Indian Geotechnical Journal, 27 (4), 1997

Bearing Capacity of Precast Concrete Piles by Stress Wave Measurement

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Introduction

The bearing capacity of precast concrete piles is generally estimated in accordance with the guidelines, laid down in IS:2911(Part-I/Sec.3)–1979 by static formula using soil properties or by dynamic formula. Though the guidelines clearly caution about the use of dynamic formula for piles in cohesive soil deposits such as saturated silts and clays, these have been still favoured for preliminary estimates. Final decision of bearing capacity, however, is made on the basis of initial load tests, conducted on few piles prior to actual execution of working piles. Further, the capacity of working piles is checked by conducting one-half to two percent routine load tests on limited number of piles in accordance with IS:2911(Part-IV)–1985. Both initial as well as routine load tests which require elaborate arrangements for obtaining reaction, are quite time consuming. For high capacity piles, these arrangements are still more problematic. Also at the projects involving large number of piles, sufficient time and energy are consumed in conducting these tests.

In view of above difficulties, for better quality assurance in recent years there has been an increasing interest in the use of stress wave measurement for assessing structural integrity, monitoring performance during driving and evaluation of static bearing capacity of driven piles. With the advancement in the application of one dimensional stress wave propagation to piles and development of stress wave measuring equipments, the technique has been used widely. However, in India, its application has been made only from towards end of last decade (Prakash et al 1989)

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At Nagarjuna Fertilizer Plant, Kakinada (Andhra Pradesh), a large number of precast concrete piles of single length as well as of two segments have been used to support various structures of the plant. The decision for their adoption and their design were based on the detailed feasibility studies alongwith initial load tests (Raju and Gandhi, 1989). At this project site, the stress wave measurements were made on some of the piles during execution by a portable computerised equipment which utilises the one dimensional stress wave solutions based on the method of "Characteristics". The data related with the integrity assessment and performance of piles during driving have already been reported elsewhere (Prakash et al. 1991 and 1992). The bearing capacity and load vs. displacement response estimation using stress wave measurement data, obtained from dynamic load tests on single length precast concrete piles, has been discussed in this paper. A comparison between estimated and observed values by static load tests has also been presented.

Testing Methodology and Principle

The Dynamic Load Test (DLT) is conducted by mounting combined acceleration and strain transducers on diagonally opposite faces of the pile, near its top (Fig. 1). The signals obtained through the mounted transducers during driving are passed on for processing to the signal conditioning sub-system and then to computer. The processed signals for each blow are stored on the fixed disk. The measured signal and other data, scaled in engineering units are also available on the screen.

The measured signals (Fig. 1), force and the velocity × impedance which is also force, are same for the pile length above ground level as there is no change in impedance for this portion. For the pile length below ground level, the deviation in velocity × impedance curve is due to change in pile impedance resulting from interaction of soil with pile. At a certain time tmax, a wave introduced at the pile top is reduced while travelling through the pile and is observed at pile top after a period 2L/C (L length of pile and C = stress wave velocity). The reduction in wave during this period which activates the soil resistance is called total driving resistance.

The principle involved in dynamic load testing is based on the solutions for one dimensional stress wave equation following the "method of characteristics". It is based on the phenomenon that stress wave in a frictionless pile propagates unaltered with a characteristic velocity and at a particular section, the following expressions for downward travelling wave $F\downarrow$ and upward travelling wave $F\uparrow$ can be derived as follows :

$$F\downarrow = (F+ZV)/2 \tag{1}$$

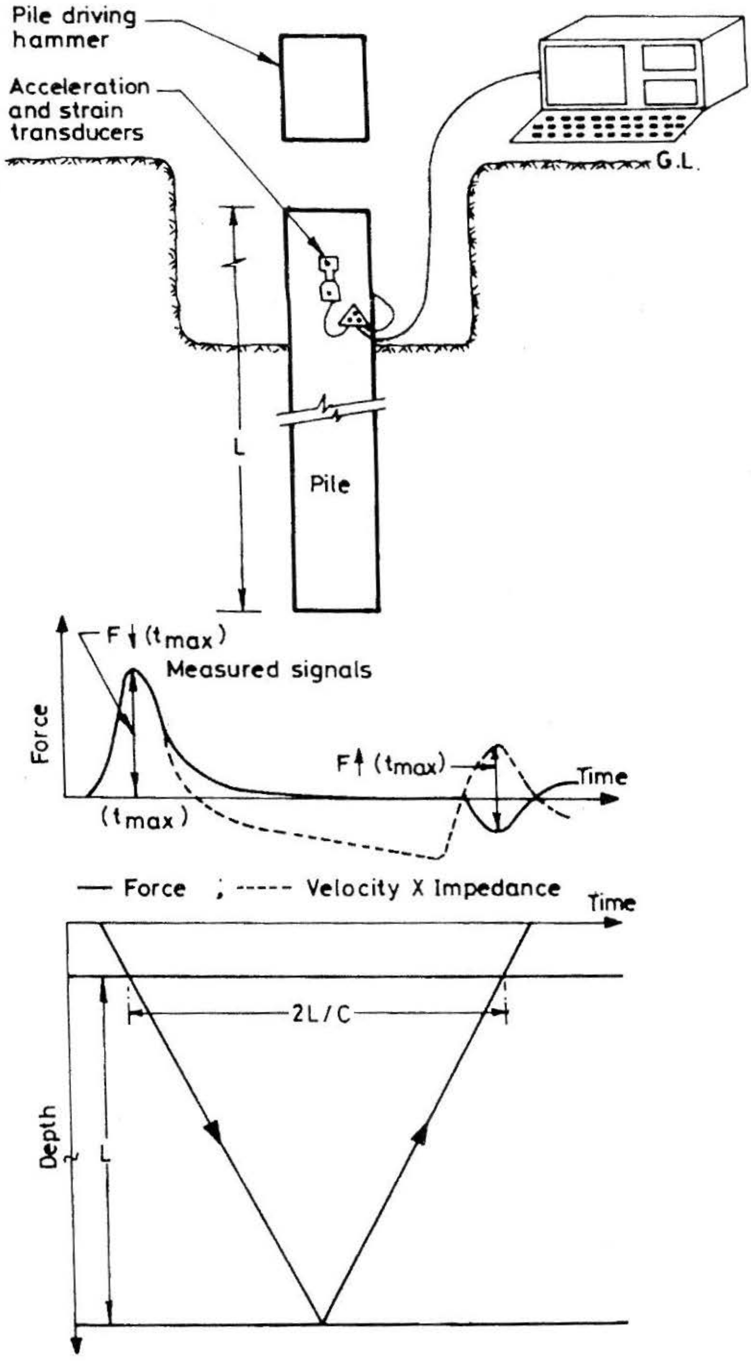

FIGURE 1 : System Concept for Dynamic Load Test

$$F\uparrow = (F - ZV)/2 \qquad (2)$$

where F = force at the section;

V = particle velocity at the section;

Z = pile impedance

= EA/C (E = Young's modulus; A = Cross-sectional area of pile).

From Eqn.s (1) and (2), it is clear that if F and V are measured, $F\downarrow$ and $F\uparrow$ can be determined. For the actual case, these equations have been modified, considering the effect of end conditions of pile, of discontinuities in pile section and of soil interaction forces acting on pile (Middendrop and Van Weele, 1986). In the model used, the skin friction which in reality acts continuously along the pile, is substituted by skin friction acting at discrete points along the pile (Fig. 2). Between any two points, the pile can be considered frictionless and so the waves leaving at one of the discrete points will propagate undisturbed to the next discrete point. At the discrete points, the equations for transmitted and reflected wave are derived by fulfilling equilibrium and continuity conditions (Fig. 3).

Sub-soil Strata and Piles

The sub-soil at the site shows large variations from place to place but generally consists of three layers. The top poorly graded fine dense sandy strata is 2m to 8m thick followed by soft marine clay strata upto 12m to 20m depth. Below this, stiff to very stiff clay layer exists. For the present study, three piles, PT1, PT56 and CT175 were selected which were put to the static load tests under normal execution programme. At the location of these piles, the top sandy strata is about 6.5m thick followed by soft clay strata upto about 14m depth. The bore log data along with penetration resistance, one near to piles PT1 and PT56 and the other near to pile CT175 are reproduced in Fig. 4. In general the strata at both these locations is almost the same.

The precast concrete piles are of 400mm × 400mm square cross-section and 22.7m length. The concrete is of M30 grade and the high strength steel bars reinforcement is about 2 percent of the pile cross-sectional area. To minimise the effect of negative drag, anticipated due to settlement of soft clay layer under the extra fill of about 2.5m height placed to raise the ground level, the piles are coated with bitumen upto 10m depth. The piles have been penetrated through dense sandy strata by water jetting and then these have been driven by 4300kg to 4500kg hammers with a height of fall about 1.2m with a total rated energy of about 50kNm per blow.

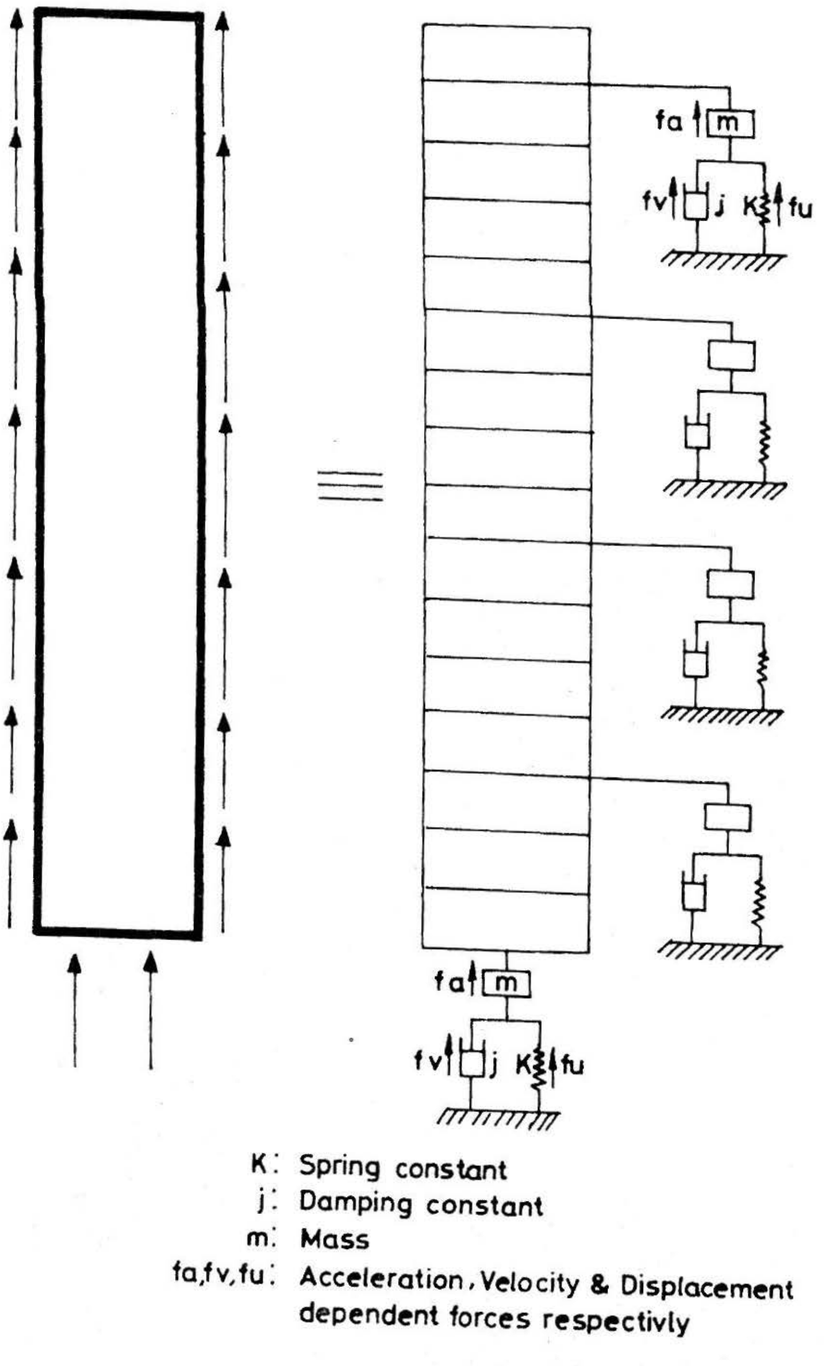

FIGURE 2 : Representation of Pile-Soil Model

Estimation of Bearing Capacity and Load vs. Displacement Response

The piles were tested after more than one month of their installation. The piles were struck by giving impact with the hammer of 4500kg with varying height of fall. The processed signals for each blow were stored on the fixed disk for further processing and signal matching.

The static bearing capacity of a pile from dynamic load test data (stress wave measurement) has been estimated by signal matching technique using a

$$f^-_{n,i} = f^+ \times \frac{Z_N - Z_{N+1}}{Z_N + Z_{N+1}} + (2 \times f^- + W_{n,i-1}) \times \frac{Z_N}{Z_N + Z_{N+1}}$$

$$f^+_{n,i} = f^- \times \frac{Z_N - Z_{N+1}}{Z_N + Z_{N+1}} + (2 \times f^+ - W_{n,i-1}) \times \frac{Z_{N+1}}{Z_N + Z_{N+1}}$$

in which : f^+ – Incident downward travelling wave ($-f^+_{n-1,i-1}$)

f^- – Incident upward travelling wave ($-f^-_{n+1,i+1}$)

$f^+_{n,i}$ – Transmitted downward travelling wave

$f^-_{n,i}$ – Transmitted upward travelling wave

Z_N – Impedance of pile element N

Z_{N+1} – Impedance of pile element N+1

$W_{n,i-1}$ – Friction force acting onnode n

n – Discrete point or node number

N – Pile element number

i – Time step number

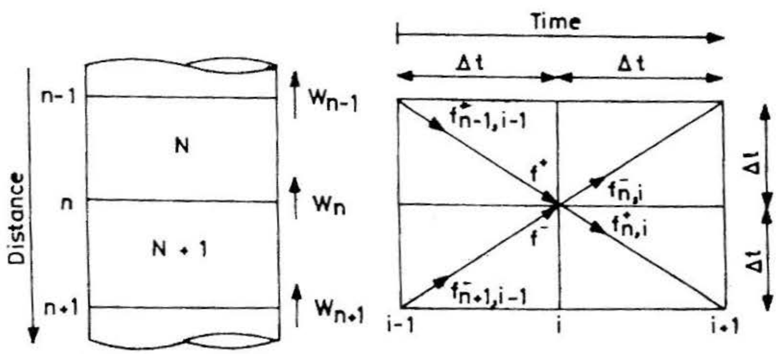

FIGURE 3 : Downward and Upward Travelling Waves (After Middendrop and Van Weel, 1986)

software (Middendrop and Bielefeld, 1993). A schematic representation to estimate static bearing capacity and load vs. displacement response is given in Fig. 5. For obtaining a reliable estimate of static bearing capacity and load vs. displacement response, it is desirable to establish relationship between the static and dynamic pile load test results at least for one pile at a particular site to derive appropriate soil model. Accordingly in the present case, pile PT1, which had already been subj ected to initial static load test prior to dynamic load test, has been taken as reference pile. First by using a program and soil parameters i.e. spring constants defined as yield strength(unit shaft/toe resistance), fy/quake (displacement), Uq, based on sub-soil investigation data and maximum total and net pile head displacements taken as 40mm and 34mm respectively from static load test results in the present case, a best fit match

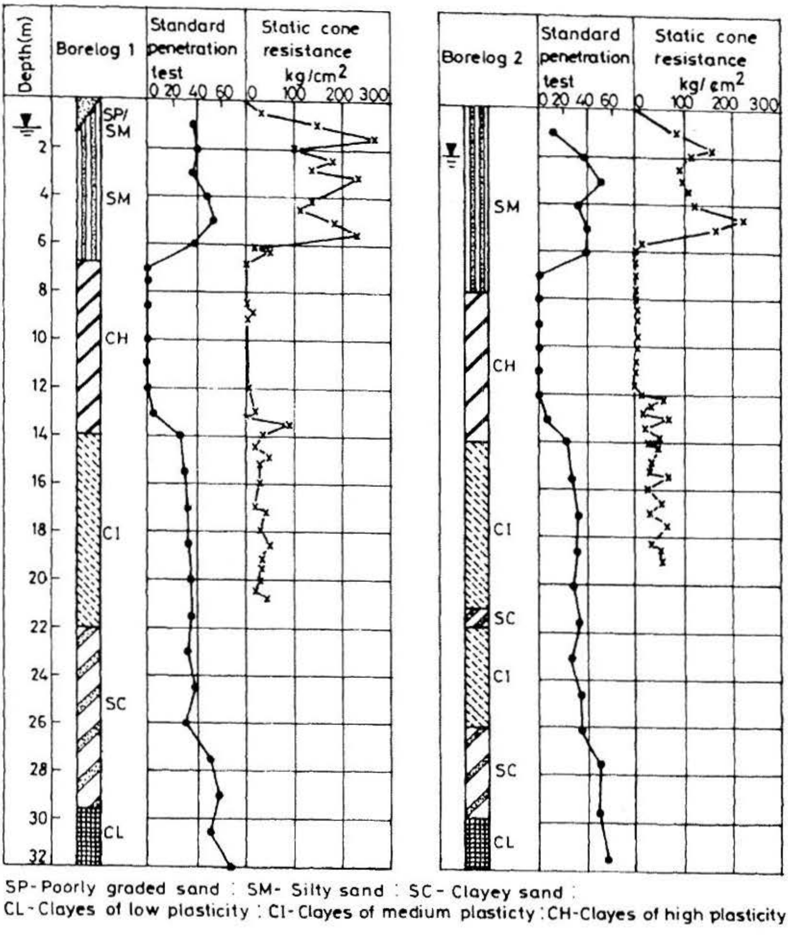

FIGURE 4 : Sub-soil Profile and Penetration Resistance

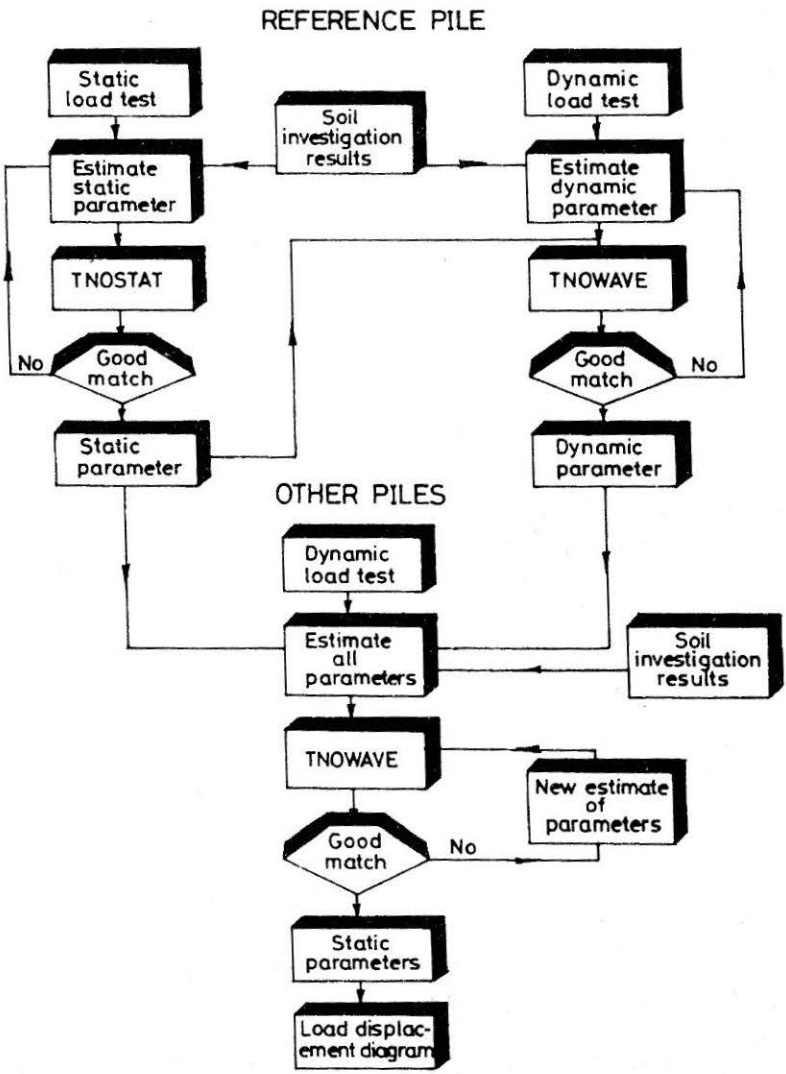

FIGURE 5 : Static Bearing Capacity Estimation Scheme from DLT Data

has been obtained between calculated and observed load vs. displacement response at pile head by adjusting soil yield strength, fy and quake, Uq, surrounding the model pile through a number of iterations. Subsequently the measured impact force during dynamic load test has been taken as input at the level of measurement for model pile with soil parameters (damping constants) along shaft and toe in the computer program. By using program, the soil parameters have been adjusted such that a best fit match is obtained between the calculated and measured upward travelling waves at the measurement level through a number of iterations. At every step, the match between calculated and observed static response has been checked. Finally, when best fit matches between calculated and observed upward travelling waves as well as load vs. displacement response have been obtained, the derived soil model has been taken as representative one.

Soil model used for signal matching considers the spring constants to represent asymmetric elasto-plastic behaviour and damping constants as velocity dependent linear behaviour (Fig. 6).

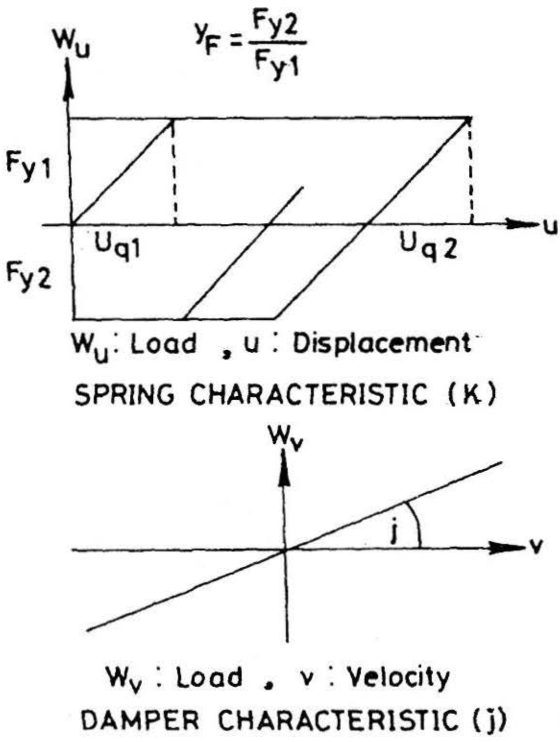

FIGURE 6 : Soil Models Characteristics

First estimate of soil parameters i.e. yield stress, quakes, yield factor and damping constants, considering sub-soil investigation data, is based on the values as given in Table 1. Best fit matches, have been obtained by varying these values considering variation in soil strata and effect of pile installation on soil properties. It has been reported (Middendrop and Bielefeld, 1993) that the quake values and the yield factors do not differ very much for each soil material. This is in contrast with the yield strength and the damping constant values which can differ substantially per soil layer and with depth.

The soil model, derived as above, has been used to estimate static bearing capacity of the piles PT56 and CT175. For these also, the measured impact force for each individual pile has been taken as input at the measuring level of model pile. The best fit matches between calculated and measured upward travelling waves for each pile have been obtained by re-adjusting primarily damping constants. In view of local variations in soil strata and pile installation effects from place to place, some readjustments in spring constant data i.e. yield strength and quakes have also been made to arrive at the best fit final matches.

For all the above three signal matches, the spring constants have been considered same both for loading and unloading cycles (Fig. 6). It may be mentioned here that the quality of signal match is not very sensitive to the variations of shaft spring constants. But it is sensitive to variations in the yield strength along pile shaft and spring constant as well as yield strength at pile toe. It is also sensitive to variations in damping constants along pile shaft as well as at toe.

Table 1
Typical Values of Soil Parameters for DLT Signal Matching

Type of Soil	Yield Strength (N/mm^2)	Quake 1 (mm)	Quake 2 (mm)	Yield Factor	Damping Constant $\times 10^6$ (Ns/m^3)
Silt and Soft clay	0.01 - 0.03	1 - 20	1 - 20	1	0.1 - 1.0
Very Stiff Clay	0.05 - 0.20	1 - 20	1 - 20	1	0.1 - 1.0
Loose Sand	0.01 - 0.03	1 - 20	1 - 20	1	0.01 - 0.1
Dense Sand	0.03 - 0.07	1 - 20	1 - 20	1	0.01 - 0.1
Dense Gravel	0.05 - 0.10	1 - 20	1 - 20	1	0.01 - 0.1

The soil model data corresponding to best fit matches provides the estimated bearing capacity and load vs. displacement response of the piles. From the soil model corresponding to best fit matches, the springs (static part) are activated and by adding the plastic resistances of both the shaft friction springs and the toe spring, the ultimate static bearing capacity of the pile is obtained. Similarly by using spring constants data both for shaft and toe, the static load vs. displacement response of the pile is calculated. The program calculates load vs. displacement response with an increasing displacement up to defined maximum total displacement limit and continues with a decreasing displacement up to a defined net displacement limit.

Results and Discussions

The best fit matches for the three piles, PT1, PT56 and CT175 are shown in Fig. 7 and the corresponding soil model data are given in Table 2. The estimated skin friction, toe resistance and the total bearing capacities are reported in Table 3. Since the top 0.5m of the piles (above the measurement level) was exposed, it has been neglected and as such the piles of 22.2m length are considered. Further the piles for about 0.5m length below measuring level were also almost exposed, no soil data has been considered for this length. In case of pile CT175, the pile was slightly in contact with loose sandy soil on one side in this portion, therefore, damping constant has been considered for this length as well. The low values of quake, less than 0.5 per cent of pile width for top 8 m length, normal range 0.5 to 1 per cent of the pile diameter/width, may probably be due to bitumen coating.

The load vs. displacement response for the three piles under static load tests is shown in Fig. 8. The trends of curves clearly indicate the behaviour of piles as predominantly friction piles. Also the slope of curves suggest the mobilisation of friction corresponding to pile head displacement, 4mm to 6mm. Further, the load vs. displacement response of these piles is identical to that reported by Raju and Gandhi (1989) for initially installed piles. The data for one of the similar instrumented pile (Raju and Gandhi 1989) indicated toe bearing component 38t (372 kN) and major portion of friction component by stiff clay layer as expected. The toe bearing component of 352kN, 384kN and 384kN for the piles PT1, PT56 and CT175 as shown by the estimates from dynamic load tests data (Table 3) is very close to this value. The skin friction component is about 1600kN for the three piles, major contribution coming from the stiff to hard clay layer.

A comparison between static load vs. displacement responses as estimated from the spring constants data both for shaft and toe corresponding to best fit signal matches as outlined above using program and those obtained from static load tests for the three piles PT1, PT56 and CT175 (Fig. 9) show a good agreement. Based on the above discussions it can be construed that

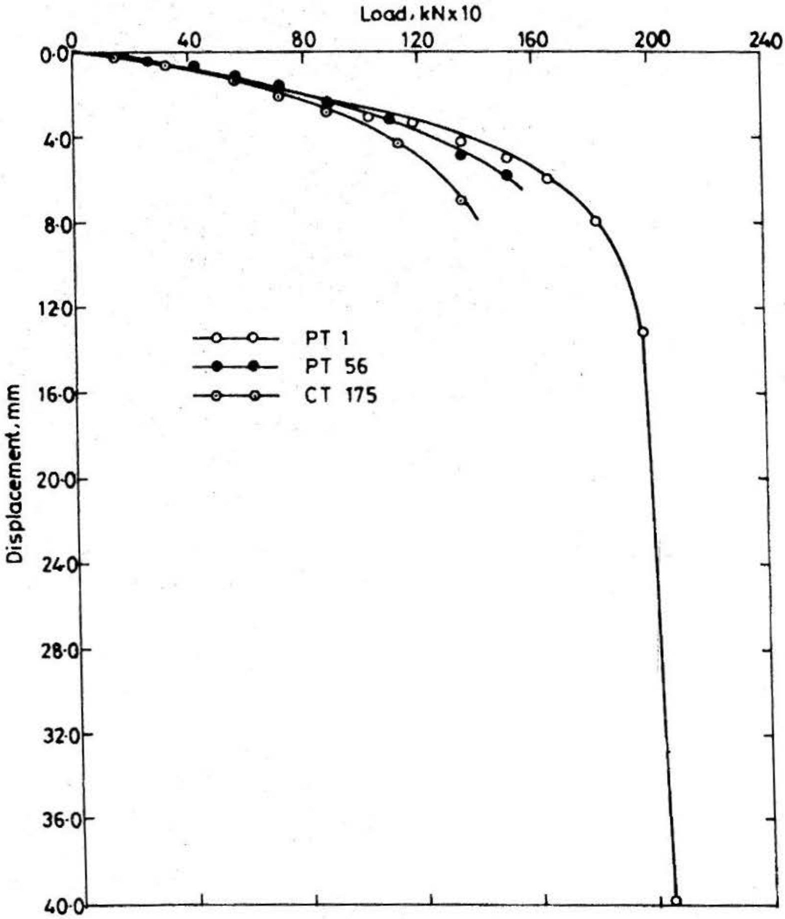

FIGURE 7 : Best Fit Signal Match of Upward Travelling Waves

Table 2
Soil Model Data

Layer No.	Skin Friction Data Layer Thickness (m)		Pile PT1			Pile PT56			Pile CT175		
			Spring Data		Damper Data	Sprmg Data		Damper Data	Spring Data		Damper Data
			fy (N/mm^2)	Uq (mm)	(Ns/m^3)	fy (N/mm^2)	Uq (mm)	(Ns/m^3)	fy (N/mm^2)	Uq (mm)	(Ns/m^3)
1	0.5	Start	0	0	0	0	0	0	0	0	1000
		End	0	0	0	0	0	0	0	0	1000
2	3.0	Start	0.01	1.5	0	0.01	1.5	0	0.009	1.4	1000
		End	0.01	1.5	1000	0.01	1.5	2000	0.009	1.4	4E+4
3	5.0	Start	0.003	1.7	2.3E+5	0.002	1.7	2000	0.003	1.7	6E+4
		End	0.003	1.7	2000	0.002	1.7	5E+4	0.003	1.7	6E+4
4	4.0	Start	0.009	2	2E+4	0.008	2	5E+4	0.008	2.1	IE+5
		End	0.009	2	2E+4	0.008	2	8E+4	0.008	2.1	4E+4

Table 2
Contd.

Layer No.	Skin Friction Data Layer Thickness (m)		Pile PT1			Pile PT56			Pile CT175		
			Spring Data		Damper Data	Spring Data		Damper Data	Spring Data		Damper Data
			fy (N/mm^2)	Uq (mm)	(Ns/m^3)	fy (N/mm^2)	Uq (mm)	(Ns/m^3)	fy (N/mm^2)	Uq (mm)	(Ns/m^3)
5	3.0	Start	0.07	2	9E+4	0.08	2.1	9E+4	0.082	2.1	1E+5
		End	0.07	2	2E+5	0.08	2.1	1E+5	0.082	2.1	6E+4
6	6.7	Start	0.09	2	2E+5	0.08	2.2	1.3E+5	1.182	2.2	9E+4
		End	0.12	2	9E+5	0.125	2.2	1.3E+5	0.118	2.2	1.8E+5
Toe Resistance		Toe	2.2	6	9.5E+6	2.4	6	2.0E+6	2.4	6	1.0E+6

Spring Constant = fy/Uq

where fy = Yield strength of soil surrounding the pile;
Uq = quake

Table 3
Static Bearing Capacities

	Layer No.	Layer Thickness	Estimated Resistance (N)		
			Pile PT1	Pile PT56	Pile CT175
Skin Friction	1	0.5	6080	6080	5470
	2	3.0	4.43E+4	4.37E+4	4.01E+4
	3	5.0	2.73E+4	1.94E+4	2.67E+4
	4	4.0	9.17E+4	9.23E+4	9.36E+4
	5	3.0	3.52E+5	3.89E+5	3.99E+5
		6.7	1.08E+6	1.06E+6	1.03E+6
	Total	–	1.61E+6	1.61E+6	1.60E+6
Toe Resistance	Toe	Toe	3.52E+5	3.84E+5	3.84E+5
Total Capacity	–	–	1.96E+6	I.99E+6	I.9SE+6

in the present case the dynamic load tests data provided a good estimate of the static bearing capacity and also of the load vs. displacement response.

The estimation of bearing capacity from dynamic load test data based on one dimensional stress wave propagation is mainly dependent on :

1. impact energy,
2. quality of measured signals,
3. soil model, and
4. one dimensional stress wave solution method.

The impact energy (hammer blow) should be concentric to the pile as the eccentricities affect the quality of signals particularly strain (force) measurement, heavily. Also the energy should be sufficient enough for pile movement to mobilise both friction and toe bearing component. The soil model should represent the pile-soil interaction phenomenon as close as possible. Further, the numerical values of various constants should be as much realistic as possible. The best way to derive these is to compare one static load test data with dynamic load test data on the similar pile in the similar soil condition. Past local experience in the similar conditions can also help to arrive at realistic values of soil parameter constants. The bearing capacity estimate is also dependent on the method (assumption made) used to solve one dimensional stress wave propagation through pile.

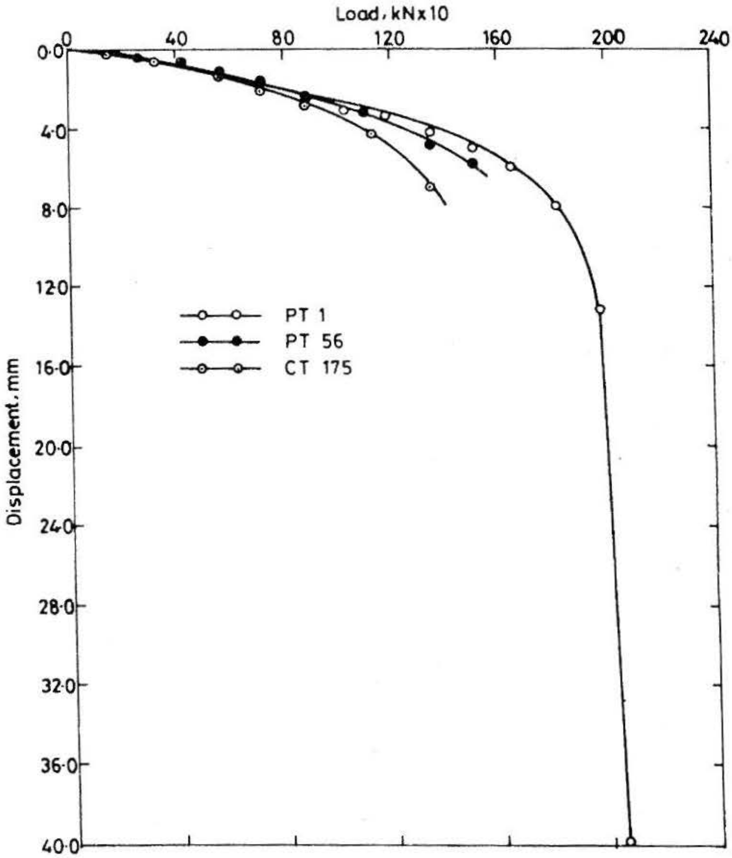

FIGURE 8 : Observed Static Load vs. Displacement Response of Piles

In addition to these, the estimates also depend on quality of signal matching. In view of large variables involved, there is not a unique solution resulting in many signal matches. Therefore, the best fit signal match is very much dependent on the experience of person in pile-soil interaction and one-dimensional stress wave propagation. In view of these facts, the technique should be used by qualified and experienced personnel. The technique may prove to be very useful and cost effective for large project sites to check the safe loads of working piles by performing PDA or dynamic load tests on these after establishing a relationship between driving resistance, obtained from PDA or dynamic load tests, and safe load from static load tests initially. Additionally, the measurements during these tests help in establishing performance of piles, i.e. stress level, blow count, integrity etc. The technique can also be used with advantage for other piles, where driving equipment is easily available and a knowledge about realistic soil model exists.

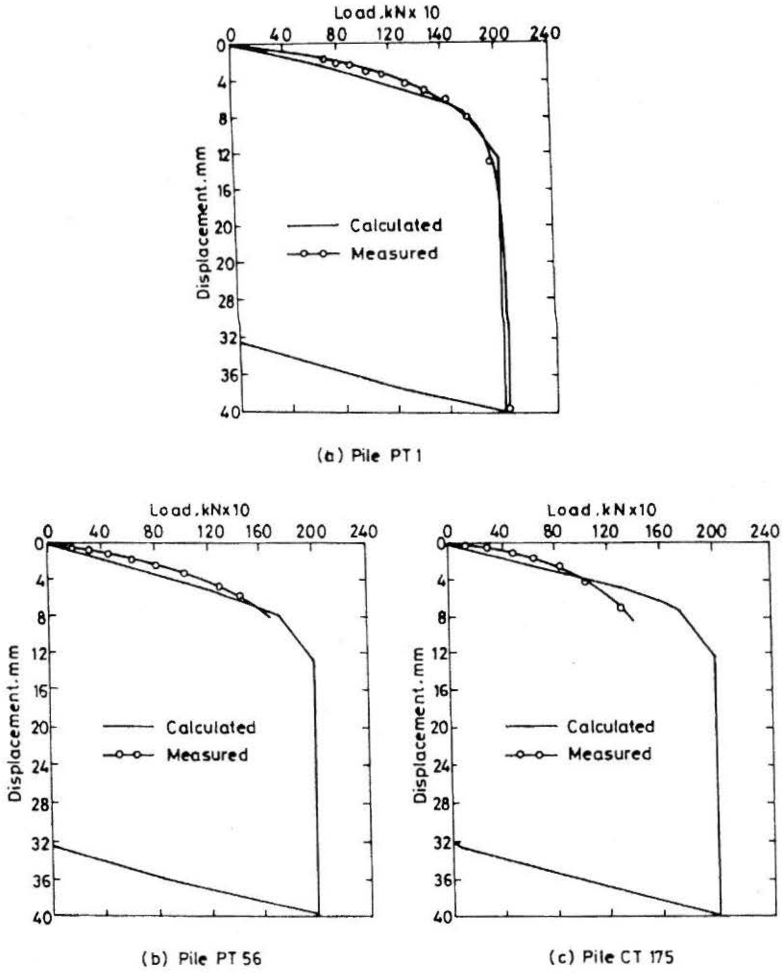

FIGURE 9 : Calculated and Measured Static Load vs. Displacement Response of Piles

Conclusions

Based on the work reported herein it can be concluded that the bearing capacity and load vs. displacement characteristics of precast concrete piles in layered clayey soil deposits can be estimated reasonably well by stress wave measurements following the signal matching technique based on one dimensional stress wave solutions applicable to piles. At large project sites, the technique can be used with advantage to monitor performance and to estimate safe loads after direct comparison between static load test and dynamic load test or PDA test atleast for one pile. The soil model used for signal matching can best be derived by using the static and dynamic load test data atleast on one pile at a particular site. An estimate of soil model may also be made on the basis of guiding past local experience on identical piles in similar soil conditions. For obtaining realistic estimate of bearing capacity and load vs. displacement response from dynamic stress wave measurements, well qualified and experience personnel having knowledge of pile-soil interaction and one dimensional stress wave propagation through piles, are required.

Acknowledgement

The work reported herein forms a normal programme of research and development of the Central Building Research Institute, Roorkee and the paper is being published with the permission of Director. The authors are grateful to Prof. V.S. Raju, Director, I.I.T., New Delhi (formerly Prof., I.I.T., Madras), Dr. S.R. Gandhi, Asstt. Prof., I.I.T., Madras and the authorities of M/s. NFCL, Kakinada for making the piles available for testing and providing facilities at site.

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