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SECTION 3.8. BEARING CAPACITY OF PILES

3.81 Determination of ultimate bearing capacity

The ultimate bearing capacity of timber, precast concrete or steel piles is most accurately determined from test loading, *vide* Item 3.16. In non-cohesive soils the probable bearing capacity may be deduced from one of the dynamic pile formulae. However, many of these are very unreliable and it is recommended that an approximate value may be obtained from the Hiley formula in accordance with Item 3.82. In cohesive soils an approximate value of the probable bearing capacity may be obtained by tests on soil samples in accordance with Item 3.811. Alternatively the bearing capacity of piles depending mainly on end-bearing resistance may be extrapolated from the results obtained from deep penetration tests as described in Item 3.811 (1).

Where piles are required to be driven in groups it may be advisable to apply test loads to groups of at least four piles placed at the intended spacing rather than to single piles. This procedure is recommended where piles are predominantly friction piles driven into cohesive soils (see Item 3.23). These tests should be carried out as described in Item 3.16. The remaining piles should be driven to the set or the depth indicated by the results of these tests.

The fundamental assumption made in all dynamic formulae is that the resistance of piles to further penetration under the permanent load has a direct relationship to their resistance to the impact of the hammer at the time of driving. Dynamic formulae may give reasonably accurate results in gravels, coarse sands, and similar deposits, which on account of their high permeability permit the free movement of their moisture content and therefore do not present a substantially different resistance to the impact forces of driving than to the subsequent permanent load.

Dynamic formulae are not applicable to deposits such as saturated silts, muds, and clays. In these soils the resistance to impact of the toe of the pile is exaggerated by their low permeability, while the frictional resistance on its sides during driving is reduced by lubrication.

The tendency of the ground to alter its resistance after driving, which is usual with the fine-grained soils, should be ascertained by re-driving the test piles and an occasional working pile after a period of rest. Loading tests on such piles immediately after driving therefore may tend to be misleading, and it is preferable to defer them for as long as possible.

Heaving of the ground or the lifting of adjacent piles already driven as the result of driving operations is an effect of low permeability in saturated soils.

The bearing capacity of friction piles embedded for their whole length in a uniform cohesive soil may be deduced approximately from laboratory tests of the soil as described in Item 3.811. That of driven cast-in-place piles may be determined as recommended above for other types of driven piles. When obtained from a pile formula, an increase in bearing capacity may be allowed where additional resistance can be developed by the friction of the finished pile against the surrounding soil after the casing has been withdrawn. However, formulae are not applicable to systems which provide an enlarged base to the foot of the pile.

The bearing capacity of bored cast-in-place piles should be obtained from test loadings or in accordance with Item 3.811. Alternatively, the safe load may be estimated from the known bearing capacity of other piles of similar dimensions and deriving their resistance from the same strata.

In practice, piles are often driven into a succession of different strata. In such cases the nature and thickness of the stratum in which the point of the pile rests will largely influence the carrying capacity. The characteristics, thickness and inclination of the strata underlying the pile points have a preponderating influence on the settlement of the structure as a whole. Reliance should not, therefore, be placed only on pile tests in such soils in estimating overall settlement. The possibility of settlement due to the consolidation of the soil below the pile points should be investigated and taken into account (see Items 1.34 and 1.35.)

In other cases the characteristics of the soil will be intermediate between those of non-cohesive and cohesive soils. In such cases considerable judgement is necessary in deciding on the means to be adopted in assessing the bearing capacity: test loadings should be carried out wherever practicable.

It is necessary to give an important warning with regard to piles driven through soft, sensitive clay. The driving of piles through such clay causes a remoulding of the material and renders it subject to settlement under its own weight. Due to this settlement the clay surrounding the piles will move downwards relative to them and will thus induce a negative, or downward-acting, skin friction. The total downward force due to this skin friction may be estimated as the cohesion of remoulded specimens of the clay multiplied by the surface area of the pile or the full weight of the soil between and around the piles, whichever is less, and should be deducted from the bearing capacity of a single pile as found by test or by formula in order to give the true bearing capacity of the pile. Provided that this precaution is taken, little trouble is likely to be experienced with piles driven through clay and into sand or gravel, but appreciable settlements may occur with driven piles embedded wholly in clay; in such cases bored piles may be preferable to driven piles and remoulding of the clay and the incidence of negative skin friction thereby avoided.

In deciding the safe load on piles driven through a recently-deposited fill, allowance should be made for the additional load which will be imposed on them by its continued consolidation. This need not exceed the full weight of the material which is likely to be transferred to a pile or pile group during the consolidation of the fill around the piles.

3.82 Dynamic pile formulae

These are based on the laws governing the dynamic impact of elastic bodies; they equate the energy of the hammer blow to the work done in overcoming the resistance of the ground to the penetration of the pile. Allowance is made for losses of energy due to the elastic contractions of the pile, cap and subsoil, as well as the losses caused by the inertia of the pile. One of the most used of these formulae is the Hiley formula.

The Hiley formula is:

$R = Wh\eta/(S + C/2)$

where

R is the ultimate driving resistance in kN

W is the weight of the ram in kN.

h is the height of the free fall of the ram or hammer (in mm), taken at its full value for trigger-operated drop hammers, 80% of the fall of normally-proportioned winch-operated drop hammers and 90% of stroke for single-acting hammers. When using the McKiernan-Terry type of double-acting hammers, 90% of the rated energy in kNmm per blow should be substituted for the product Wh in the formula. The hammer should be operated at its maximum speed whilst the set is being taken.

S is the final set or penetration per blow in millimetres.

C is the sum of the temporary elastic compressions (in mm) of the pile, dolly, packings, and ground, calculated or measured as prescribed below.

η is the efficiency of the blow, representing the ratio of energy after impact to striking energy of ram.
Where W is greater than Pe and the pile is driven into penetrable ground:
$\eta = (W+Pe^2)/(W+P)$
Where W is less than Pe and the pile is driven into penetrable ground:
$\eta = (W+Pe^2)/(W+P) - ((W-Pe)/(W+P))^2$
Values of η in relation to e and to the ratio P/W are tabulated in Appendix B.

P is the weight of the pile, anvil, helmet and follower (if any) in kN. Where the pile finds refusal in rock, 0.5P should be substituted for P in the above expressions for η.

e is the coefficient of restitution of the materials under impact as tabulated in Appendix B.

Where single-acting or drop hammers work in inclined leader guides, the percentages given in Table 4 should be deducted from the calculated bearing value.

Table 4. Reduced Bearing Values for Raking Piles

Rake	Per cent reduction
1 in 12	1.0
1 in 10	1.5
1 in 8	2.0
1 in 6	3.0
1 in 5	4.0
1 in 4	5.5
1 in 3	8.5
1 in 2	14.0

The temporary compression of the pile and ground occurring during driving should be determined by site measurements whenever possible, especially when the set is small, as described in Appendix C. The compression of the dolly and packing, as tabulated in the Appendix, should be added to the measured compression. When measurements cannot be taken, the temporary compressions of the pile and ground may be estimated from the values tabulated in that Appendix.

In calculating the value of the driving resistance from the Hiley formula it is first necessary to assume a value for the cross-sectional stress and to obtain the corresponding value of C. When R has been obtained, the cross-sectional stress should be checked with that previously estimated and the calculation repeated if necessary until agreement is obtained by trial and error. Another method of determining R directly is by expressing the value of C in terms of R and solving by the aid of the quadratic equation given in Appendix D.

To facilitate computations of driving resistance for various conditions in driving timber and reinforced concrete piles, tables and graphs* may be consulted for values based on the Hiley formula.

* R. V. Allin, MICE, *Resistance of Piles to Penetration* (2nd Ed.), E & FN Spon, 15 Bedford St., London WC2.

3.83 Driving stresses

The driving stresses in the pile are likely to be greatest at the head of the pile and can be estimated approximately by dividing the driving resistance by the cross-sectional area of the pile and multiplying this by $(2/\sqrt{\eta}) - 1$, where η is the efficiency of the blow, to obtain the peak head-stress value. For reinforced-concrete piles reference should be made to B.R.S. Technical Paper No. 20. (See also Item 3.71.)

As far as possible, raking piles should be supported during driving right down to the level at which they enter reasonably solid ground. Failing this, the additional stresses due to their spanning beyond the bottom end of the leaders and the further stresses due to the fact that the blow comes on the pile when it is deflected, should be considered.

3.86 Factor of safety

The factor of safety should be chosen after considering (a) the reliability of the ultimate resistance, (b) the type of superstructure and loading, and (c) the allowable settlement, both differential and total.

The ultimate resistance should be obtained whenever practicable from test loadings, as recommended in Item 3.16. If a sufficient proportion of the piles are tested in this manner, the data obtained from the tests may be used for adjusting the coefficients in the pile formula, which can then control the driving of the remaining piles. The ultimate resistance determined on this basis can be regarded as reliable.

When ultimate resistance is determined from the pile formula without loading tests, a greater factor of safety should be chosen. A still greater factor of safety is desirable if redriving tests show reduction in resistance, the reliability of the formula then being more doubtful.

The following Table is a guide of suitable values of the factor of safety for average conditions provided the allowable settlement is not thereby exceeded.

Table 6. Factors of Safety for Average Conditions

Type of ground	**Ultimate resistance determined from**		
	Test loading	**Formula only, resistance not reduced on redriving**	**Formula only, resistance reduced on redriving**
Rock	-	1½	-
Non-cohesive soil	1½-2	2	2½
Hard cohesive soil	1½-2	2	2½ or more*
Soft cohesive soil	1½-2	Not applicable	Not applicable

* A test load should be used in these circumstances.

When using a dynamic formula and if the resistance on redriving is reduced, it is particularly important that the temporary compressions in the pile and ground should be determined from field measurements and not from Table 8 in Appendix C.

The factors of safety tabulated above should be increased in unfavourable conditions, such as where:

(*a*) settlement must be limited or unequal settlement avoided, as for accurately-aligned machinery or a superstructure with fragile finishings;

(*b*) large impact loads are expected;

(*c*) piles derive their resistance mainly from skin friction and are driven in large groups, but the test loads were applied to single piles;

(*d*) the properties of the soil may be expected to deteriorate with time;

(*e*) the live load on a structure carried by friction piles is a considerable portion of the total load and approximates to the dead load in its duration.

On the other hand, a smaller factor of safety may be used for temporary work and for permanent work where large settlements are permissible.

APPENDIX B

Efficiency of Blow

The value of the coefficient of restitution, *e*, has been determined experimentally for different materials and conditions and is approximately as follows:

Piles driven with double-acting hammer

Steel piles without driving cap 0.5
Reinforced-concrete piles without helmet but with packing on top of pile 0.5
Reinforced-concrete piles with short dolly in helmet and packing 0.4
Timber piles 0.4

Piles driven with single-acting and drop hammer

Reinforced-concrete piles without helmet but with packing on top of piles 0.4
Steel piles or steel tube of cast-in-place piles fitted with driving cap and short dolly covered by steel plate 0.32
Reinforced-concrete piles with helmet and packing, dolly in good condition 0.25
Timber piles in good condition 0.25
Timber piles in poor condition 0.0

The efficiency of the blow given by the formula in Item 3.82 can be obtained from the following Table for various combinations of *e* with the ratio P/W, provided that W is greater than P*e* and the piles are driven into penetrable ground. For other cases and if the point of the pile is on rock, the efficiency should be calculated as specified in Item 3.82.

Table 7. Efficiency of Blow

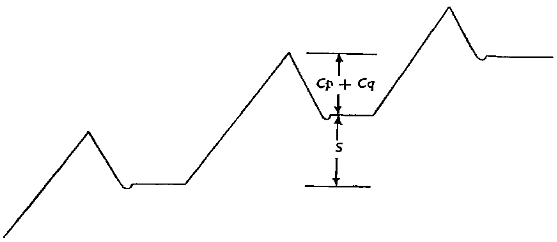
Ratio P/W	***e* = 0.5**	***e* = 0.4**	***e* = 0.32**	***e* = 0.25**	***e* = 0**
½	0.75	0.72	0.7	0.69	0.67
1	0.63	0.58	0.55	0.53	0.5
1½	0.55	0.5	0.46	0.44	0.4
2	0.5	0.44	0.4	0.37	0.33
2½	0.45	0.4	0.36	0.33	0.28
3	0.42	0.36	0.33	0.3	0.25
4	0.36	0.31	0.28	0.25	0.2
5	0.31	0.27	0.25	0.21	0.16
6	0.27	0.24	0.23	0.19	0.14

APPENDIX C
Temporary Compression

The total temporary compression C in the denominator of the Hiley formula in 3.82 is the sum of the elastic compressions in the pile head or dolly Cc, in the pile itself Cp and the quake of subsoil surrounding and under the pile Cq. Those in the pile and in the ground should be determined by field measurements whenever possible, especially in soft or peaty soils or where there is soft ground below the toe of the pile, as quake may then be much greater than tabulated values.

The observations are recorded on a card attached to the face of the pile while it is being driven, by slowly drawing a pencil along a straight edge placed against it. The straight edge should be held or fixed to two posts or piles at least 1.2 metres from the pile under observation, so as to be outside the zone of ground movement. The diagram obtained will be as sketched below in Fig. 2.

Fig. 2. Temporary Compression



Temporary compression Cc in the pile head and cap cannot be determined by field measurements and is obtained from the Table. If the record card is a distance below the pile top, elastic compression in this length of pile (calculated from the Table) must be added to Cc.

For preliminary calculations of total compression C or when measurements are not made, the compressions Cp in the pile itself and Cq of the ground must all be obtained from the Table. (No allowance for quake of the ground need be made if the pile has penetrated to rock.)

The comparative hardness of driving is expressed in terms of the compressive stress on the pile or shoe:

Easy driving 3.5N/mm²
Medium driving 7N/mm²
Hard driving 10N/mm²
Very hard driving 14N/mm²

For steel piles, tubes, or mandrels the stress is governed by the steel cross-sectional area and the four driving stresses are taken as 50, 100, 150 and 200N/mm² respectively. When such piles are driven by a double-acting hammer without driving cap, Cc is zero. If a dolly is used, the stress in the steel must be divided by the ratio of total area of pile to net area of steel to determine the hardness of driving for finding Cc.

In calculating elastic compression of the pile, Cp, L is the length to the assumed centre of driving resistance, i.e. for a pure friction pile driven through homogeneous material it is the distance between the head of the pile and half the penetration; for a purely end-bearing pile it is the whole length of the pile. The modulus of elasticity for concrete is given for Portland-cement concrete and will be greater for rapid-hardening or HAC concrete.

APPENDIX D Equation for Calculating Resistance

When the set S in the 3.82 formula is small or zero, resistance to penetration R is approximately proportional to temporary compression C. Thus if $R = mC$, m is approximately constant for any particular pile.

Substituting for C in the formula

$$R = Wh\eta/(S + (R/2m))$$

The solution of this equation for R is

$$R = \sqrt{(Wh\eta \times 2m) + (mS)^2} - mS$$

When the pile is driven to refusal, S is zero, so

$$R = \sqrt{Wh\eta \times 2m}$$

To determine m, the total compression C is first obtained from the Table in Appendix C for a stress corresponding to the anticipated resistance R. m is then found by dividing R by C. When possible the compressions in the pile and in the ground, Cp and Cq, are determined by observation and only the compression in the pile cap Cc is taken from Table 8.

Table 8. Temporary Compressions (mm)

Form of compression	Material	Easy driving	Medium driving	Hard driving	Very hard driving
Pile head and cap, ***Cc***	Head of timber pile	1.3	2.5	3.8	5
	Short dolly in helmet or driving cap.*	1.3	2.5	3.8	5
	75mm packing under helmet or driving cap.*	1.8	3.8	5.6	7.6
	25mm pad only on head of reinforced-concrete pile.	2.0	1.3	1.8	2.5
Pile length, ***Cp***	Timber pile (E = 10kN/mm²)	0.33L	0.67L	1.0L	1.3L
	Pre-cast concrete pile (E = 14kN/mm²)	0.25L	0.5L	0.75L	1.0L
	Steel pile, steel tube, or steel mandrel for cast-in-place pile (E = 205kN/mm²)	0.25L	0.5L	0.75L	1.0L
Quake ***Cq***	Ground surrounding pile and under pile point.	1.3	1.3 to 2.5	3.8 to 6.4	1.3 to 3.8

* If these devices are used in combination, the compressions should be added together.
Pile length L is measured in metres