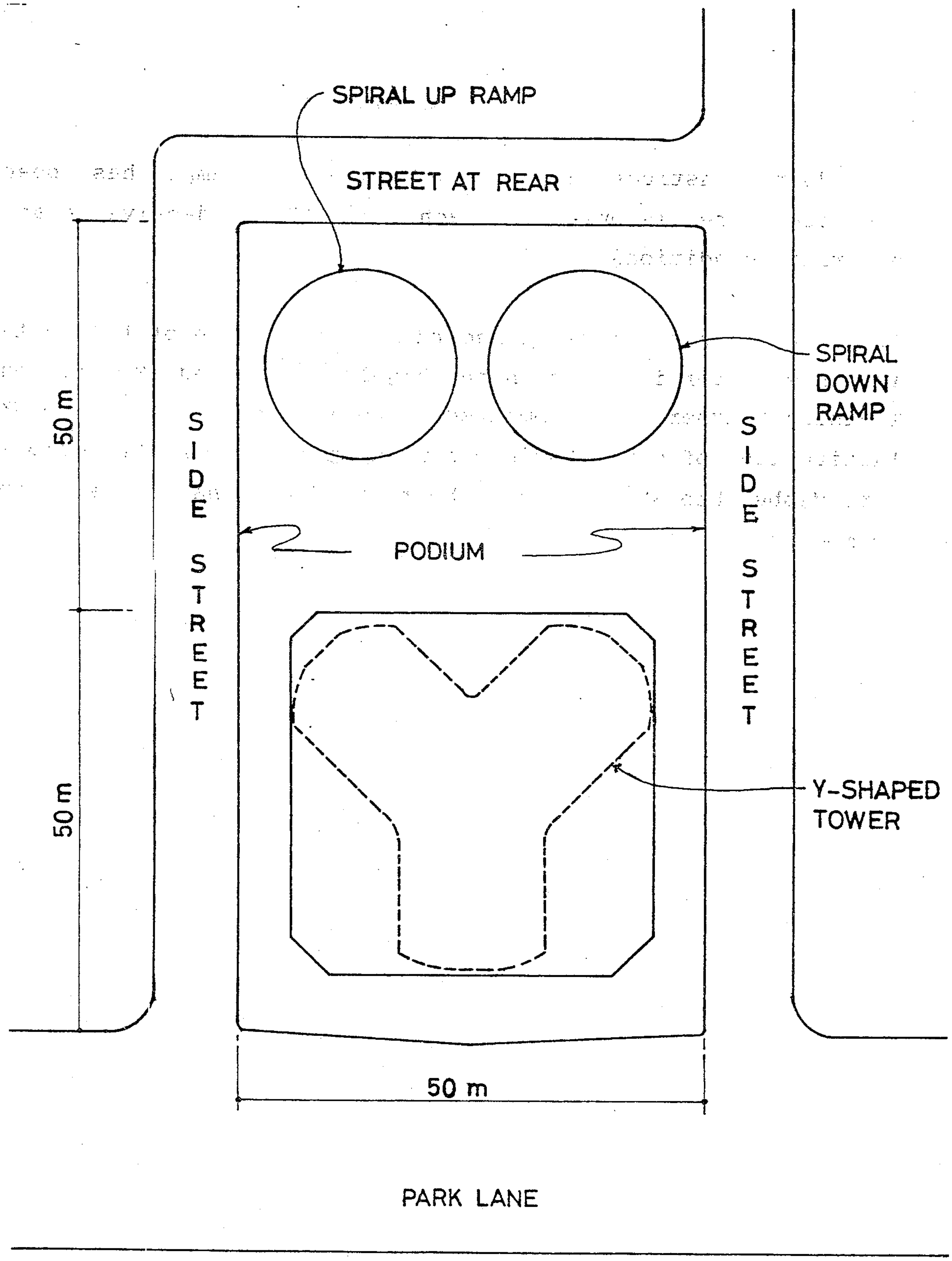
The Tower raft was then constructed and work proceeded upwards in the normal manner.

CONCLUDING REMARKS

There is a great deal to be said for trying to utilize parts of the permanent works to reduce the cost and complexity of temporary works. Hence, there is no way in which the structural designer of the permanent works can say that his job is confined only to the permanent works and that all temporary works are a matter for the contractor, who would be obliged to take full responsibility for the latter. A good designer must, from the outset, consider the methods, procedures and temporary works which are likely to be adopted for the construction of the permanent works so that any cost-saving, synergistic benefits are not ignored.

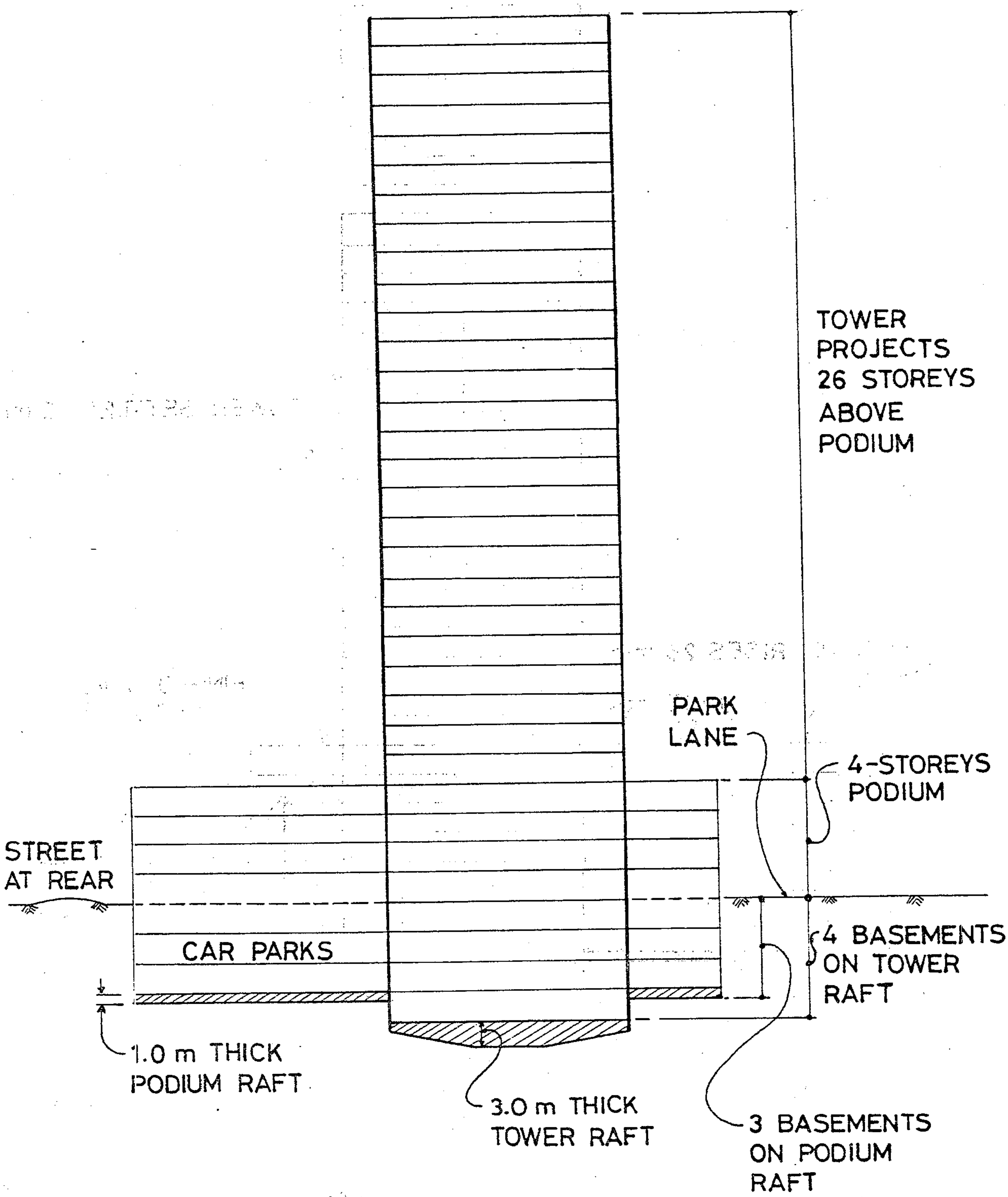
"Top-down construction", of which an early example has been described here, is one way of achieving this objective in appropriate conditions.

It is germane to place on record here that some of the data in this paper is based on the highly-compressed Report on Training & Experience submitted by me in April 1963 to the UK Institution of Civil Engineers when applying for its Corporate Membership whereas, for the rest, I have had to rely on my memory.



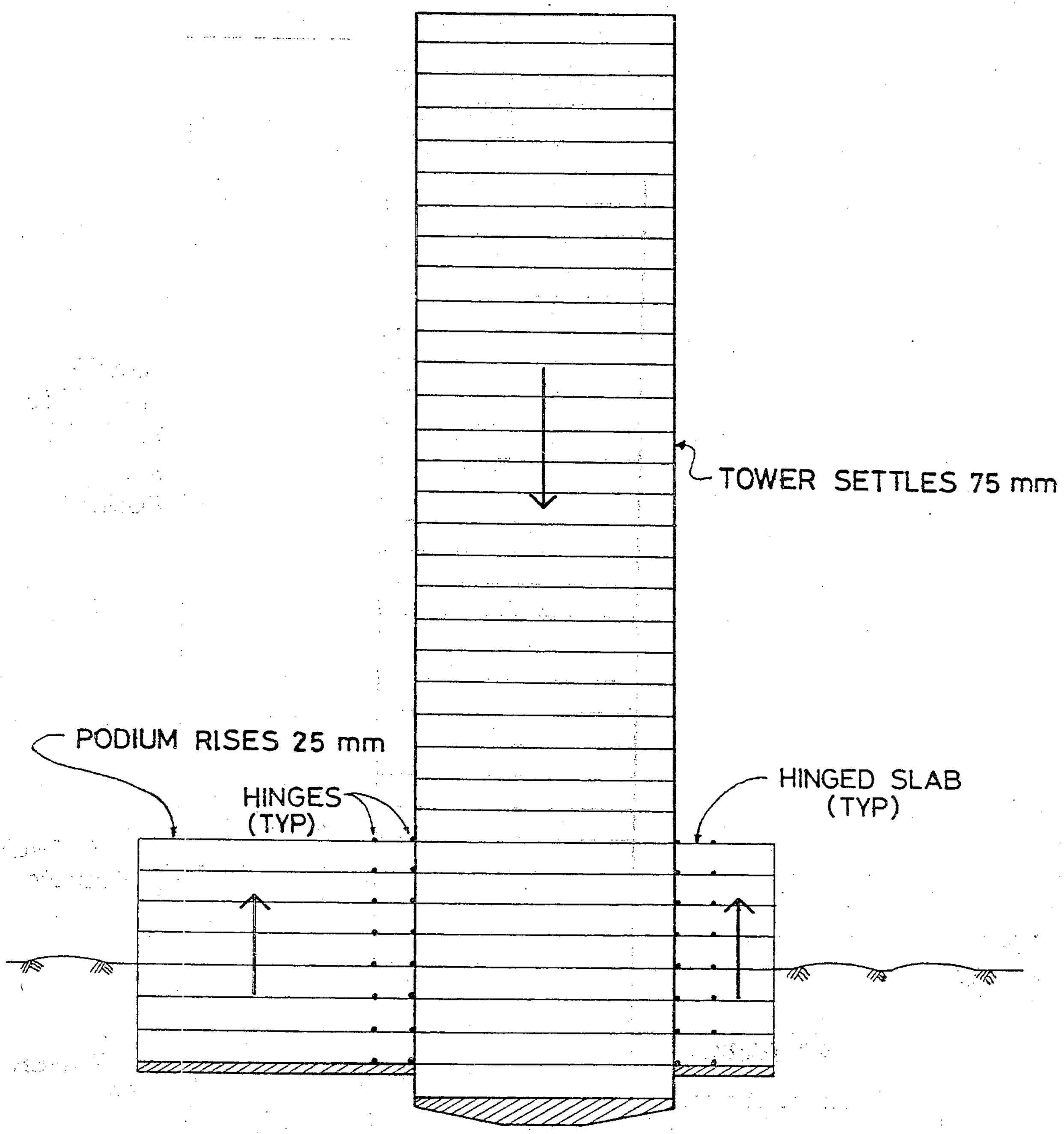
SCHEMATIC PLAN

FIG 1



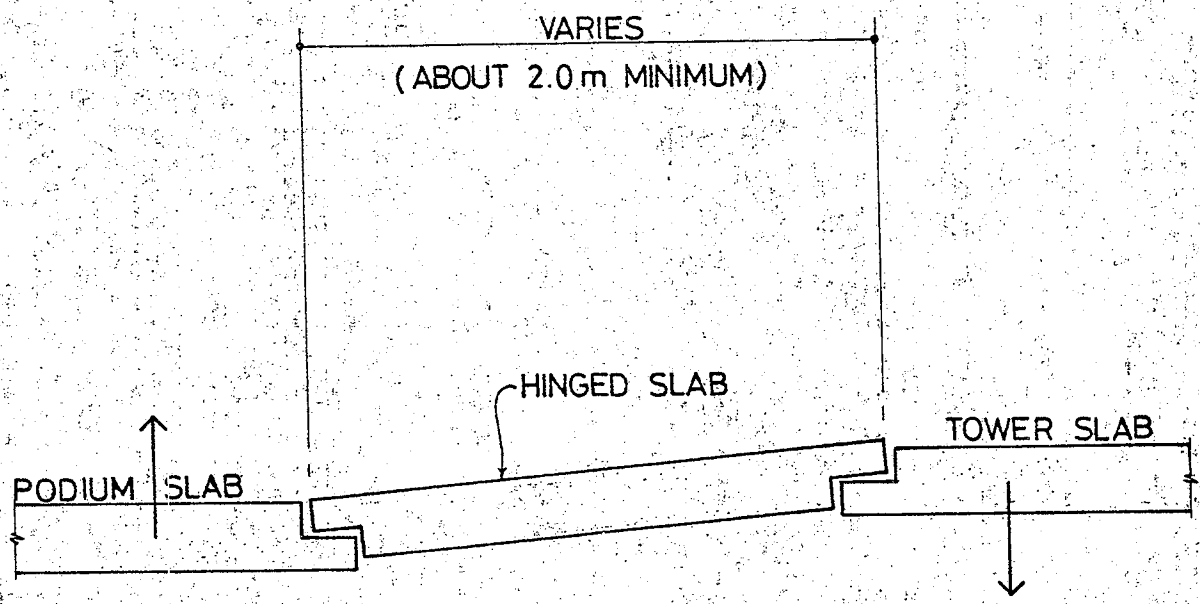
PARK LANE HILTON
SCHEMATIC CROSS-SECTION

FIG 2



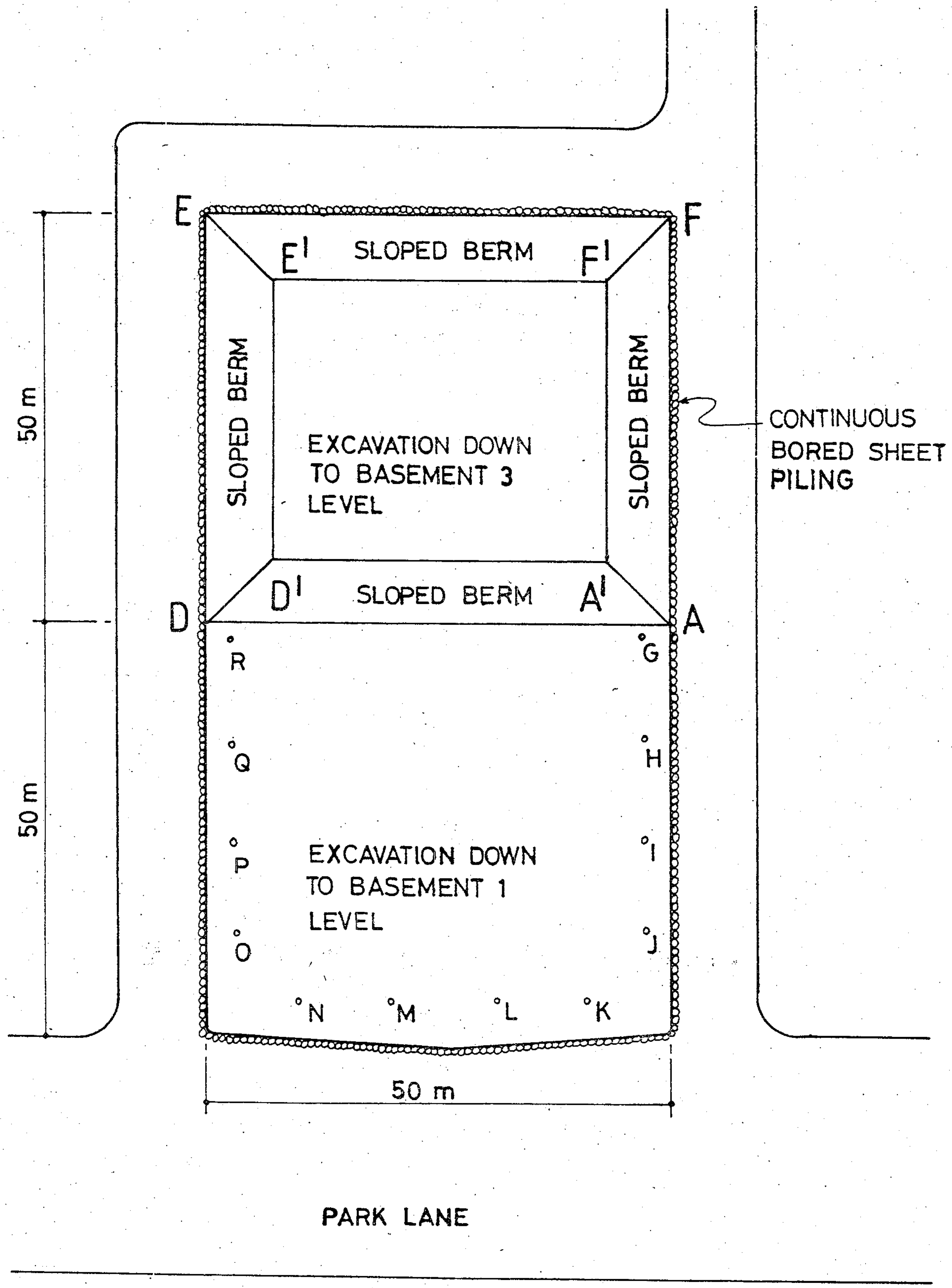
PARK LANE HILTON
SCHEMATIC CROSS-SECTION

FIG 3



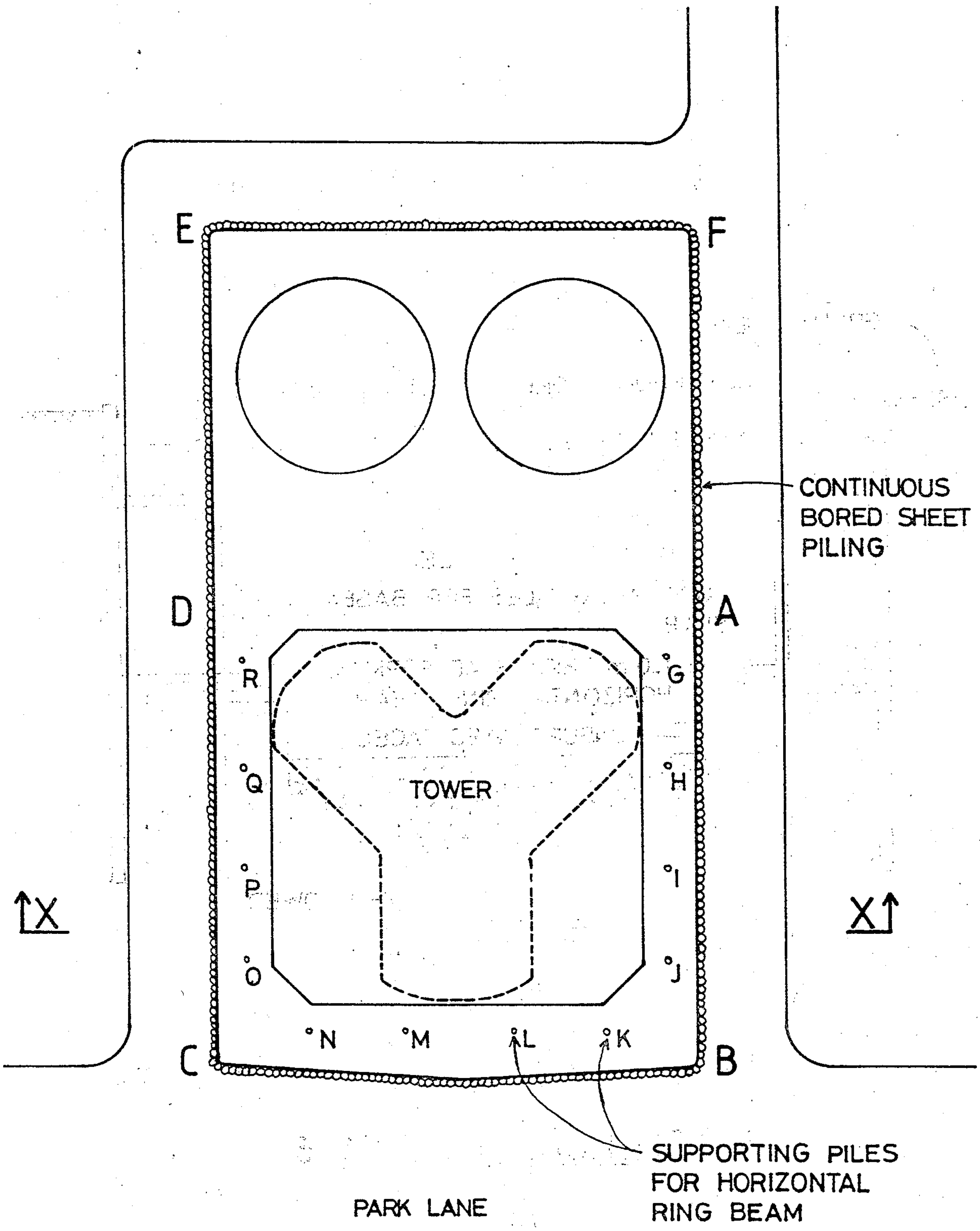
SCHEMATIC DETAIL OF HINGED SLAB

FIG 4



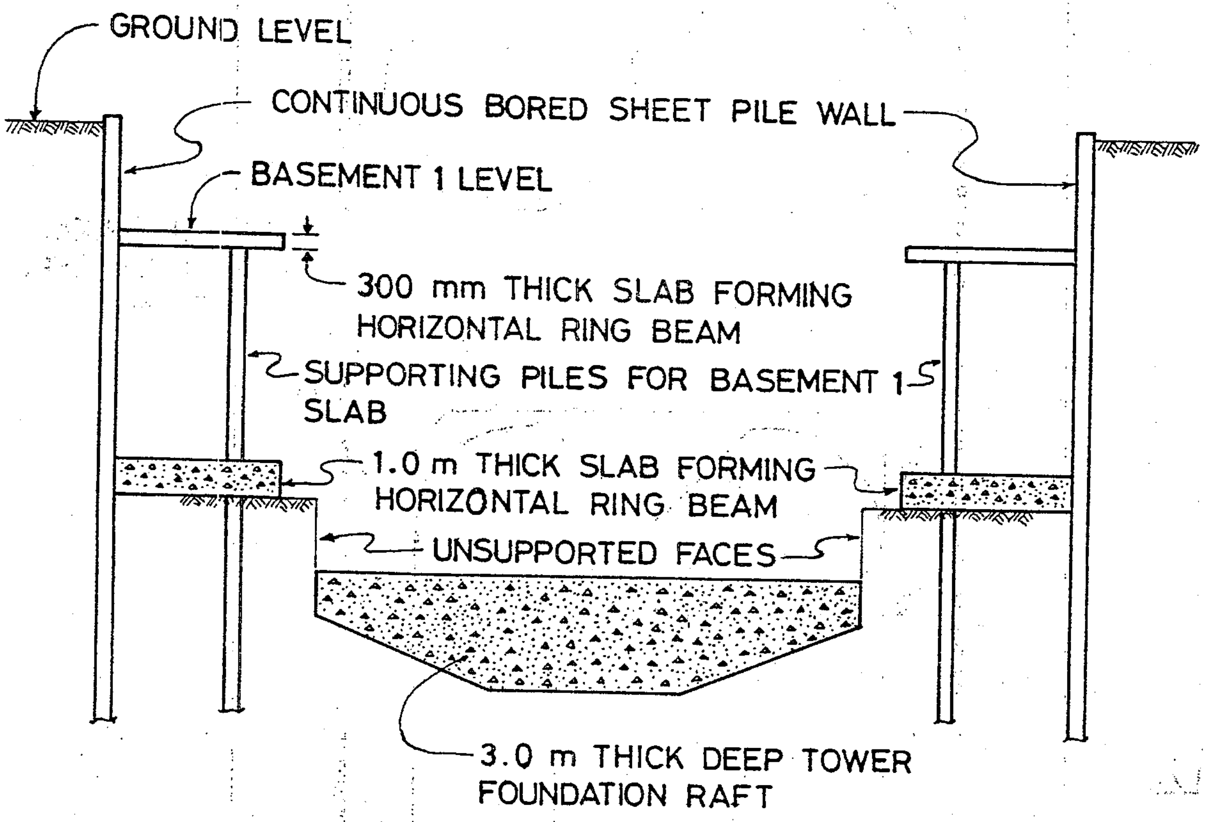
INITIAL EXCAVATION FOR PODIUM

FIG 5



SCHEMATIC PLAN

FIG 6



SECTION X-X OF FIG 6

FIG 7

CONSTRUCTION EXPERIENCES IN BORED CAST-IN-SITU PILE FOUNDATIONS

T.R.Fernando

Engineering Manager
Bauer Ceylon (Pvt) Ltd

Introduction:-

In keeping with the topic of this Seminar " Experiences in Deep Foundations", the contents of this paper relate to the experiences gathered during the construction of Pile Foundations, by the writer in several construction Projects executed recently.

An attempt is made to highlight the importance of certain aspects of the soil investigations and pile construction.

1.0 Boring Techniques & Boring Equipment

The most common techniques are:-

1.1 Percussion Drilling

The essential feature of this method is the use of drop-tools to loosen the soil in the bore hole and then bailer buckets to remove the soil from the bore.

The disadvantages of this method is listed as follows:-

a) Very slow drilling rates and consequently output

b) Non-uniform diameters in uncased piles.

c) Difficulty in achieving verticality of the borehole, in uncased piles.

d) Cannot be used in all types of soils, if uncased.

1.2 Rotary Drilling

In this method the borehole is excavated by the use of a rotary drilling machine. The loosened soil could be removed either:

(i) By the use of drilling buckets and/or bailers.

(ii) By coupling the drilling machine to a mud pump. The cuttings are brought to the surface by the drilling medium (Bentonite) which circulates through the borehole.

The main advantages of this method are:-

a) High production rates

b) Can be used in all types of soil.

c) Drilling possible without causing vibrations.

d) Can be done with or without casings.

1.3 Drilling Medium

Bentonite or Water could be used as the drilling medium. The use of bentonite is very common in this country, and less likely to give problems of collapse of borehole. In soil conditions where the soil structure consists of peat, soft clays, sandy clay, clayey sands, water can be used as the drilling medium. In conditions where non- cohesive soils are present, water as a drilling medium may cause problems of collapse of borehole. Nevertheless, in soils where water can be used as the drilling medium, the Borehole should be maintained with water, without allowing a drawdown of not more than a meter from ground level. In this case, " Reduction of waiting time" discussed under (4.1) becomes extremely important.

1.4 <u>Boring Equipment</u>

The type of equipment deployed by the contractor needs careful consideration, from the point of view of:

a) The ability to obtain a borehole of uniform diameter.

b) A borehole of required verticality.

c) Speed of drilling and/or output - which may seriously have an effect on the project deadlines.

d) Rock socketing - The performance of the boring tools in weathered rock, moderately weathered rock, slightly weathered rock and hard rock (bed-rock), needs very careful evaluation.

e) The mobility of the machines or its suitability to surface conditions as available at site.

f) Whether ground improvements should be carried out at additional cost.

g) The effect of vibrations to near by structures.

If above aspects are not evaluated at the pre- contract stage, problem are likely to crop up later. This will delay the project, which is not in the best interests of the client.

2.0 **Upward Movement of Reinforcement Cages**

The problem mainly occurs in piles where pile cut-off levels are at depths greater than 1 m. below ground level. In buildings which involve basement construction the cut-off levels are generally between 3.5 to 4.0 meters below ground level. The above problem could be overcome in two ways, namely:-

a) Weld cross- bars at the bottom of the pile cage See Fig.(6).

The tremie pipes will effectively stop the upward movement of the cage, when the first charge of concrete is placed in the pile, by bearing against the cross - bars.

b) Extend at least 30% of the longitudinal reinforcement, upto the ground level. See Fig.(7). In this case the contractor will have to incorporate the cost of extra reinforcement in his piling rates.

3.0 Use of Ready-Mixed Concrete in Pile Foundations

3.1 Quality of Concrete

The parameters considered most important in respect of quality of concrete used in pile foundation are:-

a) Workability
b) Strength

For concreting a pile with tremie pipe, workability of the concrete in the range of 150 mm to 200 mm is considered satisfactory, and generally an average value of 175 mm had been found to be suitable in most cases.

In the case of the concrete strength the commonly usec design mixes have been Grade 25 & 30. The cement content would therefore be the major factor that would contribute to the achieving of the design strength of a high-slump concrete. A minimum limit for cement content of 400 kg/m³, have shown very satisfactory results in almost all cases.

The piling contractor has to be extremely cautious in ordering of concrete from the Ready-mix concrete suppliers, and one has to make sure that, the mix is of required workability and minimum cement content.

Quoted below are examples, where these parameters have been lacking, and another case a special mix - design has been done.

Supplier-A

Eg.1. Grade Designation	25	30
Slump	10 cm	10 cm
Cement Content	360 kg/m^3	402 kg/m^3
Fine Aggregate	726 kg/m^3	738 kg/m^3
Coarse Aggregate	1120 kg/m^3	1142 kg/m^3
Water Cement Ratio	0.50	0.46
Admixture (pozzalith)	1.08 lit/m^3	1.20 lit/m^3
Mixing Reagent (AEA 303A)	21.6 ml/m^3	24.0 ml/m^3

Eg.2. **Supplier-B**

Grade Designation	25	30
Slump	N.A	N.A
Cement Content	349	387
Fine Aggregate	806	977
Coarse Aggregate	987	977
WC Ratio	N.A	N.A
Admixture	N.U	N.U
Mixing Reagent	N.U	N.U

Abbrev:- N.A:- not available
N.U:- not used

Eg.3. **Supplier - C**

Grade Designation	30
Slump	20 cm
Cement Content	502 kg/m^3
Fine Aggregate	680 kg/m^3
Coarse Aggregate	975 kg/m^3
W/C Ratio	0.40
Admixture	150 lit/m^3
Mixing Reagent	20 ml/m^3

The above are typical cases of ready- mixed supplier's standard mix-designs.ie. Supplier (A), Supplier (B), Supplier (C)

In case of Supplier (A), Gr. 30 Concrete satisfied the minimum cement content although the slump was not upto requirement. In such situations there is a likelihood of more water being added while all other design parameters remain the same.

3.2 Wastage of Concrete due to Over -break & Other Cause

In general the wastage factor due to over-break has been at around 15%. In cases where the soil strata consist of layers of peaty soils, the wastage factors have reached upto a maximum of 50% depending on the thickness of the peat layer. This is due to the tendency of the concrete to get pushed out through the peat layer, in to the surroundings.

To contain the above problem and reduce the effects to a minimum, a Geotextile membrane may be wrapped around the reinforcement cage. This will be at the zone where the peat layer is encountered, which could be established during the boring process.

4.0 Concreting of Piles

4.1 Reduction of Waiting Time

In the first instance, it is always very good practice to carryout concreting as soon a the installation of the reinforcement cage and insertion of tremie pipes have been completed. It has been observed that delays occur due to the following problems:-

a) Ready-mixed supplier does not deliver the concrete to site on time.

b) The contractor runs into difficulties during the process of installation of reinforcement cage or the tremie pipes, thus prolonging the time period between completion of boring and commencement of concreting.

4.2 Experienced Labour Crew

An experienced labour crew, well versed in the concreting of piles with tremie pipes, should be deployed by the contractor. The essential features of the concreting operation could be summarized as follow:-

a) Discharge of the first load of concrete from the hopper so as to displace the layer of bentonite from the bottom of the bore.

b) To ensure that the bottom of the tremie is well within the concrete inside the borehole. Generally the tremie pipe should be kept at least a minimum of 2 m from the top level of the concrete at any given time. Constant monitoring of surface levels of the concrete should be ensured during the concreting process to achieve this condition.

c) Removal of tremie pipe lengths as and when required.

With an experienced crew only, that one could successfully complete the above operation, within the shortest possible time, so that the quality and integrity of the piles could be ensured at all times. The labour crew is also expected to be well versed in the art of stacking and maintenance of treme pipes after each concreting operation.

4.3 Effects of the Quality of Concrete on the Concreting Process

There will be very bad effects on the whole concreting process if the ready mixed supplier's concrete is not satisfactory for the tremie pipe concreting method.

This problem mainly occurs from the point of view of workability of the concrete. The workability itself may vary between the point of mixing concrete, and point of receipt, depending upon

a) The weather conditions

b) The distance between the ready mixed supplier's plant location and the project site.

Therefore the contractors are well advised to be careful when stipulating the workability, that the ready-mixed supplier should keep at the batching plant.

5.0 Importance of Site Investigations

The problems that have occurred in several sites in the recent past, point out to the inadequacy of the site investigations programs that have been carried out.

The instances that had surfaced recently point out towards following causes:-

i) Inadequacy of the number of boreholes done at the site. The object would always be to cover the variations of soil conditions over the entire site.

ii) Selection of locations of boreholes, had not been as desired.

iii) Non availability of data on rock-cores upto a reasonable depth into bedrock. There have been cases where the rock condition itself had not been identified at all in terms of weathered, moderately weathered, slightly weathered or bed rock etc.

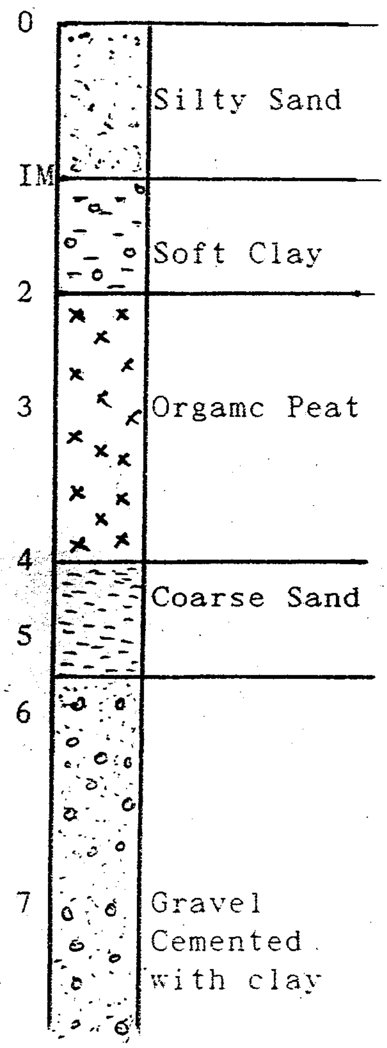
One of the reasons for such deficiencies could be due to client's reluctance to spend funds on site investigations, but if this is the case, this has to be corrected, with timely advice.

In very recent times, the amount of information, provided in these reports, have improved tremendously. Rock cores have been analyzed upto several metres into the rock, and information such as core recovery (CR) and rock quality designation (RQD) have been provided.

This provides prior information to the contractor and to the Supervising Engineers, thus eliminating the conditions that may lead to technical problems, contractual disputes and delays in projects.

6.0 Soil Profiles in recent pile foundation Sites

6.1 Housing Complex - Bloemendhal



Data

Type of piles - Bored files

No of Buildings - 06
No of Boreholes done- 02

Depth of borehole as per report - 11 m

Water Table - 0.3m below G.L

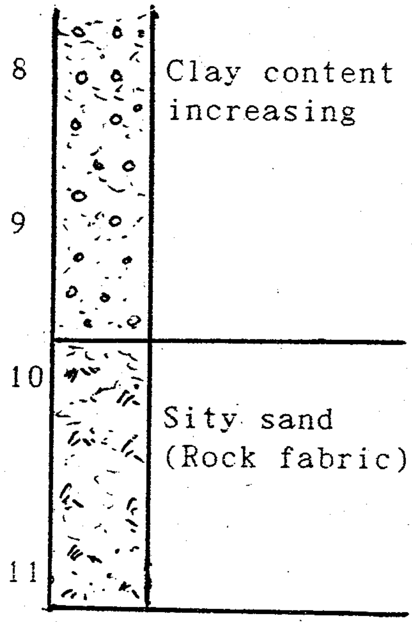
No of piles per Building - 16

Total No.of piles - 96
Diameter of pile - 525 mm

Observations

Max. depth of pile - 27.27m
Min. depth of pile - 08.15m

No. of load tests - 05
No. of failed tests - 03



Depths of failed piles -
A_5 2_b - 15.10 m
A_3 2_c - 16.40 m
A_5 2_e - 20.00 m

Rock Conditions - Not indicated in Soil Report

Comments

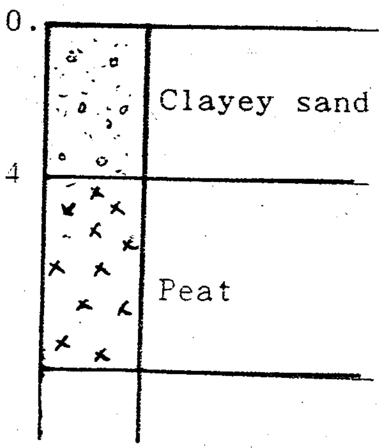
1) It was noted that the 2 bore hole investigations done were confined to only one half of the site.

2) The pile depths were more than double in the other half of the site.

3) At least a minimum of 6 borehole tests should have been done one at each building which might have indicated a much more accurate picture of the rock strata depths

6.2 Factory Project- Mabole.Wattala.

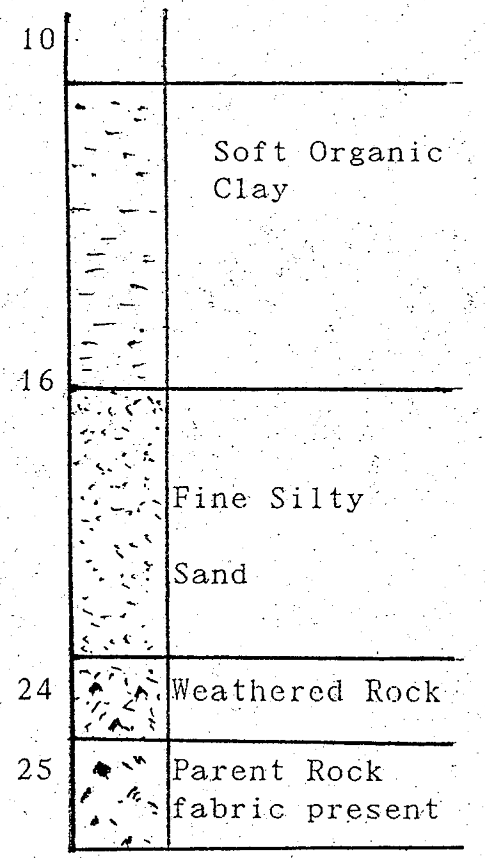
Data



Type of piles - Bored piles
Bo. of Buildings - 03
No of Boreholes - 02

Depths of borehole - 24.85 m
Water table - 1m below G.L

No. of piles - 93
Dia. of piles - 400 mm



Observations

Max.m depths of pile-26.2 m
Min.m depths of pile-22.4 m
Load tests -Not specofoed in Contract

Comments

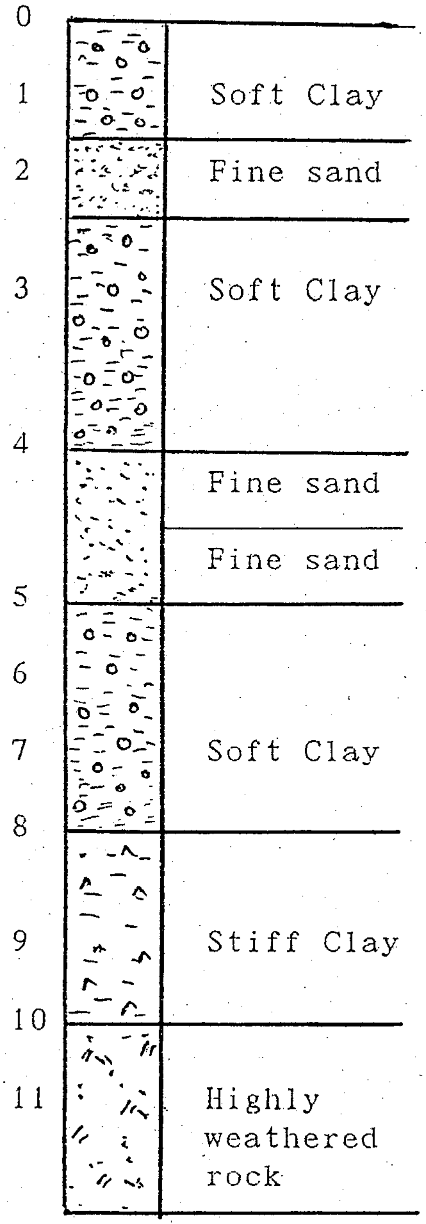
(i) In this project the contractor was asked to construct 17m piles, which were to terminate on the silty sand layer. Subsequently when the works were commenced, this decision was changed and contractor was asked to terminate boring on rock.

(ii) The second borehole done during investigations was terminated at 16.95m below G/L on the fine silty sand layer. The SPT value at this termination level was only 14. The BOQ was done to suit 17 m piles and the contract.

(iii) Load tests were not specified thus ignoring the very importance of carryout load tests.

(iv) The Contrator brought these to the notice of the client.

6.3 Shops & Office Complex - Kotahena



Data

Type of piles	-	Bored piles
Building	-	1
Depth of Borehole	-	11 m
Water Table	-	1.7m below G.L
No of piles	-	20
Dia of piles	-	500 mm & 600 mm

Observations

Max pile depth	-	18.0 m
Average pile depth	-	14.0 m
Load tests	-	Not specified
Rock condition	-	Identified as highly weathered rock

Comments

In this site, the borehole had been done towards one end of the site. The maximum depth was recorded at the other end of the site and there was a gradual increase in depth from the shallow end.

In this chapter, with the above examples it has been illustrated that there existed "significant differences" between the details indicated in the soil investigations reports, and what was actually achieved in the field.

If the borehole exploration program is not planned carefully, as highlighted in this chapter serious contractual & technical problems may occur. This is due to the soil investigators report being used as the main guiding factor in the preparation of

(a) The Bill of Quantities,

(b) Pile Designs

(c) Contract specifications & other special conditions.

7.0 Case - Study of Pile Depth Variations in a Particular Project

The depth of piles within the project site varied by two fold. There were six buildings planned on the site. The variations in pile depth was very marginal within the area occupied by building 1A, 1B, 1C. The depths varied from about 9 m to 13 metres.

The depths of piles varied very rapidly and reached a maximum depth of 27 metres in the area occupied by buildings 2A, 2B, 2C. (See figures 2&3).

The dip of the rock strata was so great from one pile position to the next adjacent one, that one - diameter socketing may not have been enough, at all resulting in a series of failures, when the piles were tested.

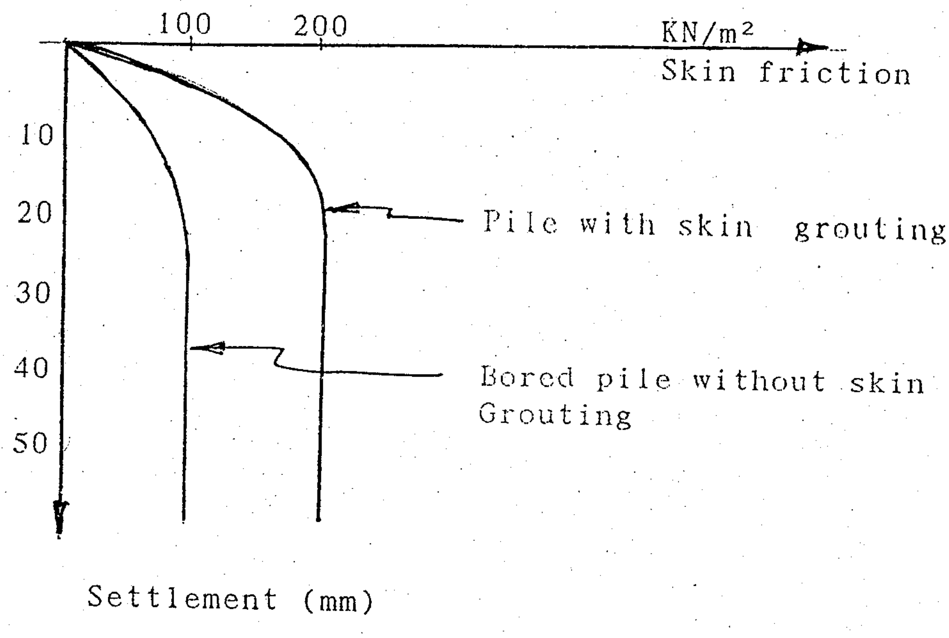
Referring to figure (4); it could be assessed that piles A3 2c A42c, A5 2c & A6 2c would have failed. The load test on A3 2c indicated excessive clastic & plastic deformations. The load test on A5 2c had to be terminated without completing the second loading cycle due to signs of increasing settlement without increment on loading. This was due to that fact that the pile was slipping at the toe, and tilt of the pile head too was observed. This condition was inevitable due to the resulting exposure, of one - half of the pheriphery of the pile at the toe on the steeping rock face. (See figure (6).

8.0 Refinements to Pile Construction Techniques

These may be listed as,
1) Pile skin grouting
2) Pile Base grouting

8.1 Pile Skin Grouting

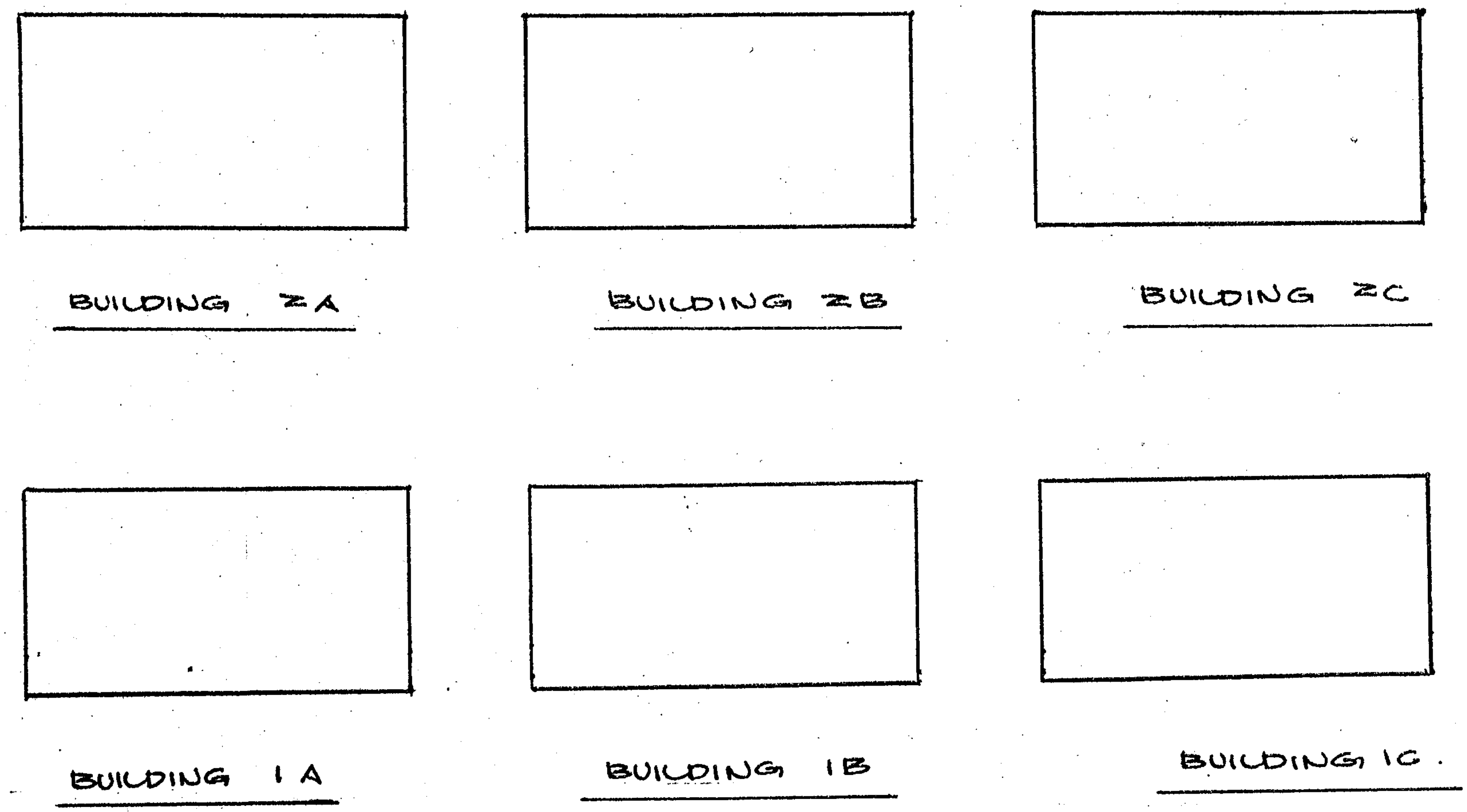
This process is used to improve the bearing behaviour of the pile. Skin grouting compresses the soil in the shaft area and produces intense interlocking of the pile concrete and the soil. The skin friction and thus the load carrying capacity of the pile can be increased considerably in this way, depending on the type of soil.



Ex:- 750 mm Dia pile, 18.80 m long, in sandy silt.

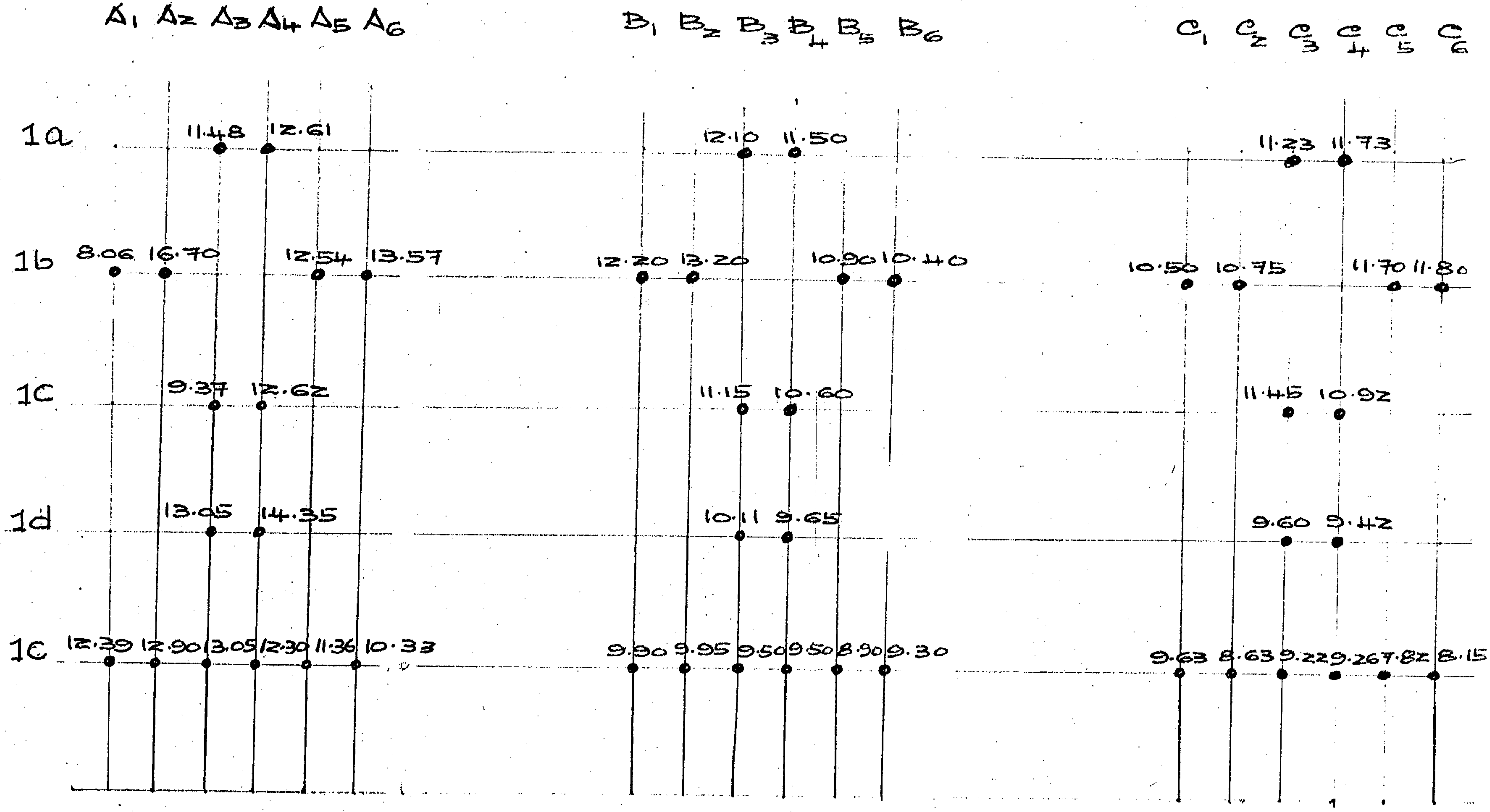
8.2 Pile Base Grouting

This must be considered to be a supplement to skin grouting. The contact area between the pile base and the soil is compacted with cement slurry. This compensates the unavoidable loosening of the base of the bore. Part of the settlement to be expected is eliminated by this method.



LAYOUT OF BUILDINGS

FIGURE (1)

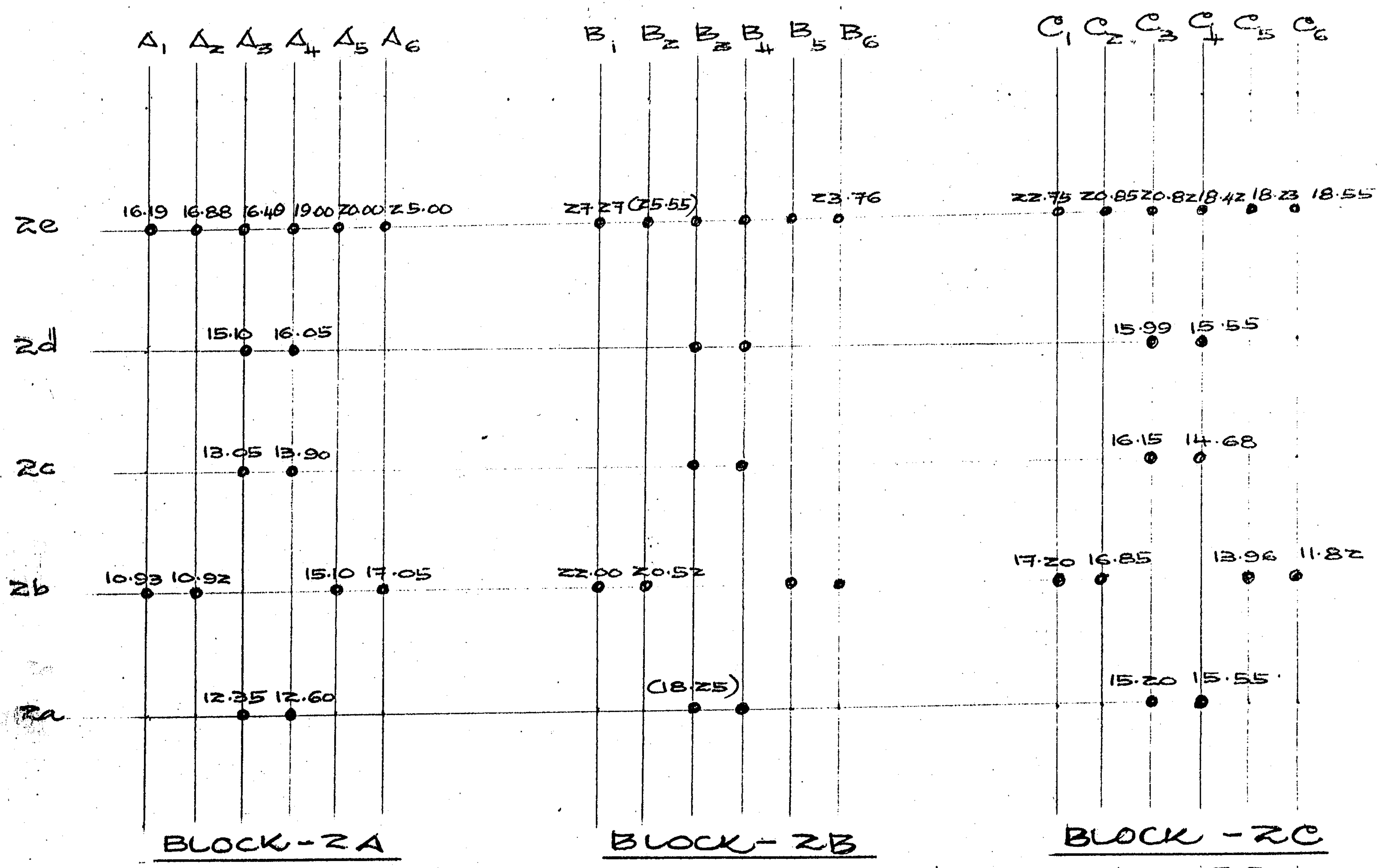


BLOCK - 1A

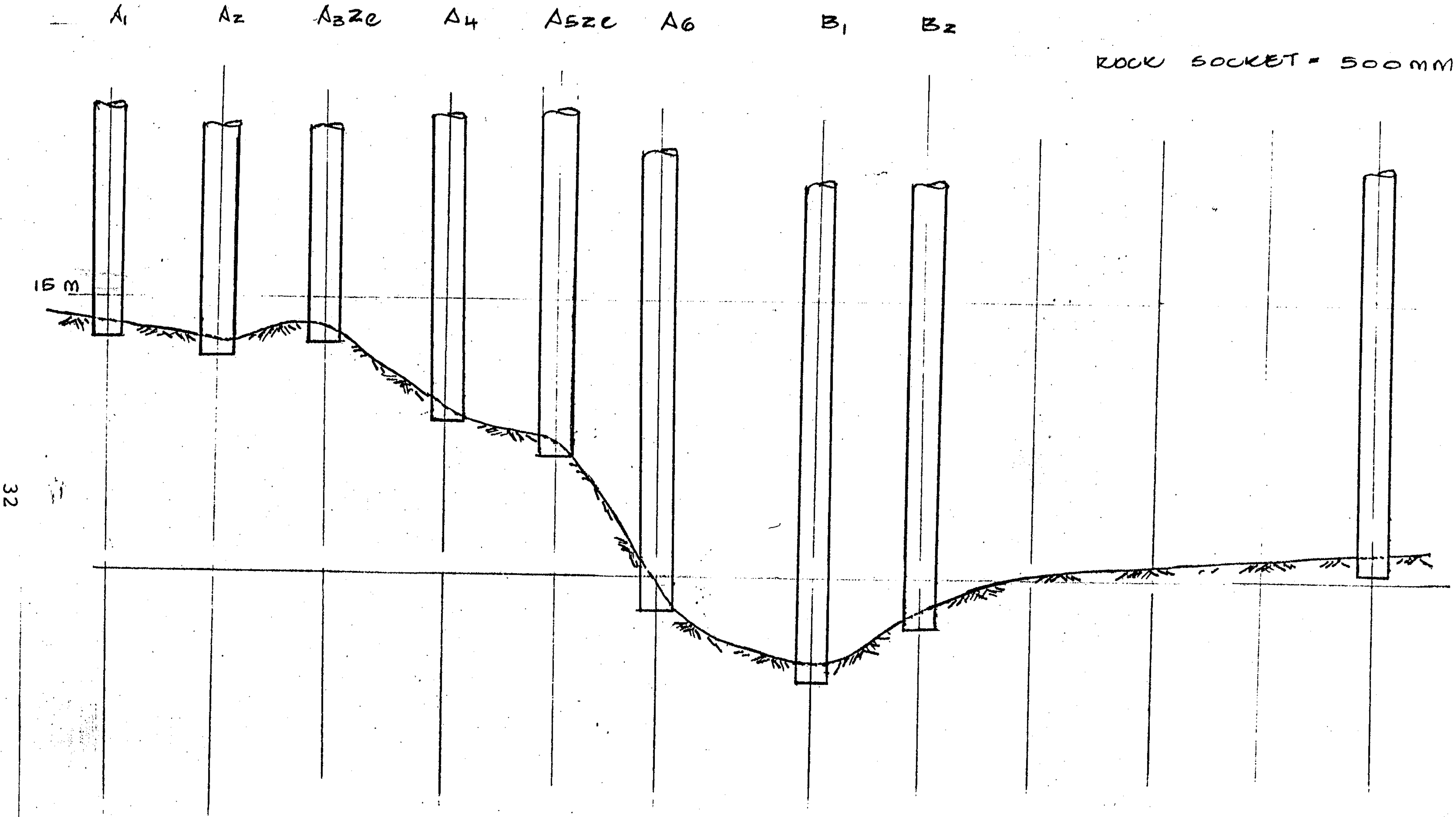
BLOCK - 1B

BLOCK - 1C

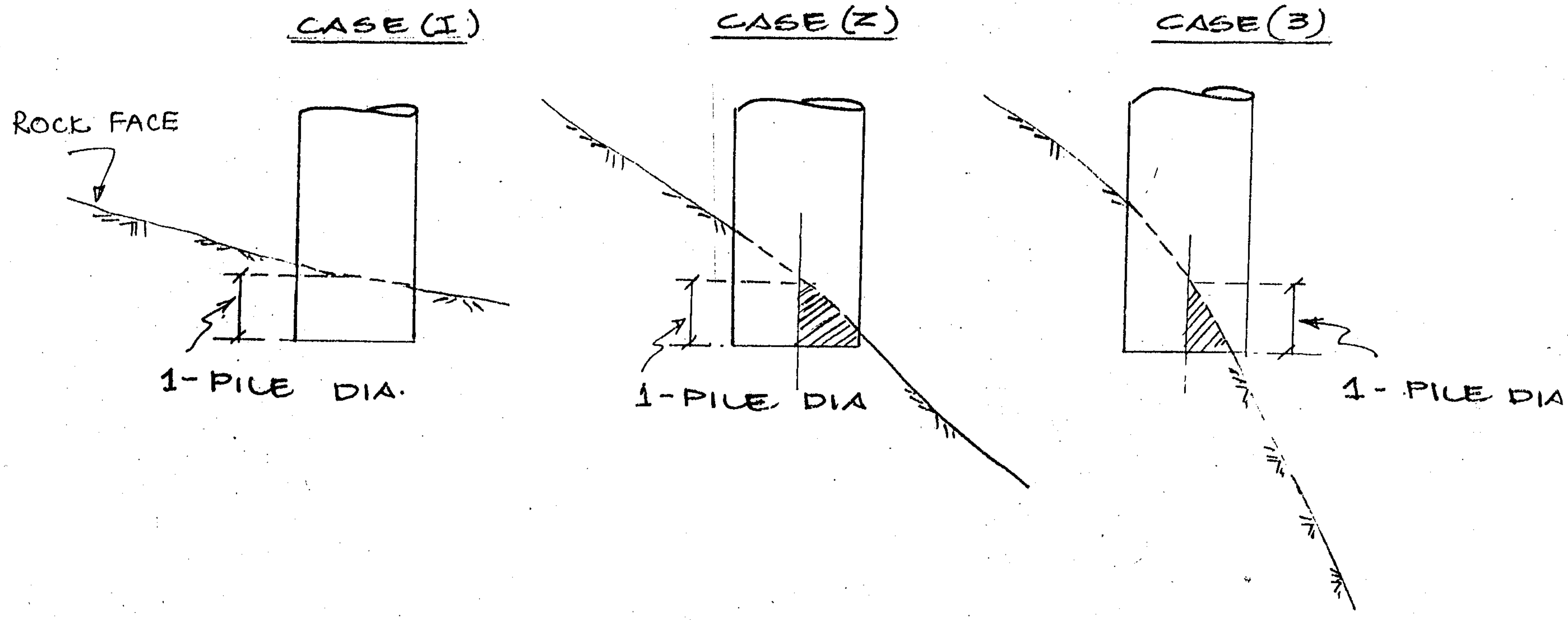
PILE DEPTHS IN BUILDINGS 1A, 1B, 1C - FIGURE 2



PILE DEPTHS IN BUILDINGS 2A, 2B, 2C - FIGURE 3

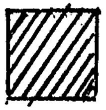


CROSS SECTIONAL PROFILE ACROSS GRID LINE 'ZC'
THROUGH THE BUILDINGS ZA , ZB & ZC
FIGURE (4)



CONDITIONS AT THE PILE TOE

FIGURE (5)

 INSUFFICIENT ROCK SOCKET AT PILE TOE

EXPERIENCES IN WELL FOUNDATIONS

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INTRODUCTION:

In Sri Lanka Well Foundation was used in many years for construction of civil engineering structures such as bridges jetties etc. Generally, well foundations are circular and diameter varies from 1.5m onwards. The main advantage of well foundation is the simple construction technique and it does not require special type of equipment and is capable of carrying large vertical and horizontal loads.

CONSTRUCTION TECHNIQUES:

Mainly two different types of techniques are used in Sri Lanka.

(1) Insitu Casting
(2) Precasting

<u>Insitu Casting</u>

The well stem had to be concreted insitu and once the insitu concrete achieved sufficient strength, sinking of the well section can be done by excavating inside the well. Sinking of the concrete stem may be assisted by adding kantiledges to increase the vertical load. The techniques used for sinking of the well stem will depend on the different type of soil strata.

CONSTRUCTION TECHNIQUE USED IN SINKING WELL FOUNDATION FOR DIFFERENT TYPES OF SOIL CONDITIONS

(1) Average soil, light clay mixed with sand.

Sinking by self weight and may be assisted by adding kantiledges, if water table is high. Partial de-watering can assist the speed of sinking. However, this has to be done with extreme care.

(2) Medium to stiff clay soil.

Excavation below cutting edge by using a heavy clam shell and cleaning around the cutting edge with under-water diverse.

Water/air jetting through blow-holes provided in the cutting edge to cut down the skin friction.

Limited quantity of explosives may be used in extreme cases.

Continuous sinking process will give better results as there is a tendency to increase the skin friction. If the sinking process is stopped for sometimes.

Partial de-watering can assist the sinking process but de-watering to be done with extreme care as there is a danger of soil failure at the bottom.

Sinking process can be accelerated by lowering the ground water level.

(3) Soft organic clay.

Self weight of the cylinder stem may be sufficient to sink the cylinder stem. Extreme care should be taken to maintain the verticality of the cylinder as the organic soft clay tends to offer varying resistance.

(4) Non cohesive medium to dense sand.

Excavation can be done by using a clam shell bucket. However if the ground water table is high, this method of excavation may not be very effective. If the water level is high sand can be pumped out and it is necessary to maintain the water level inside the well to prevent any sand flowing into the well under the cutting edge.

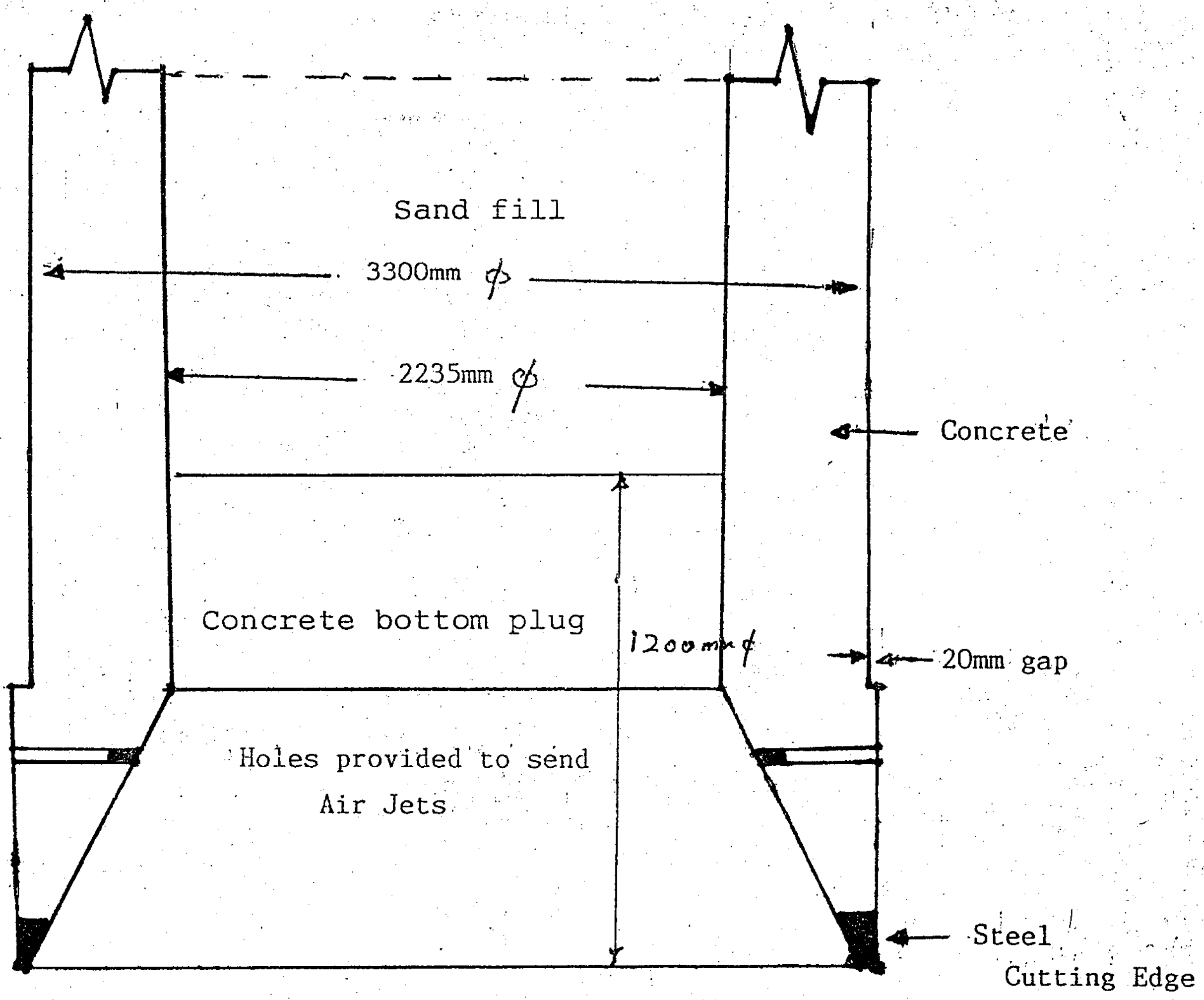
COMMON PROBLEMS ENCOUNTERED IN WELL FOUNDATION

Clearing of obstructions such as bolders, rock, rock fragments and timber logs. This type of obstruction can be removed by sledging, drilling, blasting, or by hammering with a heavy girder. If initial investigation indicates any obstruction within 5-7m below ground level, ground can be excavated to remove obstruction and fill back prior to sinking operation. Very often in bridge construction in coastal areas old steel girder and debris of old bridges are found even at the depth of 10m below ground level.

If the bed rock is dipping it is difficult to anchor the concrete cassion in to the bed rock. When a layer of granular material are present immediately above the bed rock it is difficult to prevent material flowing into the well under the cutting edge. Most common method of avoiding such a situation is to maintain higher water level inside the well. In extreme cases low pressure cement grout can be used to grout outside the cylinder stem at the bottom level of the bed rock to prevent material flowing into the cylinder.

(2) Precasting

In certain cases insitu casting of well stem may not be possible. For example well foundation for a pier in mid stream of a deep river where the formation of a temporary island is not possible. In such a situation precast segments of well stem can be used to form a well. Precast segments can be connected by mechanical means. Usually pre stress bars/wires with a coupling arrangement can be used to connect precast segments, Usually sinking technique does not differ much whether the well stem is precast or insitu cast. Generally for precasting techniques heavy handling equipment will be required to place precast segments.



TYPICAL WELL FOUNDATION USED IN BRIDGE CONSTRUCTION

SKIN FRICTION IN PILES

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1. INTRODUCTION

The load transfer of a pile load from the pile to the ground take place both by skin friction and end bearing. Because of the availability of good sound rock at relatively shallow depths in this country, not much attention has been paid to the carrying capacity of a pile by skin friction. In such a situation, the strength of rock in end bearing is considered so much stronger than the strength of the pile material that the carrying capacity of a pile is determined based on the strength of the pile material. However, in the recent past several instances have occurred where the zone of weathering of the rock was very deep, and economic foundations could be obtained by relying on the skin friction.

Similarly, there are instances where negative skin friction increases the load on piles and could cause the pile to fail, if not accounted for.

This paper deals with some case histories where skin friction was considered important.

2. THEORETICAL COMPUTATION OF SKIN FRICTION LOAD

The ultimate skin friction load of a pile is computed as

$$P_{\text{skin friction}} = f_u \cdot A_s$$

where f_u = ultimate value of skin friction coefficient per unit surface area of pile shaft,

and A_s = surface area of pile shaft.

The skin friction coefficient f_u is estimated from the equation

$$f_u = c_a + K_s \gamma d \tan \delta$$

where c_a = cohesion between soil and pile

δ = angle of friction between pile and soil.

K_s = coefficient of lateral earth pressure on shaft.

d = depth along the pile.

For saturated clays with $\phi_u = 0$,

$$\delta = 0 \quad \text{and} \quad f_u = c_a = \alpha \; c_u$$

$\alpha \approx 1$ for soft N.C. clays, peats, etc.

$\alpha < 1$ for stiff over consolidated clays.

For cohesionless soils with c = 0,

$$c_a \doteq 0 \quad \text{and} \quad f_u = k_s \, \gamma \, d \tan \delta$$

Some typical values given for δ are given in Table 1.

Material of pile		δ / ϕ	
		dry sand	saturated sand
Timber	parallel to grain	0.76	0.85
	perpendicular to grain	0.88	0.89
Concrete	Smooth	0.76	0.80
	rough	0.88	0.90

Table 1.

In the case of cohesive soils, the strength parameters (c,ϕ) are determined from laboratory tests on undisturbed samples taken from boreholes. In the case of cohesionless soils, the soil parameters are often determined from the results of the SPT test.

Empirical correlations have been established between f_u and the SPT value (N).

e.g. The Singapore Standard of the Building Control Division gives

$$f_u = 2\,N \;\; kN/m^2$$

with the proviso that f_u should not exceed 200 kN/m^2.

Another empirical relationship is given by Decourt (1982);

i.e. $f_u = (N/3 + 1) \text{ tonnes/m}^2 = (3.27\,N + 9.81) \text{ kN/m}^2$

with the proviso that all N values greater than 50 should be taken as 50.

3. ESTIMATION OF SKIN FRICTION LOAD FROM RESULTS OF PILE LOAD TEST

Tomlinson(1978) discusses the results of a typical 'load-settlement' curve from a maintained load test. Such a curve is shown in Fig.1.

Initially, due to elastic behaviour of the pile, there is a straight line relationship upto some point'A' on the curve. If the load is released from any point on OA, the pile head will rebound to its original level. When the load is increased further, yielding occurs and a point 'B' is reached when the maximum skin friction on the pile shaft will have been mobilised. The movement required to mobilise the maximum skin friction is quite small, generally 0.3% to 1.0% of the pile diameter. If the load is released at this stage, the pile head will rebound to a point 'C'. The distance OC represents a permanent set or residual settlement. Mobilisation of maximum base resistance requires much higher movements. It is stated that movements of about 10% to 20% of pile diameter would be required in soils. (However, when piles are end bearing on rock, the movements required would be less.)

The best method of estimating the skin friction load is from a pull-out test on a pile. In the absence of such test results, analytical methods have to be used to estimate such loads.

An analytical method of using the results of a load test to obtain separately the ultimate load carried in skin friction and the ultimate load carried by end bearing has been proposed by Chin (1978). In this method, the settlement (Δ) at each loading stage P is plotted against (Δ/P) as shown in Fig. 2. For an end bearing pile, the plot is a single line as shown in Fig. 2(a), and the ultimate carrying capacity is said to be given by its slope.

A pile carrying load both by friction and end bearing, produces two straight lines which intersect as shown in Fig. 2(b). The slope of the upper line given the ultimate skin friction whilst that of the lower part gives the total pile resistance.

For a broken pile, the plot is said to take the shpae as shown in Fig. 2(c).

This method of analysis was used to interpret the results of pile load tests at several sites. In this section, the results are reported of two test piles:

(i) A short pile at which most of the load would have been carried in end bearing.

(ii) A long pile at which both skin friction and end bearing could be expected to carry pile load.

3.1 Short pile carrying load in end bearing.

This case history concerns a site in which the profile of the sub-surface conditions is as shown in Fig. 3a. The site is found to contain some soft alluvial deposits overlying rock which is located at a relatively shallow depth. The piles used were 500 mm diameter bored and cast in-situ RC piles end bearing on rock. The piles were designed to carry a working load of 85 tons based on concrete strength.

One of the piles was test loaded to 1.5 times the working load; i.e. to a maximum load of 127.5 tons at which load the maximum settlement was 4.953 mm and the residual settlement was 1.21 mm.

The graph of vs (Δ/P) is shown in Fig. 3b. Using Chin's method of analysis it is noted that a single straight line has been obtained, and the slope of this straight line gives an ultimate carrying capacity of 685 tons. Malepahirane et al.(1994) have shown that Chin's method tends to over-estimate the load carrying capacity of a pile.

3.2 Long pile carrying load in both skin friction and end bearing.

This case history concerns a site in which the profile of the sub-surface conditions is as shown in Fig. 4a. The site is found to contain a sandy soil followed by highly weathered rock and weathered rock. The piles used were 675 mm diameter bored and cast in-situ RC piles which were terminated in the weathered rock.

The piles were designed to carry a working load of 125 tons based on concrete strength.

One of the piles which was 26 m long was test loaded to 1.5 times the working load; i.e. to maximum of 187.5 tons at which the maximum settlement was 0.44 mm and the residual settlement was 0.04 mm.

The graph of Δ vs. (Δ /P) is shown in Fig. 4 b. Two straight lines have been constructed- the upper line showing a slope of 111 tons, and the lower line showing a slope of 258 tons. Considering the very small settlements that have been measured, one possibility is that the ultimate load carrying capacity of 258 tons estimated is entirely due to skin friction.

4. CASE HISTORIES

This section will cover 2 case histories of

(i) where the piles were terminated in weathered rock without reaching sound rock as specified by the Consultants; and

(ii) where the negative skin friction load in piles could have caused the piles to fail.

4.1 Bored and cast in-situ RC piles for a multi-storey building

This case history concerns an office building where the Consultants had proposed the use of bored and cast in-situ RC piles for the foundations. The piles were of 18", 20" 24" and 27" diameter; i.e. 457 mm, 508 mm, 610 mm and 685 mm diameter respectively. The piles had been designed to be end bearing on sound rock. The specification applied was that the piles should be taken to at least one pile diameter into sound rock, with sound rock being defined as that having a minimum unconfined strength of 50 N/mm^2 and 100% core recovery.

During the site investigation, five boreholes were advanced at the site. Rock coring was carried out in the bed rock, and it was noted that the weathering was generally deep and core recovery very low even at depths beyond 7.0 m into rock at some locations. These results are evident from Table 2.

Borehole No.		BH-1	BH-2	BH-3	BH-4	BH-5
Depth at which coring was begun in bed rock (m)		24.00	23.53	19.30	24.00	19.35
Depth at which coring was terminated in bed rock(m)		31.60	29.63	28.60	29.70	26.50
Rock quality	Core Recovery (%)	51	100	13.3	91	20
	RQD (%)	51	97	0	32	0

Table 2

During the construction of the piles, it was found that boring through the weathered rock was slow and difficult, and even after several metres of drilling in the weathered rock sound rock was not reached.

The results are reported in this section of a 685 mm diameter pile which was to be constructed near BH-1. The sub-surface conditions at BH-1 are shown in Fig.5. Hard rock which had to be penetrated by coring was reached at a depth of 24.0 m.

In the case of the pile borehole, the rate of advancement of the borehole kept decreasing in the zone of moderately weathered rock, and when the borehole had reached a depth of 25.67 m it had slowed down to 0.08 m per hour as indicated in Fig. 6. Not only had sound rock of core recovery of 100% not been reached, the specification required going a further 0.685 m after sound rock had been reached. Considering both the increase in cost involved of advancing the pile borehole further as well as the time delay as a result of further drilling, it was decided to investigate the possibility of terminating the pile at 25.67 m depth. This would be possible if part of the load could be carried by skin friction on the pile shaft.

Calculations given in Appendix 1 show that for assumed values of f_u as indicated in Sec. 2, the working load in skin friction for this pile is approximately 156 tons.

However, it was necessary to verify that the pile could carry this amount of load by skin friction especially since the pile at this site was formed by supporting the borehole with bentonite during the drilling process. In the past, there had been a view that RC bored piles formed by the bentonite method were unable to mobilise much skin friction. But as indicated by Farmer et. al.(1971) there now seems to be a consensus that high values of skin friction could be developed even in RC piles formed by the bentonite process.

In this case, it was decided to check the adequacy of the pile to carry the design working load of 200 tons by carrying out a load test, the results of which were as follows:

For a load of 200 tons, the maximum settlement was 2.13 mm and the residual settlement on unloading was 0.42 mm.

For a load of 250 tons, the maximum settlement was 4.45 mm and the residual settlement on unloading was 1.31 mm.

These results established the adequacy of the pile to carry the design working load.

Another significant feature of these results is that the elastic compression of the pile under 250 tons, (when computed assuming that the full structural load is transferred by end bearing), was 6.13 mm. This exceeds the measured settlement of 4.45 mm. Therefore, it is concluded that some or all of the structural load has been transferred to the soil by skin friction.

The graph of Δ vs (Δ /P) is shown in Fig. 7. This shows a single straight line graph having a slope of 519 tons. Similar to the result of Sec. 3.2, the possibility is that the entire load is carried by skin friction.

4.2 Bored and cast in-situ RC piles for a swimming pool

This case history concerns a swimming pool whose floor slab was supported by several 375 mm diameter bored and cast in-situ RC piles designed to be end bearing on rock.

The profile of the sub-surface conditions is shown in Fig. 8. The original ground level was about 3.4 m below the existing ground level and the site consisted of a low lying marshy area. The site had then been filled by about 1.5 m using a compacted lateritic fill. Piling for the pool had been done from this level. Subsequently, the pool was constructed, and the ground outside the pool was raised to the existing level.

Distress to the pool slab was first observed during the initial filling of the pool. Several years later some parts of the pool had shown settlements as large as 300 mm. These settlements are so large, the only inference possible is that some of the piles have failed. Thus some of the load from the failed piles will be re-allocated to the adjacent piles (thereby possibly causing even these to fail) and the balance load will be transferred directly to the ground. Hence consolidation of the organic clay and peat will take place both due to the fill load and due to the load transferred directly to the ground from the base of the pool.

Checking of the design calculations showed that the piles had been designed for a working load of 35 tons of which only 1.75 tons had been allowed for negative skin friction.

Negative skin friction at this site arises out of

(i) the dragdown caused by the consolidating layer of organic clay and peat.

(ii) the dragdown caused by the settlement of all the soils above this layer as they also settle as a result of the settling clay.

Detailed calculations done according to Sec. 2 showed that

(a) the dragdown caused by the consolidating layer of organic clay and peat could vary between 13.8 tonf and 92 tonf depending on the strength characteristics of this layer which were found to be in the range of $c_u = 0.15$ kgf/cm^2 and 1.0 kgf/cm^2.

(b) the dragdown caused by the settlement of the soil above the consolidating layer was about 23.5 tonf.

Therefore, one strong possibility for the failure of the piles was the incorrect estimation of the negative skin friction load.

5. ACKNOWLEDGEMENTS

The Author wishes to thank the many Consultants and Piling Contractors who made available their piling records for study.

Part of this work was carried out by Miss. M.D. Malepathirana, Mr. D.L. Amarasinghe and Mr. J. Bungiriya who did the work as an undergraduate project under the Author's supervision.

Thanks are due to Mr. V. Somaratne for typing the paper and Mrs. L.I. Wickramarachchi for undertaking the tracings for the figures.

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2. Decourt, L.(1982): "Prediction of the bearing capacity of piles based exclusively on N values of the SPT". Proc. of 2nd European Symposium on Penetration Testing.

3. Farmer, I.W., Buckley, P.J.C. and Sliwinski, Z.(1971); "The effect of bentonite on the skin friction of cast in place piles". Paper published in ICE Publication "Behaviour of Piles".

4. Malepathirane, M.D., Amarasinghe, D.L. and Bungiriya, J.(1994): "Evaluation of load test data of pile foundations", Project report of the University of Moratuwa.

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Estimation of Skin Friction Load in Case History given in Sec. 4.1

Pile diameter = 685 mm, Pile length = 25.67 m

The sub-surface conditions near the test pile are given in Fig. 3.

Depth (m)	N	f_u (kN/m^2)	Ultimate load carried by skin friction (kN)
1.5 - 8.0	5	10	140
8.0 - 24.0	17	34	1170
24.0 - 25.67	> 100	200	720
Total			2030 kN

Assuming a factor of safety of 1.3,

Working load in skin friction = 1560 kN - 156 tonf.

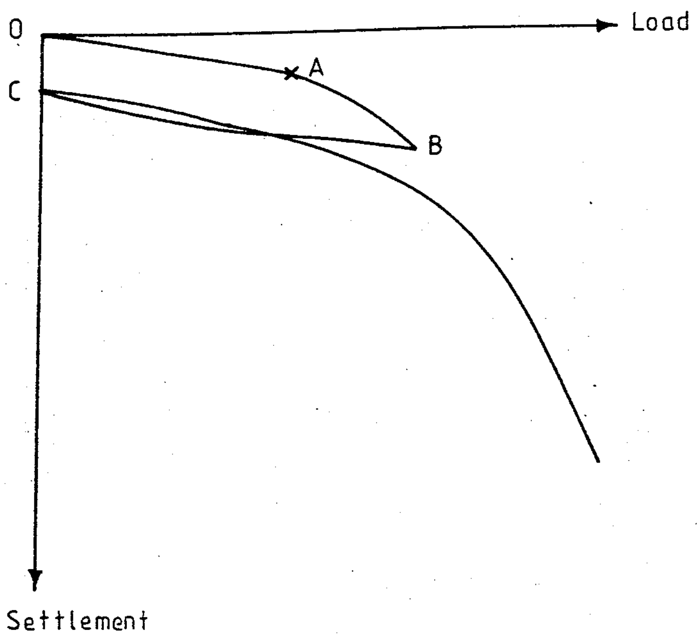
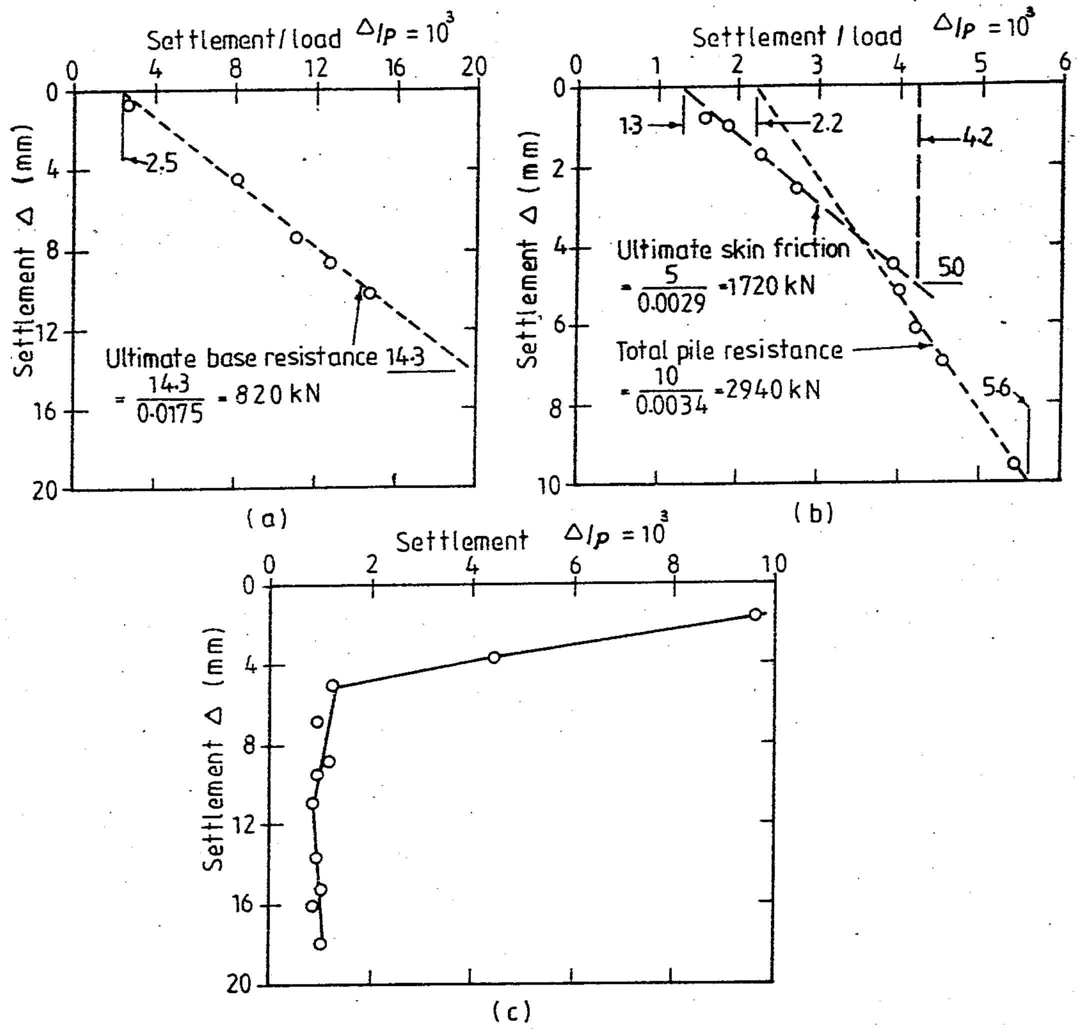


Fig. 1 - Typical Load - Settlement curve from Pile Load Test



(a) End bearing pile

(b) Friction and end bearing pile

(c) Broken pile

Fig. 2 - Analysis of Load - Settlement curves
(Ref. Chin, 1978)

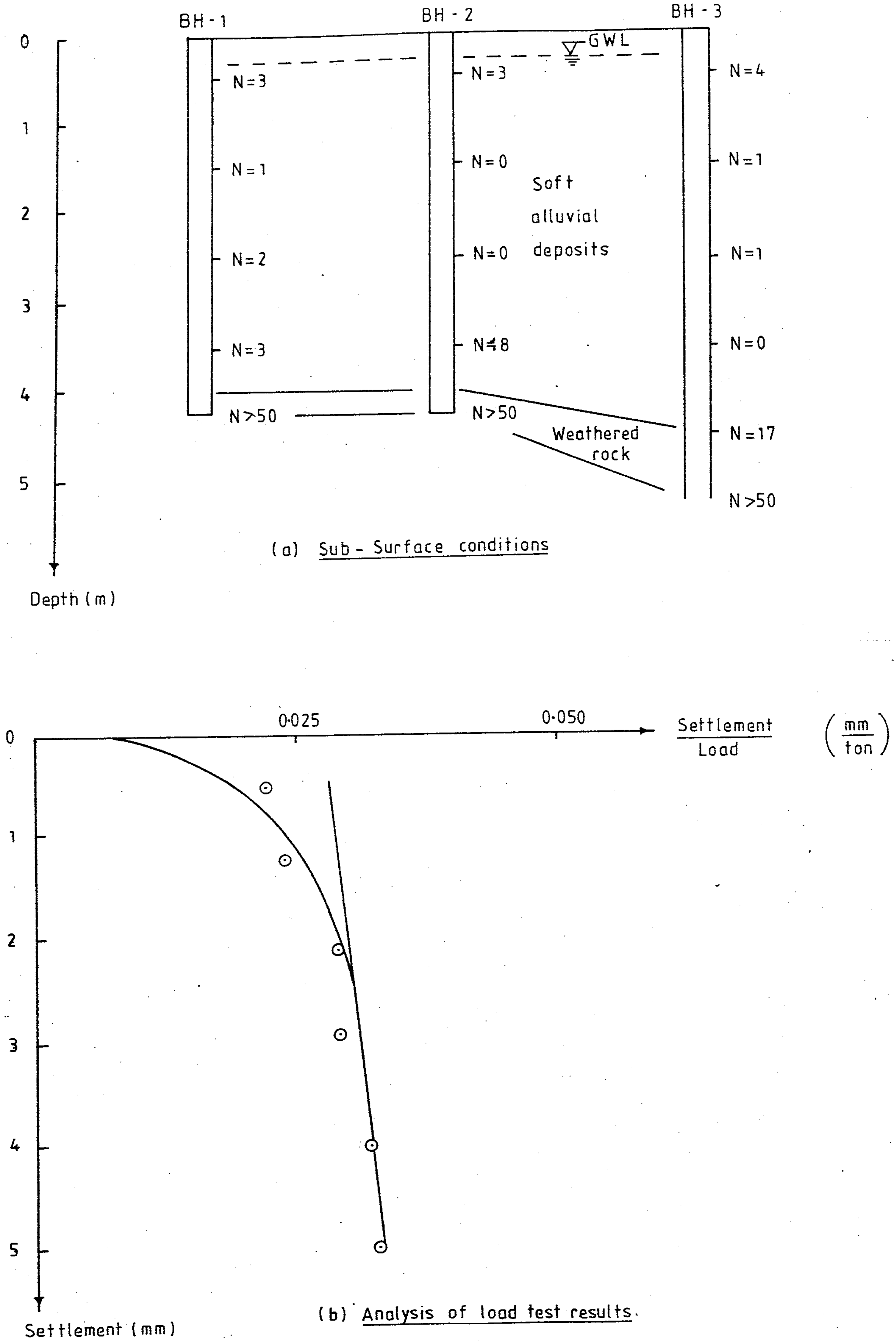
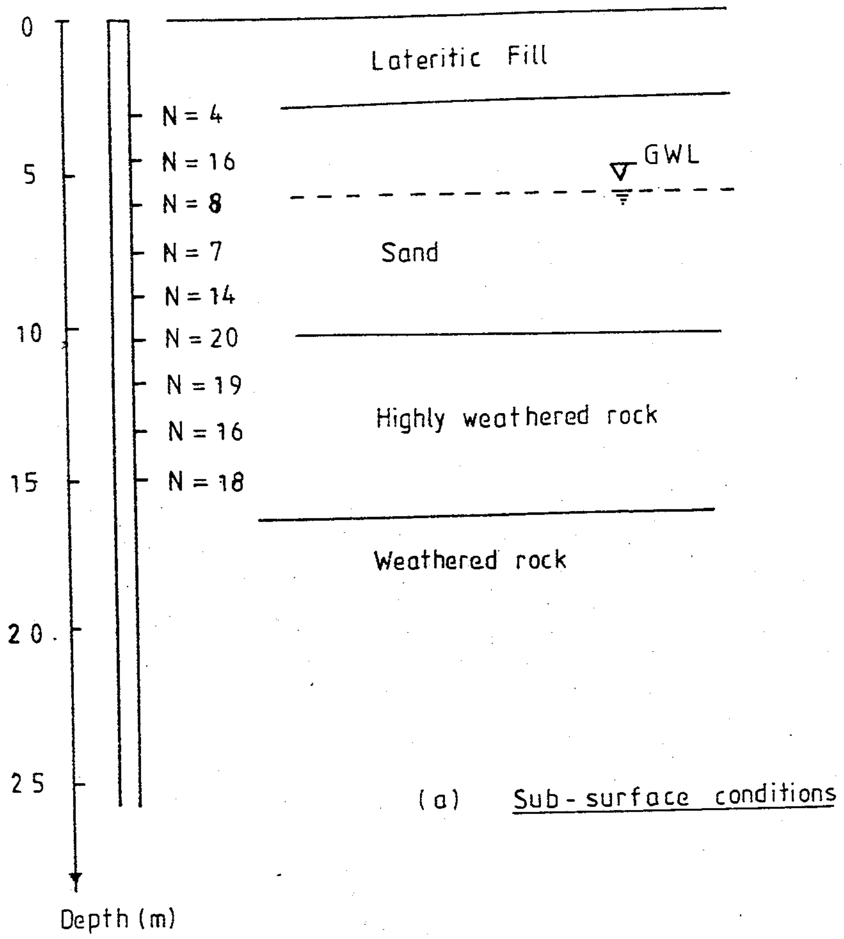
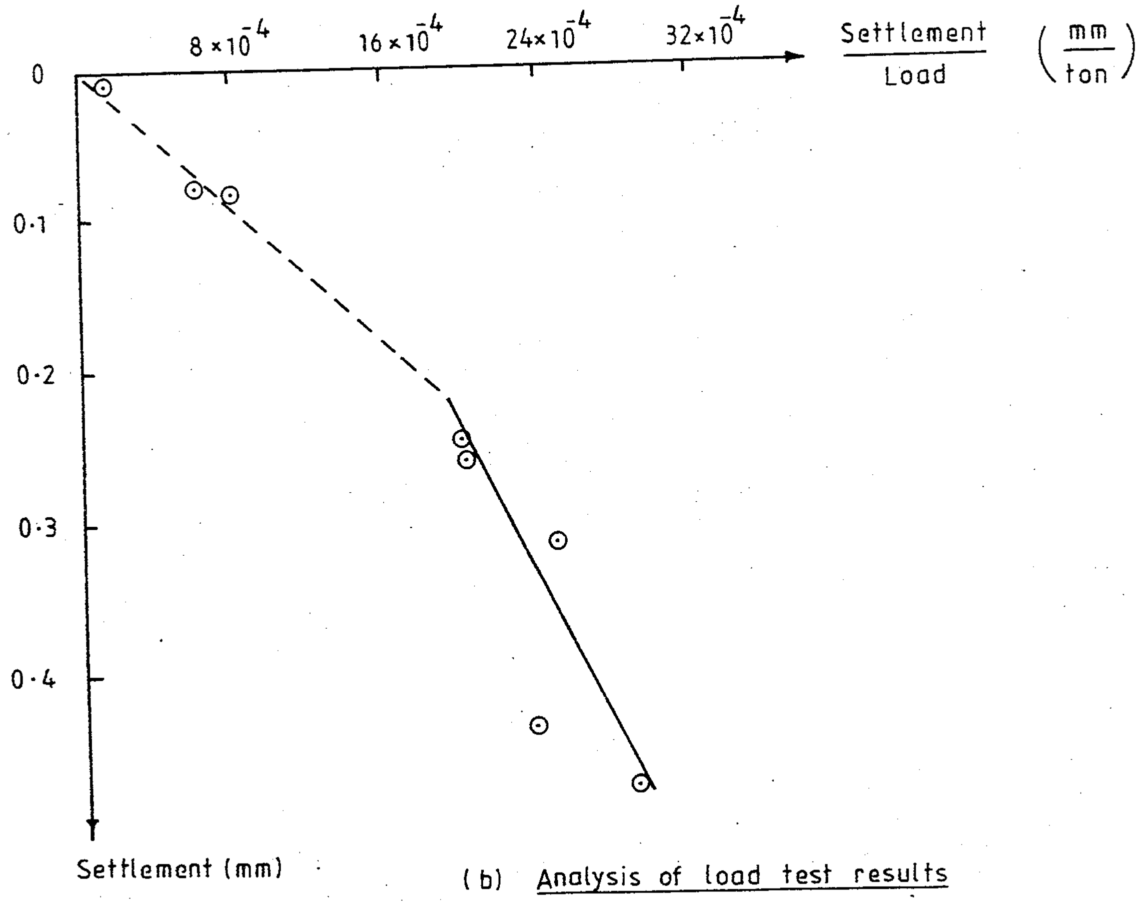


Fig. 3 - Case History referred to in sec. 3·1



(a) Sub-surface conditions



(b) Analysis of load test results

Fig. 4 - Case History referred to in sec. 3·2

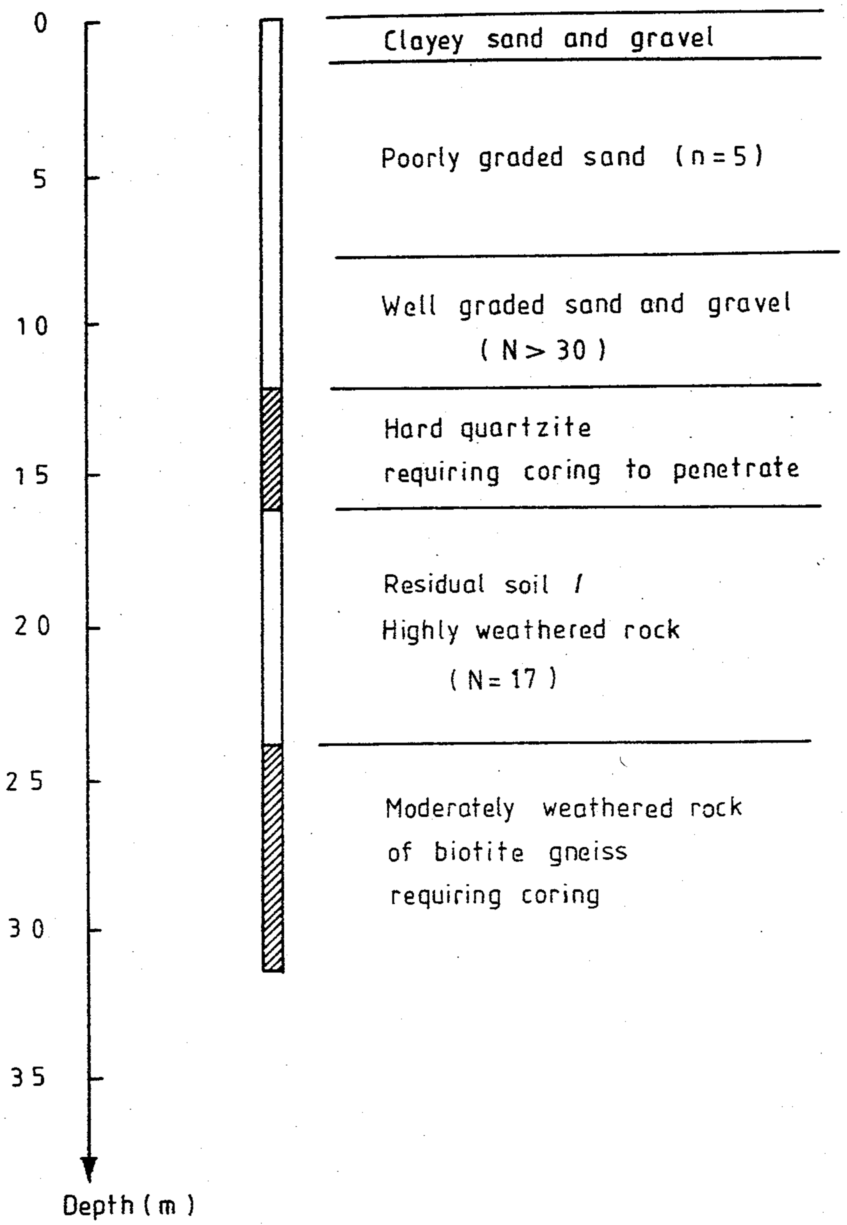


Fig. 5 - Sub-surface conditions near Test Pile
(Case History of sec. 4·1)

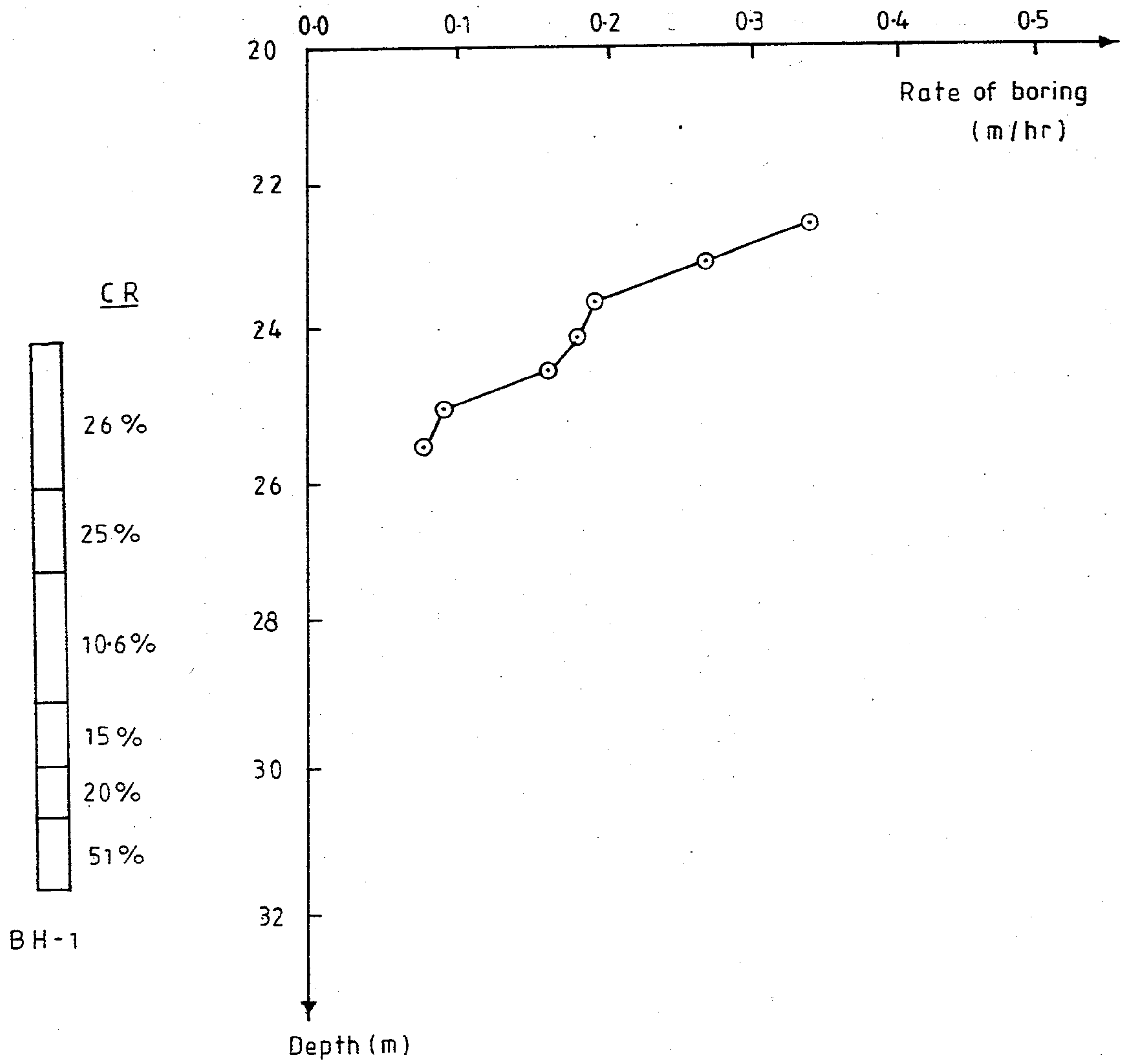


Fig. 6 - Comparison of Rate of Boring of 685 mm dia. pile with rock coring in adjacent borehole BH-1 (Case History of Sec. 4·1)

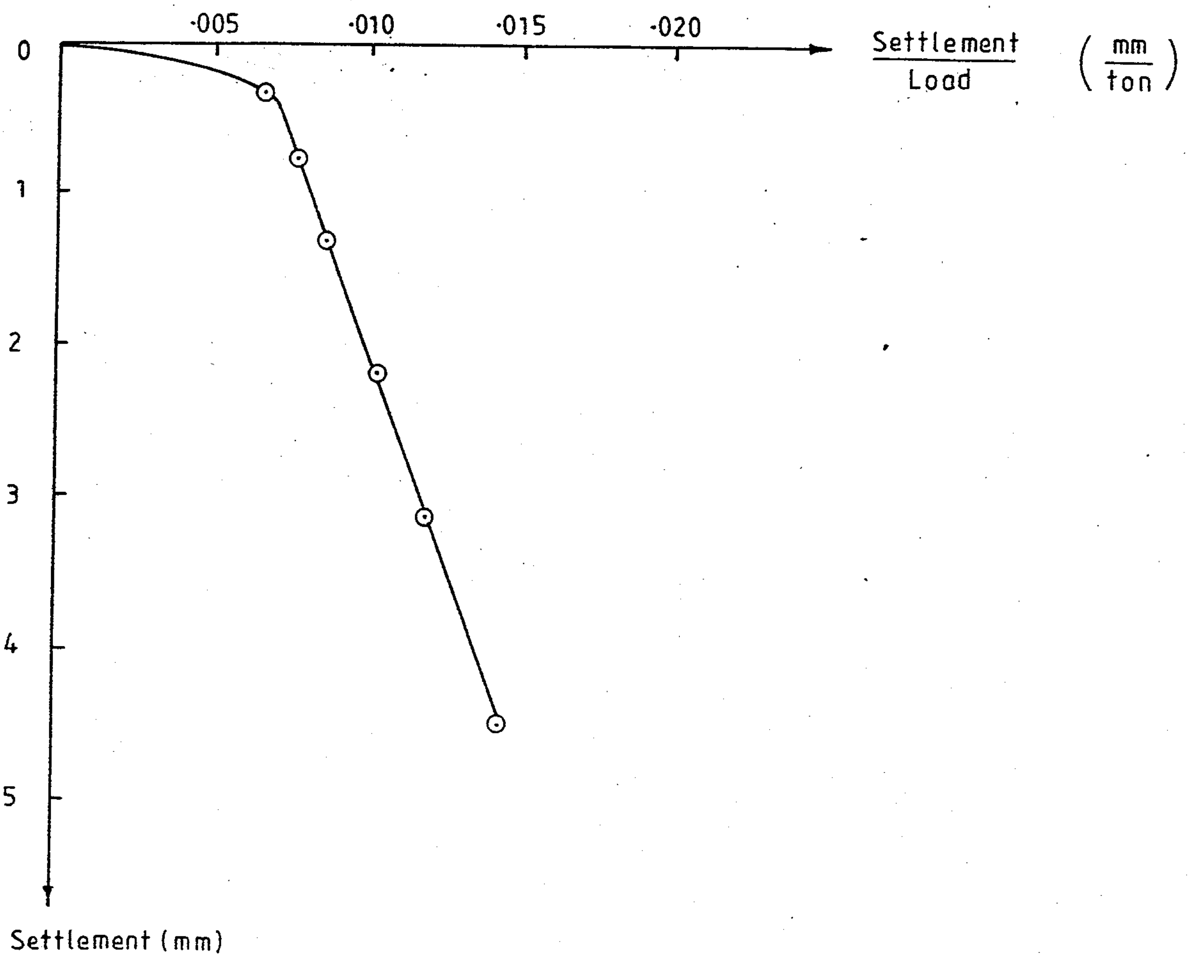


Fig. 7 - Analysis of load test results
(Case History of sec. 4·1)

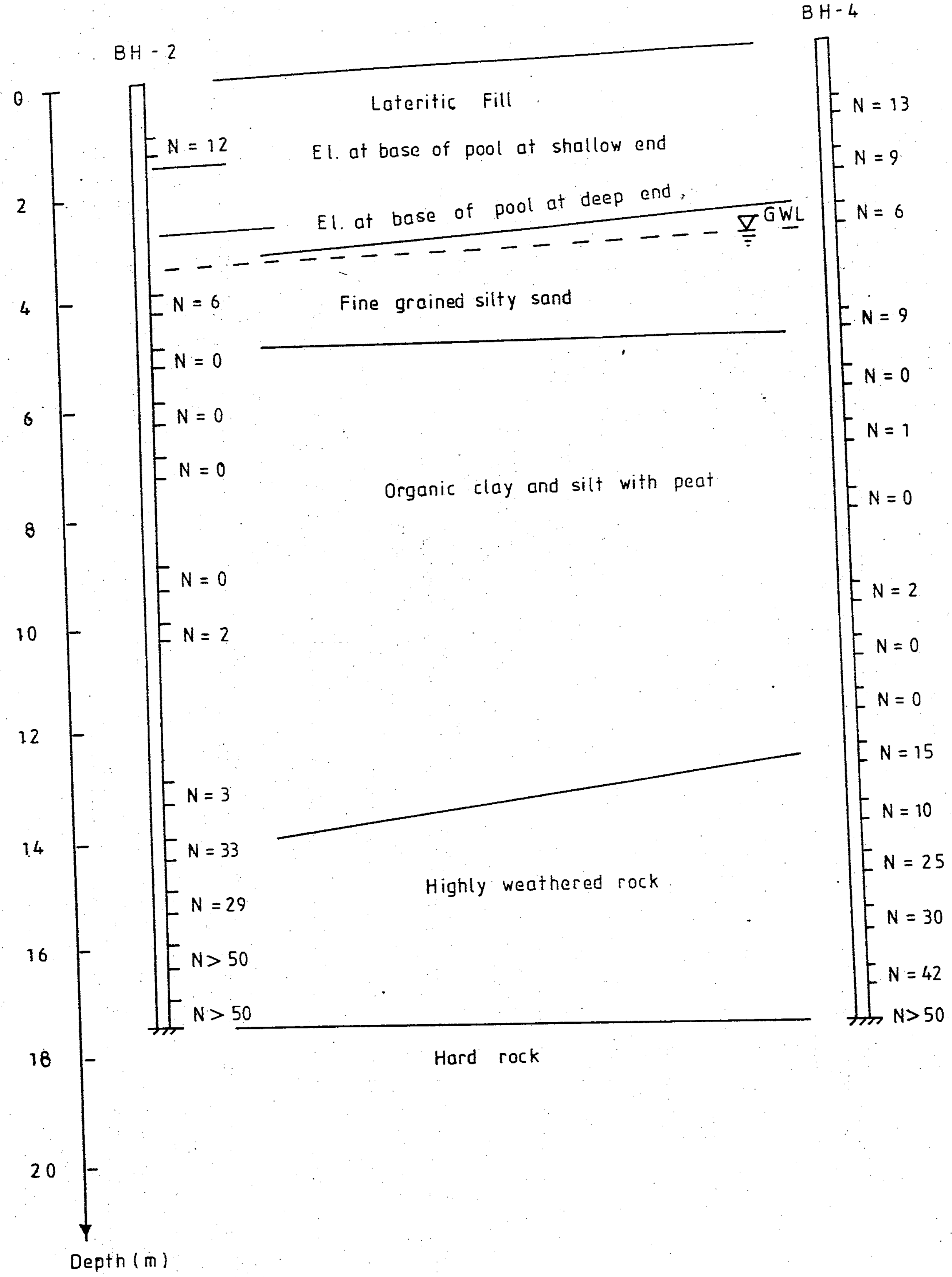


Fig. 8 - Sub-surface conditions
(Case History of Sec. 4·2)

IDENTIFICATION OF TYPE OF PILE FAILURE FROM RESULTS OF LOAD-SETTLEMENT CURVES

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1. INTRODUCTION

Whilst pile failures are detrimental to a structure and are not wished for by anyone, yet when they do take place they provide important information from which many lessons can be learnt.

This paper discusses the results of pile 'failures' which were identified at the time of pile load tests. By this is meant that the pile could not carry the load for which it was designed. However, since the superstructure had yet to be constructed, correction action in terms of a re-design was possible.

To identify possible causes of failure, a thorough understanding of factors such as the structural design, the method of construction along with the control measures adopted, the ground conditions, etc, are required. There are so many uncertainties in the design and construction of piles that good engineering practice requires 2 types of load tests to be done.

(i) Preliminary pile load tests in which the piles are tested to its ultimate bearing capacity or at least 2 times the working load of the pile. These preliminary piles are installed at the commencement of the main piling wok, and after testing are not to be used as working piles.

(ii) Proof load tests on some selected piles which will normally be loaded upto 1.5 times the working load. These piles will later be used for supporting the superstructure.

Unfortunately, because of time constraints put on a project by the Client, and sometimes because of the additional expenditure involved, in most piling projects carried out in this country, no preliminary pile load tests are done. Therefore,

(a) there are instances of uneconomical design; and

(b) there are instances of pile failure, as discussed above.

Based on his experience, Tomlison(1978) has indicated how the load-settlement curves from a pile load test can be used to interpret the mode of failure of a pile. He has identified different types of load-settlement curves which are given in Fig. 1, and the cause of failure in each case is indicated against each of the figures. It should be noted, that whilst for failure of a given type, the load-settlement curve is as shown, the converse is not exactly correct. i.e. for a given shape of load-settlement curve, the mode of failure specified is only one of the possibilities.

A similar application is adopted in this paper to identify the type of pile failure from the results of the load-settlement curve.

2. CASE HISTORIES

2.1 Bored and cast in-situ RC piles for blocks of flats in Colombo

This case history concerns a site where bored and cast in-situ RC piles were to be used to support the superstructure. The piles were to be end bearing on rock. The piles which were 528 mm diameter were designed for a working load of 100 tons based on the concrete strength.

The site was over 1 acre in extent, but only 2 boreholes had been advanced during the site investigation. The sub-surface conditions at these 2 locations are as shown in Fig. 2. The depth to rock at these 2 boreholes were 10.95 m and 11.75 m. However, when actually constructing the piles, it was found that the depth to rock was quite variable. The depth to rock at pile locations in two of the blocks are shown against their positions in Fig. 3. This shows that some of the piles were founded as deep as 25.0 m, and that the rock slope is dipping very steeply in this region. (Contours of equal depth to bed rock have been marked in Fig.3.). It was observed that in the areas where the rock was very deep, the piles passed through a large thickness of organic clay and peat.

Proof load tests were carried out on several piles according to the "Maintained Load Test" procedure in which the pile was loaded in two cycles. In the first cycle, the pile was loaded to 1.0 times the working load and unloaded; whilst in the second cycle the pile load was increased to 1.5 times the working load before unloading. Adequacy of a pile to carry a design working load was provided by a specification as follows:

For 1.5 times working load,

(i) maximum settlement < 25 mm
and (ii) residual settlement on unloading < 6 mm.

The results of the load test, on 3 piles which failed to meet the above specification will be discussed in this section. The results of these 3 piles are given in Table 1.

Pile No.	Length(m)	Settlement (mm)			
		1st Cycle		2nd Cycle	
		Total	Residual	Total	Residual
A5 - 2b	15.1	19.80	12.39	24.42	16.80
A5 - 2e	20.0	70.05	63.10	106.60*	99.81*
A3 - 2e	16.4	32.32	27.69	57.63	52.20

* Maximum test load was only 1.25 times working load.

Table 1

Pile A5-2b - The load-settlement curve for Pile A5-2b is given in Fig.4a. The significant feature of the load test results on this pile is that although the pile has shown a considerable residual settlement after the first cycle of load, during the second cycle the pile behaves as a sound pile.

i.e. considering only the 2nd cycle, for 1.5 times working load,

maximum settlement = 24.42 - 12.39 = 12.03 mm
residual settlement = 16.80 - 12.39 = 4.41 mm.

(It is here assumed that the residual settlement during the 1st cycle has already occurred.)

It is necessary to identify the cause for such a large residual settlement during the first load cycle. One of the possibilities which could explain this situation is that it could arise if the bottom of the borehole has not been properly cleaned prior to concreting the pile. If loose soil deposits are found below the pile at the bottom of the borehole, then on the first loading the pile could be pushed down by the test load past the sedimentated soil to the new bearing stratum of hard rock.

Cleaning the bottom of a borehole prior to concreting is a control measure that has to be strictly enforced at site. This is usually done using a specially designed bucket or by circulating bentonite slurry under pressure through a tricone bit at the bottom of the borehole. Sometimes it happens that after the bottom of the borehole is cleaned and the reinforcement cage lowered into the borehole, work at the site comes to a standstill until the ready mix concrete arrives at the site. The Author has noticed sometimes several hours of such a delay, and during this period collapse of a part of the borehole is possible especially if the sides are not supported by steel casing. Such a situation should be avoided, and the bottom of the borehole should always be cleaned just prior to placing the concrete.

Pile A5-2e - The load-settlement curve for Pile A5-2e is given in Fig. 4b. The significant feature of the load test results on this pile is that from a test load as small as 25 tons, the pile starts showing disproportionately large settlements which increase linearly with load. A settlement of over 25 mm was recorded under 50 tons of load.

Referring to Fig.3 which shows the bed rock contours, it is noted that the pile is end bearing on rock where it is highly dipping. The angle of dip of the rock at this location was 59.2°. Therefore, a probable cause of failure for this pile is that it is slipping on the rock. Whitaker(1970) states that if the slope of rock surface exceeds about 45°, then seating of the piles is difficult.

The site investigation of advancing only 2 boreholes in an area of over 1 acre is clearly inadequate to show the variations in sub-surface conditions.

However, even with a better site investigation program, there is a limit to the number of boreholes that could be advanced. Therefore, the results of the site investigation should always be supplemented with the additional information being obtained during the boring at each pile location during construction.

The lesson to be learnt from this result is that when piles are found to be end bearing on highly dipping rock,

(i) a proof load test should be carried out on one such pile to determine its carrying capacity. This not only provides information on whether the pile is well keyed to the rock, but also it provides information on the reduced bearing capacity associated with sloping rock.

(ii) other piles in the vicinity to be constructed later should be well keyed into the rock.

Pile A3 - 2e - The load-settlement curve for Pile A3 - 2e is given in Fig. 4c.

The significant feature of the load test results on this pile is that it has a shape similar to Fig. 1(a) which corresponds to ground failure conditions.

The working load of this pile based on concrete strength has been estimated to be 100 tons. Since the working load of a pile has a factor of safety of 2 or more on the ultimate load based on concrete strength, the failure load of the pile based on concrete strength should be more than 200 tons.

This pile was loaded to a maximum of 150 tons.

The failure load of the pile was determined according to the ASCE method given by Ackley and Sanders(1980), and the British Method given by Tomlinson(1978).

These methods have been summarised by Tennekoon(1994 a). The failure load for Pile A3 - 2e was obtained as 80.6 tons by the ASCE method and 75 tons by the British Method. Such a load is very much smaller than the ultimate load computed based on concrete strength.

There has been some discussion on whether the load carrying capacity of a pile based on concrete strength should be reduced by consideration of buckling, and reference is made to the recommendation in Reynolds and Steedman(1974). Whilst the buckling of slender piles is possible, the Author is not aware of any literature where piles of 300 mm diameter and higher have been considered as slender piles. Further, at this site the adjacent pile A4-2e, which is 19.0 m long and therefore longer than pile A3-2e, has been test loaded successfully to 1.5 times working load. (For pile A4-2e, under 1.5 times working load, the maximum settlement was 6.40 mm and the residual settlement was 1.60 mm.) Therefore, it is concluded that the pile has failed as a result of the failure of the ground.

At this site, the design specification was that the pile should be end bearing on hard rock. Sound, hard rock usually has undergone a weathering process so that above it there are various grades of rock depending on the amount of weathering, and above the weathered rock is a residual soil which has also been produced by the weathering process. In such a profile, the quality of the rock generally improves with larger depth, are therefore, the founding level of a pile is important.

The SPT value (N) can be used as a measure for the amount of weathering.

An empirical formula relating N to the ultimate bearing capacity of rock (q_{ult}) is

$$q_{ult} = 40 \text{ N kN/m}^2$$

For a hard rock stratum on which the SPT hammer rebounds without any penetration, a N value of 250 could be taken as a conservative estimate giving an ultimate bearing capacity of 10,000 kN/m^2 (10 N/mm^2). The corresponding failure load for a 528 mm diameter pile is 2189 kN; i.e. approx. 218 tons.

The failure load of pile A3-2e is less than this value. This indicates that the bearing stratum of this pile is weaker than designed, and the pile should have been taken down to a greater depth.

The depth of termination of a borehole for a pile is clearly a decision which requires engineering judgement, and should usually be made by a combination of the visual inspection of the rock chips being recovered from the bottom of the borehole, and the rate of advancement of the borehole. The rate of development of a borehole is another parameter related to the degree of weathering.

2.2 Driven pre-cast concrete piles to support a pipeline in Kotte

This case history concerns a project where driven pre-cast concrete piles were used to support a pipeline over a marshy area.

The piles had a cross-section 300 mm x 300 mm and length 10 m. They were designed for a working load of 30 tons based on the concrete strength.

The sub-surface conditions at one of the locations where a proof load test was done is shown in Fig. 5a.

The piles were driven by a hammer weighing 1.2 tonnes falling through a height of 150 cm. The final set measured for the pile under study was 0.60 cm per blow.

In the load test that was carried out, the pile was first loaded upto 30 tons and unloaded. The maximum settlement was 8.1 mm and the residual settlement 7.6 mm.

When the pile was re-loaded, it was found that the pile began to settle very fast at a load of 39 tons; i.e. the ultimate carrying capacity of the pile was only 39 tons.

The load-settlement curve for the pile is shown in Fig. 5b.

From the above results it is concluded that similar to Pile A3-2e, the carrying capacity of the pile is governed not by the strength of concrete but rather by the strength of the ground.

(It is worth recording that in another similar pile which was driven to a set of 0.24 cm per blow, a maximum load of 40 tons was applied with the pile showing only small settlement. These were (i) maximum settlement of 6.32 mm, and (ii) residual settlement of 2.51 mm.)

This also shows that the carrying capacity of the pile is governed by the strength of the ground. In this case, the piles were re-designed for a working load of 19.5 tons which gave a factor of safety of 2 against the ultimate carrying capacity.

2.3 Driven timber piles for a 2-storey building in Ratmalana.

This case history concerns a site whose sub-surface conditions are as indicated in Fig. 6a. This shows that the site has two highly compressible layers of organic clay and peat separated by a sand stratum.

Although the loads from a 2-storey building are relatively small, after consideration of the Client's requirements the Consultants decided to support the building on timber piles passing through the first layer of organic clay and peat, and end bearing on the sand stratum. 170.3 mm diameter Hora timber piles which were 3.0 m long were selected for this purpose. The working load of a pile was estimated at 140 kN based on the timber strength.

The piles were driven using a drop hammer of 1000 kg falling through 3 ft (914.4 mm). The final set measured was 3.0 cm per blow.

In the load test that was carried out, the pile was first loaded upto 140 kN and unloaded. The maximum settlement was 1.6 mm and the residual settlement was 0.05 mm.
The pile was then re-loaded to 210 kN and unloaded. For this cycle, the maximum settlement was 15.2 mm and the residual settlement 12.1 mm. The failure load was determined as 180 kN by the ASCE method, and 170 kN by the British method.

The ultimate carrying capacity of the pile was determined as 240 kN according to the analysis given by Chin(1978) and summarised by Tennekoon(1994 b).

The load-settlement curve for the pile is shown in Fig. 6b.

From the above results it is concluded that in this case also the carrying capacity of the pile is governed not by the strength of the timber but rather by the strength of the ground. In this case, the piles were re-designed for a working load of 90 kN which gave a factor of safety of 2 on the failure load by the ASCE method, and of 2.6 against the estimated ultimate carrying capacity.

(In this case, a further computation was necessary to check the compression of the second layer of organic clay and peat as a result of the load transfer by the piles.)

3. ACKNOWLEDGEMENTS

The Author wishes to thank the many Consultants and Piling Contractors who made available their piling records for study.

Much of this work was carried out by Miss. M. D. Malepathirana, Mr. D. L. Amarasinghe and Mr. J. Bungiriya who did the work as an undergraduate project under the Author's supervision.

Thanks are due to Mr. V. Somaratne for typing the paper and Mrs. L. I. Wicramarachchi for undertaking the tracings for the figures.

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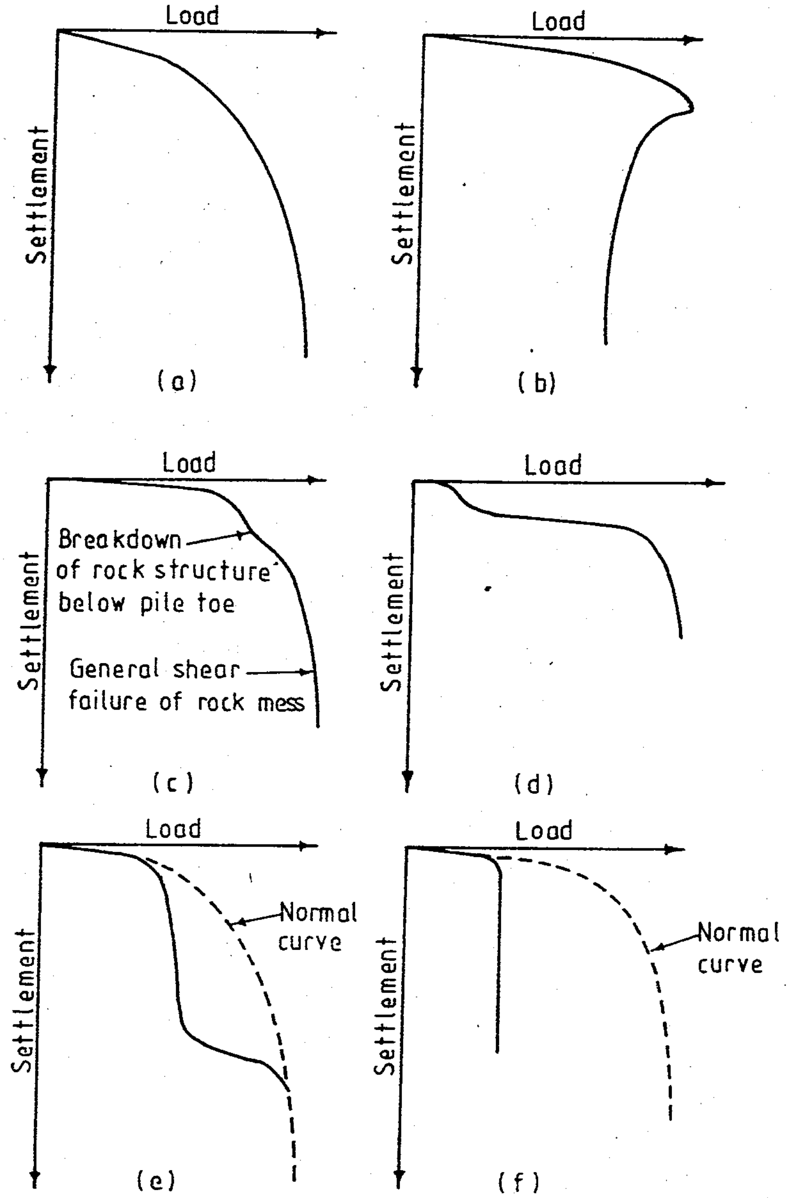


(a) Friction pile in soft clay or loose sand

(b) Friction pile in stiff clay

(c) Pile end bearing on weak porous rock

(d) Pile pushed down to new bearing on rock by Test Load

(e) Gap in pile shaft closed by Test Load

(f) Weak concrete in pile sheared completely by Test Load

Fig.1 - Typical load-settlement curves for pile load tests

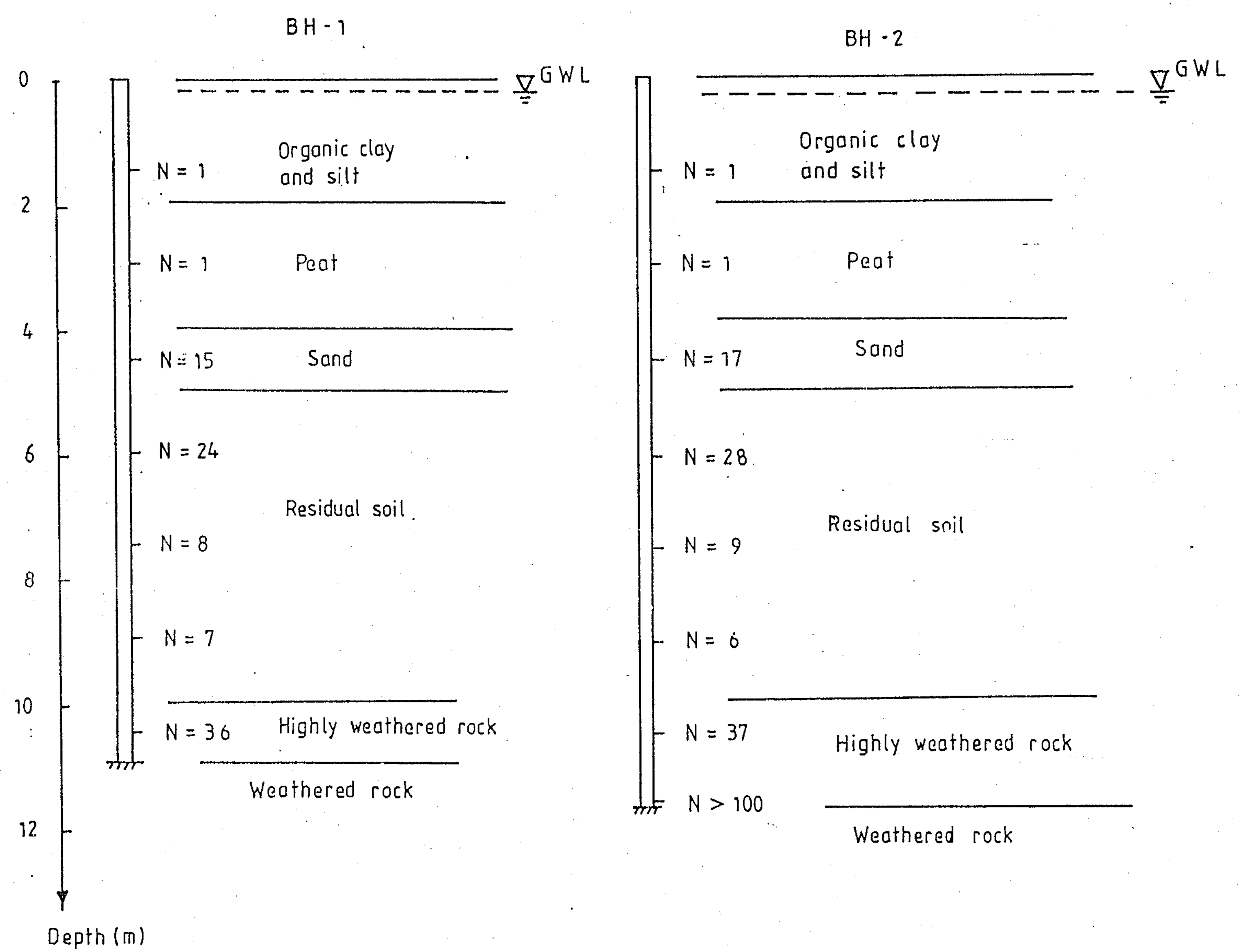
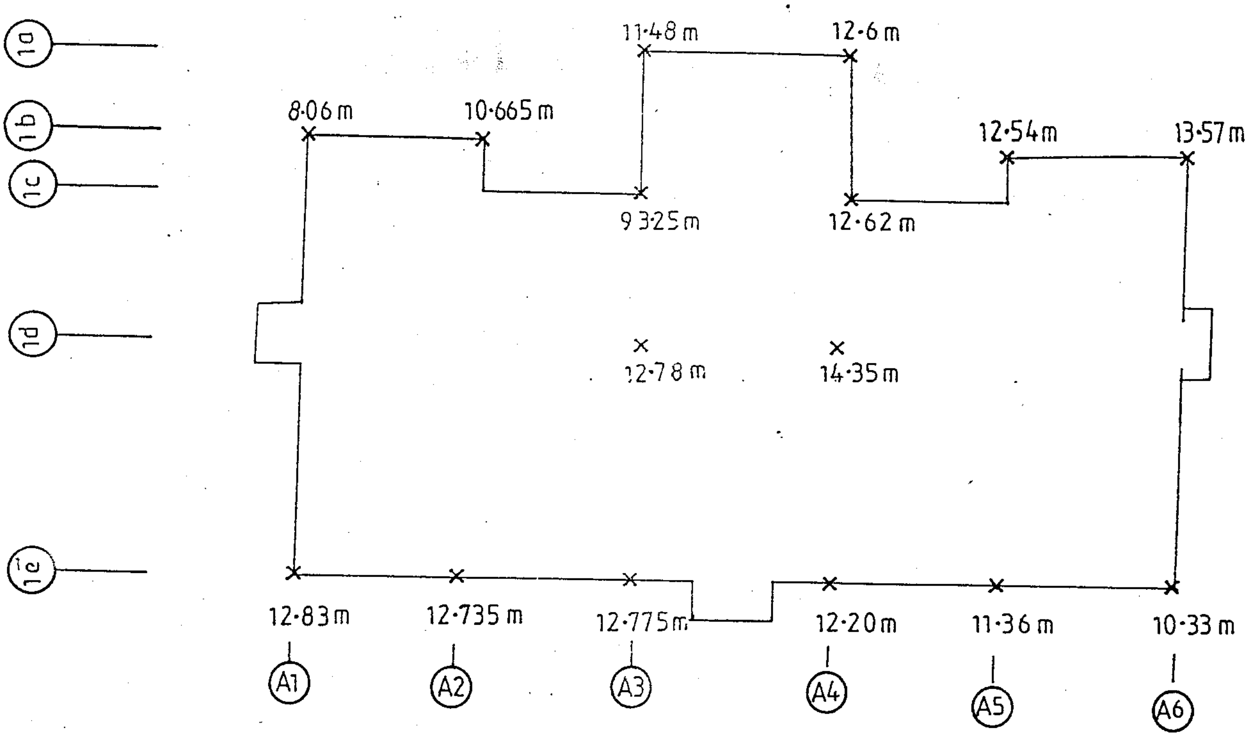
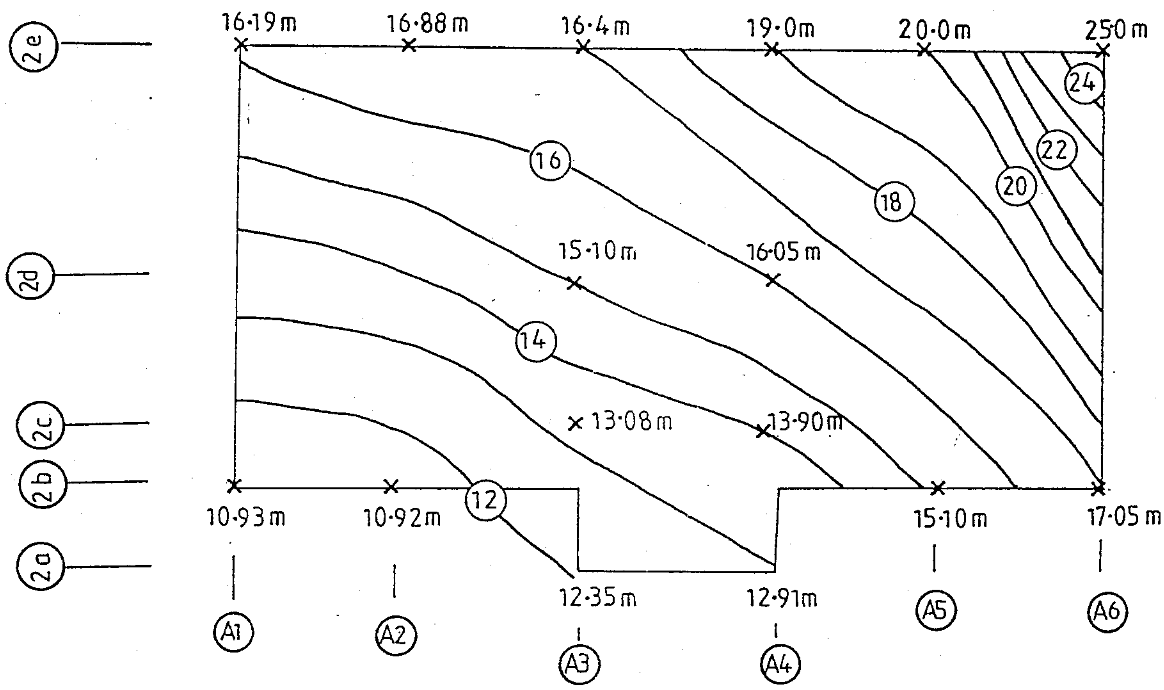


Fig. 2 - Sub-surface conditions

(Site for bored and cast in-situ RC piles)



Depth of Piles in Block 1A



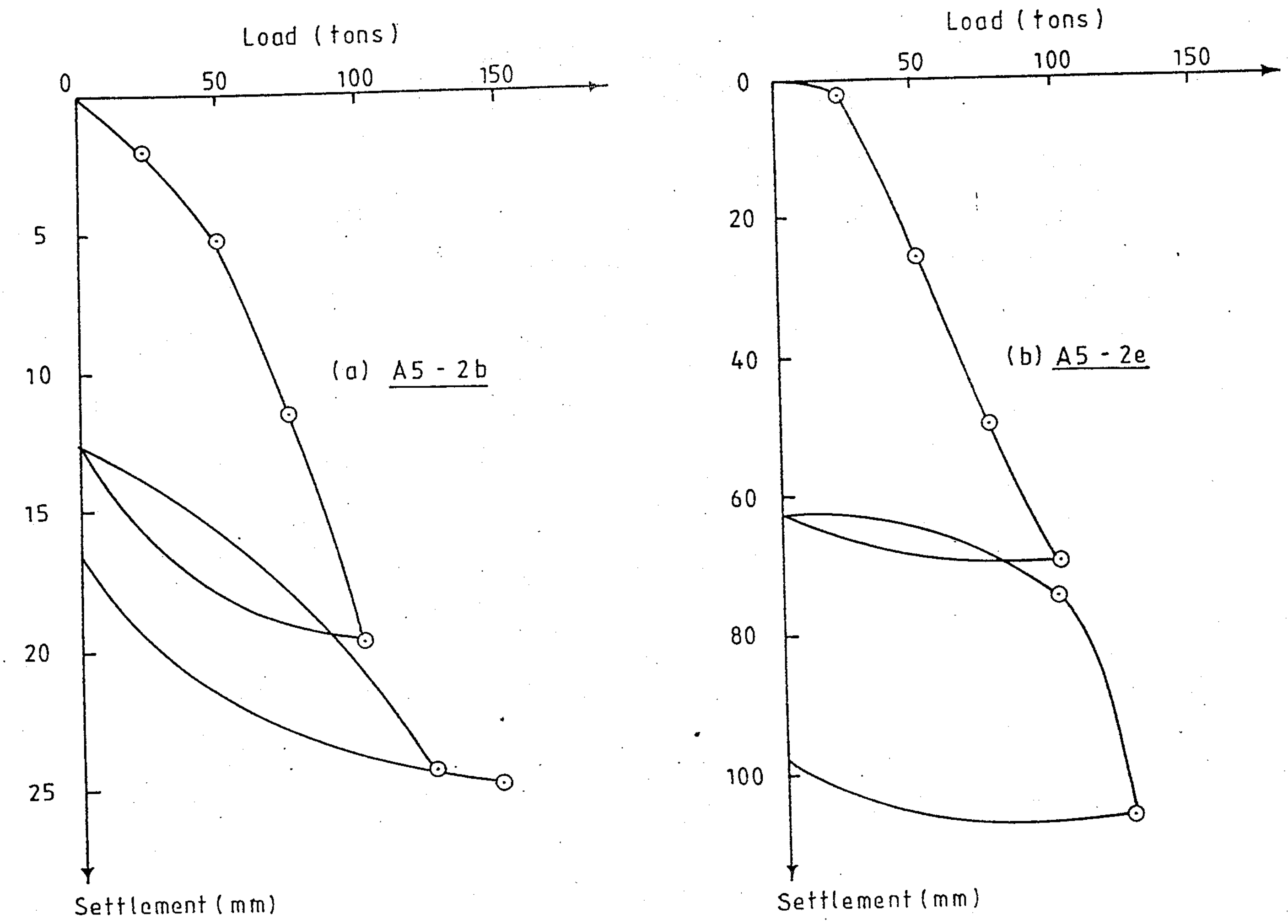
Depth of Piles in Block 2A

× Pile location

Contours of equal depth to bed rock

Fig. 3 - Depth to rock at pile locations

(Site for bored and cast in-situ RC piles)



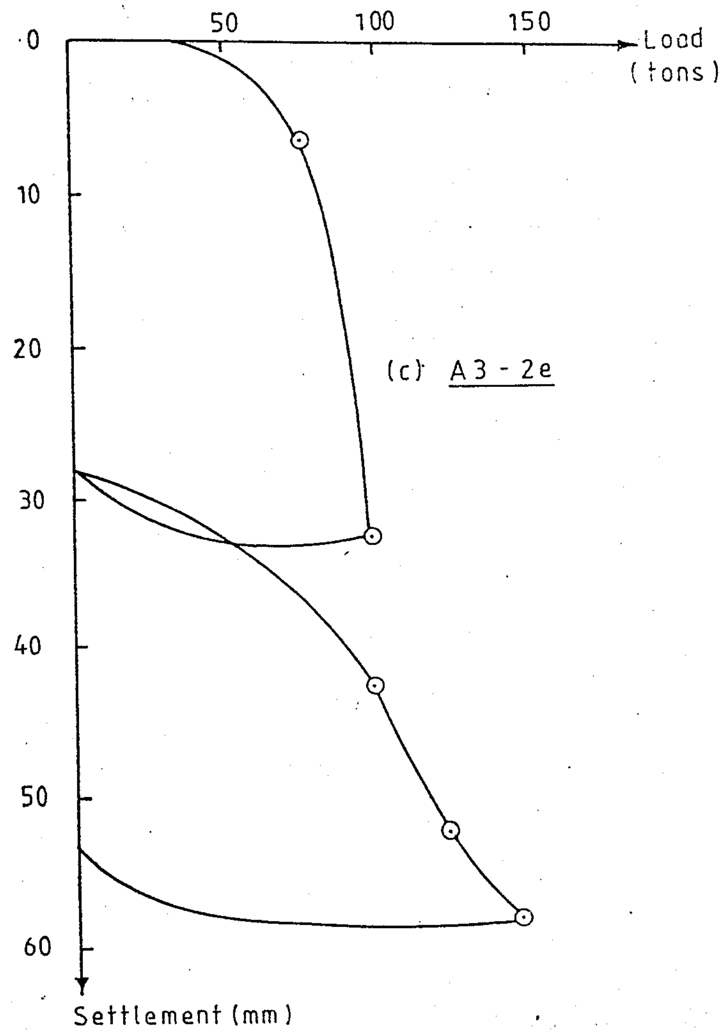


Fig.4 - Load - Settlement curves
- Bored and cast in-situ RC piles

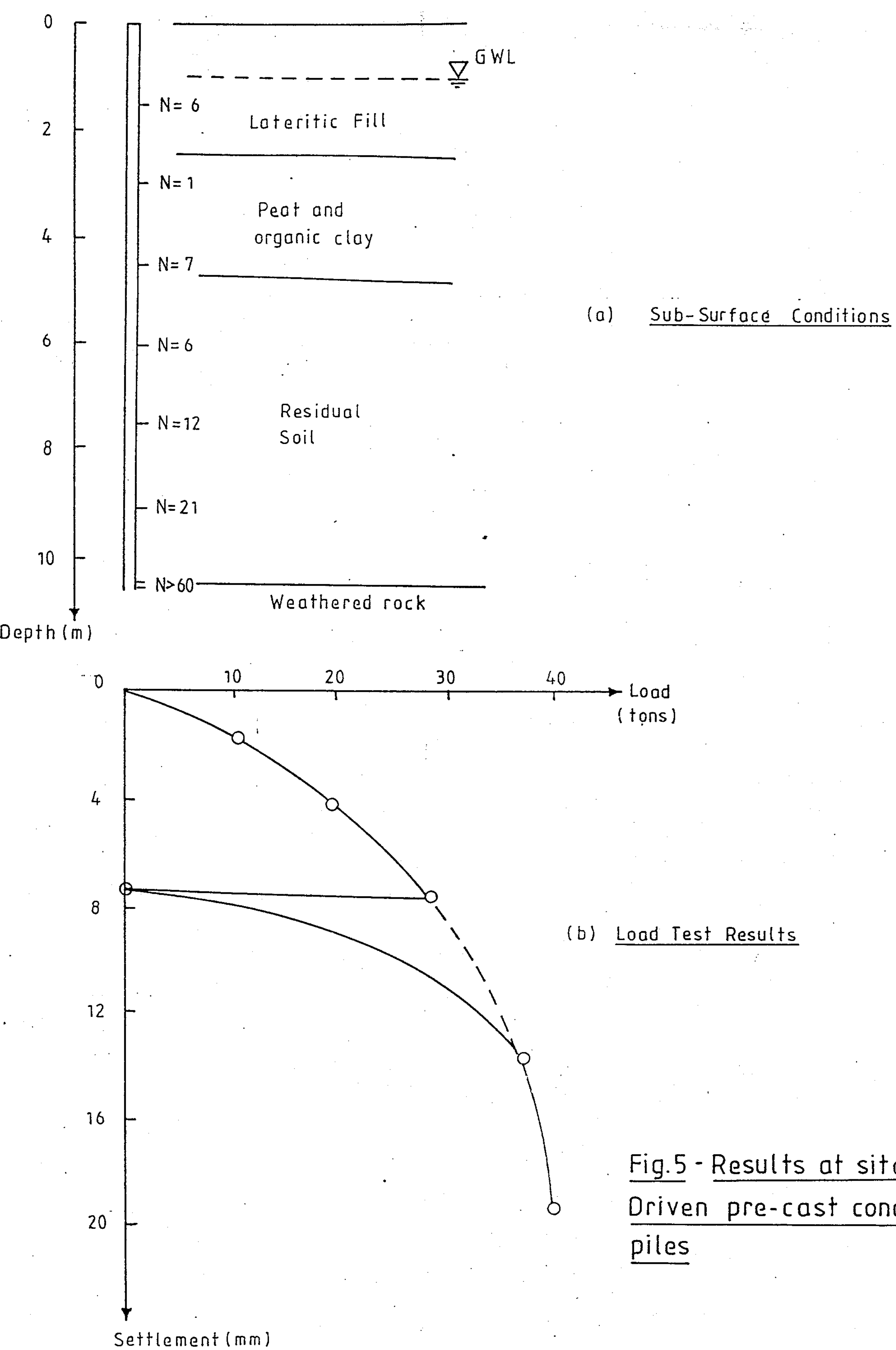
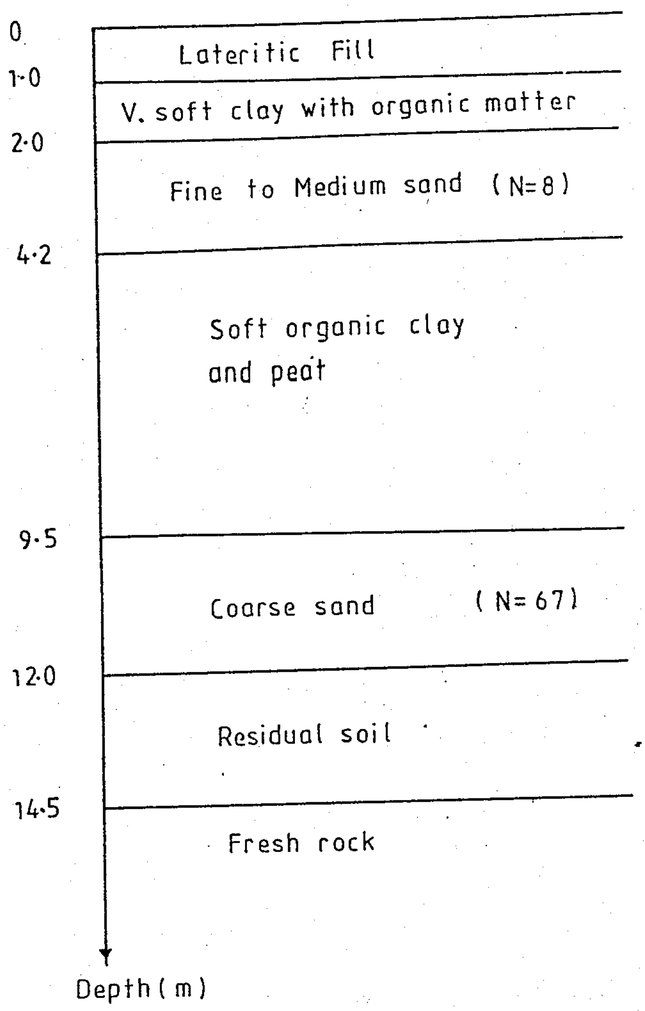
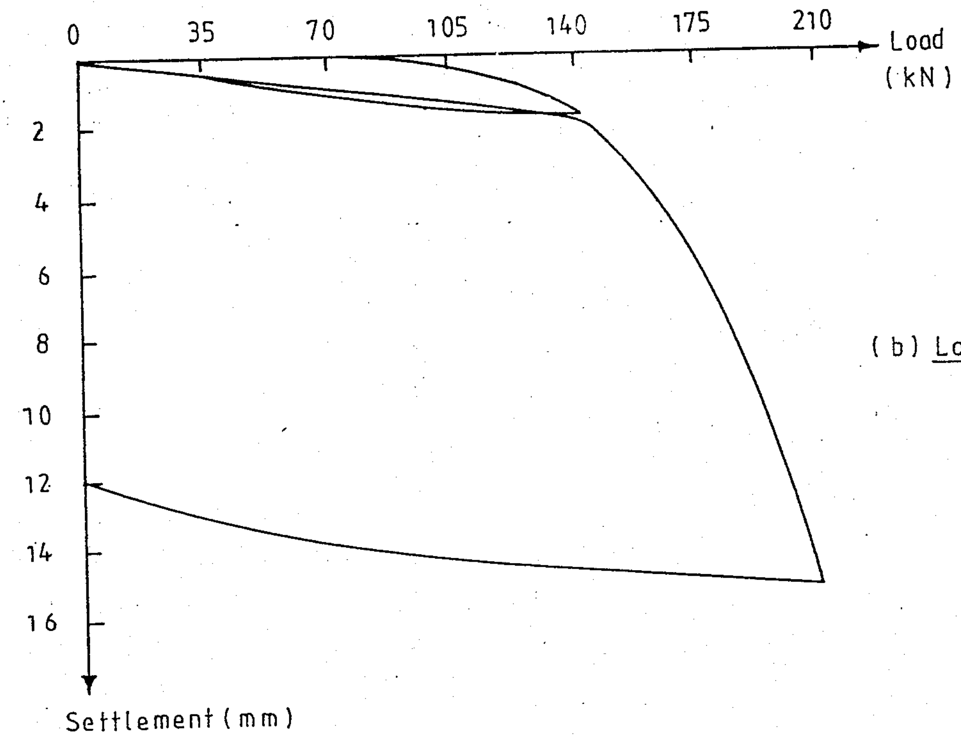


Fig.5 - Results at site of Driven pre-cast concrete piles



(a) Sub-surface Conditions



(b) Load Test Results

Fig. 6 - Results at site of Driven timber piles

EVALUATION OF LOAD CARRYING CAPACITY OF DRIVEN PRE-CAST CONCRETE PILES

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1. INTRODUCTION

The ability of a pile to carry load depends both on the material properties of the pile material as well as the properties of the ground on which the pile is founded. In Sri Lanka, because of the relative shallow depths at which rock is encountered at many locations, the practice has been to have the piles end bearing on rock. The rock is considered very strong compared to the pile material and therefore the carrying capacity of a pile is determined based on the strength of the pile material. This assumption may not be valid when piles, especially driven piles, are founded in soils or weathered rock. This situation often occurs when a decision is taken to terminate a pile without extending it by a full pile length. The assumption could also fail when using high strength concrete mixes or steel, as the pile material. This paper evaluates the critical factors which are used for predicting the load carrying capacity of driven pre-cast concrete piles using driving formulae.

2. PILE DRIVING FORMULAE

Pile driving formulae are often used for predicting the working load of a pile. These formulae provide an empirical relationship between the Resistance (R) to penetration, and the Set (s) which is defined as the net penetration per blow.

The working load (P) of a pile is then obtained as

$$P = R/F \qquad \text{Eq. (1)}$$

where F is a suitable factor of safety.

A scientific cloak is provided to the pile driving formulae by deriving them analytically either by means of the Work-Energy relationship, or by the Impact-Momentum theory, or a combination of both.

Two of the commonly used formulae are

(i) Engineering News Formula , which gives

$$W h = R\ (s + 0.5\ c) \qquad \text{\_\_\_\_\_\_} \qquad \text{Eq. (2)}$$

where R, s are as defined earlier;

W = weight of falling hammer;

and c = elastic compression of pile system during driving.

(ii) Hiley's Formula , which gives

$$R[s + 0.5(c_1 + c_2 + c_3)] = \frac{(W + e^2 p)}{(W + P)} W(e_f h) \qquad \text{\_\_\_\_} \qquad \text{Eq, (3)}$$

where the parameters not defined earlier are

P = weight of pile;

e = coefficient of restitution between the two striking surfaces;

e_f = hammer efficiency ; and

c_1, c_2, c_3 = elastic compression of pile cap, pile, and ground, respectively.

The main criticism levelled against the use of driving formulae is that the assumptions used in deriving them pay little regard to the actual forces and movements occurring during driving of a real pile, or to the nature of the soil and its behaviour. There is no basis at all for assuming that the resistance to penetration is equal to the static load bearing capacity.

However, engineers claim that within a limited context of pile and soil type, some particular driving formula may rank as 'good'. e.g. The Engineering News Formula is often used for computing the working load of timber piles.

The British Standard Code of Practice on Foundations (BSCP 8004 of 1986) recommends the Hiley formula as being the most reliable and widely used driving formula in UK. Although, not explicitly stated so, it is probably the most widely used driving formula for pre-cast concrete piles.

The soils in this country are very different from the soils in UK, USA, etc for which conditions these formulae have been derived. Whereas in those countries, the soils are transported sedimentary soils, in Sri Lanka the piles often terminate in residual soils which have been formed by the in-situ weathering of the parent rock lying beneath. Therefore, a need exists to predict the load carrying capacity of piles driven into residual soils or highly weathered rock based on the resistance to penetration.

In this study, the Hiley formula has been assumed to be the most appropriate driving formula for pre-cast concrete piles.

3. RESULTS OF A TEST PROGRAM

Results are reported here of a test program where 3 pre-cast concrete piles TP-1, TP-2 and TP-3 were driven into the residual soil/weathered rock.

Each of the piles had a cross-section 300 mm x 300 mm and was of length 10 m.

The piles had been designed for a working load of 30 tons based on the concrete strength.

The sub-surface conditions at the 3 locations are as shown in Fig.1.

The piles were driven by a hammer weighing 1.2 tonnes falling through a height of 150 cm. The piling hammer was winched up along the pile guide to the required height and then made to fall by the release of the engine clutch and brake. The final set measured for the 3 piles are given in Table 1.

Pile No.	TP-1	TP-2	TP-3
Final set (cm/blow)	0.50	0.24	0.60

Table 1

It is noted that the set in TP-2 is the lowest.

A maintained load test was then carried out on each of the these piles.

The following loading procedures were used.

Pile TP-1 was loaded to the design working load of 30 tons and unloaded; and then re-loaded again to a load of 40 tons and unloaded.

Pile TP-2 was loaded to 40 tons and unloaded.

Pile TP-3 was loaded to 30 tons and unloaded. It was then required to re-load the pile to 40 tons. However, this could not be achieved because the pile began to settle very fast at a load of 39 tons.

The results of the (load-settlement) curves for the 3 test piles are shown in Fig. 2.

4. ESTIMATION OF THE ADEQUACY OF A PILE TO CARRY A DESIGN WORKING LOAD

The adequacy of a pile to carry a load is usually determined from the results of the load test on the pile.

Although no Sri Lankan Standards are available as yet in this respect, some of the Consultants use a specification as follows;

For 1.5 times working load,

(i) maximum settlement < 25 mm

(ii) residual settlement on unloading < 6 mm.

The Housing and Development Board of Singapore has a slightly different specification as follows:

(i) For 1.0 times working load, max. settlement < 12 mm

For 2.0 times working load, max. settlement < 30 mm

For 2.5 times working load, max. settlement < 40 mm.

(ii) In all the above cases, an overall recovery of more than 50% should be observed on unloading.

The results of the pile load test gave the following results:

<u>Pile TP-1</u>

When load = 30 tons,

max. settlement = 4.485 mm

residual settlement = 1.23 mm

Hence the pile can carry this load safely.

When load = 40 tons,

max. settlement = 19.2 mm

residual settlement = 15.5 mm

From consideration of the residual settlement, the pile is inadequate to carry this load.

<u>Pile TP-2</u>

When load = 40 tons,

max. settlement = 6.32 mm

residual settlement = 2.51 mm.

Hence the pile can carry this load safely.

Pile TP-3

When load = 30 tons,

max. settlement = 8.1 mm

residual settlement = 7.6 mm

From consideration of residual settlement, the pile is inadequate to carry this load.

When load = 39 tons, very large total settlements took place.

Hence the load of 39 tons may be considered as the ultimate carrying capacity of this pile.

5. ESTIMATION OF FAILURE LOAD OF A PILE

Two methods have been used to estimate the failure load of a pile.

5.1 Method recommended by ASCE (American Society of Civil Engineers)

Details of this method are given by Ackley and Sanders (1980). In this method, the failure load is estimated using the elastic offset criterion.

The method assumes that the pile is a fixed base, free standing column of length L, area A, Young's modulus E_p and subjected to a load P.

The elastic deflection is calculated by

$$\delta = PL/A\ E_p$$

The scale of the 'load vs. settlement' curve is then selected such that the elastic deflection line plots an angle of about 20^{0} from the load axis. An off-set line is then drawn parallel to the elastic line at a distance equal to (0.38 + B/120) cm, where B is in cm.

This construction is shown in Fig. 3.

Failure load is defined as the load at which the 'load-deflection' curve intersects the off-set line.

The results when applied to the test piles are as follows:

<u>Pile TP-1</u> - Failure load of pile = 37.4 tons.

<u>Pile TP-2</u> - Settlements undergone by the pile are not adequate to determine the failure load.

<u>Pile TP-3</u> - Failure load of pile = 31.2 tons.

(It should, however, be noted that pile TP-1 was loaded to 40 tons without the ultimate condition being reached; and in pile TP-2, the ultimate load was 39 tons.)

5.2 <u>Method based on the British Practice and reported by Tomlinson(1978).</u>

In this method, the 'load vs. settlement' curve, and the 'load vs. net-settlement' curve are drawn. Typical shapes of these curves are shown in Fig. 4.

Several criteria are then considered to define failure. These are discussed below with reference to the 2 graphs of Fig. 4.

1. The load at which settlement continues to increase without any further increase of load. (Point A)

2. The load causing a gross settlement of 10% of the least pile width. (Point B).

3. The load beyond which there is an increase in gross settlement disproportionate to the increase of load (Point C).

4. The load beyond which there is an increase in the net-settlement disproportionate to the increase of load. (Point D).

5. The load that produces a plastic yielding or net settlement of 6 mm. (Point E).

6. The load indicated by the intersection of tangent lines drawn through the initial flatter portion of the gross settlement curve and the steepest portion of the same curve. (Point F).

7. The load at which the slope of the net-settlement curve is equal to 0.25 mm per 10 kN of test load.

8. One half of the load at which, when maintained for 24 hours, the net-settlement after the removal of the load does not exceed 0.25 mm per 10 kN of test load.

The minimum of these different criteria is then taken as the failure load.

These criteria when applied to the test piles gave the following results.

<u>Pile TP-1</u> - failure load of pile = 35 tons

Pile TP-2 - settlement undergone by the pile are not adequate to determine the failure load.

Pile TP-3 - failure load of pile = 28 tons

6. EVALUATION OF RESISTANCE TO PENETRATION

The resistance to penetration has been evaluated using Hiley's Formula (Equation(3) of Sec. 2). Several parameters have to be evaluated in using this formula, and these are discussed below.

6.1 Efficiency (η) of the striking blow

This is given as $\eta = \dfrac{W + e^2 P}{W + P} = \dfrac{1 + e^2 (P/W)}{1 + (P/W)}$

for this test, W = 1.2 tonnes and P = 1.4 tonnes.

Judgement is required in selecting the coefficient of restitution (e) which depends on the properties of the striking surfaces.

Poulos and Davis (1980) report values for e from a table given by Housel, and these are shown in Table 2.

Pile Type	Head Condition	value of e for drop hammers
Reinforced Concrete	Helmet with composite plastic or greenhart dolly, and packing on top of pile	0.4
	Helmet with timber dolly, and packing on top of pile.	0.25

Table 2

Prakash et al.(1987) report on the recommendations of the Indian Standards where values similar to Table 2 are given. The Indian Standards gives one further 'Head Condition' - viz. for a deteriorated condition of the head of pile or of dolly for which e=0 is recommended.

For the test piles used, e was selected as 0.25. This gives

$$\eta = 0.495.$$

(It should be noted that if e=0 is selected, η changes to 0.462.)

6.2 Hammer efficiency (e_f)

Poulos and Davis (1980) report values for e_f from a table given by Chellis, and these are shown in Table 3.

Hammer Type	e_f
Drop hammer released by trigger	1.00
Drop hammer actuated by rope and friction winch	0.75

Table 3

The Indian Standards recommends values of 1.0 and 0.8 respectively for the 2 cases given above.

For this test program, a value of $e_f = 0.8$ was selected, thus giving $H = e_f\,h = 0.8 \times 150 = 120$ cm.

6.3 Elastic compression of c_1, c_2, c_3 - i.e. of pile cap, pile, and ground respectively.

Poulos and Davis (1980) recommend the following for the elastic compressions:

$c_1 = (0.381 \text{ to } 0.508)$ cm

$c_2 = RL/AE$ where
- L = length of pile unit
- A = cross-section area of the pile
- E = Young's modulus of pile material
- R = Resistance to penetration.

$c_3 = (0.0 \text{ to } 0.05)$ cm

The Indian Standards has an alternate recommendation:

$c_1 = 9.05\ R/A$ when driving is with short dolly upto 60 cm long, helmet and cushion upto 7.5 cm thick ; or

$= 1.77\ R/A$ when driving is without dolly or helmet, and cushion is about 2.5 cm thick.

$c_2 = 0.0657\ RL/A$

$c_3 = 3.55\ R/A$

In the above equations of the Indian Standards,

R is measured in tonnes,
L is measured in m,
and A is measured in cm^2.

The results c_1, c_2 and c_3 are in cm.

For this test program, the recommendations of the Indian Standards was used with c_1 being taken as 9.05 R/A. This method has the advantage of then solving algebraically for R for any given value of the set (s).

e.g. for TP-3 where the set (s) = 0.60 cm,
the value of R as computed from Eq. (3) is 65.75 tonnes.

This value of R can now be substituted to find the values of c_1, c_2, c_3.

These are obtained as $c_1 = 0.661$ cm, $c_2 = 0.048$ cm, $c_3 = 0.259$ cm.

Further $c_1 + c_2 + c_3 = 0.968$ cm.

6.4 Sensitivity analysis

The above analysis has shown that much subjectivity was involved in the selection of the parameters for use with the Hiley formula. Therefore, a sensitivity analysis was carried out for the parameters η, c_1, c_2 and c_3 in the Hiley formula around the equilibrium point.

$$\bar{\eta} = 0.495,\ \bar{c}_1 = 0.661 \text{ cm},\ \bar{c}_2 = 0.048 \text{ cm and } \bar{c}_3 = 0.259 \text{ cm}$$

Details of this analysis are given in Malepathirana et al.(1994). The change in each parameter required to cause a 10% change in R was calculated and these results are given in Table 4.

Parameter	Variation required to cause a 10% change in R	
η	0.055	11.1 % of $\bar{\eta}$
c_1	0.19 cm	28.7 % of $\bar{c}_1$
c_2	> 0.10 cm	> 200 % of $\bar{c}_2$
c_3	0.209 cm	80.7 % of $\bar{c}_3$

Table 4

From these results it is concluded that the parameter η has the biggest influence on R. Therefore, the selection of e (and e_f) correctly must take the highest priority. The next important parameter which should be assessed accurately is c_1 - the elastic compression of the pile cap. The accuracy of c_3 and c_2 are less significant.

7. DETERMINATION OF THE PILE WORKING LOAD

The working load on a pile can be assessed by dividing the ultimate load carrying capacity of the pile by a suitable factor of safety. When the pile fails by failure of the pile material, a factor of safety of 2.5 is usually used for concrete piles. However, no standard has been specified in this country nor in the British Code of Practice for the factor of safety against ground failure.

The use of too high factors of safety will result in uneconomical designs, whilst too small factors of safety should also be avoided.

In the tests reported in this paper, 2 test piles failed - viz. TP-1 and TP-3. Test pile TP-1 was loaded to 40 tons, whilst TP-3 had an ultimate load carrying capacity of 39 tons. All the test piles had a design working load of 30 tons based on concrete strength. Therefore, it is concluded that the piles which failed have most probably failed as a result of ground failure.

7.1 Assessment of working load based on settlement criteria

As indicated previously in Sec. 4, a specification often used by some Consultants is that for 1.5 times working load,

(i) maximum settlement < 25 mm
(ii) residual settlement on unloading < 6 mm.

Using this specification,

For TP-1, 1.5 times working load should lie between 30 and 40 tons.
i.e. working load will lie between 20 and 26 tons.

for TP-3, 1.5 times working load is very slightly less than 30 tons.
i.e. working load will be very slightly less than 20 tons.

7.2 **Assessment of working load based on factor of safety against ultimate failure.**

If a factor of safety of 2.0 is used against the maximum load applied in the pile,

for TP-1, working load = 20 tons

for TP-3, working load = 19.5 tons

It should be noted however, that the failure loads estimated by the ASCE method and the British Method is less than the maximum load applied to the piles; e.g. the failure loads estimated by the ASCE method are 37.4 tons and 31.2 tons respectively for TP-1 and TP-3. The corresponding factors of safety are given in Table 5.

Pile No.	Failure load (tons) - ASCE Method	Assumed working load (tons)	F
TP-1	37.4	20.0	1.87
TP-3	31.2	19.5	1.6

Table 5

These results show that a factor of safety of about 1.6 should be used against the failure load as determined from the ASCE Method to establish the working load of a pile.

7.3 Relationship of working Load (P) to Penetration Resistance (R) as determined from the Hiley formula

The relationship of Working Load (P) to Penetration Resistance (R) for the 2 piles which were loaded to failure is shown in Table 6.

Pile No.	Penetration Resistance (R) tons	Working Load (P) tons	F = R/P
TP-1	70.1	20.0	3.5
TP-3	65.75	19.5	3.4

Table 6

These results show that a factor of safety of about 3.4 should be used to predict the working load of a pile from the Hiley formula.

Calculations for pile TP-2 show a Penetration Resistance of 83.4 tons, which if used with the above recommended factor of safety will yield a pile working load of 24.5 tons.

8 CONCLUSIONS

This paper sets out the results obtained on 3 pre-cast concrete driven piles where for each pile information was available on (i) sub surface conditions, (ii) driving data, (iii) load test data.

The Hiley formula is generally accepted as the most suitable driving formula for estimating the carrying capacity of a pre-cast concrete pile. It is shown that even for the use of this formula, judgement is necessary in selecting several parameters of which the most critical are (i) the coefficient of restitution between the striking surfaces, (ii) the hammer efficiency, and (iii) the elastic compression of the pile cap.

It is shown that for driven piles which terminate in residual soil/weathered rock, the failure of the pile takes place by failure of the ground, and therefore it would be incorrect and dangerous to determine the pile carrying capacity from consideration of concrete strength.

From the results of the 3 tests, the following conclusions were made:

(i) the working load of a pile can be determined from a pile load test when a specification is placed that for 1.5 times the working load,

(a) the maximum settlement should be less than 12 mm, and

(b) the residual settlement an unloading should be less than 6 mm.

(ii) the failure load of pile can be estimated from the results of a load test most simply using the ASCE method. This failure load is less than the ultimate load carrying capacity of the pile. A factor of safety of 2 against the ultimate load is equivalent to a factor of safety of 1.6 against the failure load estimated by the ASCE method.

(iii) the carrying capacity of a driven pre-cast concrete pile in residual soil/weathered rock can be estimated using a factor of safety of 3.4 with the Hiley Formula.

It is necessary to point out that these conclusions have been obtained on the basis of only 3 tests. It is recommended that many more similar results be published so that sufficient data would be available for adopting a suitable standard for use in Sri Lankan ground conditions.

9. ACKNOWLEDGEMENTS

The Author wishes to thank Messrs. Foundation and Waterwell Engineering (Pvt.) Ltd., for making available the data.

Much of the work on this paper was carried out by Miss. M.D. Malepathirana, Mr. D.L. Amarasinghe and Mr. J. Bungiriya who did the work as an undergraduate project under the Author's supervision.

Thanks are due to Mr. V. Somaratne for typing the paper and Mrs. L.I. Wickramarachchi for undertaking the tracings for the figures.

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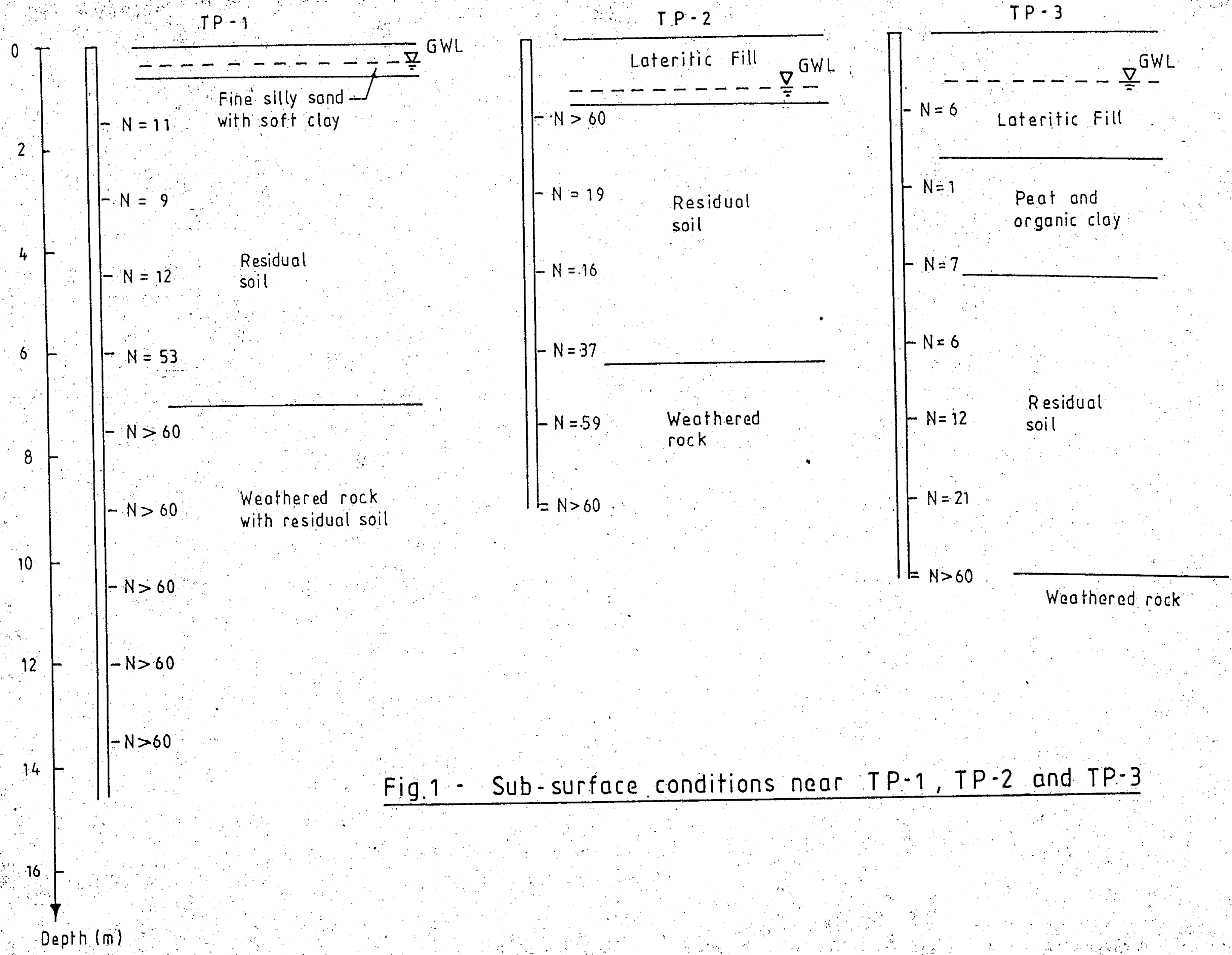


Fig.1 - Sub-surface conditions near TP-1, TP-2 and TP-3

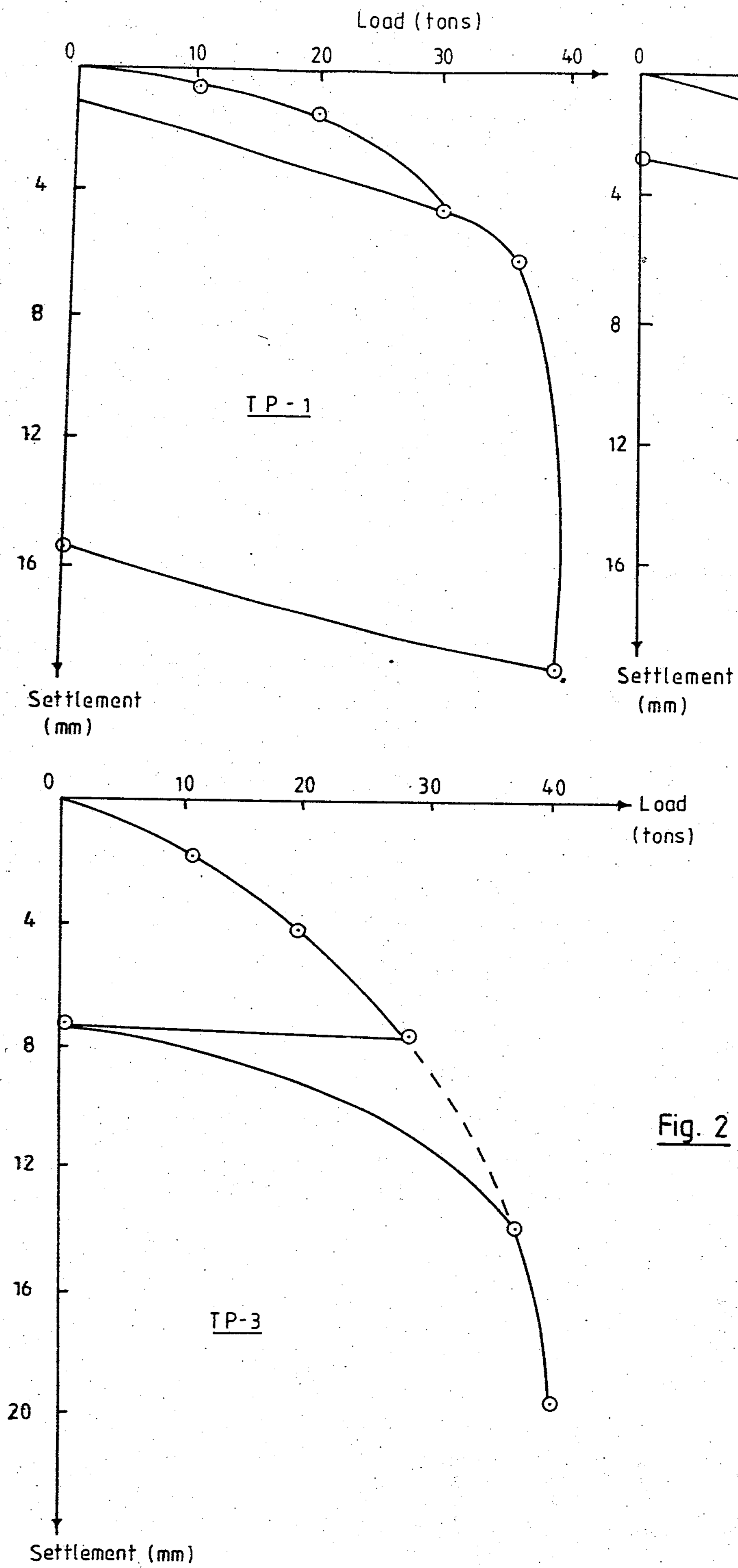


Fig. 2 - Results of maintained load tests

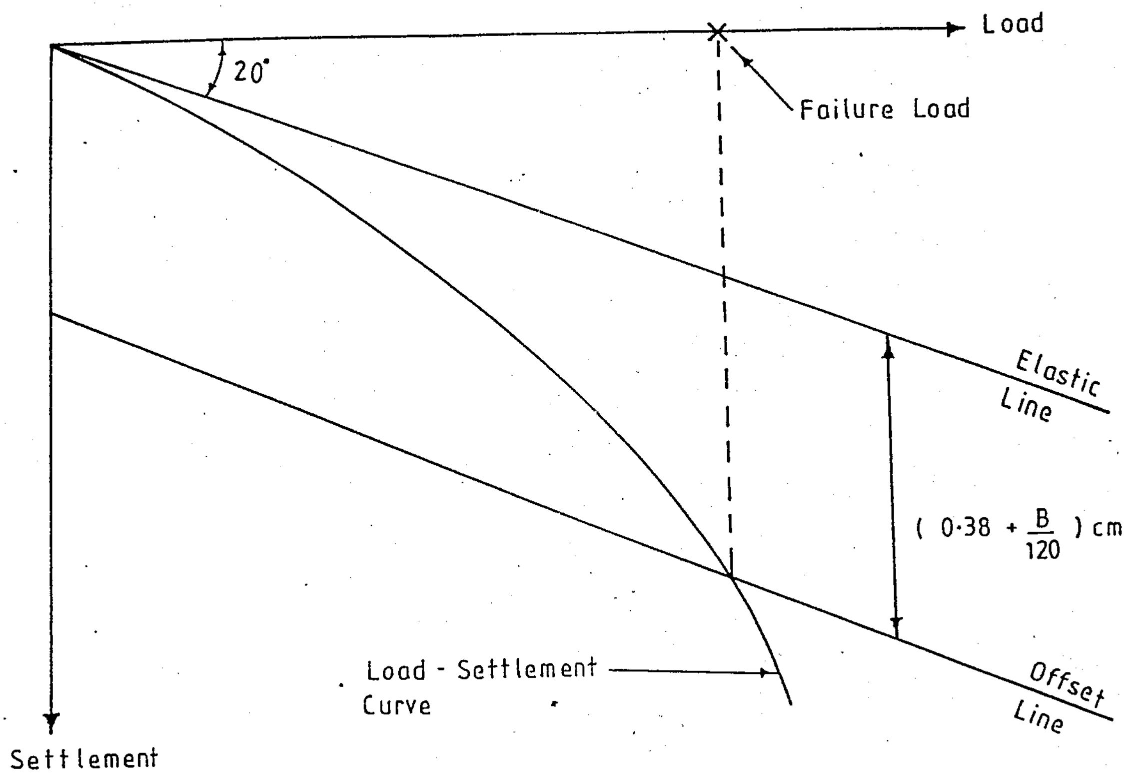
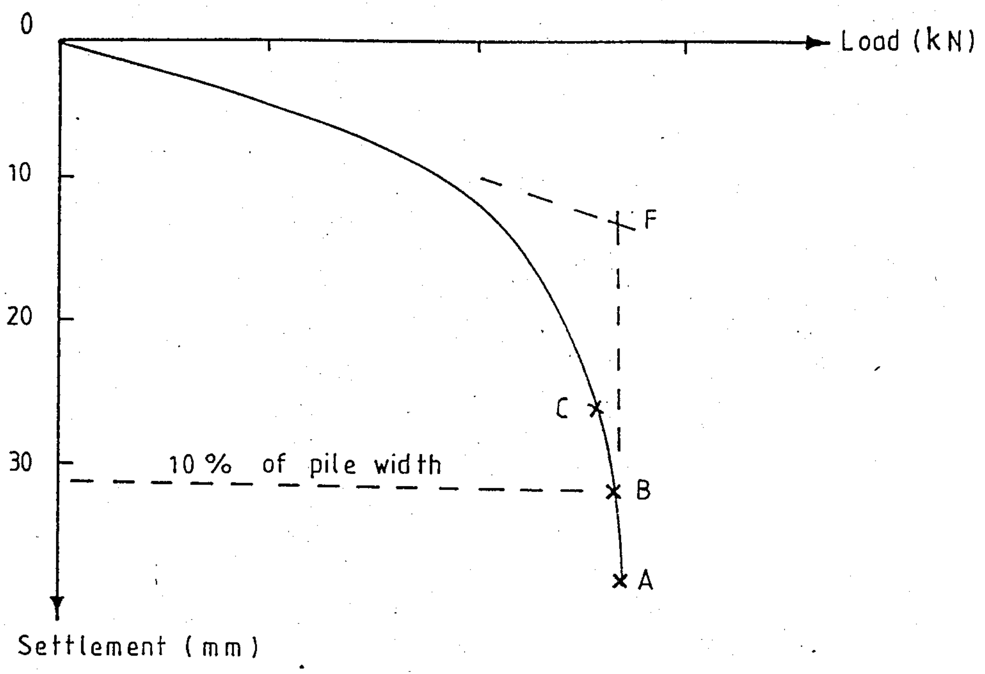


Fig. 3 - Estimation of Failure Load by ASCE Method



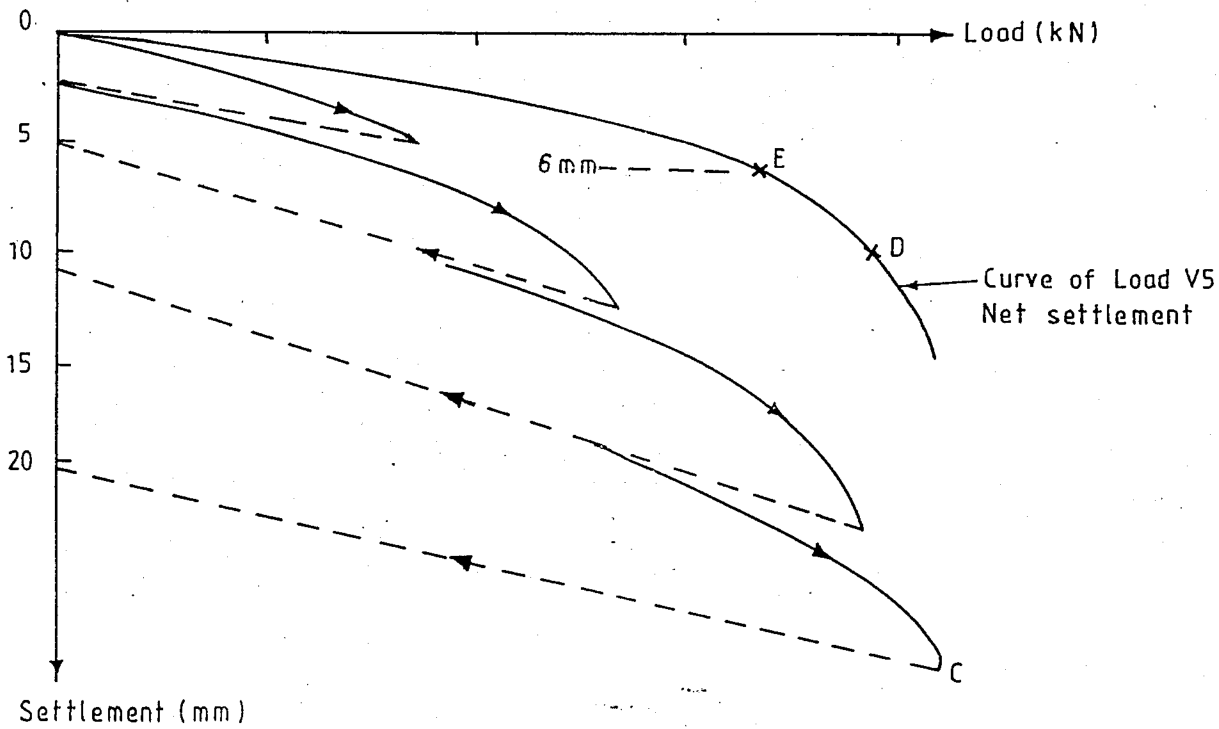


Fig.4 - Estimation of Failure Load based on British Practice