For Official use only

सत्यमेव जयते

GOVERNMENT OF INDIA
MINISTRY OF RAILWAYS
(Railway Board)

INDIAN RAILWAY STANDARD

MANUAL ON THE DESIGN AND CONSTRUCTION OF WELL AND PILE FOUNDATIONS (1985)

(WELL AND PILE FOUNDATION CODE)

ADOPTED – 1941
REPRINTED - 2004

ISSUED BY

RESEARCH DESIGNS AND STANDARDS ORGANISATION
LUCKNOW - 226011

INTRODUCTION

This manual covers the design and construction of well foundation and pile foundations for Railway bridges, which generally form part of the permanent foundations for long span bridges. These foundations are commonly used for transferring heavy loads to deep strata in river bed from piers and abutments of bridges.

This manual finalised by RDSO has been approved by the Bridge & structure Standards Committee. The Chief Engineers may issue supplementary instructions from time to time to suit local working conditions.

CONTENTS **PAGE NO**

MANUAL ON THE DESIGN AND CONSTRUCTION OF WELL AND PILE FOUNDATIONS

DEEP FOUNDATIONS

1. WELL FOUNDATION

1.1. Depth Of Foundations The depth of deep foundations below the high flood level shall be determined as indicated in clause 6.10.1. For substructures in sandy strata the depth of foundations may be determined from Fig 1 which is based on Technical Paper No 153 PL: XIII. The choice of type and shape of well foundation will depend upon the soil, type, the size and shape of pier or abutment, depth of foundation and available construction material. Where major obstructions such as uneven rocky strata are likely to be encountered, provision for pneumatic sinking may be made. Small obstructions can be removed either with the help of divers or by chiselling.

1.2. Shape And Cross-Section Of Wells: The horizontal cross-section should satisfy the following requirements:

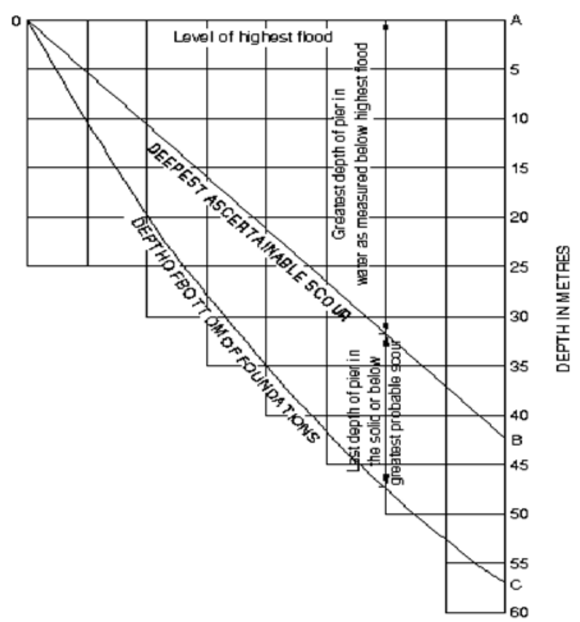
(a) The dredge hole should be large enough to permit dredging.

(b) The steining thickness should be sufficient to enable sinking without excessive kentledge and provide adequate strength against forces acting on the steining, both during sinking and service. The well steining should also be designed to withstand the earth pressures acting only on two opposite sides or only on diametrically opposite quadrants under conditions of sand blowing. The effect of heap of earth dumped near the well during sinking shall also be taken into account.

(c) It should accommodate the base of the substructure and not cause undue obstruction to the flow of water.

(d) The overall size should be sufficient to transmit the loads safely to the soil without exceeding its allowable bearing pressure.

FIG.1 DIAGRAM OF THE DEPTH OF BRIDGE PIERS IN WATER AND IN RIVER BED RESPECTIVELY.



Explanation Of The Diagram: The intention of the diagram is to offer something definite in place of the rather fortuious method now centrally practiced.

OA-Represents highest known flood level

OB-Represents deepest ascertainable scour.

OC-Represents depth to which foundation should be sunk.

Note:

1. The diagram applies only to sandy bottom. If the river bed is soft, a greater depth is necessary. Piers are always presumed to have enough stone around them to prevent local pier formed swirls from scooping pot-holes at pier base.
2. This diagram is based upon Technical paper No 153 PI: XIIIA.

(e) It shall allow rectification of the tilt and shift of the well without damaging the well.

The shapes normally used are circular, double D. Dumb-bell, hexagonal or octagonal, square, rectangular and any of the above shapes with multiple dredge holes.

1.3. Allowable Bearing Pressure And Modulus Of Sub-Grade Reaction

1.3.1. The allowable bearing pressure may be determined in cohesion less soils on the basis of the penetration test results as given in IS: 3955 and reproduced below:

$Q = 9.8 \{5.4\ N^2B + 16(100+N^2)\ D\}$ in Newton/m^2

[$Q = 5.4\ N^2B + 16(100+N^2)\ D$ in Kg/m^2

where,

Q = Bearing capacity of soil under the well foundation in N/m^2 (Kg/m^2)

N = Number of blows per 30cm in the standard penetration test.

B = smaller dimension of the well cross-section in metre.

D = Depth of foundation below scour level in metre.

The capacity worked out by the above formula is applicable only for safety against shear failure. For well foundations, settlement governs the allowable bearing capacity in most cases. The permissible value of settlement is generally kept within 25mm and the allowable bearing pressure q_a for such settlement can be obtained approximately by the following equation:

$q_a = 9.8 \times (1+ 0.3/B)^2\ N$ for $B > 1.2$m - in KN/m^2

[$q_a = (1+ 0.3/B)^2\ N$ for $B > 1.2$m - in tonne/m^2]

$q_a = N$ in tonne/m^2 (approximately irrespective of B)

$q_a = 9.8 \times 1.4\ N$ for $B \le 1.2$ m – in KN/m^2

[$q_a = 1.4\ N$ for $B \le 1.2$ m – in tonne/m^2]

Where,

N=corrected standard penetration resistance

(No of blows per 30 cm)

If larger settlement can be tolerated, the allowable bearing pressure could be increased accordingly. For clayey strata settlement should be worked out for full load based on consolidation test results. For wells constructed in cohesion less soils where full settlement due to dead load will take place by the time construction is completed and the necessary adjustments in the final level can be made before erection of girder, dead load due to well and the substructure can be ignored. In such cases, settlement shall be evaluated only for superstructure, live load and loss of friction in the well due to scour.

1.3.2. The passive pressure and skin friction shall be taken only for soil below the level of scour. In seismic areas relief due to skin friction should be ignored.

The average value of skin friction may be adopted as per following equation.

$$F = 9.8\left(\frac{1}{2}K_a \gamma Z - 2C\sqrt{K_a}\right)\tan\frac{2}{3}\varphi \quad \text{in } N/m^2$$

$$F = \left(\frac{1}{2}K_a \gamma Z - 2C\sqrt{K_a}\right)\tan\frac{2}{3}\varphi \quad \text{in } kg/m^2$$

Where,

F = Skin friction in N/m^2 (kg/m^2)

K_a = Active earth pressure coefficient.

C = Half of unconfined compressive strength.

Ø = Angle of shearing resistance of soil.

= Submerged weight of soil below scour line.

Z = Depth of foundation level below bed level.

In the absence of any data, the following values may be adopted; these are based on observations made during sinking of wells:

Soil	**Value of skin friction KN/m^2**	**Value of skin friction Kg/m^2**
Silt & soft clay	7.16 to 28.73	730 to 2930
Very stiff clay	47.86 to 191.52	4880 to 19530
Loose sand	11.96 to 33.54	1220 to 3420
Dense sand	33.54 to 67.08	3420 to 6840
Dense gravel	47.86 to 95.71	4880 to 9760

1.3.3 Modulus of sub-grade reaction may be adopted as per IS: 2950.

1.3.4 In case the well is found on rock, its suitability to take load shall be found by testing cores. If the rock bed is inclined, it is advisable to seat the steining evenly on the rock foundation.

1.4 Loading

1.4.1 Wells shall be designed to resist the worst condition due to possible combination of the following loads, as may be applicable, with due regard to their direction and point of application.

(a) Vertical Loads:

i) Self-weight of well.

ii) Buoyancy

iii) Dead load of superstructure, substructure.

iv) Live load, and

v) Kentledge during sinking operation

(b) Horizontal Forces:

i) Braking and tractive effort of moving vehicles.

ii) Forces on account of resistance of bearings.

iii) Forces on account of water current or waves.

iv) Centrifugal force, if the bridge is situated on a curve.

v) Wind forces or seismic forces.

vi) Earth pressure.

vii) Other horizontal and uplift forces due to provision of transmission line tower (broken wire condition) etc.

1.5 Tilt And Shifts As far as possible wells shall be sunk without any tilt and shift. A tilt of 1 in 100 and shift of D/40 subject to a minimum of 150 mm shall be taken into account in the design of well foundation (D is the width or diameter of well).

If greater tilts and shifts occur, their effects on bearing pressure on soil, steining stresses, change in span etc. should be examined individually.

1.6 Cutting Edges Cutting edge shall be properly anchored to the well curb.

When there are two or more compartments in a well the bottom of the cutting edge of the intermediate walls may be kept about 300 mm above the cutting edge of the outer wall to prevent rocking.

1.7 Well Curb It should transmit the superimposed load to the bottom plug without getting overstressed and it should offer minimum resistance to sinking. The slope to the vertical of the inner faces of the curb shall preferably be not more than 30 degrees. In sandy strata, it may be upto 45 degrees. An offset on the outside (about 50 mm) may be provided to ease sinking. The curb shall invariably be of reinforced concrete with a minimum reinforcement of 72 kg/m^{33} excluding bond rods. In case blasting is anticipated, the inner face of the curbs shall be protected by steel plates or any other means to sufficient height.

1.8 Well Steining Well steining shall be built cement concrete not weaker than M-15 grade. Sufficient bond rods shall be provided to bond the units of the steining during the progress of construction. Bond rods shall be distributed evenly on both faces of steining and tied up by providing adequate horizontal hoop reinforcement. For masonry steining and for concrete steining of small thickness, bond rods may be provided in one row in the centre only and tied up by providing plates or hoop reinforcement.

For plain concrete wells, vertical reinforcements (whether mild steel or deformed bars) in the steining shall not be less than 0.12% of gross sectional area of the actual thickness provided. The vertical reinforcements shall be tied up with hoop steel not less than 0.04% of the volume per unit length of the steining."

1.9 Bottom Plug A bottom plug shall be provided for all wells and its top shall be kept 300 mm above the top edge of the inclined face of the curb. The concrete used for the bottom plug when placed under dry conditions shall generally be of 1:3:6 proportions and it shall be placed gently in one operation. When the concrete is placed under water, the quantity of cement shall be increased by 10% and it shall be placed by tremie or skip boxes under still water condition.

1.10 Top Plug A 300 mm thick plug of cement concrete M-10 grade shall be provided over the hearting which shall normally be done with sand.

1.11 Well Cap The bottom of the well cap shall, as far as possible, be located 300 mm above low water level. All the longitudinal bars from the well steining shall be anchored into the well cap. The well cap shall be designed as a slab resting on the well.

1.12 Pneumatic Sinking Of Wells Where boring data indicate pneumatic sinking, it will be necessary to decide the method of such sinking and location of air lock.

1.12.1. The side wall and roof of the working chamber shall be designed to withstand the maximum air pressure envisaged with the use of pneumatic sinking equipment. The design air pressure for design shall be higher than the pressure due to the depth of water above the bottom of the well.

1.12.2. In case the concrete steining is used and the tension in concrete exceeds three-eighths of the modulus of rupture, the section of the steining shall be changed to keep the tensile stress within this limit or mild steel reinforcement shall be provided suitably over the width of the steining. The following further points shall be kept in view.

(i) Extra hoop reinforcement, if required to be provided, shall overlap at least one bond length below the section from where MS plates are provided for protection against blasting or other reason.

(ii) The pneumatic platform and the weight of the steining and kent ledge, if any, shall be sufficient to resist the uplift of air from inside.

(iii) If at any section of steining the uplift pressure is more than the total weight acting downwards, then the platform and the steining can be weighed down by kentledge and also anchored to the steining, if necessary.

(iv) The well steining shall also be checked at different sections for any possible rupture against the uplift force and upto the height at which the uplift force is balanced by the self weight of the steining and any superimposed load on it.

2. PILE FOUNDATIONS

2.1 Piles may be divided into the following categories depending upon the manner of transference of load:

(i) Friction Piles
(ii) Bearing Piles
(iii) Bearing-cum-friction piles

2.1.1. Friction Piles: These piles transfer the load primarily by skin friction developed along their surface.

2.1.2. Bearing Piles: These piles transfer the load primarily by bearing resistance developed at the toe.

2.1.3. Bearing-cum-frication piles: These piles transfer the load both by bearing and friction.

2.2. Piles may also be further divided into the following categories, depending upon the method of construction.

(i) Pre-cast driven piles.
(ii) In-situ driven piles (these are normally not used for Railway Bridges).
(iii) In-situ bored piles (only large diameter bored piles are normally used for Railway Bridge construction.)

2.2.1. Selection of type of pile.
The type of pile shall be selected by considering broadly the following factors:

(i) Availability of space. Driven piles require large areas and head room since it needs larger and heavier driving rigs. Bored piles, however, require comparatively smaller space.

(ii) Proximity to structure:
Driving causes vibration of the ground which may damage nearby structures.

(iii) Reliability:
Precast driven piles ensure good quality of material, uniform section of piles and give a valuable guide to the load carrying capacity. In cast-in-situ piles, segregation of concrete is possible in water-logged areas.

(iv) Compaction of cohesion-less soil is effected if driven piles are used.

(v) Cast-in-situ piles can be formed to any desired length and no cutting of pile or addition in length is required.

2.3. Spacing of Piles

2.3.1. The spacing of piles shall be considered in relation to the nature of the ground and the manner in which piles transfer the load to the soil. The spacing is also decided by group behavior for total carrying capacity and settlement.

Normally, centre-to-centre spacing shall not be more than 4 d where d is the diameter of the piles. In case of piles of non-circular section, `d' will be the diameter of the circumscribing circle.

2.3.2. Friction piles shall be sufficiently far apart to ensure that the zones of influence surrounding them do not overlap to such an extent that their carrying capacities are appreciably reduced. Generally, the spacing shall not be less than 3 d.

2.3.3. For end-bearing piles passing through relatively compressible strata, the spacing shall not be less than 2.5 d to avoid heaving of soil.

2.4. LOAD CARRYING CAPACITY OF A PILE

2.4.1.

(a) The ultimate bearing capacity of a pile may be assessed by means of a dynamic pile formula, using data obtained during driving of piles or by a static formula on the basis of soil-test results or by a load test.

(b) For non-cohesive soils, Hiley's formula is more reliable than other formulae. This formula is given in Appendix `E' of IS: 2911 Part-I Section -I-1979.

(c) Hiley's formula is not reliable in cohesive soils.

(d) The static formula should be used with careful judgment as the mechanics of load transfer from pile to soil is very complex. This judgment is employed in selecting appropriate multiplying factors.

(e) In unknown areas, load test is therefore most desirable.

(f) Where scour is anticipated, resistance due to skin friction will be available only below the scour line and this must be taken into account, in all the three methods.

2.4.2. When piles are installed through compressible fill or sensitive clay into underlying hard stratum, a drag down force is generated in the fill or the clay stratum. This must be added to the load.

This can be roughly estimated as cohesion of the remoulded clay multiplied by the surface area of pile shaft. The underlying hard stratum shall not be considered for assessing the downward drag and the skin friction expected to be mobilised by the strata will be assessed on the basis of para 2.4.3.

2.4.3. Load Carrying Capacity - Static Formula:

2.4.3.1. Piles in non-cohesive Soil:

The ultimate bearing capacity Q_u of a pile in homogenous sand may be represented by

$Q_u = Q_p + Q_s$ Where Q_p = Point resistance
Q_s = Skin resistance

$= q_p A_p + f_s A_s$ where $q_p = p_o N_q$ :::q_L

Where q_p = Unit bearing capacity of pile point of area A_p.

p_o= Effective overburden pressure at pile point.

N_q=The bearing capacity factor with respect to Overburden pressure

f_s = Average unit skin friction on shaft of area As

q_L= Limiting value of unit point resistance in 100 Kn/m^2 for D/B� Dc/B, where B= width of pile, D= Depth, Dc= critical depth of penetration of pile.

The maximum value of q_L is to be equal to 0.5 N_q tan .

Note: From a given initial value of ø (angle of internal friction) bored piles have a unit point resistance of only about $^1/_6$ to ½ of that of driven piles and bulbous piles driven with great impact energy have upto twice the unit point resistance of driven pile of uniform section. The extant methods of evaluation of ultimate bearing capacity indicate that both point resistance and average skin friction of a pile would increase with greater depth of penetration.

However, large scale experiments and field observations have shown that the theoretical relationships hold good only upto a certain depth below which the point resistance and average skin friction remain practically constant in a homogenous sand deposit due to effects of soil compressibility, crushing, arching and other factors. The depth is termed as critical depth. The semi-empirical relationships between the bearing capacity factor, N_q, for driven circular or square piles with various depth ratios D_b/B in the bearing stratum of depth D_b and the angle of internal friction, of the soil before pile driving shown in Fig. 2.

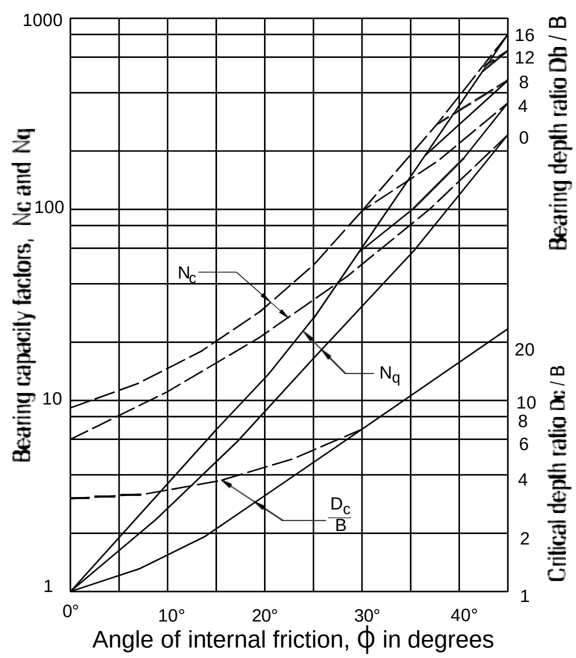


Fig.2 Bearing Capacity Factors and Critical Depth Ratios For Driven Piles.

Critical depth ratios (D_c/B) for various ø values and N_c, N_q factors; at different values for different D_b /B values in q case of driven piles. The average ultimate unit skin friction, f_s in homogenous sand may be expressed by

$f_s = k_s p_o \tan$ o :::f_1 In which
k_s= the average coefficient of earth pressure on pile shaft,

p_o = Average effective overburden pressure.
o = Angle of skin friction and
f_1 = the limiting value of average unit skin friction for D/B�Dc/B which roughly approximates that for the point resistance, the reliable values of k_s and f_1 can only be deduced from load tests on piles at the given site.

Analysis of the result of load tests on short piles above the critical depth in generally homogenous normally consolidated sand show that the value of k_s for a given initial friction angle ø can scatter considerably from a lower limit of roughly K_o for bored piles to about 4 times, this value or more for piles driven into dense sand, due to dilatancy effects and other factors. The values of K_s for bored and driven piles are shown in Figure 3

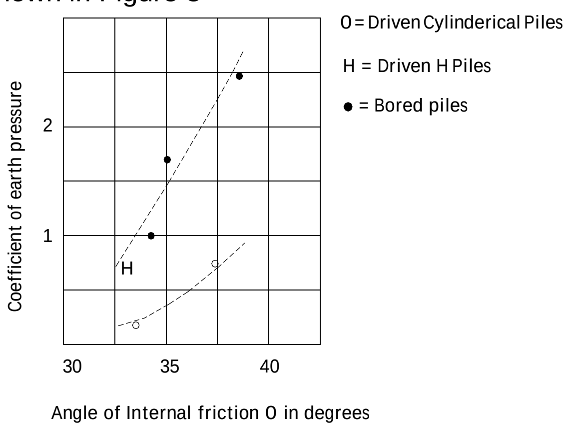


Fig. 3 Coefficient of Earth Pressure on shaft of piles above critical depth in sand.

The conventional shaft capacity theory in terms of Ks cannot be used for piles longer than about 15 to 20 pile diameter because the corresponding value of f_s in case of such long piles does not exceed the critical value f_1. The empirical relationship between the limiting value of f_s=f_1 and the friction angle ø of sand indicates a wide variation from a lower limit for bored piles to an upper limit for piles driven in over consolidated sand, as shown in figure 4.

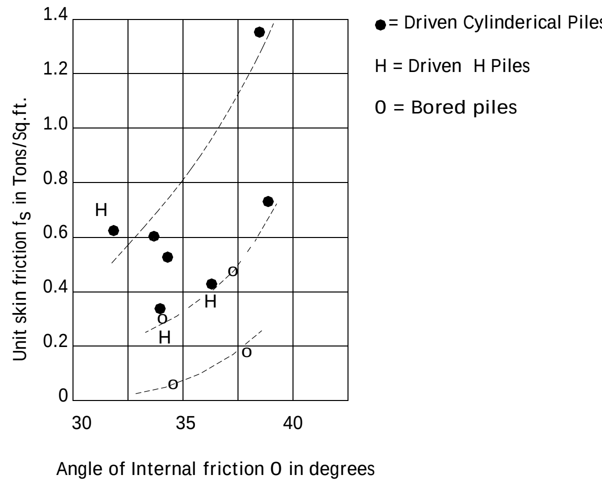


Figure 4 Ultimate Skin Friction of Piles in Sand.

<u>Use of penetrometer results for evaluation resistances</u>

When the pile point is above the critical depth in the bearing stratum a small variation of considerably influences resistance. The unit point resistance shall preferably be estimated from static cone penetration results. While the pile point is above the critical depth, the unit point resistance has to be deduced from limiting static cone resistance, q_c in proportion to the embedment ratio D_b /B in this stratum, the suggested correlation is

$$q_p = \frac{q_c \times D_b}{10B} ::: q_L$$

The ultimate skin friction of a driven pile can be known by unit resistance of local friction sleeve of static penetrometers. Value of f_s/q_b generally varies between ½% to 1% for driven cylindrical piles. For bored piles one third to one half values as applicable to driven piles shall be used. For piles driven upto a depth D_b into a sand stratum the ultimate unit point resistance in t/s mm may be taken in terms of Standard Penetration Test (SPT) as under:

$$q_p = \frac{4.3747\ N \times D_b}{B} ::: 43.75N$$

In which N = Average standard penetration resistance in blows per 0.3 m near the pile point.

The average ultimate skin friction of driven displacement pile in t/sq.m.

f_1 = (limiting value) = .22 N

In which N = the average standard penetration resistance in blows per 0.3 m within embedded length of pile.

One-half of the value may conservatively be used for piles with small soil displacement such as `H' piles. For piles driven into non-plastic silt, better agreement is obtained by using an upper limit of approximately q_p=32.81N (in t/sq.m). The value of N shall be corrected for overburden pressures to determine unit point resistance and not for frictional resistance.

Example showing use of "Static" formula:
Problem
–To work out the ultimate bearing capacity of a single pile.
- Given data
- Diameter of the pile ------ 1.2 m
- Type of pile ------ Driven-
- Embedded Depth below the deepest scour level ------ 9 m
- Soil strata ------ sandy
- ø = Angle of internal friction for soil at 9m depth below scour level --- 25°
Standard penetration test (SPT) value (Corrected for overburden & water level) - 55
(Average value from scour level to toe)

Now N = 55
We get ø = 41º 45' from Fig 4
- Saturated density of soil - 1.8 t/cum.
- Submerged density of soil-0.8 t/cum.

Design $Q_u = Q_p + Q_s$

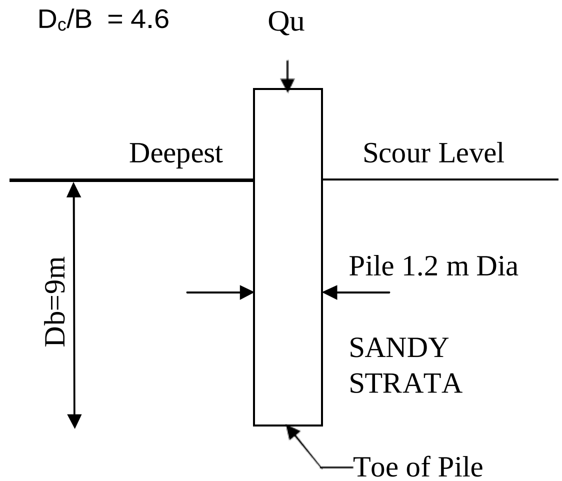
Where
Q_u=Ultimate bearing capacity of the pile
Q_p=Ultimate point resistance
Q_s=Ultimate skin friction resistance

(a) Ultimate point resistance: Q_p

D_b/B = 9/1.2 = 7.5

For ø = 25º using ø – D_c/B curve in Fig 4.

D_c/B = 4.6



A_3 D_b/B D_c/B, the condition for limiting value of q_1 is satisfied.

Using ø – D_b/ B-N_q curves of Fig 4.

We get N_q = 22 for = 25°

Therefore, q_L = 0.5 x Nqx tan ø
= 0.5 x 22 x tan ø
= 5.1 t/sq.m
Effective over-burden pressure = p_0
= Submerged density of soil x bearing depth= 0.8 x 9 = 7.2 t/sq.m

$q_p = p_0 \times N_q$
= 7.2 x 22 = 158.2 t/sq.m
since q_p ::: q_L

Therefore, q_1 is adopted for calculating the unit point resistance i.e. q_p will be taken as 5.lt/sq.m

Therefore, the ultimate point resistance
$Q_p = q_p \times A_p$
= 5.1 x ρχ/4 x $(1.20)^2$
= 5.7 tonnes

(B) Friction resistance
From the formula, the limiting value of unit skin friction
f_1 = .22 N

Here, N is 55. Therefore, the unit skin friction

$= .22 \times 55 = 12.1$ t/m^2

Also, $f_s = K_s\, p_o \tan o$

For ø = 41º 45' where fig 2 (for cylindrical driven pile)

K_s is found to be 0.75 (taking the value at ø=37º.5

since no value is available at ø=41º 45')

The average over burden pressure
$p_o = 9 \times 0.8/2$
$= 3.6$ t/sq.m
o = Soil concrete interface friction can be assumed as ø

Therefore, ø = 41º 45'
$f_s = K_s p_o \mathrm{Tan}\ o = 0.75 \times 3.6 \times \tan 41º\ 45'$
=2.410t/sq.m

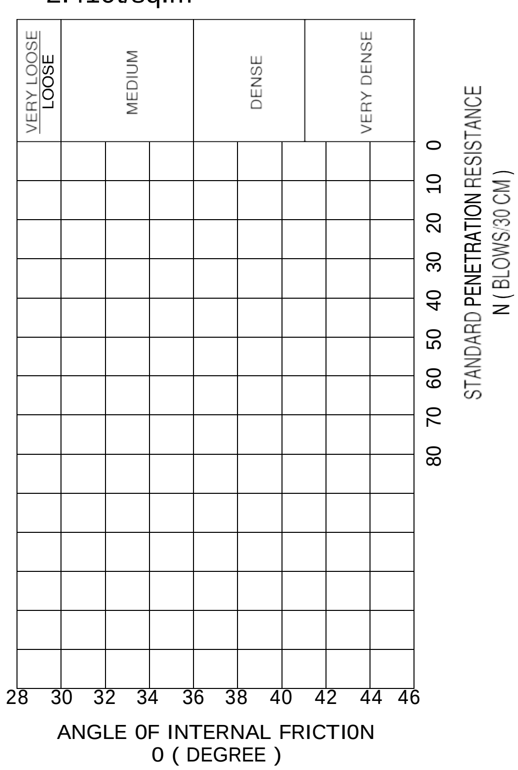


Fig 4 'A' Relationship between ø and N

Now, as f_s ::: f_1, the value of frictional resistance will be 2.41 t/sq.m

Therefore, the skin friction $Q_p = f_s \times A_s$
= 2.410 x ρχ x1.2 x 9
= 81.77 tonnes

Therefore, the ultimate bearing capacity of the pile–$Q_u = Q_p + Q_s = 5.7 + 81.77 = 87.47$ tonnes

2.4.3.2 Piles in cohesive soils

$Q_u = A_p \,.\, N_c \,.\, C_p + \alpha\, \overline{C}\quad As$

Where,
A_p (cm) ^{2} = cross sectional area of pile toe.

N_c = bearing capacity factor usually taken as 9+

C_p(kg/cm^2) = average undrained cohesion at pile tip

$\overline{C}$ (kg/cm^2) = average undrained cohesion along the embedded length of pile
α = reduction factor
As (cm^2) = Surface area of pile shaft.
+ (although its value varies from 5 for very sensitive brittle normally consolidated clay to 10 for insensitive stiff over-consolidated clay. The value of N_c for different value of ø is also shown in Fig 2.)

Note 1- The value of α decreases rapidly with the increase in shear strength. In the case of bored piles, α is 0.5 for stiff clay. For driven-cast-in-situ pile α may be taken to be 1.0 for soft clay to 0.5 or less for stiff clay.

Note 2- Static Formulae may be used as a guide only for bearing capacity estimates. Better reliance may be placed on results of load tests on piles.

2.5 Factor Of Safety For Pile Foundation

2.5.1. The factor of safety shall be judiciously chosen after considering the following:

(a) Reliability of the soil parameters used in the computation.

(b) Type of superstructure and nature of loading.

(c) Possible reduction in the strength of the sub-soil strata arising out of the installation technique.

(d) Experience of similar structures near the site.

2.5.2 The minimum factor of safety of static formula shall be 2.5. The final selection of the factor of safety shall take into consideration the total settlement and differential settlement of the structure.

2.5.3 The ultimate safe load capacity shall be obtained wherever practical from a load test (initial)(as per IS: 2911 Part4- 1985). Factor of safety for assessing safe load on piles from load test data should be increased in unfavorable conditions such as:

(a) settlement is to be limited or unequal settlement avoided as in the case of accurately aligned machinery or a superstructure with fragile finishing.

(b) large impact or vibrating loads are expected.

(c) the properties of the soil may be expected to deteriorate with time, and

(d) the live load on a structure carried by friction piles is a considerable portion of the total load.

2.5.4 The maximum permissible increase over the safe load of a pile on account of Wind load is 25%. In the case of loads and moments arising out of earthquake effects, the increase of safe loads on a single pile may be limited to the provisions contained in IS: 1893-1984. For transient loading arising out of superimposed loads, no increase in the safe load is generally permitted.

2.5.5 Overloading

When a pile designed for a certain allowable load is found to be short of the load required to be carried by it, due to change in design during construction stage or due to construction inaccuracies or due to outcome of the actual load test, an overloading upto 10%. of the pile capacity may be allowed on each pile. For a group of piles, the maximum overloading on the group shall be restricted to 40% of the allowable load on a single pile of the group. This overloading shall not be allowed at the initial design stage.

2.6 Pile Grouping

2.6.1. The bearing capacity of a pile group may be worked out as under:

Strata	**Type of Pile**	**Bearing capacity of the Pile group**
1. Dense sand not underlain by weak deposit.	Driven	No. of piles x SPC *
2. Loose sandy soil		½ (Nos. of piles x SPC)
3. Sand not underlain by weak deposit	Bored	2/3 (No of piles x SPC)

* SPC = Single pile capacity

2.6.2. The bearing capacity of a group of piles is generally evaluated by multiplying with efficiency factor. A large number of equations giving the efficiency factor are in use and it is very difficult to establish the accuracy of these equations as the behaviour of pile group is dependent on many complex factors. It is, therefore, desirable to consider each case separately on its own merits. Full scale trials have shown that for piles driven into soft and medium clays with 3 to 4d spacing, the ultimate group capacity may be only 2/3 of the sum of single pile capacities.

2.6.3. A group of piles deriving their support mainly from friction and connected at the top by a rigid pile cap may be visualised to transmit load to the soil from a column of soil enclosed between the piles. The ultimate capacity of the group may be computed taking into account the friction capacity along the sides of the column of soil and the end bearing of the soil column. The ultimate capacity of the group computed in this manner shall, however, not be taken as more than the capacity obtained by multiplying the capacity of individual piles by the number of piles.

2.6.4. When the cap of the pile group is cast directly on reasonably firm stratum which supports the piles, it may contribute to the bearing capacity of the group. This additional capacity along with the individual capacity of the piles multiplied by the number of piles in the group shall not be more than the capacity worked out as per Para 2.6.3.

2.7 Settlement Of Pile Foundation The total settlement of a group of driven or bored piles can generally be estimated roughly from equivalent pier foundation. The method of estimation for different types of piles are indicated below:

(a) **Friction Pile Groups** The load on a group of friction piles is usually assumed to be acting at an effective depth of 2/3 of pile embedment in the bearing stratum Fig 5 (a).

(b) For end bearing pile groups, the pile group may be replaced by a fictitious footing at the top of the bearing layer. The load is assumed to be uniformly distributed and spread at 2:1 slope or at 60º to the horizontal Fig 5 (b).

(c) Friction-Cum-Point Bearing Pile Groups

For pile groups which transmit the loads partly through friction and partly through point bearing, the stress in the compressible upper strata may be computed assuming that the frictional load acts on a fictitious footing at H/3 above the bearing stratum where H is the thickness of the compressible strata Fig 5(c). If, for some depth from ground level, the soil is poor and cannot provide any friction, that portion shall be neglected in arriving at the location of the fictitious footing. When point bearing pile groups in sand are subjected to negative skin friction from an upper consolidating clay or silt stratum, the corresponding down drag per unit of the pile group has to be included, in the net foundation pressure for settlement estimation.

After ascertaining the load taken by skin friction and point resistance (if any) the total settlement is given as under:

$$S_f = S_s + S_I + S_{II}$$

Where S_f = final settlement

Ss =elastic compression of the foundation of structure.

$$= \frac{P + P_b}{2} \times \frac{D}{AE_P}$$

where,

P=Average load on each pile foundation

P_b = Average point resistance of each pile of foundation.

D = Length of pile

A = Area of cross-section of pile

E_p= Young's modulus of pile material

S_I and S_{II} are settlement along the embedded length and below pile tip respectively. The deformation and compressibility of the soil can be determined from consolidation tests on undisturbed samples of cohesive soil or from empirical correlations with penetration or pressure meter tests. In all cases the settlement must be estimated for normal and scoured conditions.

2.8 Load Test This shall be done as per Appendix D of IS: 2911 (Part IV) – 1979.

2.9 Capacity Of Pile Against Lateral Loadings:

2.9.1 The lateral load due to tractive/braking effort is transferred to the cap level along with a moment. The bending moment transferred at the pile cap level is shared by the piles in the group.

2.9.2 The piles should be considered as partially restrained at the pile cap level.

2.9.3 The deflection and the slope below scour depth can be calculated by

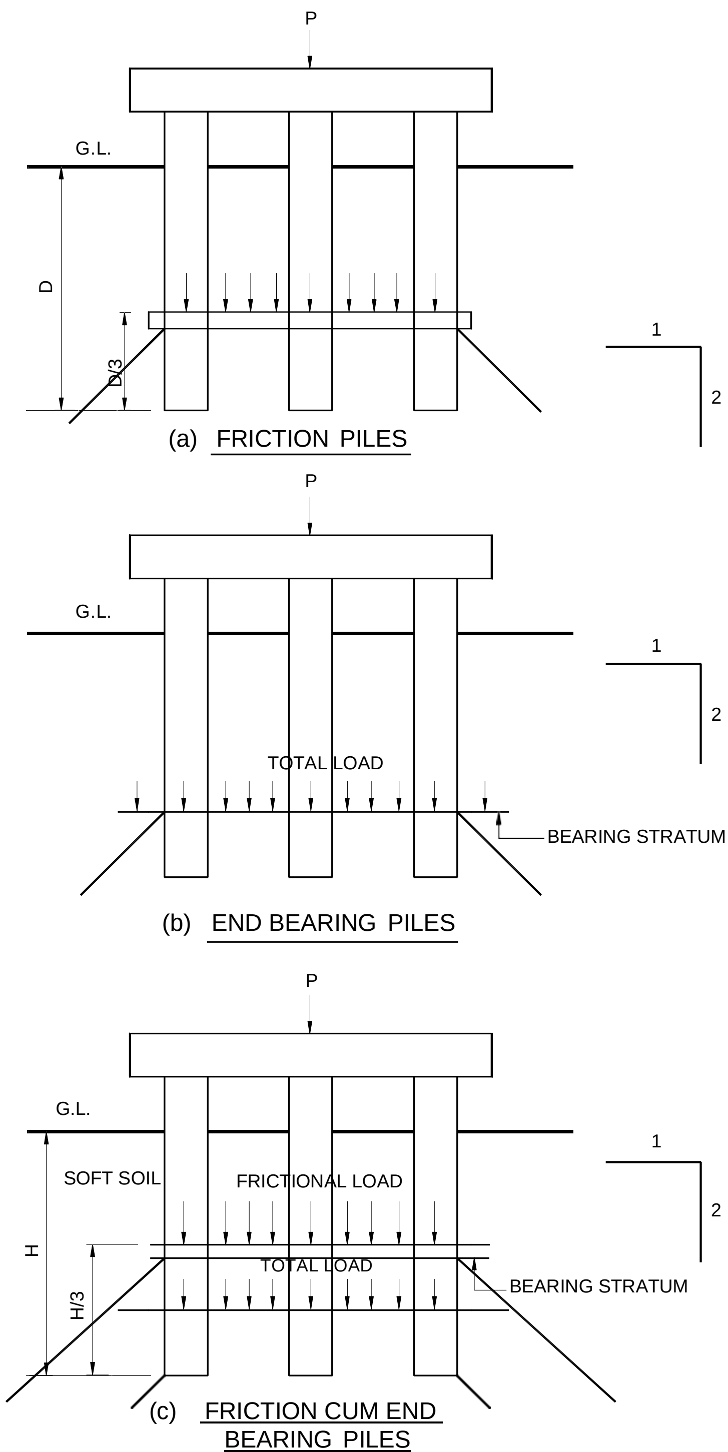


FIG. 5- APPROXIMATE SOLUTION

referring Reese and Matlock Curves (presented in 8th Texas Conference, 1956).

(These are included in the Hand Book on Soil Mechanics for Railway Engineers issued by RDSO).

2.9.4 The piles which are founded on rocks shall be designed as per method recommended by Poules in his article "Behaviour of Laterally Loaded Piles-III socketed piles" published in ASCE Vol 98-1972.