

7764 From theory to field experience with the non-destructive vibration testing of piles

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The Paper gives a brief outline of the principle of a non-destructive pile testing method by vibration and presents a summary of the parameters which may be obtained from velocity response graphs for cylindrical concrete piles excited by a vibrator. Consideration of the behaviour of ideal piles provides a means of interpreting these parameters. The effect of important defects in piles on the shape of their response graphs is considered by computer analysis of an electrical analogue of their mechanical behaviour. Examples of vibration test results on piles at five sites are given (two in France and three in Great Britain). With a knowledge of the geotechnical conditions of the site and of the type of pile used, it is shown that information can be obtained from the vibration tests about the length and continuity of the piles, their concrete quality and the quality of the anchorage of the pile in the ground. This information is obtained from comparison of test results across any given site, and not from unique tests. A correlation between measured pile head stiffness and pile shaft diameter is suggested for end bearing piles 5-15 m long passing through soft deposits.

Control testing of bored cast in situ piles

Pile loading tests have, until now, been accepted as the normal method of control testing piles on a site. However, they suffer from the major disadvantages that

- (a) they are expensive and time consuming
- (b) because of (a) only a few piles are usually tested and this number is often not large enough to give statistically significant results
- (c) because of (b) load tests can be regarded as measuring the performance of the test piles only and do not serve to locate faulty piles
- (d) test piles are seldom loaded up to failure
- (e) they are seldom carried out in such a way that the relative contributions of end resistance and skin friction can be determined
- (f) they yield no information on the dimensions of the pile under the ground and only confirm that the pile is structurally strong enough to carry the test load without giving any measure of concrete quality

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- (g) more care may be taken by the contractor in installing test piles than other piles so that results can be misleading
- (h) the pile is often discarded after test and not included as part of the foundation.

2. Although also suffering from many disadvantages and limitations of use, non-destructive tests offer control measurements which are not possible with pile load tests. However, they should not be regarded as offering an alternative to pile load tests. It is often possible to ascertain a great deal about the whole population of piles on a site by a combination of destructive and non-destructive tests and an intelligent assessment of geotechnical conditions.

3. The Centre Expérimental de Recherches et d'Études du Bâtiment et des Travaux Publics (CEBTP) of France has been carrying out various types of non-destructive pile tests on sites in Western Europe and other French speaking countries. Two of their techniques—the sonic sounding method and the vibration control method—have become generally accepted there. In Britain

Notation

a	amplitude, m
A_0	cross-sectional area of concrete pile, m^2
C	capacitance, F
E_0, E_B	modulus of elasticity of concrete and soil respectively, N/m^2
E'	pile head stiffness, N/m
E'_{max}	pile head stiffness when pile is supported on rigid base, N/m
E'_{min}	pile head stiffness when pile is unsupported at depth, N/m
E'_{∞}	pile head stiffness of an infinitely long pile embedded in soil, N/m
E_t	voltage, V
F	sinusoidally varying force applied to head of pile, N
F_0	maximum vertical force applied to head of pile, N
I_t	input current analogous to F , A
L	length of pile, m
L', l, \mathcal{L}	inductance, H
M_v	mass of moving part of electrodynamic vibrator, kg
M_p	mass of pile, kg
N	mechanical admittance of a free pile, $1/\rho_0 v_0 A_0$, s/kg
P	$N \coth(\sigma L)$, s/kg
Q	$N \tanh(\sigma L)$, s/kg
r	radius of pile, m
d	diameter of pile, m
R	resistance, Ω
v_0	velocity of plane wave propagation in the pile, m/s
V	vertical velocity of the pile head, m/s
V_0	maximum vertical velocity of the pile head, m/s
$ V_0/F_0 $	mechanical admittance of the pile head, s/kg
α	acceleration of the mass M_v , m/s^2
β'	velocity of propagation of transverse waves in soil, m/s
ρ'	bulk density of soil, kg/m^3
ρ_0	density of concrete, kg/m^3
σ	damping factor, m^{-1}
ν	Poisson's ratio of soil
ω	angular velocity, rad/s

these techniques have been used on a number of sites. The sonic sounding method is perhaps better known and its application has been described by Levy¹ and Gardner and Moses.² The vibration method has also been used and described by Gardner and Moses² but this technique has not been exploited to the extent that usefully it might have been by British engineers, probably because of a lack of knowledge of the technique or possibly because of a degree of mysticism associated with the interpretation of the results.

4. Since vibration testing of piles was first started by the CEBTP a considerable amount of theoretical work has been done which has shed light on interpretation, and the experience of testing many thousands of piles has led to the technique being applied with more confidence to piles on sites which lend themselves to being tested in this way.

5. The vibration method is most often used to test cast in situ piles. As no special inclusions are normally required in the piles to be tested, any piles may be selected for testing provided that the pile heads are accessible and unattached to any structural linkage. The main function of the test is to detect any major defect such as an open fracture or an important strangulation of the concrete, particularly in the upper portion of the pile. Theoretically, in ideal conditions it is possible to infer much more than this from the results but the field experiences described in this Paper show the versatility and limitations of the vibration method.

Vibration apparatus

6. An electrodynamic vibrator is placed on the head of a pile and in intimate contact with it, as illustrated in Fig. 1. This vibrator is driven by the current supplied by a sine wave generator, the frequency of which may be varied from 0 to 1000 Hz. The vibrator therefore applies a sinusoidally

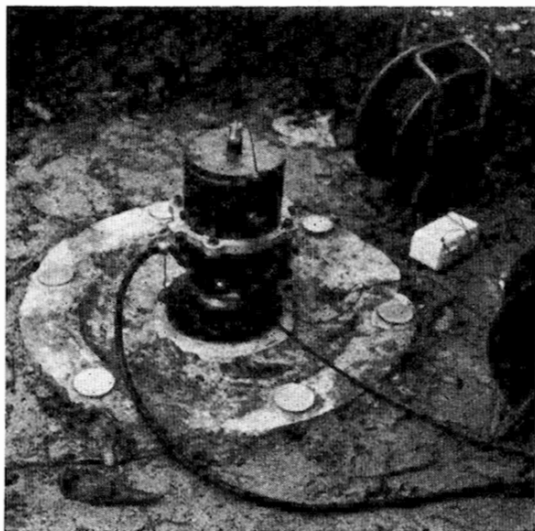


Fig. 1

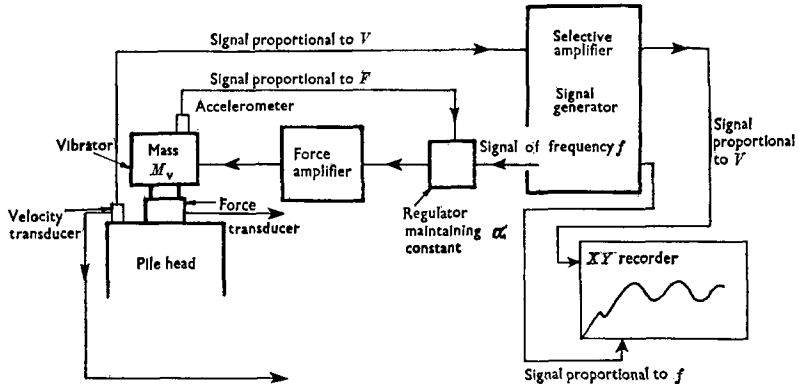


Fig. 2. Diagram of vibration testing equipment

varying vertical force $F = M_v \alpha$, where α is the acceleration of the moving part of mass M_v . The maximum force applied is $F_0 = M_v \alpha_0$, where $\alpha_0 = \frac{1}{2} \omega^2 a$, a being the amplitude of vibration. The output of an accelerometer attached to the moving mass is fed back into a regulator unit which adjusts the amplitude of vibration so that α_0 and F_0 are kept constant irrespective of the frequency of vibration (see Fig. 2). A load cell placed under the vibrator may be used to monitor the force being applied to the pile head. A velocity transducer mounted separately on the head of the pile is used to monitor the vertical velocity of the head.

7. The movement of the head of the pile is a measure of the response of the pile head to excitation. The impulse travels to the bottom of a continuous pile and is reflected back again towards the head at a speed v_0 which is dependent only on the density of the concrete of the pile. The velocity of the head of the pile V at any instant is the resultant of the imposed and reflected stress waves.

8. An X - Y recorder is used to plot a signal proportional to mechanical admittance $|V_0/F_0|$ against a signal which is proportional to f as the frequency is slowly varied from about 20 Hz to 1000 Hz. Interpretation of the resulting shape of the graph so obtained enables a number of physical properties of the pile to be inferred.

9. If the frequency of pulsation applied to the pile is such that the reflected stress wave is continually reaching the head of the pile at the same instant as the applied stress wave (acting in the same sense) then the amplitude of vibration increases, resonance occurs and $|V_0/F_0|$ reaches a maximum. However, the amplitude of vibration is limited because it is damped by the energy absorbed by the soil surrounding the pile. To clarify the interpretation of the $|V_0/F_0|$ response curves, the behaviour of ideal piles is considered.

Initial preparation of piles for testing

10. The head of the pile should be made roughly horizontal and preferably level with the surrounding ground surface. An area at the centre of the pile

and two smaller outer areas diametrically opposite are bush hammered to a reasonably smooth finish. A 150 mm dia. steel plate is levelled and fixed at the centre of the pile head using a thick epoxy resin. Plates 60 mm in diameter are similarly attached to the adjacent smaller areas. When a test is carried out, the vibrator is placed on the central plate and levelled precisely by means of three thumb screws at its base. The velocity transducer may be placed on either of the smaller plates. A rubber hood is then used to cover the whole assembly to reduce the effect of wind.

Principles of response curve interpretation

11. Consider first the case of a perfect free pile (i.e. one which is laterally unrestrained) of length L resting on the surface of an elastic foundation (Fig. 3(a)). By applying excitation at constant maximum force F_0 and observing the maximum velocity V_0 of the head of the pile at varying frequencies it is observed that frequencies at which resonance occurs are equally spaced at intervals of frequency

$$\Delta f = \frac{v_c}{2L} \dots \dots \dots (1)$$

where v_c is the velocity of plane wave propagation along the pile.

12. In the case where the elastic base is infinitely rigid, the lowest frequency of resonance has a value of $v_c/4L$ as shown in Fig. 4(a). In contrast, when the pile rests on an infinitely compressible base, resonance first occurs at very low frequency (Fig. 4(c)). When the base is an elastic one of normal compressibility the lowest frequency of resonance lies in an intermediate position between that of the rigid and infinitely compressible bases as shown in Fig. 4(b).

13. When the pile is embedded in soil as shown in Fig. 3(b), the movement of the pile is damped by the presence of the soil. The vibration response curve of $|V_0/F_0|$ is attenuated and the shape of this curve looks like that shown in Fig. 5. The denser the soil and the longer the pile, the greater is the attenuation so that the amplitude between maxima and minima is reduced.

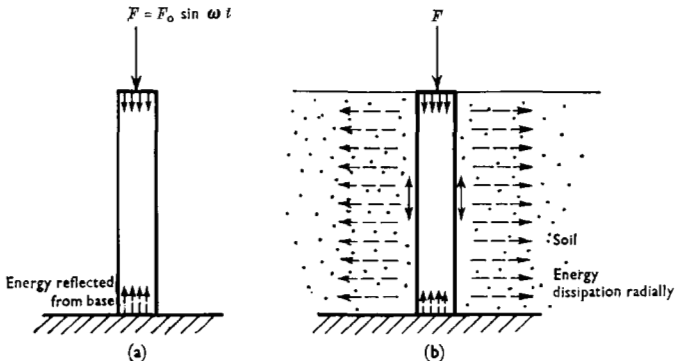


Fig. 3. (a) Free pile on elastic base, (b) embedded pile on elastic base

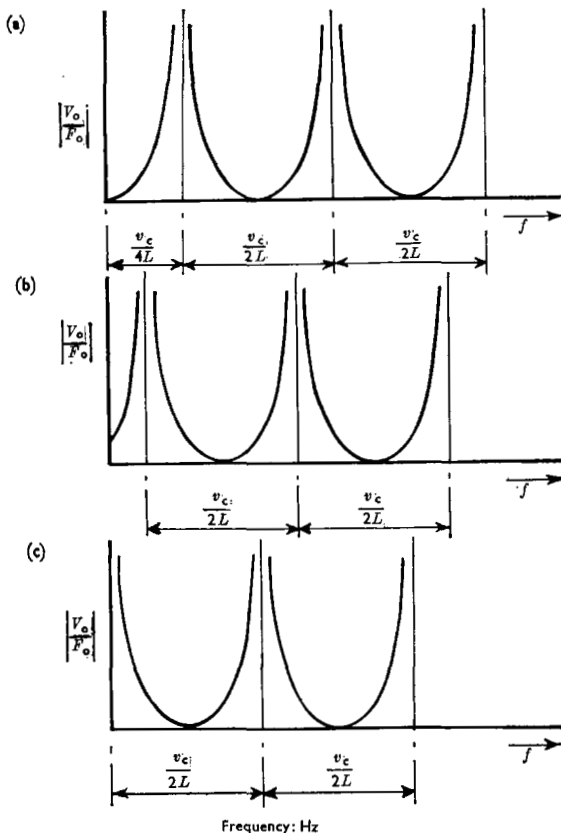


Fig. 4. Effect of base compressibility on resonance frequency, (a) rigid base, (b) intermediate elastic base, (c) infinitely compressible base

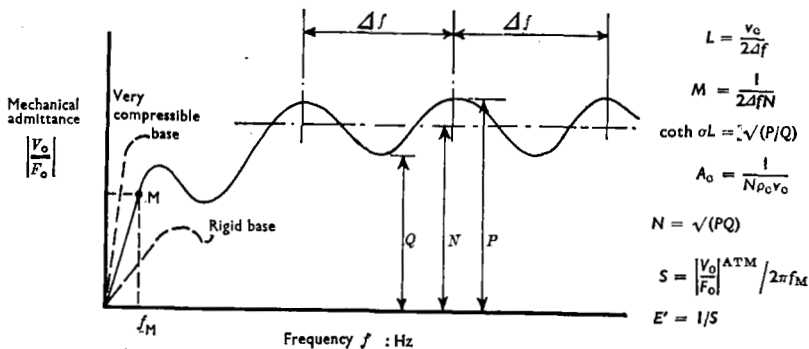


Fig. 5. Response curve for cylindrical pile

14. A factor which gives a measure of the soil damping effect was given by Briard³ as

$$\sigma = \frac{\rho' \beta' 1}{\rho_c v_c r} \dots \dots \dots (2)$$

15. The mean value of $|V_o/F_o|$ in the steady state region of the response curve is known as the mechanical admittance N and theoretically

$$N = \frac{1}{\rho_c v_c A_c} \dots \dots \dots (3)$$

(This is the inverse of mechanical impedance defined by Paquet.⁴) The maximum value P and the minimum value Q of $|V_o/F_o|$ provide a measure of the damping effect of the soil where

$$P = N \coth(\sigma L) \dots \dots \dots (4)$$

$$Q = N \tanh(\sigma L) \dots \dots \dots (5)$$

From equations (4) and (5) N can be expressed in terms of P and Q as

$$N = \sqrt{PQ} \dots \dots \dots (6)$$

and σL may be calculated from the expression

$$\coth(\sigma L) = \sqrt{P/Q} \dots \dots \dots (7)$$

16. If the exact length of a pile is known, it is possible to determine v_c by measuring Δf and substituting it in equation (1). The propagation velocity is related to the compressive strength of the concrete and so a determination of v_c gives a measure of concrete quality. Conversely, if the average concrete quality is known for example by determination of velocity measurements on representative concrete cores, it becomes possible to determine the effective length of a pile by measuring Δf . This can then be checked against specified length.

17. If the pile is discontinuous or severely broken then the length down to the discontinuity will be measured. Sometimes in constructing bored cast in situ piles, a bulb or a local enlargement of the diameter of the pile may be formed in soft ground at the interface with an underlying more rigid stratum. In such cases the value of Δf measured indicates the length of the pile down to the base of the bulb and the actual length of the pile is not correctly measured. However, it is debatable in such cases which part of the pile is effectively giving base support.

18. Owing to the cumulative effect of soil damping on a long pile, a test on a pile of length/diameter ratio greater than 20 is unlikely to be very definitive, unless the pile passes through a very soft deposit on to a rigid stratum.

19. The area of cross-section of the pile may be calculated from equation (3) provided that ρ_c and v_c are known. If the length determined from Δf is nearly equal to that specified, then the value of v_c will be known. The measured value of N is predominantly a function of the properties of the upper portion of the pile. If N is much greater than its calculated value, it is likely that the upper part of the pile is defective either because of restricted cross-sectional area or inferior concrete.

20. If equations (1) and (3) are combined an expression for the mass of the pile is obtained which includes only parameters actually determined from the response curve

$$M_p = LA_c \rho_c = \frac{1}{2\Delta f N} \dots \dots \dots (8)$$

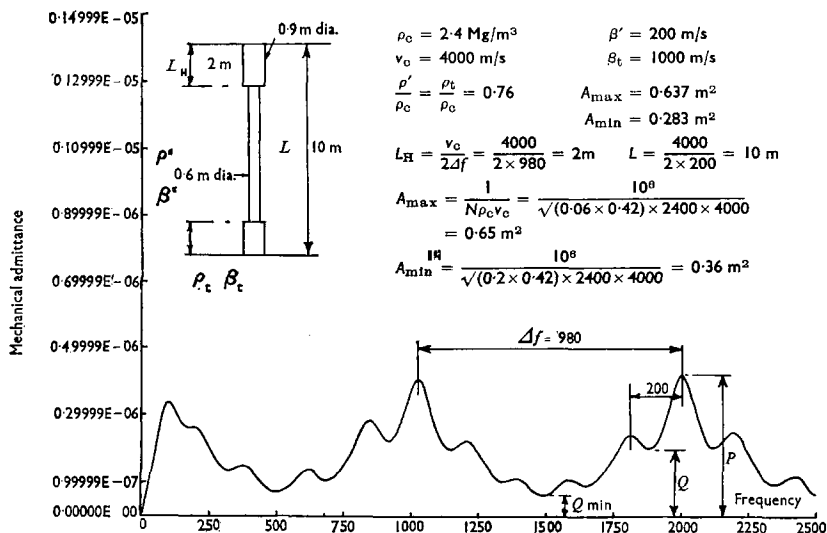


Fig. 6(a). Pile with variable diameter

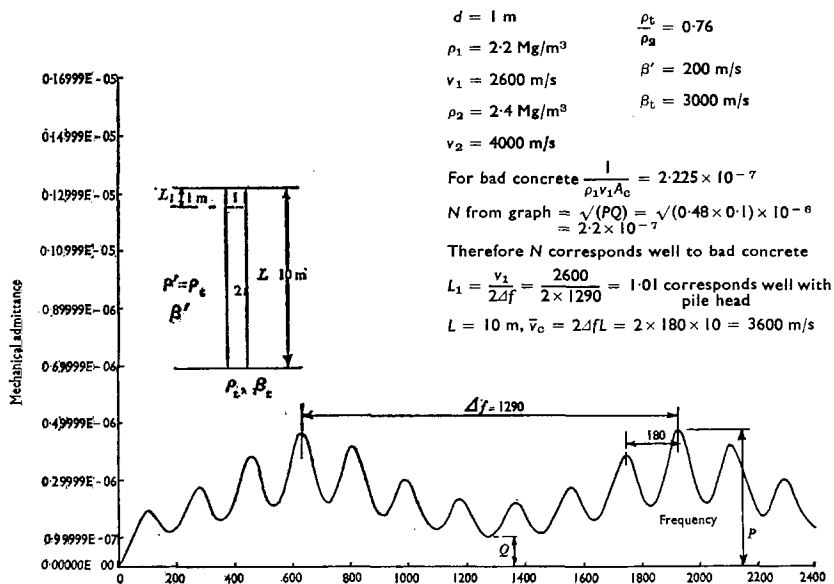


Fig. 6(b). Pile with poor quality concrete at head

Measurement of pile stiffness

21. When the pile is excited at low frequencies, the inertia effects are insignificant and the pile-soil unit, behaving like a spring, gives a straight line response at the start of the $|V_0/F_0|$ curve. The inverse of the slope of this straight line measures the apparent stiffness of the head of the pile. This stiffness is given by

$$E' = \frac{2\pi f_M}{|V_0/F_0|_M} \text{ in units of N/m} \quad (9)$$

where f_M and $|V_0/F_0|_M$ are the co-ordinates of the point M on the response curve. E' corresponds to the slope of the initial tangent modulus to a load-displacement graph obtained from a static load test on the pile. Although it cannot be pretended that the measurement of E' can be precise, it does give a good indication of the ability of a pile to carry load.

Factors affecting the stiffness measured at the head of the pile

22. The apparent stiffness of the pile head is a function of

- (a) the stiffness of the concrete constituting the pile E_c
- (b) the stiffness of soil surrounding the pile as indicated by the attenuation of the oscillations of the $|V_0/F_0|$ response curve and measured by the damping factor
- (c) the stiffness of the soil supporting the base of the pile.

23. Knowing the first two properties it is in principle possible to determine the apparent stiffness measured at the head in the following two extreme cases.

24. A pile supported on a rigid base would give the maximum possible stiffness that the pile could have and may be calculated from the equation

$$E'_{\max} = E'_{\infty} \coth(\sigma L) = E'_{\infty} \sqrt{(P/Q)}(\text{N/m}) \quad (10)$$

where

$$E'_{\infty} = \frac{A_c E_c \sigma L}{L} (\text{N/m}) \quad (11)$$

A_c is the area of cross-section of the pile (m^2), E_c is the modulus of elasticity the concrete (N/m^2) and σL is determined from equation (7).

25. A pile with no base support behaving as if it had been cut off at depth would give the minimum possible stiffness that the pile could have and may be calculated from the equation

$$E'_{\min} = E'_{\infty} \tanh(\sigma L) = E'_{\infty} \sqrt{(Q/P)}(\text{N/m}) \quad (12)$$

26. These two values provide upper and lower limiting values of stiffness with which the actual measured stiffness E' may be compared.

Influence of secondary effects on response curve

27. The $|V_0/F_0|$ response curves obtained from real piles on site are seldom as simple as the theoretical curve for the perfect pile. This may be due to numerous factors among which the most common are

- (a) variations in the diameter of the pile
- (b) variations in the quality of concrete within the pile
- (c) variations in stiffness of the soil through which the pile passes
- (d) the top part of the pile being exposed above the ground.

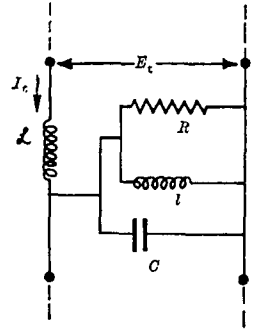


Fig. 7. Electric analogue of pile segment

The general effect of these on the response curve is to produce an oscillation which is superimposed on the normal oscillation of the $|V_0/F_0|$ response curve such as that shown in Fig. 6. This makes interpretation more difficult.

28. In order to appreciate better the effect of such variables on response and to improve the methods of interpretation, the response curve of any pile with a known defect may be predicted by making use of the analogue which exists between mechanical wave propagation and an electrical transmission line, the mathematics of which are understood.³

29. Consider a perfect cylindrical unit length segment of a pile. The force F_t applied due to excitation transmitted to the top of the segment may be represented by current I_t and the velocity of the top of the segment by the voltage E_t as shown on the electrical analogue in Fig. 7. This means that the mechanical impedance of the pile segment corresponds to the electrical admittance I_t/E_t . The other electrical analogues may be summarized as follows.

30. The mass of the pile per unit length is represented by capacitance $C = \pi r^2 \rho_c$. The soil resistance per unit length is represented by $1/l = \pi r' \beta'^2$, where l is the inductance of a coil. The energy lost per unit length by the damping effect of the soil is represented by the admittance $(1/R = 2\pi r \rho' \beta')$ of a resistance R . The stiffness per unit length of pile is represented by an inductance \mathcal{L} where $1/\mathcal{L} = \pi r^2 E_c$.

31. As an example of how a defective pile may be represented by an electrical analogue, Fig. 8 shows the electrical elements which could be used as a first approximation to represent each unit length of the different sections of a pile. The base stiffness is represented by an inductance L' where the base stiffness is given by

$$\frac{1}{L'} = 1.84r \left(\frac{E_B}{1 - \nu^2} \right)$$

Using a computer to solve the matrix which takes the general form

$$\begin{vmatrix} V \\ F \end{vmatrix} = \begin{vmatrix} |a| & |b| & |c| & |d| & |e| & |f| \end{vmatrix} \begin{vmatrix} V_B \\ F_B \end{vmatrix}$$

where V_B/F_B is the admittance of the soil at the pile base, it is possible to calculate the form of the response curve for any pile.

32. Figure 6(a) shows a computer plot of a theoretical response curve for a hypothetical pile whose cross-sectional area varies. The distance between the primary maxima enables the length of the enlarged head to be estimated and the distance between the secondary maxima enables the total length of pile to be estimated.

33. The diameter of the enlarged head may be approximately determined from equations (3) and (6) by taking Q as the minimum and P as the maximum values of $|V_o/F_o|$. However, the reduced diameter of 600 mm can only be approximately estimated by taking Q as a minimum just before the maximum peak and P as the maximum. No information can be ascertained about the diameter at the toe of the pile from inspection of the response curve.

34. Figure 6(b) shows a response curve for a pile whose top portion consists of poor quality concrete. It is possible to estimate the length of poor concrete and, given the total length of the pile, the approximate average value of v_o .

35. It is evident from these two examples that if a combination of defects or variable conditions exists in practice, it can be difficult to interpret the actual response curve without the facility of being able to study theoretically the effects of varying geometrical and material parameters on simulated response curves.

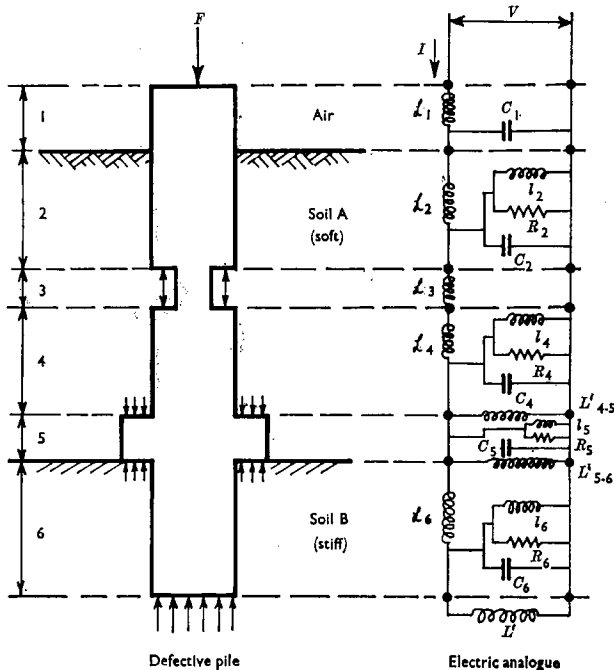


Fig. 8

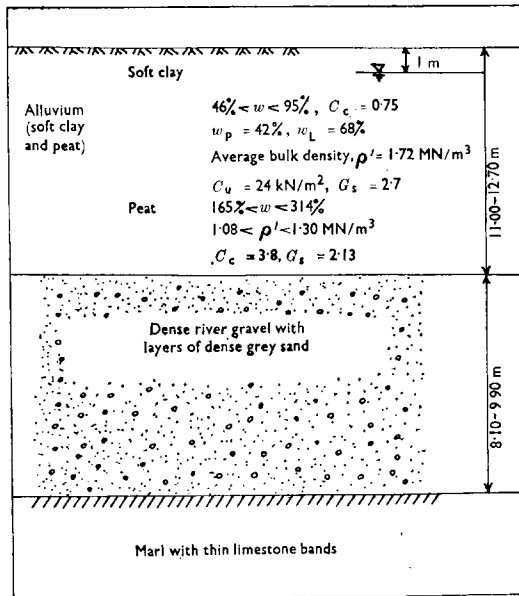


Fig. 9. Ambès : soil profile

Field experience

Electricity generating station, Ambès, near Bordeaux

36. The electricity generating station at Ambès is located on estuarine deposits on the banks of the River Garonne. The soil profile across the site is relatively uniform, consisting of alluvium (soft silty clays and peats) overlying a thick layer of river gravel, which in turn rests on interbedded limestones and marls. The profile is shown in Fig. 9, together with the geotechnical properties of the respective beds.

37. Bored, cast in situ piles were selected, with varying diameters to allow for differing loads. They were designed purely as end bearing piles and their average lengths were $15 \text{ m} \pm 1 \text{ m}$. Twenty rectangular piers were constructed to the same depth using diaphragm walling techniques. Bentonite was used throughout, and concrete was placed by tremie. Reinforcement was limited to the upper 3 m of the pile.

38. The results of the vibration tests carried out in June, July, August and November 1971 are presented in Table 1. In all fourteen piles where probable faults were located, the signal was reflected from the fault itself, resulting in an increased distance between peaks on the chart (see Fig. 10).

39. Values of measured $1/(\rho_c v_c A_c)$ were always equal to or less than the theoretical range for good concrete. It can be inferred that the quality of concrete was good, with some piles being oversize. (A_c measured $> A_c$ theoretical.) This has been confirmed by the concreting charts, which invariably showed more concrete being poured per pile than was allowed for in

NON-DESTRUCTIVE VIBRATION TESTING OF PILES

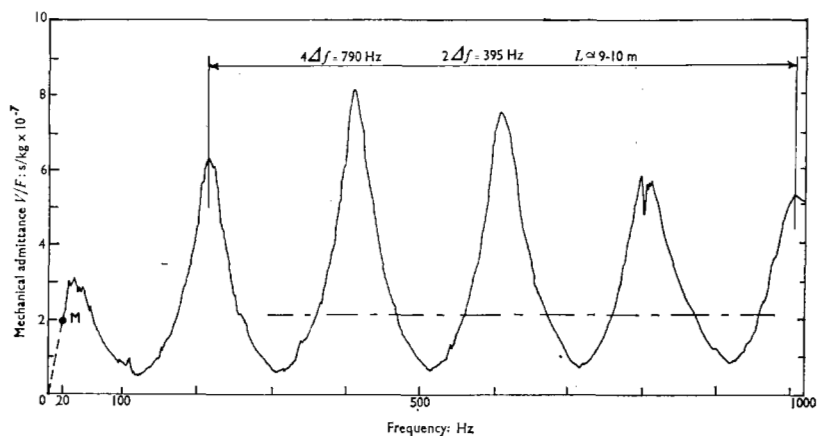


Fig. 10. Ambès : pile 1223

design. As the average concrete quality appeared to be satisfactory, the nature of the faults in the fourteen shorter piles was deduced to be planes of separation (cracks) or complete discontinuities. The relative importance of these faults could then be evaluated by comparing the stiffness $|F_0/V_0|2\pi f$ for the faulty pile with the average stiffness for piles of the same diameter for the low frequency range with linear response.

40. The pile with the lowest stiffness value was pile 1308 (see Fig. 11), with a stiffness of 0.25×10^3 MN/m; this is nearly one fifth of the average stiffness for 770 mm dia. piles on this site. It was decided to core this pile to observe the nature of the fault. A complete gap was found in the concrete at a depth of 4.2 m below the pile head. The size of the gap was difficult to determine, but was estimated from the cores as greater than 50 mm.

41. Two test piles with built in anomalies were placed outside the foundation area. The first was constructed with a length of 15 m and diameter of

Table 1

Pile dimensions	Number of piles tested	Average stiffness,* MN/m	Number of piles with faults
0.56 m dia.	44	0.867×10^3	1
0.77 m dia.	45	1.198×10^3	9
0.92 m dia.	41	1.614×10^3	4
1.07 m dia.	13	2.177×10^3	0
1.80 × 0.60 m	16	2.817×10^3	0
1.80 × 0.80 m	4	3.067×10^3	0

* The average stiffness is calculated only for piles where the design length has been confirmed by the vibration test.

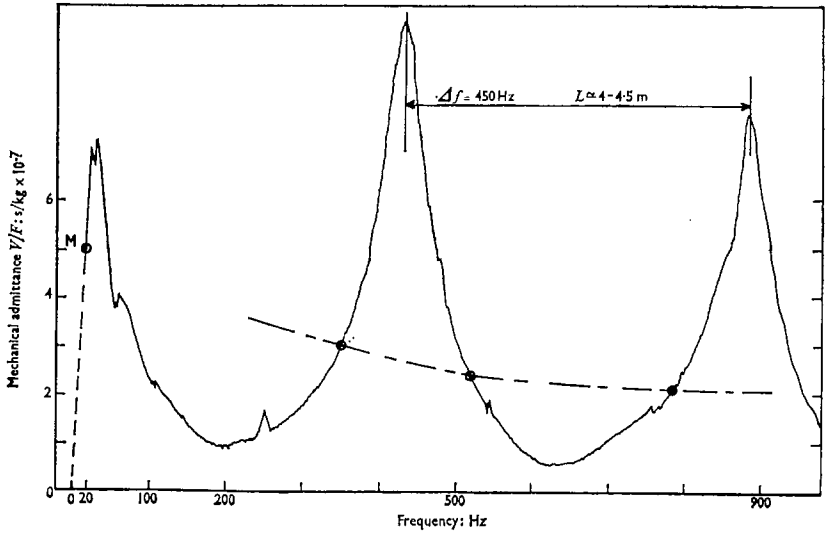


Fig. 11. Ambès : pile 1308

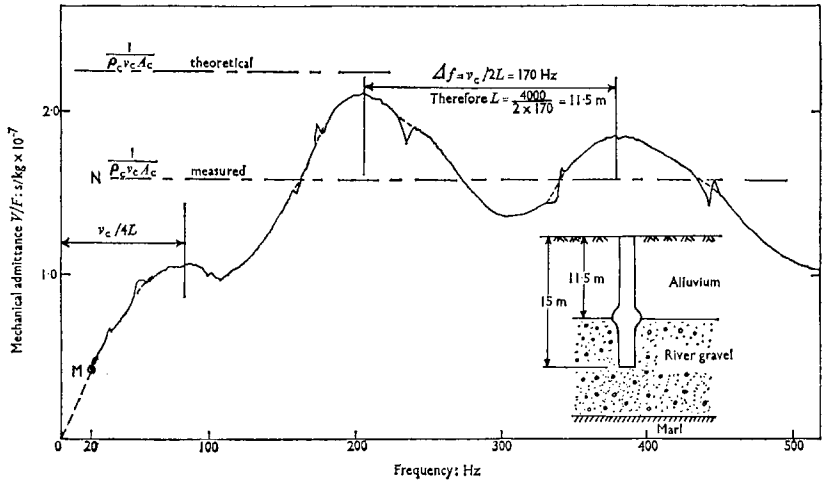


Fig. 12. Ambès : test pile 1

770 mm, but a bulb was formed at the interface between the alluvium and the gravel at a depth of 11.5 m approximately. The formation of this bulb was confirmed by the concreting chart. The vibration test result is given in Fig. 12. The measured value of $1/(\rho_c v_c A_c)$ was considerably lower than the theoretical value, indicating overboring of the shaft with a correspondingly bigger pile section. The bulb reflected the signal completely, giving a calculated length from $v_c/2L = \Delta f$ of $L = 11.5$ m for a velocity $v_c = 4000$ m/s.

42. A high stiffness value of 3.45×10^3 MN/m was measured—four times the average for piles of this diameter on this site. In fact, 76 piles reflected the signal from a level corresponding to the interface between the gravel and the alluvium, while giving above average stiffnesses. In these cases it has been assumed that a bulb was formed at this level. In some of these tests reflexion from the toe of the pile was also detected.

43. The second test pile was constructed to a length of 10 m and a diameter of 770 mm. The toe of the pile was located in the alluvium, about 3 m above the top of the gravel. The measured length L deduced from the distance between peaks $v_c/2L$ was 10 m for a value of $v_c = 4000$ m/s. The measured value of $1/(\rho_c v_c A_c)$ was much less than the theoretical value, indicating a larger pile shaft than that designed, due to overboring. This was confirmed by the concreting records. The stiffness for test pile 2 was 0.82×10^3 MN/m—approximately two thirds of the average stiffness value for 770 mm dia. piles at this site (see Fig. 13).

44. Other information to be deduced from the vibration curves for test piles 1 and 2 includes the relative position of the first resonant peaks and the relative M/N ratios (M is the value of $|V_0/F_0|$ at 20 Hz and N is the geometric mean for the oscillating portion of the curve). For test pile 1 a hump is observed at 85 Hz, which corresponds to $v_0/4L$, indicating a very stiff anchorage;

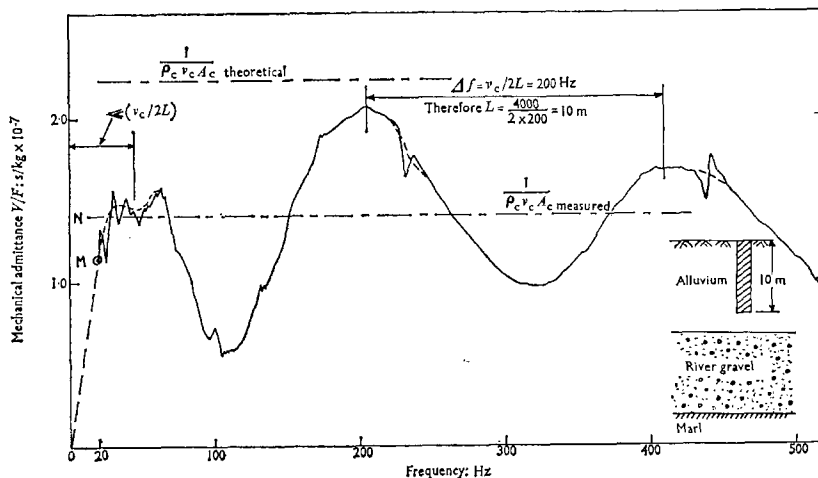


Fig. 13. Ambès : test pile 2

test pile 2 shows a hump at 45 Hz, which is much less than $v_c/4L$, indicating a broken pile. Confirming these deductions, the values of M/N for TP1 and TP2 are 0.25 and 0.8 respectively.

45. A loading test up to 20 t was performed on pile 1082 (770 mm dia.), which had given a stiffness of 0.42×10^9 MN/m in the vibration test, together with a major break at 5–6 m depth. Two comparative loading tests were carried out on adjacent piles with proven continuous lengths and average stiffnesses; the settlement for pile 1082 was 3–4 times as great as that for the sound piles.

Foundations for generators at steel works, Fos-sur-mer

46. A site for generators at Fos-sur-mer is located on estuarine deposits at the mouth of the River Rhone. The soil profile across the site is relatively constant, consisting of 6–11 m of medium to medium dense silty sand overlying well-rounded dense gravels, which are sometimes cemented as conglomerates. The water table ranged between 1 and 3 m below ground level.

47. Bored, cast in situ piles of various diameters were founded in the gravel, with casing for the first 2–3 m. Pile lengths ranged between 10 m and 14 m. Flight augers were used, with bentonite support and the concrete was placed by tremie pipe. Reinforcing was continuous to the base of the piles. Vibration testing of 329 piles and piers was carried out in April and May 1972. The results are summarized in Table 2. Four major categories of faults were noticed (see Table 3).

- (a) If the head of a pile had a cross-sectional area greater than the pile shaft below, then $1/(\rho_c v_c A_c)$ measured, decreased with increasing frequency (see Fig. 14). Similarly, an increasing shaft diameter with depth resulted in $1/(\rho_c v_c A_c)$ measured, increasing with frequency.

Table 2

Pile dimensions	Number of piles tested	Average stiffness, MN/m	Number of piles with faults
0.60 m dia.	104	1.111×10^9	15
0.80 m dia.	156	1.550×10^9	24
1.00 m dia.	84	2.041×10^9	7
2.3 × 0.6 m	4	4.65×10^9	—
4 × 0.8 m	8	6.45×10^9	—

Table 3. Faults observed at Fos-sur-mer

Fault type	Number of piles	Remarks
1	2	Not necessarily detrimental to future pile performance
2	17	Serious effect on future pile performance
3	13	No serious breaks observed
4	14	Some piles were broken completely

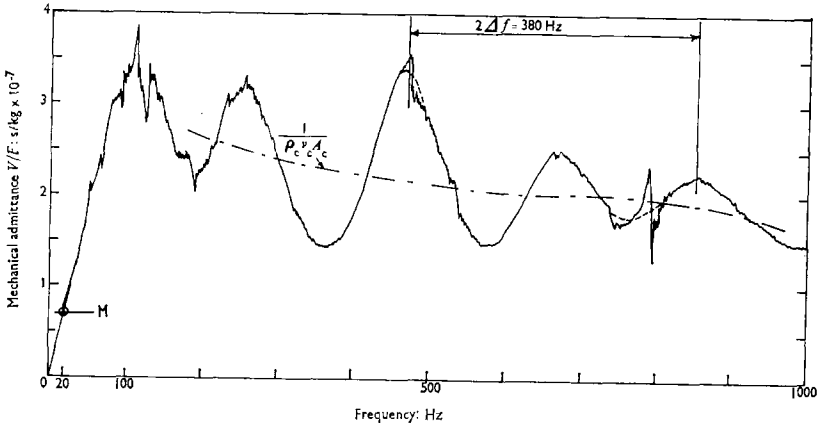


Fig. 14. Fos-sur-mer : pile 357

- (b) When the pile was continuous with the correct length L but with reduced cross-sectional area A_c , then the stiffness was reduced below the average stiffness by at least 20%, $\Delta f = v_c/2L$ corresponded to the theoretical value and the measured value of $1/(\rho_c v_c A_c)$ was greater than the theoretical value.
- (c) If the pile were broken or fissured, then it would be expected that the stiffness of the pile would be reduced, $\Delta f = v_c/2L$ would be increased and the measured value of $1/(\rho_c v_c A_c)$ would correspond approximately to the theoretical value. At this site the first and last of these conditions were encountered, but for no pile was there an increase in Δf above the theoretical, which shows that any fissures or cracks in the piles were of minor importance.
- (d) If the fault was very great or very close to the pile head, then the vibration curve was heavily damped, resulting in greatly reduced stiffness values and no measurement possible for Δf .

48. The vibration survey was followed by excavation of all the faulty piles to a depth of 4 m below ground level. Nearly all showed faulty concrete around the shaft perimeter, with reinforcing bars appearing on the shaft surface in some cases. One pile had been broken just below the pile head by construction plant and offered little resistance to horizontal loading. Many of the faults were attributed to an overdose of bentonite in the drilling mud. Many of the piles had pile head diameters greater than the shaft diameters, because of the temporary casing down to 3 m below the pile head.

Paisley College of Technology

49. Piles at the Paisley College of Technology site were tested by the sonic method¹ as well as the vibration method. The site and the results of the non-destructive testing have been described by Gardner and Moses.²

50. The foundations are supported on cast in situ bored piles, 480 mm in diameter, formed in raised beach deposits of soft to firm laminated clays.

Reinforcing is continuous to the base of the piles. Altogether 94 piles were tested, with lengths varying between 2.5 m and 8 m.

51. Three piles were shown by both methods to have considerable honeycombing at levels where a rapid cross-flow of ground water existed, resulting in the washing out of fines. The vibration test results for these three piles gave stiffnesses of between 0.42×10^3 MN/m and 0.59×10^3 MN/m—less than 60% of the average for piles of similar length on this site.

52. For comparison with results taken from other sites, the average stiffness for 13 piles with proven lengths between 5.5 m and 8.1 m was 0.76×10^3 MN/m.

New offices, Greater London Council

53. At the site of new offices for the Greater London Council 61 piles were tested by the sonic method;¹ one was seen to have a slight fault at 5.2 m below the pile head (slowing down of the concrete wave propagation velocity at this level). This pile was subsequently tested by the vibration method.

54. The pile tested was a bored, cast in situ pile with a diameter of 1.22 m and under-reamed for the bottom 1.70 m. The total pile length was 20.64 m and it was founded in London clay (see Fig. 15). No comparison of stiffness values with other piles on this site could be made, but the M/N ratio of 0.39 was typical of piles of this nature and size tested on other sites. The absolute stiffness value was 4.25×10^3 MN/m³.

55. The fault located by the sonic method at 5.2 m was not visible on the vibration test result. At relatively low frequencies (up to 250 Hz) a value of $2\Delta f = 180$ Hz was measured. This corresponds to a value of $v_0 = 3740$ m/s for a pile length of 20.64 m. However, $2\Delta f$ increases with increasing frequency, until at 400 Hz a value of $2\Delta f$ gives a length L of 18.60 m for a value of $v_0 = 3740$ m/s. This corresponds to the length of straight-sided shaft down to the top of the under-ream.

56. For a value of $A_0 = 1.17$ m², $\rho_0 = 2400$ kg/m³ and $v_0 = 3740$ m/s, the

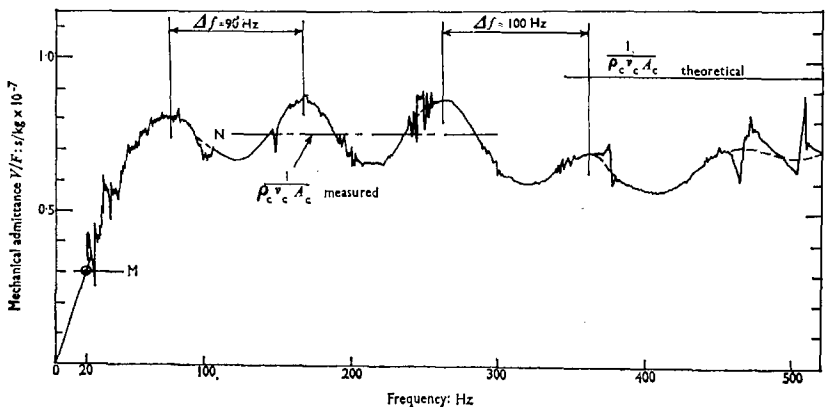


Fig. 15. Greater London Council : pile 49

theoretical value of $1/(\rho_c A_c v_c) = 0.95 \times 10^{-7}$ s/kg; the measured value was 0.76×10^{-7} s/kg. The most probable explanation for this difference is that the actual pile diameter was greater than the design diameter because of over-boring by approximately 125 mm.

Fawley, near Southampton

57. The oil tank farm site at Fawley together with the events leading to failure of one oil tank (tank 1) during water test loading and the incipient failure of a second tank (tank 2) have been described in reference 5.

58. Driven, cast in situ 420 mm dia. piles with expanded bases were founded in river gravel through approximately 2 m of gravel fill overlying 6–8 m soft silty clay and peat. The piles were reinforced to the base, and the casing was withdrawn during concreting. Vibration testing was carried out on 43 piles selected from both tank foundations during March 1973—almost five years after the tank test loadings. Some of the piles tested had been seen to be broken from visual observations in headings driven under the tanks in the gravel fill after the test loadings. For testing purposes, each pile head had to be separated from the reinforced concrete raft spanning the piles.

59. For eleven of the piles tested it was possible to confirm the original design lengths to the top of the bulb from the measured values of $2Af$. Combining these lengths with the measured values of $1/(\rho_c v_c A_c)$, the average value of v_c was deduced for these eleven piles as $v_c \approx 3900$ m/s. From previous correlations between v_c and concrete strength with silico-calcareous aggregates as used at Fawley, it can be deduced that the likely average concrete crushing strength in the eleven continuous piles is 20 MN/m², with a possible minimum value of 15 MN/m².

60. The measured stiffnesses E' for these eleven piles are given in Table 4 together with three limiting values of stiffness for each pile deduced from the vibration test curve as follows

- (a) a completely broken pile, i.e. with zero stiffness of the soil under the pile toe (E'_{\min}) (equation (12))
- (b) an infinitely long pile surrounded by the soft lateral soil only (E'_{∞}) (equation (11)) ($E_c = 30\,000$ MN/m²)
- (c) a pile with an infinitely stiff anchorage at the toe (E'_{\max}) (equation (10)).

The values of E'_{\min} in Table 4 show that the alluvial silty clay surrounding the piles under tank 1 is considerably stiffer than that under tank 2. This is also reflected in the higher values of E'_{∞} for tank 1. The measured values for E' for tank 1 are close to E'_{∞} , and most of them tend towards E'_{\min} ; for tank 2 E' measured lies between E'_{∞} and E'_{\max} .

61. For the limiting case of a column with no lateral support $E'_{\min} \rightarrow 0$ and $E'_{\infty} \rightarrow 0$. At this site the anchorage gives a finite value to E' measured which falls between E'_{∞} and E'_{\max} . The latter value represents the stiffness of the concrete column, given directly by $E_c A_c / L$. It can therefore be concluded that the alluvium under tank 2 is considerably weaker than that under tank 1.

62. When excavated, the twelfth pile in Table 4 was seen to suffer from necking 2.5 m below the pile head. However, this constriction does not

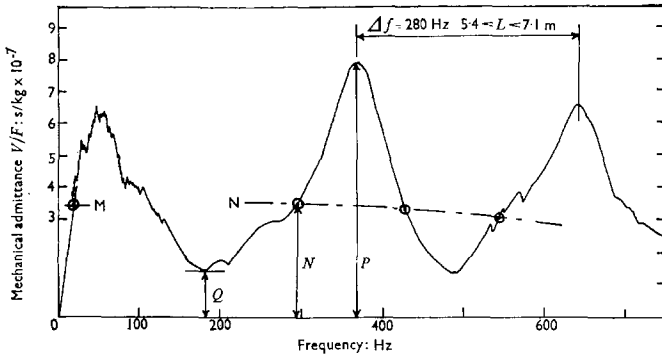


Fig. 16. Fawley : pile 12

Table 4

Pile		E' measured, $MN/m \times 10^3$	E'_{min} , $MN/m \times 10^3$	E'_{∞} , $MN/m \times 10^3$	E'_{max} , $MN/m \times 10^3$
Tank 1	1	0.417	0.385	0.500	0.625
	2	0.333	0.175	0.294	0.500
	3	0.625	0.333	0.476	0.667
	4	0.278	0.167	0.286	0.500
	5	0.294	0.172	0.303	0.526
	6	0.263	0.182	0.303	0.500
	7	0.294	0.145	0.270	0.500
Tank 2	8	0.167	0.063	0.152	0.370
	9	0.278	0.069	0.159	0.370
	10	0.294	0.067	0.167	0.417
	11	0.192	0.067	0.167	0.417
	12 (broken)	0.100	0.093	0.250	0.685

NON-DESTRUCTIVE VIBRATION TESTING OF PILES

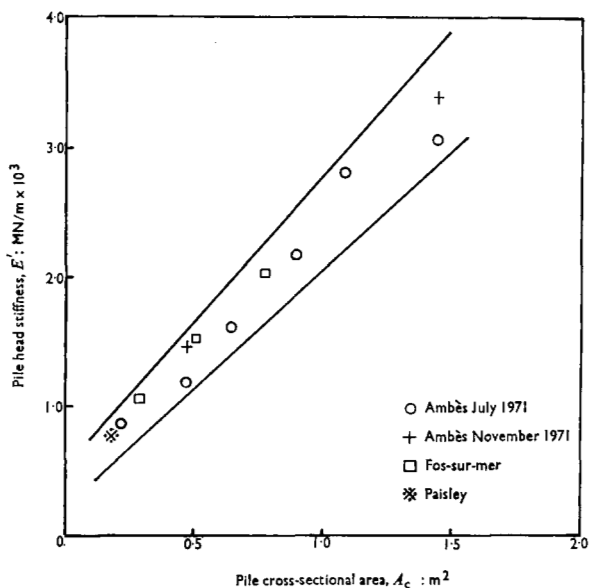


Fig. 17

Table 5

Variable	Site		
	Ambès	Fos-sur-mer	Paisley
1	15 m \pm 1 m	10–14 m	5.5–8.1 m
2	0.215–0.898 m ² (circular) 1.08–1.44 m ² (rectangular)	0.283–0.786 m ² (circular) 1.38–3.2 m ² (rectangular)	0.18 m ² (circular)
3	Cylindrical L/d from 14/1 to 27/1	Cylindrical L/d from 10/1 to 23/1	Cylindrical L/d from 11.5/1 to 16.5/1
4	2 m for $d=0.60$ m 0.50 m for all others	0.50 m approximately	0.50 m approximately
5	3400–3700 m/s	3550–3700 m/s	3100–3600 m/s
6	150–350 m/s	500–600 m/s	Not measured*
7	Dense river gravel	Dense river gravel with cobble	Firm boulder clay and coal measures

*Typical values for raised beach deposits of this nature are $300 < \beta' < 900$ m/s.

affect the vibration response; Fig. 16 gives the fault at between 5.4 m and 7 m. If the necking at 2.5 m were responsible for the maxima and minima observed in Fig. 16 then the value of v_0 for the concrete in the pile would have to be as low as 1500 m/s, with the Young's modulus E_0 for the concrete above the necking as low as 4500 MN/m², which is most unlikely.

Comparison of results

63. In order to compare the pile-soil stiffness E' derived from the vibration test for a given site with the results from any other site, it is necessary to enumerate the variables which affect the absolute value of stiffness measured. These are the pile length L , the cross-sectional area of the pile A_0 , the shape of the pile and the ratio L/d , the length of free pile above ground surface, the concrete longitudinal wave propagation velocity v_0 , the velocity of propagation of transverse waves through the ground surrounding the pile shaft β' , the stiffness of the ground under the toe of the pile, and the relative stiffnesses of the soil under the toe and the soil around the shaft, i.e. whether the pile is designed as an end bearing or a skin friction pile.

64. For the sites described in this Paper, it is possible to compare the results from Ambès, Fos-sur-mer and Paisley. Ranges of values for the first seven variables are given in Table 5. The piles at Ambès and Fos-sur-mer were designed as end bearing piles only, and the ratios of stiffnesses between the soil under the toe and the soil around the shaft are similar. The soil conditions are different at Paisley, and it is probable that the relative soil stiffnesses at this site are lower than those for the other two sites. Counter-acting this is the likelihood that skin friction is contributing more to total pile resistance at Paisley.

65. Figure 17 shows pile stiffness measured at these three sites plotted as a function of pile cross-sectional area. All the values used were measured on piles of confirmed length. A trend of pile stiffness increasing linearly with cross-sectional area is shown. It is suggested here that end bearing piles constructed through soft to firm strata to a relatively stiff bearing layer can be expected to yield a similar relationship at other sites, provided that pile lengths fall between 5 m and 15 m approximately.

Conclusions

66. The fact that faults exist in some piles after installation on many sites cannot be disputed. The number and seriousness of these faults on any site depend to a great extent on the geotechnical conditions encountered, such as groundwater problems and contrasts in strata penetrated. However, what is not clear is how these faults, if detected, would affect the future performance of any structure founded on the pile system.

67. The vibration method has been shown to be a useful tool for the detection of faults in piles, within certain limitations. It is possible to infer the importance of any faults detected by comparing values of pile head stiffness E' with the average values for piles of the same dimension on the same site. It is suggested that the pile head stiffness is related to the early slope of the load-settlement curve for a pile loading test; for this to be shown, more loading tests are required on piles which have been vibration tested.

68. Limitations on the use of the vibration test include the limiting length

to diameter ratio of 20 to 1, and the requirement for the pile to be a right cylinder, although end bearing piles through soft alluvial deposits can be successfully tested with L/d ratios of up to 30/1 (see Table 5). It has been necessary to test with no extraneous vibrations caused by site plant, which necessitates testing at night on some sites. Also, if bulbs are formed at depth it is usually impossible to learn anything about the state of the pile concrete below the bulb.

69. These limitations apart, it is considered that the vibration test is a useful complement to load testing, and when used in conjunction with other information derived from standard site investigation procedures, pile loading tests and possibly seismic surveys can give assurance to engineers responsible for piling contracts, which they previously lacked. The testing schedule need not interrupt the site programme as 25–40 piles can be tested per day, from pile ages of four days upwards.

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