The Adhesion of Piles Driven in Clay Soils

L'Adhérence des Pieux Battus dans des Sols Argileux

by M. J. TOMLINSON, A.M.I.C.E., George Wimpey & Co. Ltd., Central Laboratory, Southall, Middlesex, England

Summary

The results of an analysis of loading test data on piles driven into clay soils show that the 'percentage adhesion', i.e. the observed adhesion expressed as a percentage of the undisturbed cohesion of the clay, falls with increasing stiffness of the clay from approximately 100 in very soft clays to 20 in very stiff clays.

The loss of adhesion is not related to loss of strength by remoulding, but is believed to be due to the presence of a partial gap between the pile and the soil. This gap may be formed by transverse vibrations during driving and by movement of the displaced soil upwards and away from the pile. Whereas in soft clays the heaved-up soil will re-consolidate and close up the gap, thus giving 100 per cent adhesion, firm and stiff clays will only partially re-consolidate and a soft clay slurry can be formed around part of the pile shaft. It is thought that tapered piles may show appreciably higher percentage adhesion than straight-sided piles.

Sommaire

L'analyse des résultats des essais de chargement sur pieux enfoncés dans des sols argileux montre que l'adhérence observée, exprimée en pourcentage de la cohésion d'une argile intacte, décroît approximativement de 100, pour des argiles très molles, à 20, pour des argiles compactes.

Cette perte d'adhérence n'est pas en relation directe avec la perte de résistance due à la modification de structure de l'argile, mais on suppose qu'elle provient de l'existence d'un vide partiel entre le pieu et le terrain, vide pouvant se produire sous l'influence des vibrations transversales en cours de battage et du mouvement ascendant du terrain déplacé qui s'écarte du pieu. Lorsqu'il s'agit d'argile molle, le terrain soulevé est partiellement consolidé à nouveau et remplit le vide, et donne ainsi 100 pour cent d'adhérence. Dans les argiles consistantes et tenaces, le terrain n'est que partiellement reconstitué et une argile molle demi-fluide peut se former autour du fût du pieu. On admet qu'il est possible qu'un pieu cônique ait un pourcentage d'adhérence plus fort qu'un pieu cylindrique.

Introduction

The ultimate carrying capacity of a pile driven into a clay soil is given by the formula:

$$Q_f = N_c A_b c_b + C.L.c_a$$

where Q_f = ultimate carrying capacity of pile; N_c = bearing capacity factor; A_b = area of base of pile; c_b = cohesion at base of pile; C = circumference of pile; L = length of pile; and c_a = adhesion between pile and soil.

It has generally been assumed that the adhesion between the pile and soil c_a is equal to the remoulded cohesion c_r when the piles are loaded soon after driving, or the full undisturbed cohesion c when the piles are not loaded until the remoulded clay has had time to regain its original strength. These assumptions were mainly based on loading tests made on full-scale piles driven in soft clays or on model piles pushed into soils of varying stiffness. Until recent years there have been little published data on the results of loading tests made on full-scale driven piles in stiff clays accompanied by details on the cohesion and other characteristics of the clays. However, the fact that the full undisturbed or remoulded strength of stiff clays is not mobilized in adhesion is well known to engineers experienced in piling. The widely used 'static' formula in Arrol's Handbook does not give values of adhesion higher than 450 lb./sq. ft. for 'compact hard clay'. Measured values of skin friction quoted by CHELLIS (1944) have a maximum of 1930 lb./sq. ft. and the majority of the values are much lower than this. As a result of a paper by MORTENSEN (1948), it was suggested by MEYERHOF (1951) that there was an upper limit of adhesion of 1 ton/sq. ft. for piles driven into clays. In a paper prepared in 1952 by RODIN and the author (1953), it was concluded that the ratio of the observed adhesion to the theoretical maximum adhesion (assuming the full cohesion of the clay on the embedded surface of the clay) deceased with increasing stiffness of the clays.

Since the publication of the last mentioned paper the results of more loading tests have become available, and the whole of the data have been re-analysed.

Analysis of Loading Test Data

An analysis of the results of loading tests made on 56 driven piles in clays is shown in Table 1. All published and unpublished data available to the author have been used in compilation, but in the interests of reasonable accuracy and reliability the data included in the Table have been limited to: (a) piles loaded to failure or sufficiently near failure to extrapolate the failure load with reasonable certainty; (b) piles known to terminate in clay; (c) piles driven wholly within clay strata, or through clays only containing relatively thin sand and gravel layers—an exception to this is the group of piles 40 to 56 which were driven through a 22 ft. thick layer of sand, and are included to give additional data on stiff clays which are generally rather scanty.

To determine the load carried by the pile in adhesion Q_a it was necessary to allow for the end resistance of the piles Q_b. This was calculated from:

$$Q_b = 9.c_b.A_b \quad \ldots. (1)$$

and

$$Q_a = Q_f - Q_b \quad \ldots. (2)$$

The bearing capacity factor N_c of 9 has been found by research and experiment to give a reasonably accurate assessment of the end resistance of bored or driven piles in clays (MEYERHOF and MURDOCK, 1953).

The theoretical adhesion Q_c was calculated by multiplying the embedded surface area of the pile shaft by the undisturbed cohesion of the soil; the undisturbed strength was used rather than the remoulded strength, since the majority of sensitive clays regain most of their undisturbed strength in a comparatively short time, and the engineer is usually interested in the final rather than the immediate carrying capacity of a pile. The percentage of the undisturbed cohesion of a clay actually mobilized in adhesion $Q_a/Q_c \times 100$ (this will be referred to henceforth as 'percentage adhesion' f) has been plotted against the undisturbed cohesive strength of the clay in Fig. 1.

The effect of the slenderness ratio L/D of the pile on the percentage adhesion was investigated by plotting actual values of Q_a/Ac against L/D in Fig. 2. This relationship is based on the equation

$$Q_a = c_a \pi D L \quad \ldots\ldots (3)$$

Since

$$c_a = \frac{fc}{100}, \quad Q_a = \frac{fc}{100} \cdot \pi D L$$

If $A = \pi D^2/4$ is the average cross-section of the pile

$$Q_a = \frac{fc}{100} \cdot \frac{4AL}{D}$$

Hence

$$\frac{Q_a}{Ac} = \frac{4fL}{100D} \quad \ldots\ldots (4)$$

or

$$\frac{\frac{Q_a}{Ac}}{\frac{L}{D}} = \frac{4f}{100} \quad \ldots\ldots (5)$$

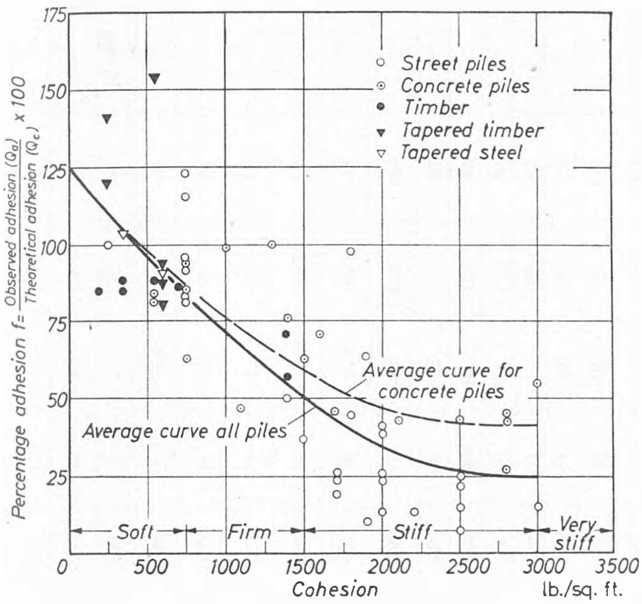

Fig. 1 Relationship of observed adhesion (expressed as a percentage of theoretical adhesion) to cohesive strength of clay
Rapport de l'adhérence observée (exprimée comme pourcentage de l'adhérence théorique) en fonction de la cohésion de l'argile

This relationship has been plotted in Fig. 2 for different values of f, and hence different values of c (from the average curve relating f and c in Fig. 1). The full line represents the relation between Q_a/Ac and L/D when the full cohesion is mobilized in adhesion, i.e. $f = 100$ per cent, corresponding to $c_a = c = 450$ lb./sq. ft. The broken lines represent equation 5 for values of f corresponding to the strength limits of soft, firm, and stiff clay.

The relationship between average observed adhesion and cohesive strength has been plotted in Fig. 3, using the average curves for all piles and concrete piles, respectively, in Fig. 1.

Discussion on Adhesion in Relation to Characteristics of Clay Soils

The soil displaced by a driven pile is not wholly heaved up at the surface, but is partly compressed elastically and partly consolidated in a zone around the pile. The elastic compression is relatively insignificant, but in a highly compressible soil the amount of ground heave may not be large in relation to the volume of soil consolidated around the pile. Ground heave is resisted by overburden pressure.

Consolidation of the soil around the pile is, of course, accompanied by expulsion of pore-water. Seed and Reese (1955) embedded pore-water pressure gauges at various depths in an experimental pile. They found that an excess hydrostatic pressure was built up at the surface of the pile which was finally dissipated 600 hours after driving the pile. The excess pore-water is dissipated partly by radial flow into the surrounding soil, partly into the material of the pile itself (if permeable) and partly into the gap which tends to form between the pile and the soil as a result of transverse vibrations during driving.

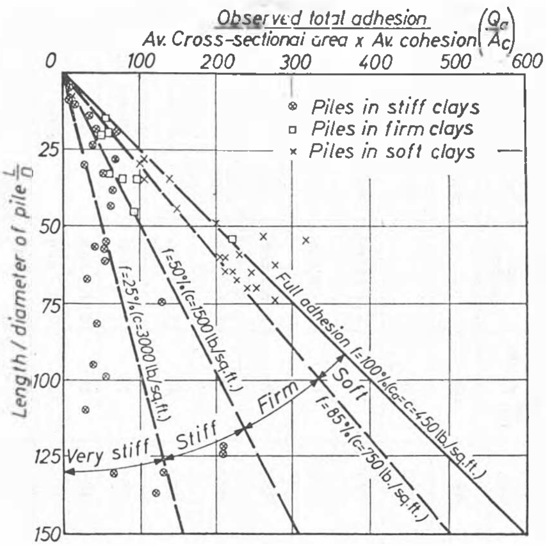

Fig. 2 Relationship between observed adhesion and pile length
Rapport de l'adhérence observée en fonction de la longueur du pieu

The soil displaced by the pile is wholly or partially remoulded close to the pile. In sensitive clays this causes a very marked loss of cohesion, but glacial clays such as boulder clay are usually insensitive to the effects of remoulding and certain stiff fissured clays (e.g. London clay) actually gain strength due to the elimination of fissuring by remoulding.

The results of pile loading tests at increasing intervals of time after driving all show that 90 per cent or more of the undisturbed

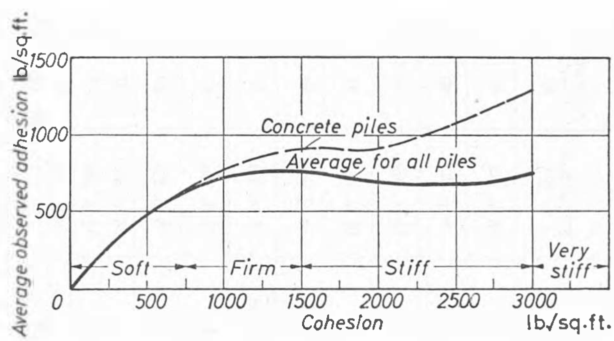

Fig. 3 Limiting adhesion for soft to stiff clays
Limites de l'adhérence de différentes argiles

strength of sensitive clays is regained 30 to 50 days after driving (Meyerhof, 1951; Seed and Reese, 1955; and Fellenius, 1955). Seed and Reese (1955) measured a 50 per cent gain in shear strength over the undisturbed shear strength of the clay close to their experimental pile.

It is evident from Fig. 1 that the percentage adhesion decreases in a marked manner with increasing stiffness of the clay. Steel piles appear to have a lower percentage adhesion

Table 1
Pile loading test data
Essais de chargement sur pieux

Pile No.	Location	Type	Size at base	Embedded length ft.	Cohesion at base c_b lb./sq. ft.	Average cohesion along shaft c lb./sq. ft.	Ultimate load Q_f ton	Estimated cohesion Q_a ton	Theoretical adhesion Q_c ton	Percentage adhesion $\frac{Q_a}{Q_c} \times 100$	Time between driving and loading	Soil type	Remarks
1	Detroit, U.S.A.	Steel pipe	16 in. dia.	65	300	300	37·5	36	35	103	—	Soft clay	Taper 1 in. in 10 ft. (*Cummings et al., 1950*)
2	Mare Island, U.S.A.	Tapered timber	8 in. dia.	80	1100	560	100	95·5	62·5	153	—	Soft clay	Taper 1 in. in 10 ft. (*Moore, 1949*)
3	Gothenburg, Sweden	ditto	6¼ in. dia.	36·5	300	250	11	10·5	9·0	117	1 yr. 11 mth.	Soft silty clay	Top dia. 10 in. (*Fellenius, 1955*)
4	ditto	ditto	6¼ in. dia.	37	300	250	13	12·5	9·0	140	2 yr. 0 mth.	ditto	Top dia. 10 in. (*Fellenius, 1955*)
5	ditto	Tapered sheet iron	7 in. dia.	49	410	600	33	32·0	35·5	90	3 yr. 2 mth.	ditto	Top dia. 14 in. *Fellenius, 1955*)
6	ditto	Tapered timber	5½ in. dia.	49	410	600	23	22·5	29·5	76	3 yr. 2 mth.	ditto	Top dia. 11 in. (*Fellenius, 1955*)
7	ditto	ditto	6 in. dia.	43	400	610	27	26·5	28·5	93	2 yr. 1 mth.	ditto	Top dia. 11 in. (*Fellenius, 1955*)
8	ditto	ditto	6 in. dia.	49	400	610	27	26·5	30·0	88	3 yr. 1 mth.	ditto	Top dia. 10¼ in. (*Fellenius, 1955*)
9	ditto	Timber	7½ in. dia.	37	400	350	11	10·5	12·0	88	2 yr. 0 mth.	ditto	(*Fellenius, 1955*)
10	ditto	ditto	7¼ in. dia.	36	400	350	10	9·5	11·0	86	2 yr. 7 mth.	ditto	(*Fellenius, 1955*)
11	ditto	Precast concrete	8 in. sq.	43	380	550	25	24·0	28·5	84	3 yr. 1 mth.	ditto	(*Fellenius, 1955*)
12	ditto	ditto	8 in. sq.	42	380	550	24	23·0	28·0	82	3 yr. 1 mth.	ditto	(*Fellenius, 1955*)
13	ditto	Timber	10¼ in. sq.	60	410	550	46	45	51·5	87	1 yr. 1 mth.	ditto	(*Fellenius, 1955*)
14	San Francisco, U.S.A.	Steel pipe	6½ in. dia.	14	250	250	2·7	2·5	2·5	100	33 days	ditto	Experimental pile (*Seed and Reece, 1955*)
15	Morganza, U.S.A.	ditto	24 in. dia.	58·5	750	750	170	160·5	130	124	—	—	(*U.S. War Dept., 1950*)
16	ditto	ditto	12 in. dia.	67·5	750	750	60	58	70	83	—	—	(*U.S. War Dept., 1950*)
17	ditto	ditto	18 in. dia.	67	750	750	90	85	104	82	—	—	(*U.S. War Dept., 1950*)
18	ditto	ditto	24 in. dia.	67·5	750	750	140	131	140	93	—	—	(*U.S. War Dept., 1950*)
19	ditto	ditto	30 in. dia.	67·5	750	750	180	165	174	95	—	—	(*U.S. War Dept., 1950*)
20	ditto	Precast concrete	22 in. sq.	64·5	750	750	140	130	156	83	—	—	(*U.S. War Dept., 1950*)
21	Richmond, Cal., U.S.A.	ditto	18 in. × 24 in.	25·5	1000	1300	115	103	103	100	—	—	(*Moore, 1949*)
22	Isle of Grain, G.B.	ditto	14 in. sq.	21·5	1900	1400	58	47·5	62·5	76	—	45 ft. soft clay 17 ft. stiff clay	—
23	ditto	ditto	12 in. sq.	18	1900	1500	38	30·5	48·5	63	—	3·5 ft. soft clay 14·5 ft. stiff clay	—
24	ditto	ditto	12 in. sq.	20	1800	750	24	17	27	63	—	13 ft. soft clay 7 ft. stiff clay	2 ft. layer of sand in soft clay
25	ditto	ditto	12 in. sq.	33	1710	1100	37	30	64	47	—	14 ft. soft clay 19 ft. stiff clay	—
26	ditto	ditto	12 in. sq.	55	1900	1000	105	97·5	99	98	—	30 ft. soft clay 25 ft. stiff clay	—
27	ditto	Steel tube	12¾ in. dia.	20	—	1800	53	53	54	98	16 days	Stiff clay	Adhesion by pull test was 53 ton
28	Grimsby, G.B.	Timber	14 in. sq.	40	2350	1400	80	67	118	57	—	17 ft. soft clay 23 ft. stiff clay	—
29	ditto	ditto	14 in. sq.	40	2350	1400	95	82	114	71	—	18 ft. soft clay 22 ft. stiff clay	—
30	ditto	ditto	14 in. sq.	8¾	200	200	4	3	3·5	85	—	Soft clay	—

(Table 1 *continued*)

31	Bruntingthorpe, G.B.	Precast concrete	14 in. sq.	45	1800	2500	112	102	235	43	—	Stiff clay with firm layers	—
32	Barnet, G.B.	ditto	12 in. sq.	26	4600	3000	78	60	142	55	9 mth.	Stiff to very stiff clay	Ultimate load 1 month after driving was 96 tons (*Meyerhof and Murdock, 1953*)
33	ditto	ditto	12 in. sq.	13·5	3000	1600	39	27	38	71	9 mth.	Stiff clay	Ultimate load 1 month after driving was 42 tons (*Meyerhof and Murdock, 1953*)
34	Ellesmere Port, G.B.	ditto	14 in. sq.	39	2500	2100	86	71·5	170	42	20 days	13 ft. soft clay and sand 26 ft. stiff clay and peat	(*Rodin and Tomlinson, 1953*)
35	ditto	ditto	14 in. sq.	36	2500	2500	60	45·5	186	24	5 days	13 ft. soft clay and sand 23 ft. stiff clay and peat	(*Rodin and Tomlinson, 1953*)
36	ditto	ditto	14 in. sq.	27	1450	1700	52	44	96	46	37 days	4·5 ft. fill 2 ft. firm clay 7 ft. peat 14 ft. stiff clay	(*Rodin and Tomlinson, 1953*)
37	ditto	ditto	24 in. dia.	16	2700	2800	68	34	125	27	—	Stiff clay	(*Rodin and Tomlinson, 1953*)
38	ditto	ditto	24 in. dia.	21	2700	2800	105	71	164	43	—	Stiff clay	Pile 37 jacked from 16 ft to 21 ft. (*Rodin and Tomlinson, 1953*)
39	ditto	ditto	24 in. dia.	10	2700	2800	70	35	80	45	—	Stiff clay	
40	Cleveland, U.S.A.	Steep pipe (open end)	10¾ in. dia.	65	2250	1800	71	66	150	44	—	22 ft. sand and silt 43 ft. stiff clay	(*Peck, 1954*)
41	ditto	ditto	10¾ in. dia.	85	6000	1900	36	21	200	10	—	22 ft. sand and silt 63 ft. stiff – stiff clay	(*Peck, 1954*)
42	ditto	ditto	10¾ in. dia.	89	6000	2500	54	39	280	14	—	22 ft. sand and silt 67 ft. stiff – stiff clay	(*Peck, 1954*)
43	ditto	ditto	10¾ in. dia.	93	6000	2500	36	21	300	7	—	22 ft. sand and silt 71 ft. stiff – stiff clay	(*Peck, 1954*)
44	ditto	ditto (closed end)	10¾ in. dia.	111	3500	2000	80	71	280	25	—	22 ft. sand and silt 89 ft. stiff – stiff clay	(*Peck, 1954*)
45	ditto	ditto	10¾ in. dia.	111	3500	2000	125	116	280	41	—	ditto	(*Peck, 1954*)
46	ditto	ditto	10¾ in. dia.	111	3500	2000	125	116	280	41	—	ditto	(*Peck, 1954*)
47	ditto	ditto (open end)	10¾ in. dia.	122	3000	2000	45	38	290	13	—	22 ft. sand and silt 96 ft. stiff clay	(*Peck, 1954*)
48	ditto	ditto	10¾ in. dia.	122	2500	2500	90	84	380	22	—	22 ft. sand and silt 100 ft. stiff clay	(*Peck, 1954*)
49	ditto	ditto (closed end)	16 in. dia.	57	2250	1500	71	59	160	37	—	22 ft. sand and silt 35 ft. stiff clay	(*Peck, 1954*)
50	ditto	ditto	16 in. dia.	61	2250	1400	90	78	160	49	—	22 ft. sand and silt 39 ft. stiff clay	(*Peck, 1954*)
51	ditto	ditto	16 in. dia.	73	2250	1700	71	59	230	26	—	22 ft. sand and silt 51 ft. stiff clay	(*Peck, 1954*)
52	ditto	ditto	16 in. dia.	73	2250	1700	71	59	230	26	—	ditto	(*Peck, 1954*)
53	ditto	ditto	16 in. dia.	76	3000	1700	62	46	240	19	—	22 ft. sand and silt 54 ft. stiff clay	(*Peck, 1954*)
54	ditto	ditto	16 in. dia.	82	5000	2000	100	72	300	24	—	22 ft. sand and silt 60 ft. stiff – stiff clay	(*Peck, 1954*)
55	ditto	ditto	16 in. dia.	89	6000	2200	80	47	370	13	—	22 ft. sand and silt 67 ft. stiff – stiff clay	(*Peck, 1954*)
56	ditto	ditto	16 in. dia.	109	3000	3000	100	84	610	14	—	22 ft. sand and silt 87 ft. stiff clay	(*Peck, 1954*)

Notes:—1 ton = 2240 lb. Estimated adhesion Q_a = Ultimate load (Q_f)−Base resistance (Q_b). Base resistance $Q_b = 9 \times$ base area $\times c_b$. Base resistance for tapered piles calculated from area at ⅓L from tip. Theoretical adhesion Q_c = surface area of shaft × average undisturbed cohesion (c).

than concrete piles in the firm to stiff clay zone, but there is little difference between steel, concrete or timber piles in the soft to firm clay zone. Since all the results for steel tube piles shown for the stiff clay zone are from the same site (PECK, 1954) it would perhaps be unwise to conclude from this limited evidence that the adhesion of steel piles is markedly inferior to that of concrete piles.

Remoulding of the clay cannot account for the low adhesion, since all the results shown in the firm to stiff clay zone for timber and concrete piles were for piles driven into London clay or insensitive boulder clays. The results for the steel tube piles (40 to 56) were in glacial clay with a sensitivity of $1\frac{1}{2}$ to 3.

The author believes that the low values of adhesion can only be accounted for by the presence of a gap between the parts of the pile and the clay.

Formation of Gap between Pile and Soil

There can be little doubt that a gap does form to some extent between the pile and the surrounding soil during driving. GLANVILLE, GRIME and DAVIES (1935) stated that transverse vibrations of a pile occur due to eccentricity of hammer blows but they found it impossible to measure them. Strain gauges embedded at intervals down the length of an experimental steel pile in researches by SEED and REESE (1955) showed that three hours after pile driving the ultimate load was only 30 per cent of the ultimate load calculated from the fully remoulded cohesion, and that the upper two-thirds of the pile carried practically no load in adhesion. However, 33 days after driving, the adhesion was equal to 102 per cent of the undisturbed cohesion, but there was little adhesion over the top 3 ft. of the pile. They concluded that 'it is possible that this soil was displaced an extra amount by the vibration of the pile and that the overburden pressure was insufficient to cause bond to develop'. This formation of a gap has been noted by other observers including FABER (1947). In soft clays the gap will readily close up due to the clay slumping down around the pile, thus giving the marked 'take up' effect well known in soft clays: however, in stiff clays the gap will not close up—an unlined bore hole in a stiff clay will remain open indefinitely. The result of tests on piles in stiff London clay (piles 32 and 33) show that the ultimate load 9 months after driving was only 80 to 90 per cent of the ultimate load one month after driving (MEYERHOF and MURDOCK, 1953).

It is possible that the lower values indicated for long steel tube piles in firm to stiff clays as compared with piles of other materials may be due to the greater flexibility of such piles. However, if the gap formed by the 'whip' during pile driving is the explanation for the low adhesion, it would be expected that the longer the pile the greater the adhesion, since the gap should be widest near the surface and the transverse vibrations should tend to be damped down with increasing depth until they are fully taken up by lateral elastic compression of the soil. This hypothesis is not, however, substantiated by Fig. 2. There is no tendency for the average curve for stiff clays to diverge to the right for values of L/D up to 100. The author believes that although the gap caused by vibration does account for some of its loss of adhesion, there is an additional cause, namely the effect of ground heave around the piles.

The Effect of Ground Heave

When piles are driven in soft and stiff clays, a heave of the ground surface occurs. In the discussion on the paper by CUMMINGS, KERKHOF and PECK (1950), Zeevaert reported a surface heave of 13 in. in the centre of a 4460 sq. ft. area into which 187 piles were driven. The soil was a soft volcanic clay and over a period of 10 months after driving the surface subsided to four in. above the original level. Thus the ground had reconsolidated around the piles. Also in the discussion on this paper, Legget remarked on the appearance of driving piles into hard Leda clay in Canada. The hard clay heaved up around the pile to a distance of about 2 ft. radially from the pile surface. The clay at the surface tended to break up into lumps and a cream-like slurry was squeezed up in a 1-in. wide annular space around the pile. He did not remark on any subsequent subsidence of the heaved-up zone, but it is evident that the broken-up clay together with the softened zone around the pile could not re-consolidate to the original undisturbed strength. In his account of piling operations in the Cuyahoga River Valley, PECK (1954) reported a heave of 12 to 15 in. in the centre of a group of 164 steel pipe piles in stiff clay. Experiments by L'HERMINIER (1953) with driving piles into sands in a glass-fronted box showed that heave took place in a damp sand as illustrated in Fig. 4. It is possible that heave in a stiff clay takes place in a similar manner with the soil being displaced upwards and away from the pile over most of its length. Thus the loss of adhesion due to the ground heave effect will be independent of the pile length. Movement of the heaved clay away from the pile is related to driving vibrations. The greater the vibrations (including longitudinal vibrations) the greater will be the tendency for masses of clay in the form of plugs to become detached from the pile as they are displaced upwards.

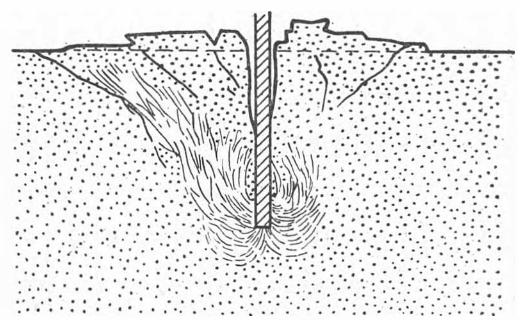

Fig. 4 Heave of damp sand due to driving of model pile (after L'Herminier)

Soulèvement du sable légèrement humide par le battage d'un pieu-modèle

The gap formed by the clay breaking away from the pile will serve as a seepage path for excess hydrostatic pressure set up at lower depths and for the percolation of water from water-bearing layers or fissures in the clay or from the surface. Thus a permanently softened zone around the pile may result.

Effect of Pile Shape

The results in Table 1 do not suggest any increasing adhesion with increasing diameter. Although by increasing the diameter of piles the amount of consolidation should be greater, in practice the amount of consolidation is small in relation to the ground heave.

Piles which displace a minimum of ground (e.g. open-ended tubes) should again in theory show less ground heave effects. However, in practice a mass of consolidated clay is taken down inside open-ended tubular piles or inside the flanges of H-beam piles. Therefore, the surface area available for skin friction is no greater than that given by the net perimeter of the pile.

On the other hand, tapered piles should show an increase in adhesion due to the continual closing up of gaps formed by transverse vibrations and the compaction of any loose heaved up soil around the pile. The results for tapered piles in Fig. 1 are unfortunately restricted to soft clays but there is a tendency to a higher percentage adhesion in these clays. The author has no data available on tapered piles in firm or stiff clays. MORTENSEN (1948) found a 50 per cent higher pulling resistance for tapered piles than for straight-sided piles both in full-scale field and in laboratory model tests.

Conclusions

It has been shown that the percentage adhesion (expressed as the percentage of the observed adhesion to the full cohesion of the clay) falls with increasing stiffness of the clay. Tentative design criteria for straight-sided piles are given in Table 2.

Table 2

Material of pile	*Cohesion of clay c* (lb./sq. ft.)	*Adhesion of pile to clay* c_a* (lb./sq. ft.)
Concrete and timber	*Soft* 0–750	0–700
Concrete and timber	*Firm* 750–1500	700–900
Concrete and timber	*Stiff* 1500–3000	900-1300
Steel	*Soft* 0–700	0–600
Steel	*Firm* 750–1500	600–750
Steel	*Stiff* 1500–3000	Inconclusive

* Calculated on whole of embedded surface of shaft

The adhesion of tapered piles may be higher than given in Table 2, but the author has no factual data to confirm this.

It is believed that the loss in adhesion in firm to stiff clay can be accounted for by the formation of a partial gap between the pile and the soil. This gap is formed partly by transverse vibrations set up in pile driving and partly by flow of the displaced clay upwards and away from the pile. In soft clays the gap will tend to close as the heaved-up soil re-consolidates around the pile. In these soft soils the ultimate adhesion may be higher than the original undisturbed cohesion of the clay due to consolidation and gain in strength of the clay around the pile. However, in firm, and particularly in stiff, clays the soil does not fully re-consolidate around the pile. In some circumstances a skin of softened clay may form around the pile and the ultimate adhesion may be no more than that of a normally consolidated clay under the same overburden pressure. There is no evidence to show that, after initial take-up, the adhesion will increase to any appreciable extent with increasing time after driving.

It is suggested that further research could be usefully done into adhesion of piles in very stiff clays and especially the adhesion of tapered piles in such clays.

References

CHELLIS, R. D. (1944). *Pile Driving Handbook*, pp. 175–178 and 190. New York; Pitman

CUMMINGS, A. E., KERKHOF, G. O. and PECK, R. B. (1950). Effect of driving piles in soft clay. *Trans. Amer. Soc. Civ. Eng.*, **115**, 275

FABER, O. (and discussion) (1947). A new piling formula. *J. Inst. Civ. Eng.*, **5**, 5

FELLENIUS, B. (1955). *Results of tests on piles at Gothenburg Railway Station.* Geotechnical Dept. Swedish State Railways, Bulletin No. 5

GLANVILLE, W. H., GRIME, G. and DAVIES, W. W. (1935). The behaviour of reinforced concrete piles during driving. *J. Inst. Civ. Eng.*, **1**, 150

L'HERMINIER, R. (1953). Remarques sur le poinçonnement continu des sables et graviers. *Annales de L'Institut Technique du Batiment et des Travaux Publics*, *6e année*, No. 63–4, p. 377

MEYERHOF, G. G. (1951). The ultimate bearing capacity of foundations. *Géotechnique*, **2**, (4), 301

— and MURDOCK, L. J. (1953). An investigation of the bearing capacity of some bored and driven piles in London clay. *Géotechnique*, **3**, (7), 267

MOORE, W. W. (1949). Experiences with predetermining pile lengths. *Trans. Amer. Soc. Civ. Eng.*, **113**, 351

MORTENSEN, A. (1948). Predetermination of the resistance of a pile on the basis of field shear and loading test values. *Proc. 2nd International Conference on Soil Mechanics and Foundation Engineering*, Vol. 2, p. 180

PECK, R. B. (1954). Foundation conditions in the Cuyahoga River Valley. *Proc. Amer. Soc. Civ. Eng.*, **80** (separate No. 513)

— HANSON, W. E. and THORNBURN, T. H. (1953). *Foundation Engineering*, p. 184. New York; Wiley

RODIN, S. and TOMLINSON, M. J. (1953). Recherches sur la frottement latéral des pieux forés et battus dans l'argile. *Annales de L'Institut Téchnique du Batiment et des Travaux Publics, 6e année*, No. 63–64, p. 343

SEED, H. B. and REESE, L. C. (1955). The action of soft clay along friction piles. *Proc. Amer. Soc. Civ. Eng.*, **81**, 842

U.S. War Dept. (1950). *Pile loading tests, combined Morganza floodway control structure.* U.S. Waterways Experimental Station, Tech. Memo 3–308