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Dynamic analyses and field observations on piles in Kolkata city

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Abstract. In the present case study, High Strain Dynamic Testing of piles is conducted at 3 different locations of Kolkata city of India. The raw field data acquired is analyzed using Pile Driving Analyzer (PDA) and CAPWAP (Case Pile Wave Analysis Programme) computer software and load settlement curves along with variation of force and velocity with time is obtained. A finite difference based numerical software FLAC3D has been used for simulating the field conditions by simulating similar soil-pile models for each case. The net pile displacement and ultimate pile capacity determined from the field tests and estimated by using numerical analyses are compared. It is seen that the ultimate capacity of the pile computed using FLAC3D differs from the field test results by around 9%, thereby indicating the efficiency of FLAC3D as reliable numerical software for analyzing pile foundations subjected to impact loading. Moreover, various parameters like top layers of cohesive soil varying from soft to stiff consistency, pile length, pile diameter, pile impedance and critical height of fall of the hammer have been found to influence both pile displacement and net pile capacity substantially. It may, therefore, be suggested to include the test in relevant IS code of practice.

Keywords: FLAC3D; CAPWAP; PDA; displacement; pile capacity; High Strain Dynamic Testing; pile impedance

1. Introduction

Pile load tests, being in–situ test, are the most reliable approach for determining the allowable loads on piles. It is implemented by applying loads of known magnitude on the pile top and measuring the pile movement and establishing the load – settlement relationship to determine the allowable load capacity of the pile. Various soil properties like cohesion, angle of friction, consistency limits and grain size distribution considerably influence the allowable load capacity of

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the pile. The static load bearing capacity of the pile is limited by the capacity of the supporting soils and structural strength of the pile shaft, which in turn is dependent upon the material properties and specification requirements of the pile. Static pile load tests are expensive, time consuming and sometimes become impossible to perform due to physically carrying the heavy equipment and accessories in many terrain.

Therefore dynamic pile testing is practiced in current days and becoming more and more popular for determining pile capacity. This test carries out dynamic measurements of pile force and motion during impact of a falling hammer, for evaluating the axial static bearing capacity of a pile. When a hammer strikes a pile head, the force or stress generated, is transferred to the pile head. It builds progressively with time up to a peak value and then decays to zero, with the entire event occurring for a few hundredths of a second. During this time, the transfer of energy initiates a compression strain wave which propagates down the pile at the speed of sound, and at the pile toe is reflected back towards the pile head. The reflected wave is in compression or tension depending upon whether the embedment of the pile toe is in dense soil or soft soil respectively.

The soil pile response and nature of loading are widely influenced by the source of dynamic loading. The piles are subjected to horizontal translation, vertical oscillations, rocking and torsion in machine foundations. The responses of pile foundations under such dynamic loadings are more complex when compared to static loads. The interaction between piles and surrounding soil, in the dynamic analyses of soil – pile foundation is a challenging topic of foundation dynamics. However very little information is available on the dynamic behavior of pile foundations because only a few tests have been conducted so far and the test is not yet included in IS code of practice.

2. Previous studies

The behavior of pile foundations subjected to static and dynamic loading in non –liquefied and liquefied soil have been analyzed by experimental methods like centrifuge testing, analytical methods, numerical techniques and simplified approaches. Poulos (1968) considered axial and lateral modes of vibration for static analyses of end – bearing and floating piles and pile groups and provided the base for the development of various pile dynamic theories. Nogami and Novak (1977) performed dynamic analyses of a single end bearing pile subjected to horizontal and vertical vibration without considering coupling between the two modes of vibration. Novak *et al*. (1978) proposed a plain strain model representing a rigid cylinder extending upto infinity in an infinite medium which underwent uniform lateral displacements or rotation when subjected to harmonic dynamic loading. Centrifuge experiments were performed by various researchers for determining the response of piles in the form of deflection and bending moment profiles when subjected to different types of loading (axial, lateral, dynamic or their combinations) in both non–liquefied soil (Choudhury *et al*. 2006, 2008) and liquefied soil (Abdoun *et al*. 2003, Brandenberg *et al*. 2005, Tobita *et al*. 2005, Knappett and Madabhushi 2009a, 2012, Haigh and Madabhushi 2011, Motamed *et al*. 2013, Shen *et al*. 2013). It was concluded that maximum bending moment occurred at the interface of non – liquefied and liquefied layers and thickness of the liquefying layer influenced the bending response of piles. Numerical modeling of both single piles and pile groups were conducted by various researchers using finite difference and finite element based software packages and behavior of piles in various soil types were obtained (Maheshwari *et al*. 2004, Liyanapathirana and Poulos 2005a, Karthigeyan *et al*. 2006, Knappett and Madabhushi 2009b, Rayhani and El Naggar 2008, Maiorano *et al*. 2009, Hussein *et al*. 2010,

Table 1 Latitudes and longitudes of the 3 different locations in Kolkata city as considerd in the present study

Locations	Latitude (°N)	Longitude (°E)
Aliah University Campus at Park Circus	22°32'43.8"	88°22'7.60"
Bandhab Nagar under Dum Dum Municipality	22°37'52.5"	88°24'56.1"
Kendriya Bihar (Phase II) at Belghoria Expressway	22°39'30.7"	88°22'27.7"

Rose *et al*. 2013, Biswas *et al*. 2013, Wu *et al*. 2013). Analytical studies on pile foundation were implemented by considering various soil models exposed to lateral dynamic loading and resulting expressions on bending moment were expressed by various researchers (Tokimatsu *et al*. 1998, El Naggar and Bentley 2000, Nikolaou *et al*. 2001, Mostafa and El Naggar 2002, Dobry *et al*. 2003, Liyanapathirana and Poulos 2005b, Basu and Salgado 2008, Basu *et al*. 2009, Seo *et al*. 2009, Choudhury *et al*. 2009, 2015, Amar Bouzid 2011, Amar Bouzid *et al*. 2013, Phanikanth *et al*. 2013a,b, Phanikanth and Choudhury 2014).

In the present case study 3 different locations at and near Kolkata city of West Bengal in India have been considered where High Strain Dynamic Pile Testing was executed. The various locations along with their respective latitudes and longitudes are given in Table 1. The test was implemented by subjecting the pile to several blows using a hammer dropped from various heights. The raw data obtained were analyzed using Pile Driving Analyzer (PDA) and CAPWAP (Case Pile Wave Analysis Programme) computer software. The distribution of shaft resistance along depth of the pile, load – settlement curve and the variation of force and velocity with time were obtained for all the 3 different locations of Kolkata city. Further the field results were compared with the results obtained from the finite difference based numerical software FLAC3D which was used for simulating field conditions by creating a similar model of a pile embedded in ground having soil properties similar to the field condition. The net pile displacement and ultimate pile capacity determined from the field tests and estimated using numerical analyses were compared and the results were found to be in close agreement. Moreover the net pile displacement and ultimate load carrying capacity of the pile at the various locations in Kolkata city were influenced by the top layers of the soil which is usually soft clay or clayey silt in nature, diameter and length of the pile and critical height of fall of the hammer.

3. Soil profile

The site conditions prevailing at 3 different locations of Kolkata city in India as considered in the present study, i.e., Aliah University Campus at Park Circus, Bandhab Nagar under Dum Dum Municipality and Kendriya Bihar (Phase II) at Belghoria Expressway were explored by sub – soil investigations and together with the average Standard Penetration Test (SPT) values (N_{avg}) have been tabulated in Table 2. It is observed that at almost all the sites, the top layers of the soil strata are composed of soft clayey silt or silty clay with the presence of kankars, rubbish, brick – bats and fly ash. The SPT N values recorded in these layers, extending for a depth of 2 m, varied from 2 to 4, thereby indicating soft soil. At Aliah University campus at Park Circus, the soil strata is generally composed of silty clay with clayey silt for a depth of 19 m, with a variation in its consistency from medium brownish to soft grey (thickness 9 m) to stiff bluish grey and finally very stiff yellowish clay. Beyond 19m and extending upto 28 m, medium to dense grey silty fine sand is observed having an average SPT N value around 30. Similar soil conditions are also observed at Kendriya Bihar (Phase II), where a gradual increase in SPT N value with depth is observed due to stiffening of the underlying soil layers with age. However at Dum Dum Municipality it is observed that the soil stratum at a depth of 6 m is composed of deep grey medium stiff silty clay with fine sand. It changes to dark grey silty sand at a depth of 12 m to whitish grey medium – dense silty sand at a depth of 19 m and finally to dark grey dense silty fine to medium grained sand at a depth of 33 m and extending to 40 m. It is observed that at Dum Dum Municipality, the primary soil type is fine to medium grained sand which is a contrast to the

Table 2 Soil profile at 3 different locations in Kolkata city as considerd in the present study

Locations	Stratum	Thickness of layer (m)	Soil description	N_{avg} *	V_s (m/s)**	G_{max} (MPa)***
Aliah University Campus at Park Circus (GWT = 1.2 m)	I	0.5	Fill of rubbish with brickpieces	5	144.16	42.40
	II	2.5	Medium brownish grey clayey silt	2	101.78	15.02
	III	9	Soft to very soft grey silty clay	2	101.78	15.02
	IV	3	Medium to stiff grey silty clay	7	163.83	45.63
	V	4	Very stiff yellowish clayey silt	15	218.87	91.02
	VI	11	Medium to dense grey sandy silt / silty fine sand	29	281.17	156.54
Bandhab Nagar under DumDum Municipality (GWT = 6.0 m)	I	1	Filling with mixed soil	1	78.21	10.70
	II	6	Grey loose silty sand with clay binders	2	101.78	15.54
	III	6	Dark grey medium stiff silty clay with fine sand	4	132.45	33.86
	IV	7	Dark grey medium silty sand with clay binder	11	194.53	71.90
	V	14	Whitish grey medium to dense silty sand	40	317.72	185.75
	VI	4	Dark grey fine to medium grained dense silty sand	46	335.05	224.52
	VII	3	Yellowish grey very dense silty sand	71	395.13	312.26
Kendriya Bihar (Phase II) at Belghoria Expressway (GWT = 1.7 m)	I	2	Filling with grayish clayey silt and fly-ash	1	78.21	10.25
	II	2	Bluish grey soft clayey silt	2	101.78	17.61
	III	9	Blackish very soft to soft silty clay	2	101.78	15.02
	IV	6	Bluish grey medium to stiff clayey silt	20	244.15	110.28
	V	6	Yellowish grey medium fine silty sand	21	248.72	120.63
	VI	5	Bluish grey very stiff clayey silt	24	261.66	139.68

* N_{avg} – Average field SPT N value of the particular soil stratum
** V_s (m/s) – Shear wave velocity of the particular soil stratum obtained by proposed formulation of Chatterjee and Choudhury (2013)
*** G_{max} (MPa) – Low strain shear modulus of the particular soil stratum obtained through Eq. (2)

typical soil profile of Kolkata city wherein the main soil type is clay. It is seen that for all the boreholes, with depth the soil layer becomes stiffer and denser and SPT *N* value increases considerably. This can be attributed to the presence of dense soil layers which has become stiffer and harder with time and due to the transfer of stress from the overlying layers. It is further observed that the location of ground water table is at a shallow depth of 1.5 m which is quite common for Kolkata city except at Dum Dum Municipality where water table depth is 6.0m respectively. Due to the lack of specialized personnel and non–availability of necessary free space, seismic Cross Hole tests and Multichannel Analysis of Spectral Waves (MASW) tests were not performed at these sites. Hence the magnitude of shear wave velocity (V_s) of each layer, required in the finite difference based numerical software FLAC3D, is evaluated using the correlations proposed between shear wave velocity (V_s) and uncorrected SPT *N* value for Kolkata city, as given by Chatterjee and Choudhury (2013)

$$V_s = 78.21N^{0.38} \tag{1}$$

As it is normally practiced, in the present study also the low strain shear modulus ($G_{\max}$) was determined using the shear wave velocity (V_s) (computed using Eq. (1)) and mass density of the soil (ρ), as indicated by the following equation

$$G_{\max} = \rho V_s^2 \tag{2}$$

4. Pile details and loading

The details of the various piles tested at the 3 different locations in Kolkata city are tabulated in Table 3. High Strain Dynamic Pile Load Tests were conducted on the vertical RCC bored cast – in – situ piles. RCC or Reinforced cement concrete piles are formed by making a hole in the ground by boring, then lowering the reinforcement cage and filling it up with concrete, at the site. These are usually straight bored piles. The reinforcement is provided in the piles as per the design requirements existing at the particular site. These piles have length extending upto 50 m with a high allowable load carrying capacity around 10,000 kN. RCC piles are generally used since they have negligible displacement and allow no risk of ground heave. The length of these piles can be varied and the soil at the site can be checked and varied with the geotechnical investigation data. The installation of these piles also involves very little noise and vibration. The weak portion of the pile shaft, i.e., the top portion of the pile above the cut off level, where the concrete is not cast with care and not much aggregate is encountered, was dismantled and the pile shaft was built upto the test level using concrete of the same grade and similar pile reinforcement. The Young's Modulus (E), specific weight (ρ) and damping coefficient (ξ) of the concrete pile were kept constant at 39.26 GPa, 24.07 kN/m^3 and 2% respectively. The velocity of the stress wave (c), propagating down the concrete pile, generated due to impact on the pile head by the hammer, was calculated using the following equation as given by Fellenius (2012)

$$c = \sqrt{\frac{E}{\rho}} \tag{3}$$

The magnitude of the material constant, impedance (Z), an important factor for wave propagation, which is a function of pile modulus (E), cross sectional area of the pile (A) and velocity of propagating wave (c) in the pile is computed using the relation also given by Fellenius (2012)

$$Z = \frac{EA}{c} \tag{4}$$

The duration of travel (t_o) which is the time when the hammer and the pile are in contact, is the time taken by the strain wave for travelling the entire length of the pile (L) twice, i.e., from the top of the pile to the bottom and back upto the pile top, is expressed as

$$t_o = \frac{2L}{c} \tag{5}$$

It is quite interesting to observe from Table 3 that the depth of embedment of the piles in all the 3 different locations of Kolkata city of India is more than 25 m. This clearly indicates that the piles could be classified as long flexible piles, since the length of the piles (L) are greater than 5 times their characteristic lengths (T) as given in Table 3 (Das 2004). The main reason of selecting long flexible piles for construction purposes in Kolkata city may be attributed to the soil conditions prevailing in the city, which does not allow to normally bear beyond foundation pressure of only 60 kPa to 70 kPa. Kolkata city is situated along the banks of the river Hoogly in the west of Kolkata. The typical soil profile of the city comprises of soft silty clay or clayey silt with organic matter upto a depth of around 13 m followed by clayey soil of stiffer consistency down to about 18 m to 20 m and this overlies a dense sand stratum. It is difficult to adopt shallow foundation for heavily loaded structures in this region because of the soft and compressible nature of the clayey or silty clay soil upto greater depth, which requires large foundation area and results in high consolidation settlement crossing the allowable limit in most cases. As a result pile foundations are required for major construction works across Kolkata city, with the most common pile types being end bearing piles and few friction piles and pile load tests are performed to confirm the design pile capacity.

Table 3 Pile details at the various sites in Kolkata city as considerd in the present study

Locations	Pile Mark	Pile length below gauges (m)	Pile embedment length (m)	Pile diameter (mm)	Pile Grade	Impedance (kN/m/sec)	Characteristic length [T] (m)
Aliah University Campus at Park Circus	P1	27.0	26.8	600	M-20	2775	2.08
Bandhab Nagar under Dum Dum Municipality	P2	26.7	26.5	500	M-25	1926.9	1.89
Kendriya Bihar (Phase II) at Belghoria Expressway	P3	25.3	25.0	450	M-25	1560.7	1.65

Table 4 Loading details on the piles at various sites in Kolkata city as considerd in the present study

Locations	Pile mark	Hammer weight (kg)	Height of fall (m)	Design load (kN)	Duration of travel [t_o] (ms)
Aliah University Campus at Park Circus	P1	2200	2.00	765	13.5
Bandhab Nagar under Dum Dum Municipality	P2	2200	1.50	740	13.3
Kendriya Bihar (Phase II) at Belghoria Expressway	P3	2200	2.00	600	12.6

In these sites under consideration dynamic pile load tests were conducted to validate the design pile capacity. The loading was applied on the cushion of pile top by dropping the hammer from various heights including 0.5 m, 1.0 m, 1.5 m and 2.0 m. The loading details on each piles is tabulated in Table 4. The cushion of pile top was composed of ply board and mild steel plate which matched the cross – sectional area of the pile head.

5. High Strain Dynamic Pile Testing

The main purpose of conducting High Strain Dynamic Pile Testing is to evaluate the static capacity and structural integrity of the pile by measuring both force and velocity. It is conducted by applying an axial impact force at the top of the deep foundation unit using a pile driving hammer or a large drop weight and measuring the subsequent force and velocity response of the deep foundation. High Strain Dynamic Pile Testing is used for evaluating various pile parameters like true static capacity of the pile at the time of testing, obtaining static load test curve, computing the total skin friction and end bearing resistance of the pile with variation of skin friction along the pile depth. Other important data like compressive and tensile stresses developed in the pile along with net and total displacement of the pile are also generated during High Strain Dynamic Pile Test.

In the present study, the dynamic pile load tests were conducted at 3 different locations of Kolkata city using a Pile Driving Analyzer (PDA) and its associated pile top force and velocity transducers. The equipments which were used for the PDA field test are:

- Pile Driving Analyzer (PDA) model PAX
- Strain transducers for measuring the strains induced under the impact of a heavy falling hammer from a pre – determined height and
- Accelerometers for recording the accelerations generated in the pile.

The PDA – PAX was equipped with PDA – W software using the Case Pile Wave Analysis Program (CAPWAP) for evaluating the static capacity of the pile. The test was done using the Pile Driving Analyzer (PDA), an instrument from Pile Dynamics Inc, USA with the Serial Number: 4009LA and testing was conducted in accordance with ASTM D4945-08 (2008).

5.1 Testing procedure

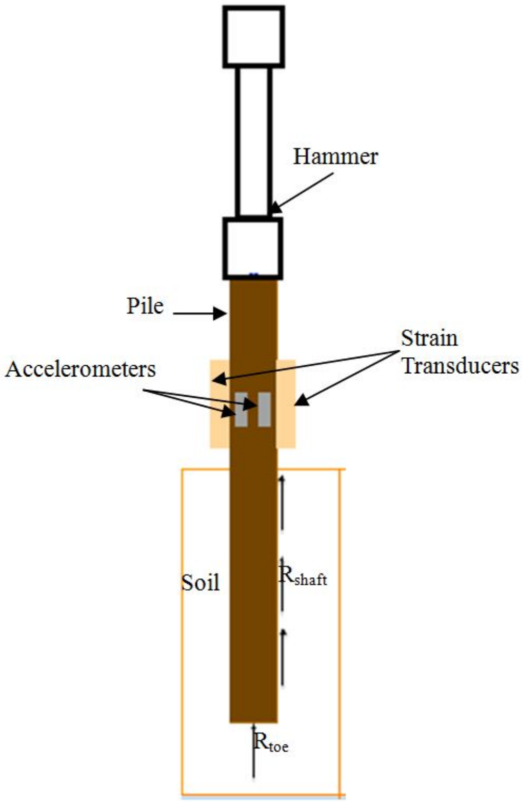


Fig. 1 Schematic sketch showing high strain dynamic testing of pile foundation as implemented in the present study

The dynamic testing on the piles were implemented by subjecting the piles to several blows during the re – striking process. During testing of the piles, dynamic measurements were accurately recorded for each hammer blow by the two strain transducers and two accelerometers that were fixed below the pile head, as shown in Fig. 1. They were mounted on opposite sides of the pile and connected to the pile driving analyzer using cables. The pile driving analyzer, which is a micro – processor based signal conditioner and digital computer, converted the strain to force and acceleration records to velocity. The signals of the velocity and force at the pile top, measured and analyzed during each strike of the pile driving hammer, were stored in the analyzer and the force and velocity time curves generated at the pile top were displayed on the laptop computer screen. The PDA also recorded the real time analogue signals of the pile top force and velocity and stored the same in the field computer unit.

The PDA measured the total resistance acting on the pile, while the portion of total resistance computed as static resistance by the analyzer was determined using the soil damping factor (J_c) which was set into the analyzer. In addition to this, a more accurate and independent measure of the applicable soil – damping factor was obtained using the CAPWAP analysis. The CAPWAP computer software analyzed a selected PDA field recording of force and velocity data for a blow which were delivered to the piles. CAPWAP analysis is based upon the principle of applying the measured velocity time record, generated at the pile top, to the top of a lumped mass and spring

Table 5 Summary of PDA and CAPWAP analyses results obtained in the present study

Pile mark	R_u (kN)	R_s (kN)	R_b (kN)	R_c (kN)	*D_y (mm)	+D_x (mm)	*+S (mm)	σ_c (kPa)	σ_t (kPa)	$E_{\max}$ (kN – m)	IF
P1	1801	1326	475	1200	7.5	9.3	1.8	16350	4724	13.97	88%
P2	1553	849	704	1035	7.4	7.9	0.5	10410	2211	5.371	81%
P3	1116	648	467	744	7.5	8.2	0.7	11220	861.5	5.213	82%

*Maximum static top displacement = D_x
+Maximum displacement at failure = D_y
*+Permanent settlement $S = D_x - D_y$

wave equation model of the pile. The pile top force time record, computed using the CAPWAP program, was compared to the actual measured force time record. The pile and soil resistance model was adjusted in an iterative procedure till the attainment of good match between both the measured and computed forces at the pile top were obtained. The pile and soil models are further used for determining the estimated static load settlement curves.

6. Results and discussions

The results obtained from the Pile Driving Analyzer (PDA) and CAPWAP analysis for the different piles located at the 3 different sites of Kolkata city of West Bengal, India have been illustrated in the form of load – displacement graphs, distribution of shaft resistance against depth and variation of measured force and velocity at the pile top against time. The magnitudes of the ultimate capacity of the pile (R_u), mobilized pile shaft resistance (R_s), mobilized pile toe resistance (R_b), maximum displacement of the piles at failure (D_y), maximum static displacement at the pile top (D_x), maximum compressive stress (σ_c), maximum tensile stress (σ_t), maximum energy transferred to the pile (E_{max}), permanent penetration of pile (set) for all impacts applied (S) and the pile integrity factor (IF) for the different piles P1, P2 and P3 have been furnished in Table 5. A factor of safety equal to 1.5 was considered for determining the safe load (R_c) on the pile. According to IS 2911 [Part1/Sec4] (1984) safe load is recommended as two-third the load corresponding to 12 mm settlement, thereby implying in a way that the factor of safety is equal to 1.5. Since the authors have made an attempt to compare the results with those of static tests, FS has been chosen as 1.5.

6.1 Pile P1 at Aliah University Campus at Park Circus

The variation of force and velocity with time, as measured using the strain transducers and accelerometers, attached to the pile driving analyzer (PDA), for Pile P1, is shown in Fig. 2(a). Initially both the force and velocity, near the top, are reaching their peak magnitude at the time of striking the hammer at the top of the pile, within a few milliseconds. Then there is decay in both force and velocity with time and depth. It is not that the force is not matching. However, the curve is not ideal due to defects in pile casting and eccentricity of the load, which can never be avoided in case of test done manually. The average capacity has been calculated by CAPWAP software analysis. Also, it is noticed that after around 20 ms, the magnitude of force is below the horizontal axes. However, in this case the force is not negative but due to unavoidable eccentricity of the

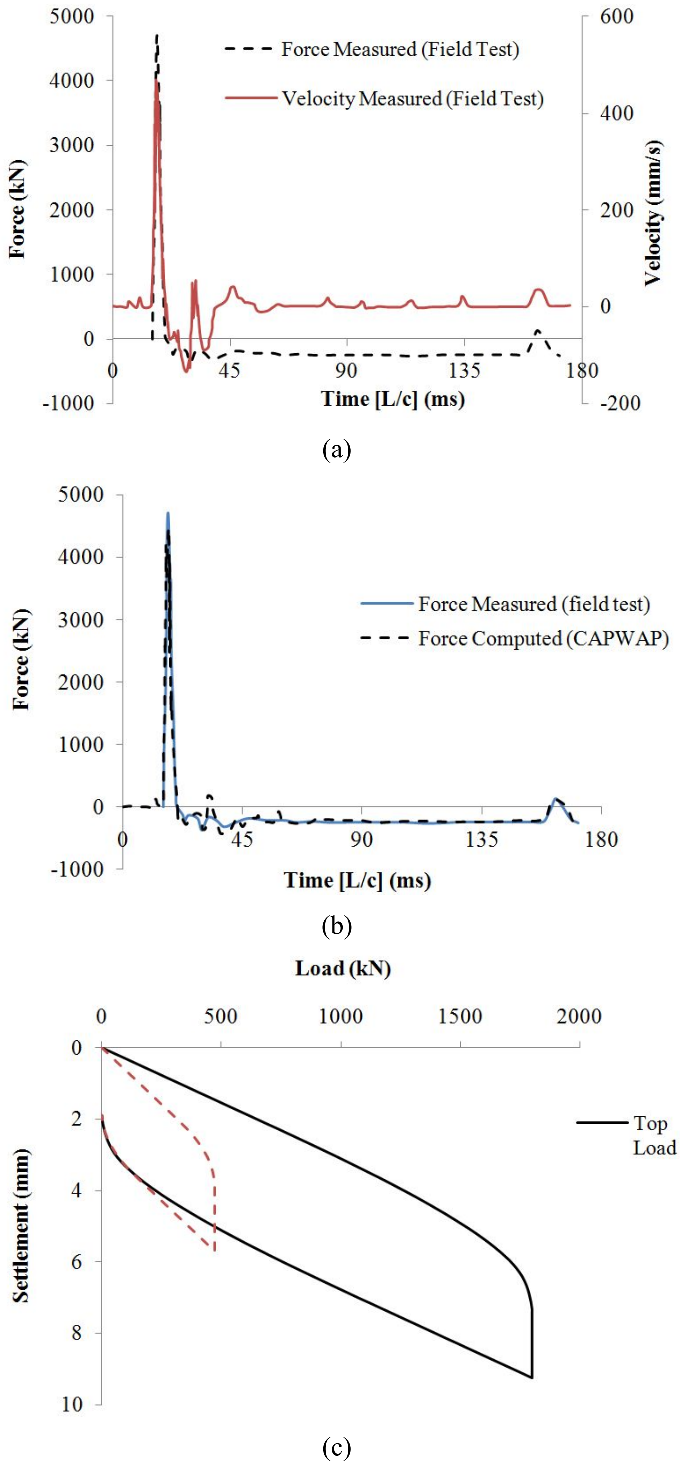


Fig. 2 (a) Variation of force and velocity against time measured using Pile Driving Analyzer for Pile "P1"; (b) Comparison of measured force using PDA with computed force using CAPWAP software for Pile "P1"; (c) Load versus settlement curve for Pile "P1" manually operated winch rig hammer without proper guide frame the base line has shifted. Further the measured value of force obtained using PDA was compared to the magnitude of force computed from CAPWAP software analysis, as shown in Fig. 2(b). It is observed that both the computed and measured forces at the pile top were in good agreement with each other, and hence the pile and soil model were used for determining the estimated static load settlement curves, as shown in Fig. 2(c). The CAPWAP analysis results for the pile P1 showed that the ultimate capacity of the pile was 1,801 kN during the time of testing. The maximum compressive stress experienced by the pile at the pile top due to the effect of the hammer impact was 16,350 kPa, and the stress was within the permissible limit. The pile integrity factor for the pile was observed to be 88% below sensors, which were mounted at a distance of twice the pile diameter below the point which was struck by hammer, implying that the pile was not damaged during the process of testing. The load against settlement curve at the pile top was obtained from CAPWAP analysis by simulating static load test and the results showed that the pile was capable of bearing 1,801 kN of loads when the pile settlement reached 9.3 mm. A factor of safety equivalent to 1.5 was considered for computing the safe load on the pile and it worked out to be 1,200 kN. This showed that the safe load on pile was greater than the design load of 765 kN, thereby implying the pile was safe for its design load.

6.2 Pile P2 at Bandhab Nagar under Dum Dum Municipality

Fig. 3(a) shows the variation of force and velocity with time respectively, measured using the pile driving analyzer (PDA) for Pile P2. It is observed that both force and velocity at the pile top reaches its maximum peak magnitude within a few milliseconds when the hammer strikes the pile. After reaching the maximum value, the magnitude of force decreases and becomes constant with time. However beyond time span of 150 ms, the variation of force shows a slight hump. After attaining the maximum positive magnitude, the magnitude of velocity reaches a negative value and thereafter shows a constant variation with time. Fig. 3(b) shows the comparison of measured value of force obtained using PDA with the magnitude of force computed from CAPWAP software analysis. It is observed that both the variation of computed and measured forces with time, at the pile top are in good agreement with each other, and hence the pile and soil model could be used for determining the estimated static load settlement curves, as shown in Fig. 3(c). The CAPWAP analysis results for the pile P2 showed that the ultimate capacity of the pile was 1,553 kN during the time of testing. The maximum compressive stress experienced by the pile at the pile top due to the effect of the hammer impact was 10,410 kPa and within the permissible limit. The pile integrity factor for the pile was observed to be 81% below sensors, which implied that the pile was not damaged during the process of testing, although there was a small change in pile impedance. The load against settlement curve at the pile top was obtained from CAPWAP analysis by simulating static load test and the results showed that the allowable capacity of the pile was 1,553 kN when the pile settlement being 7.9 mm. A factor of safety equivalent to 1.5 was considered for computing the safe load on the pile and it worked out to be 1,035 kN, which was more than the design load of 740 kN.

6.3 Pile P3 at Kendriya Bihar (*Phase II*) *at Belghoria Expressway*

In Fig. 4(a) the variation of force and velocity with time, as measured using the strain transducers and accelerometers, attached to the pile driving analyzer (PDA), for Pile P3, is

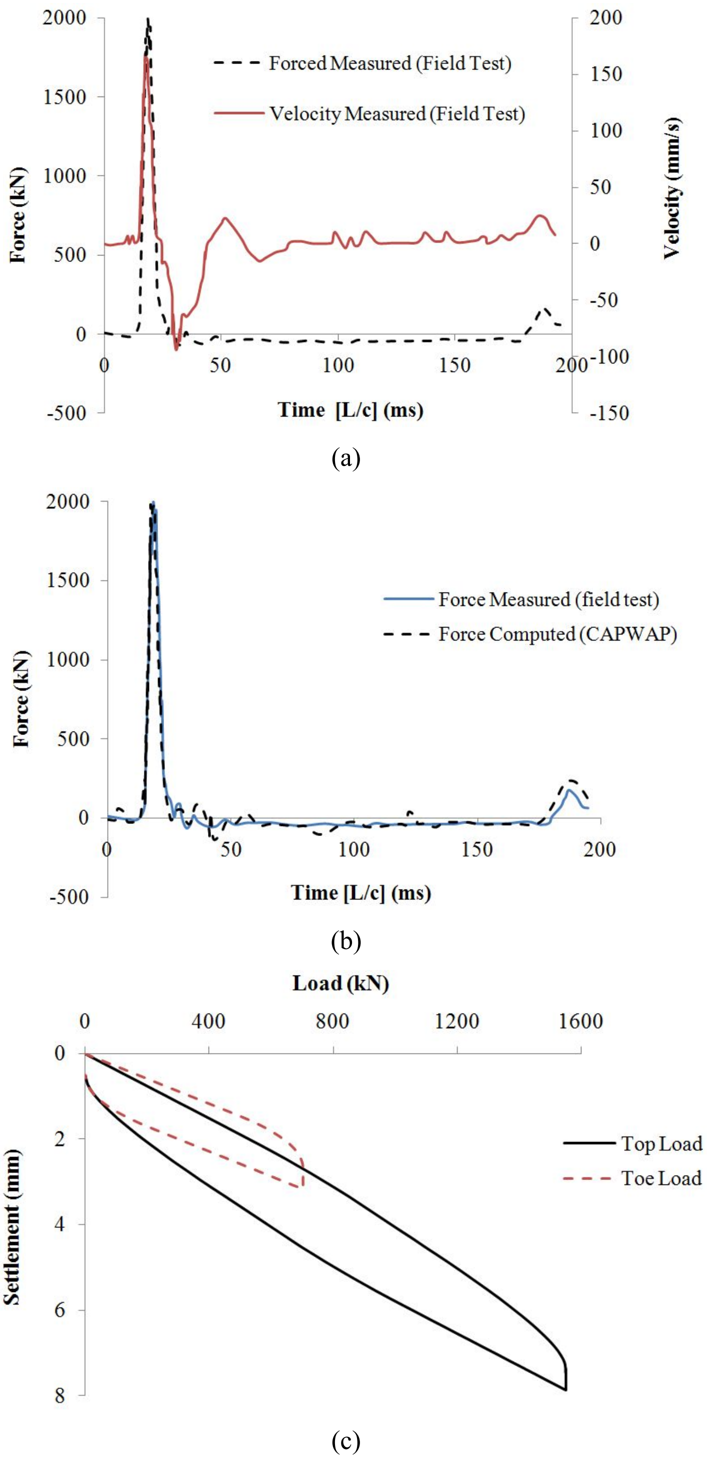
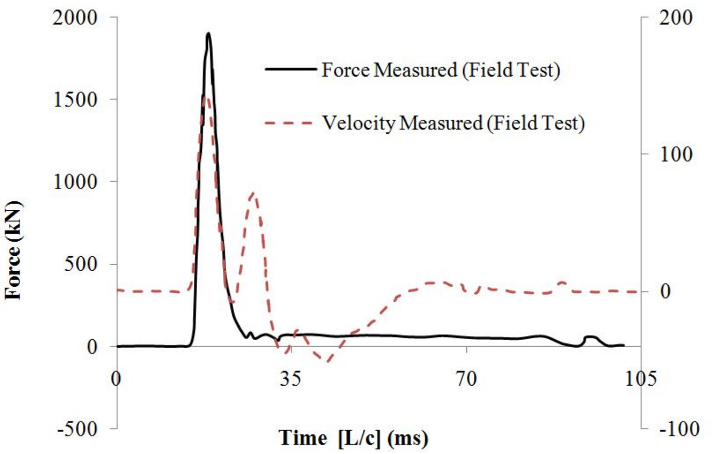
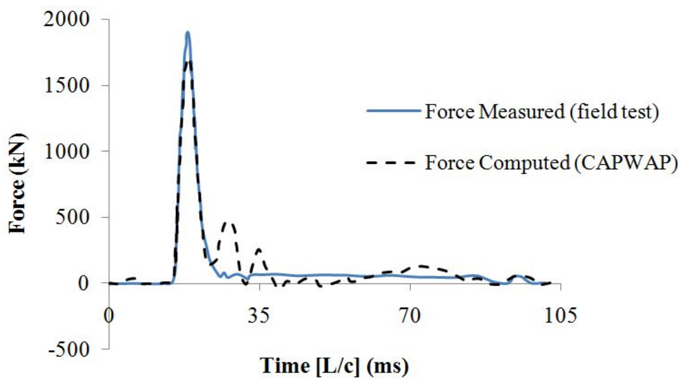


Fig. 3 (a) Variation of force and velocity against time measured using Pile Driving Analyzer for Pile "*P2*"; (b) Comparison of measured force using PDA with computed force using CAPWAP software for Pile "*P2*"; (c) Load versus settlement curve for Pile "*P2*"



(a)



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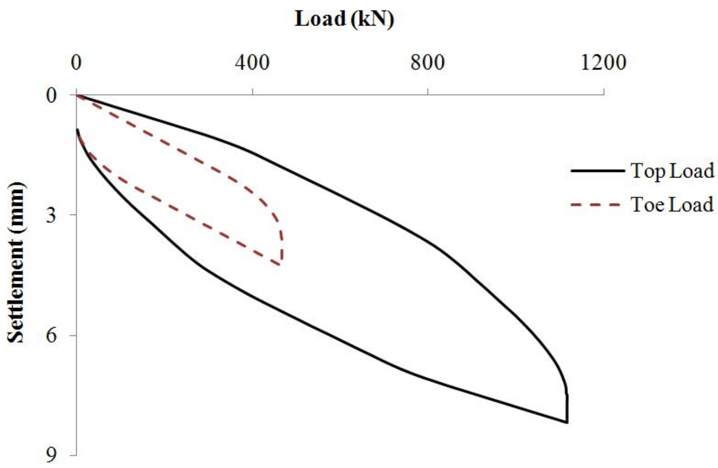


(c)

Fig. 4 (a) Variation of force and velocity against time measured using Pile Driving Analyzer for Pile "P3"; (b) Comparison of measured force using PDA with computed force using CAPWAP software for Pile "P3"; (c) Load versus settlement curve for Pile "P3" illustrated. It is observed that the magnitude of force measured using PDA reaches the maximum value around 20 ms, and after which remains constant with time. However the variation of velocity with time shows an unique trend, which is quite different as compared to the other piles. 2 peaks of decreasing magnitude are observed at 20 ms and 30 ms, after which the velocity drops to a negative value before becoming constant with time beyond 70 ms. In this study the pile capacity has been considered and the variation of force and velocity with time has not been studied in details. The variation is presented only. Further the curves are not ideal due to manual application of impact with winch rig hammer without proper guide frame and also there may be defects in pile casting. The average capacity obtained by output of CAPWAP analysis is presented only for comparison. In Fig. 4(b), the magnitudes of force measured and computed using PDA and CAPWAP software is illustrated. It is observed that although the maximum magnitudes of both the plots are almost similar, the measured force remains constant with time after 35 ms, while the force computed using CAPWAP software shows humps of decreasing magnitude at regular intervals. The estimated static load settlement curve for the Pile P3 is shown in Fig. 4(c). The CAPWAP analysis results for the pile P3 showed that the maximum soil resistance of the pile was 1,116 kN during the time of testing, which was quite low as compared to the other piles. The maximum compressive stress experienced by the pile at the pile top due to the effect of the hammer impact was 11,220 kPa, and within the permissible limit. The pile integrity factor for the pile was observed to be 82% below sensors, which implied that although the pile was not damaged during the time of testing, however there was a small change in pile impedance due to a change in cross – sectional area of the pile. The load against settlement curve at the pile top was obtained from CAPWAP analysis by simulating static load test and the results showed that with the pile settlement being 8.2 mm, the allowable load carrying capacity of the pile was 1,116 kN. A factor of safety equivalent to 1.5 was considered for computing the safe load on the pile and it worked out to be 744 kN, which was greater than the design load of 600 kN.

6.4 Variation of shaft resistance with depth

The variation of shaft resistance per unit area with depth for the piles P1, P2 and P3 tested at

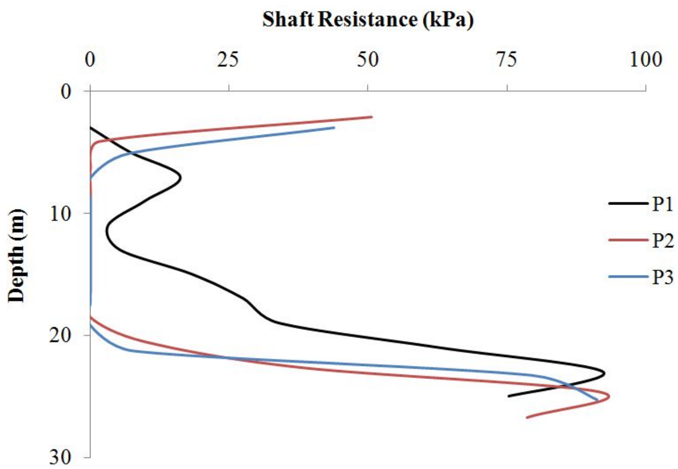


Fig. 5 Variation of shaft resistance per unit area with depth of the pile

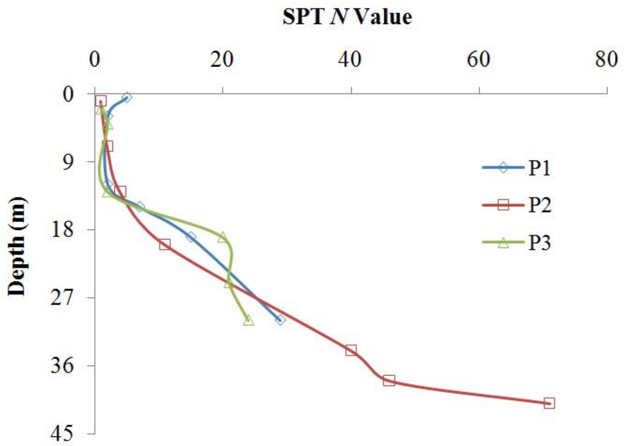


Fig. 6 Variation of SPT *N* value with depth at the 3 locations where High Strain Dynamic Pile Testing was conducted in Kolkata city

3 different locations of Kolkata city of India is illustrated in Fig. 5 and the variation of SPT *N* value with depth at these locations is plotted in Fig. 6. It is observed from Fig. 5 that for pile P1, the shaft resistance at the top of the pile is zero. Beyond a depth of around 4 m, the shaft resistance increases and reaches a considerably large magnitude of 95.3 kPa at a depth of 23 m below ground level. Pile P2 and P3 shows an unique variation of shaft resistance against depth. At the pile top, a shaft resistance in excess of 45 kPa is computed for both the piles, which gradually becomes zero at a depth around 5 m. From 5 m and extending down to a depth of 20 m, the shaft resistance for both piles P2 and P3 is found to be zero. This might have occurred due to non – recording of strain by the transducers which were attached to the pile driving analyzer. Otherwise it might happen that the strength of soil was lost to a great extent during operations of construction. Beyond 20 m depth, a shaft resistance of magnitude close to 100 kPa is recorded at the pile tip. It is observed from Fig. 6 that at a depth of around 27 m, the SPT *N* values for P1, P2 and P3 are quite identical, thereby giving rise to a similar magnitude of shaft resistance. Beyond 30m ground exploration was stopped at sites for pile P1 and P3, while for pile P2, a high SPT *N* value of 72 was recorded at depth of 40 m, thereby indicating the stiffness of the underlying soil layers.

7. Comparison of field data with FLAC3D results

In this section a single pile is modeled in FLAC3D which is a 3 – dimensional “explicit, finite difference program” that performs a “Lagrangian analysis” for engineering mechanics computation. A well documented case history, the evaluation of pile capacity in Aliah University Campus at Park Circus is taken here for comparing the results obtained from FLAC3D. A pile of length 27 m and 600 mm diameter is modeled in layered soil profile simulating the soil condition of the considered case study.

7.1 Modeling procedure

The coordinate axes for the FLAC3D model are located with the origin at the bottom left corner of the model and z axis being oriented upward along the pile axis. The model grid is shown in Fig. 7. A soil block of 35 m × 35 m × 35 m (further any increase in the dimensions of soil block will not affect the results) is generated and the entire soil block is divided into 42875 zones and assigned the properties of soil by layer wise according to the site condition as given in Table 6.

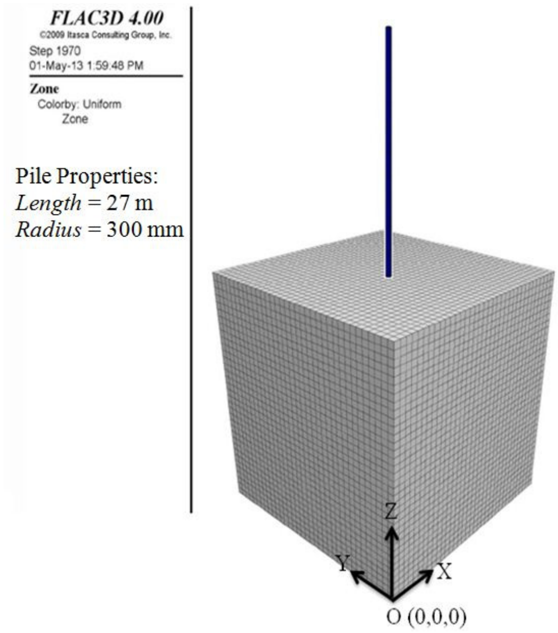
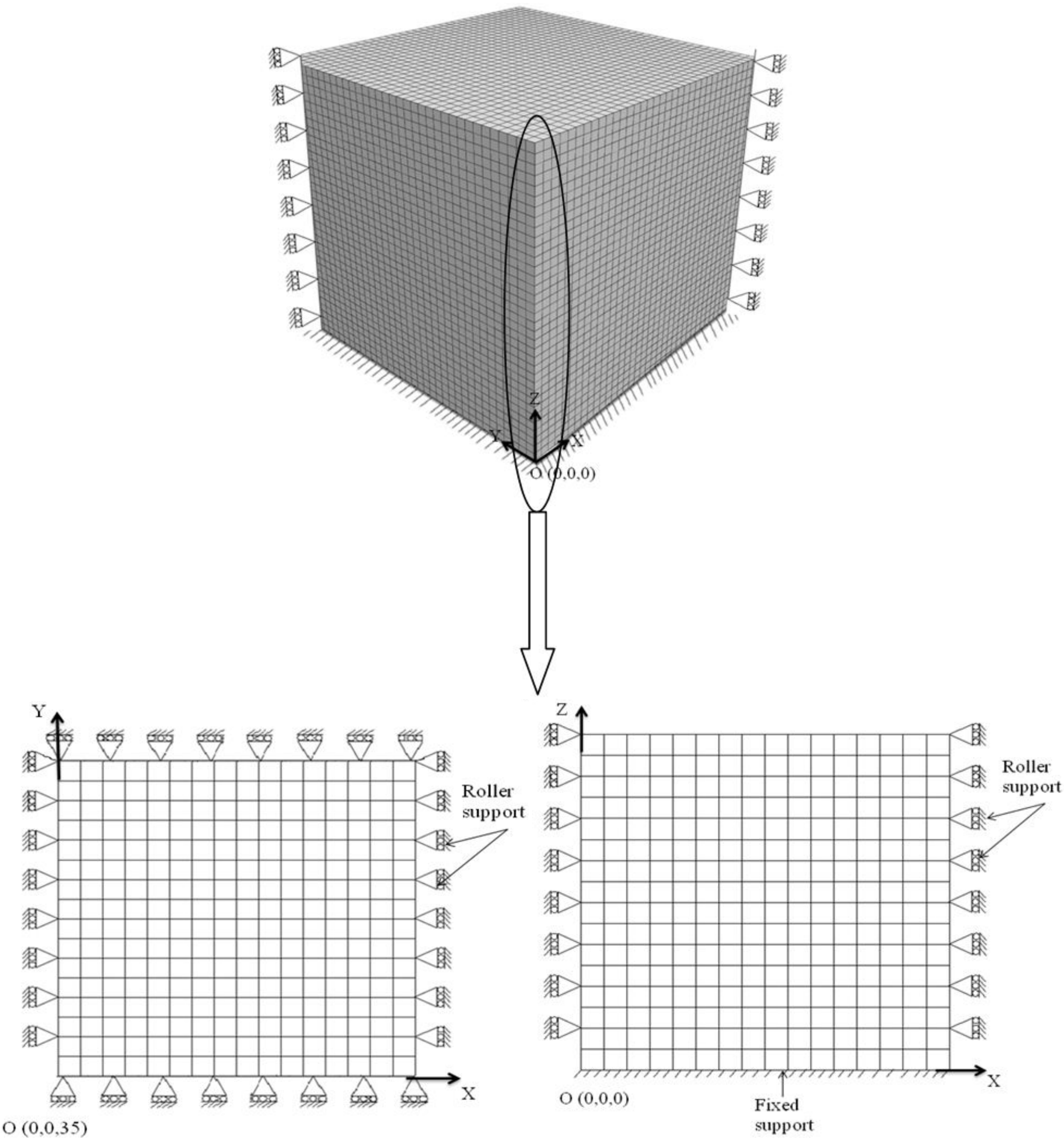


Fig. 7 Basic model showing the insertion of the pile in soil using FLAC3D

Table 6 Soil properties considered for numerical modeling in FLAC3D [Bowels (1997)]

Description	Layer thickness [t] (m)	SPT [N]	Density [ρ] (gm/cc)	Poisson's ratio (μ)	Cohesion [c] (kPa)	Soil friction angle [ϕ] (°)
Fill of rubbish mixed with brick pieces.	0.5	5	2.04	0.25	30	20
Medium brownish grey clayey silt / silty clay	2.5	2	1.45	0.3	30	5
Soft to very soft grey silty clay with decomposed wood	9	2	1.45	0.3	32	5
Medium to stiff grey / bluish grey silty clay with kankars	3	7	1.7	0.4	40	5
Very stiff yellowish silty clay/clayey silt	4	15	1.9	0.38	90	8
Medium to dense grey sandy silt / silty fine sand, % of sand increases with depth	14	29	1.98	0.35	5	40



(a) (b)

Fig. 8 Boundary condition of the model in FLAC3D and (inset) boundary conditions along (a) x - y plane; and (b) along x - z plane respectively

Mohr – Coulomb failure criteria is chosen for the analysis. The top of the model, at z = 35 m, is a free surface. The base of the model, at z = 0, is fixed in x, y and z directions, and roller boundaries are given on the sides of the model, x = y = 0 and x = y = 35 m (i.e., it is fixed in both x and y directions), as illustrated in Fig. 8.

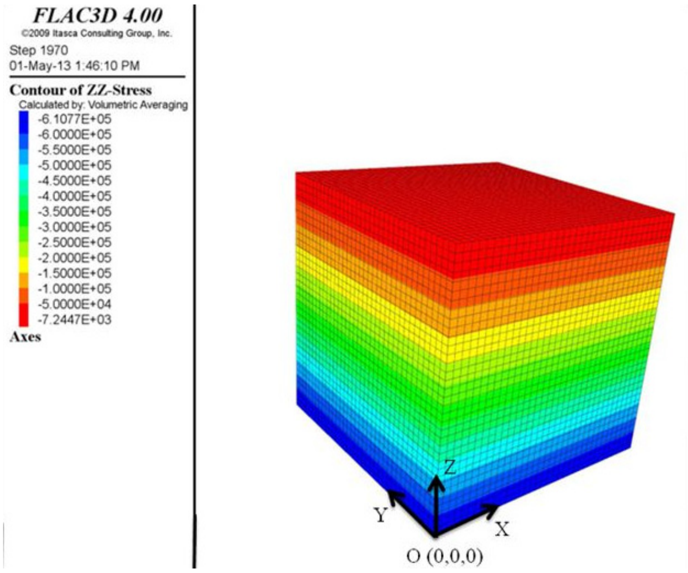


Fig. 9 Vertical stress distribution contours in the free field obtained using FLAC3D

The model is first brought to equilibrium state under gravity loads before the pile is installed. In the next step, the model is brought to equilibrium after installing the pile in the soil. The distribution of vertical stress (s_{zz}) at the equilibrium state, inclusive of the weight of the pile, is shown in Fig. 9. The actual vertical stress, computed according to the equation $s_{zz} = \gamma h$, where γ is the unit weight of the soil and h is the height of the soil block, at the initial condition are -581.85 kPa at the bottom of the soil block ('-'implies compression) and 0 kPa at the top. From Fig. 9, it is observed that the magnitude of vertical stresses computed using FLAC3D at the bottom of the soil block is -610.77 kPa, which is almost similar to the values calculated analytically, as specified.

A pile of length 27 meters and 0.6 meter diameter is generated at the centre of the soil block and assigned the properties of M20 grade concrete as per IS 456 (2000) and tabulated in Table 7. Elastic model is chosen for the analysis. Pile has been modeled as structural elements (pileSELs) and divided into 32 segments. Pile structural elements are two nodded, straight, finite elements with six degrees of freedom per node. The stiffness matrix of a pileSEL is same as that of a beamSEL; however, in addition to providing the structural behavior of a beam, both a normal – directed (perpendicular to the pile axis) and a shear – directed (parallel with the pile axis) frictional interaction occurs between the pile and the soil grid. In addition to skin friction effects, end – bearing effects can also be modeled by deleting the link between soil grid and pileSEL at the tip of pile and replacing it with a new link containing a normal – yield spring in the axial direction of stiffness equal to normal stiffness of the interface at the pile tip.

The capacity of a pile is a function of the skin friction resistance along the pile shaft and the end bearing capacity at the pile tip. This skin friction resistance is modeled by placing an interface between the pile walls and the soil. The friction angle at the interface is taken same as the soil friction angle as per IS 2911 [Part1/Sec4] (1984). The adhesion between pile and soil is taken as per IS 2911 (1984). A second interface is placed between the pile tip and the soil. The normal stiffness (k_n) and shear stiffness (k_s) at the interfaces are calculated as per Timoshenko and Goodier (2002), and the interface properties are given in Table 8.

Table 7 Pile properties considered for numerical modeling in FLAC3D as per IS 456 (2000)

Pile properties	Values
Young's modulus [E] (MPa)	22,400
Poisson's ratio [μ]	0.2
Bulk modulus [K] (MPa)	12,400
Shear modulus [G] (MPa)	9,320
Unit weight [γ] (kN/m^3)	25

Table 8 Soil – pile interface properties (IS 2911 [Part1/Sec4] 1984, Timoshenko and Goodier 2002)

Description	Normal stiffness [k_n] (MN/m)	Shear stiffness [k_s] (MN/m)	Interface angle [δ] (°)	Adhesion [αc] (kPa)
Fill of rubbish mixed with brick pieces	61.59	5.26	20	15
Medium brownish grey clayey silt / silty clay	22.22	1.9	5	21
Soft to very soft grey silty clay with decomposed wood	22.22	1,9	5	21
Medium to stiff grey/bluish grey silty clay with kankars	67.77	5.78	5	20
Very stiff yellowish silty clay / clayey silt	129.32	11.04	8	36
Medium to dense grey sandy silt/silty fine sand, % of sand increases with depth	219.51	18.73	40	1.5

$$k_n = \frac{4Gr_o}{(1-\mu)} \tag{6}$$

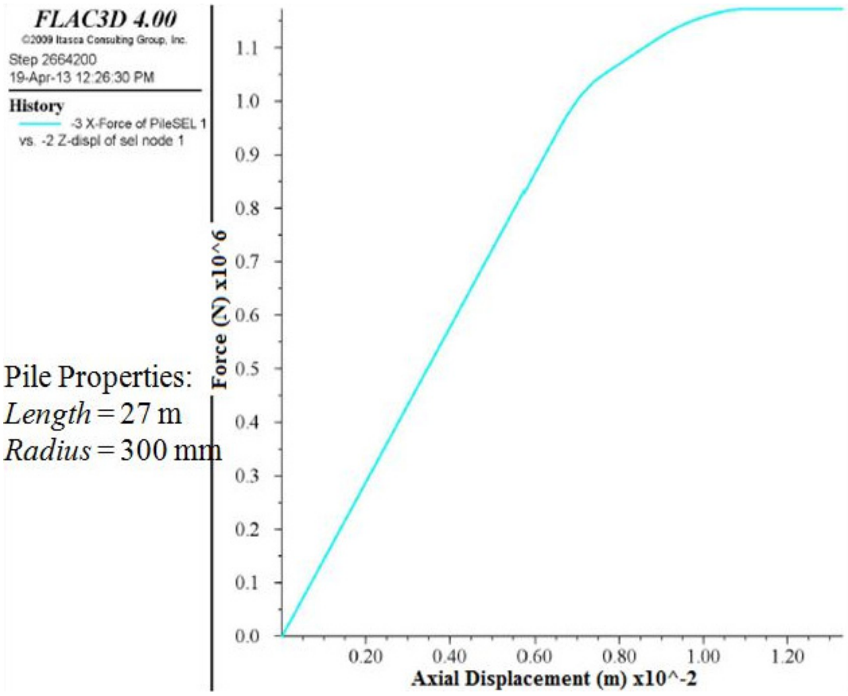
$$k_s = \frac{32(1-\mu)Gr_o^3}{(7-8\mu)} \tag{7}$$

where, G is the low strain shear modulus, r_o is the radius of the pile and μ is the Poisson's ratio.

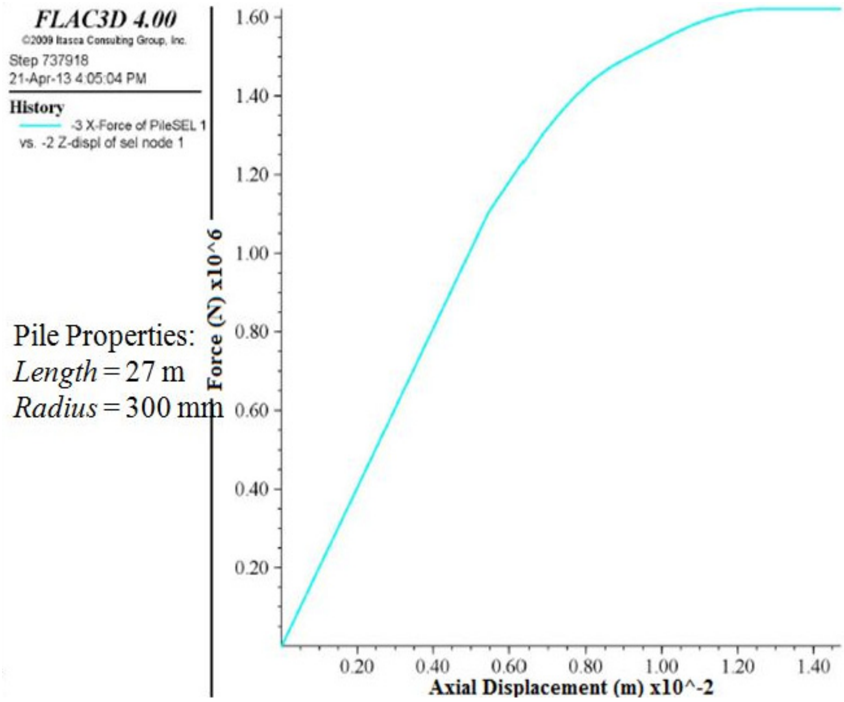
The model is first brought to equilibrium state under gravity loads after all properties are assigned to soil, pile and soil – pile interface. The ultimate bearing capacity of the pile in the axial direction is calculated by applying a vertical downward velocity of 1×10^{-8} m/step at the top of the pile. If high velocity is applied, then the inertial effects will dominate initially, thereby making it more difficult to identify the steady – state response of the system.

7.2 Analysis and results

A plot of axial load versus axial displacement for the present friction cum end bearing pile for skin friction component and ultimate pile capacity, obtained using numerical analyses in FLAC3D, is



(a) Mobilized skin friction component of the pile

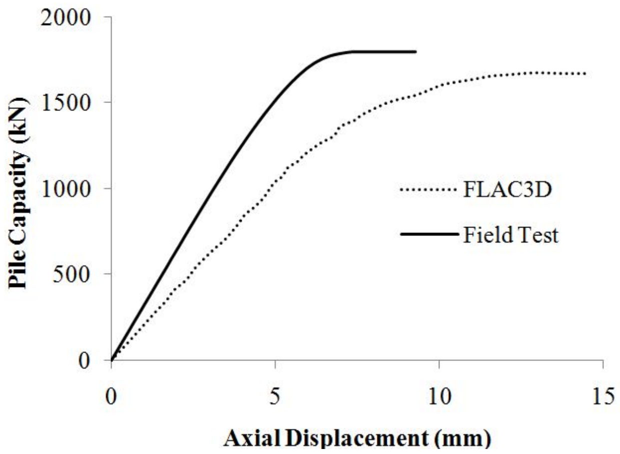


(b) Ultimate pile capacity

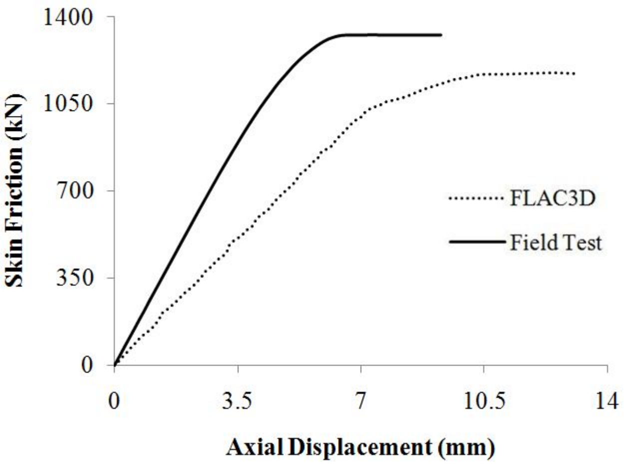
Fig. 10 Variation of axial load versus axial displacement at pile top

Table 9 Comparison of ultimate pile capacity at 3 different locations of Kolkata city

Locations	Pile mark	Ultimate pile capacity (kN)		% difference
		Field Test	FLAC3D	
Aliah University Campus at Park Circus	P1	1801	1654	8.2
Bandhab Nagar under Dum Dum Municipality	P2	1553	1399	9.92
Kendriya Bihar (Phase II) at Belghoria Expressway	P3	1116	1016	8.97



(a)



(b)

Fig. 11 Comparison of: (a) ultimate pile capacity; and (b) mobilized skin friction against axial displacement for measured field test results and computed numerical analyses results using FLAC3D shown in Figs. 10(a)-(b), respectively. From Fig. 10(a), it is observed that the mobilized pile shaft resistance is equal to 1170 kN. Similarly, from Fig. 10(b), the ultimate capacity of pile, i.e., skin friction + end bearing, from FLAC3D is computed to be 1,654 kN. A graphical comparison showing the variation of ultimate pile capacity versus axial displacement and mobilized skin friction against axial displacement, for measured field test results and numerical analyses results computed using FLAC3D, is shown in Figs. 11(a)-(b), respectively. It is observed from Figs. 11(a)-(b), the nature of the curves for both ultimate pile capacity and mobilized skin friction obtained using field test and numerical analyses are quite identical. However the ultimate pile capacity and mobilized skin friction resistance is computed to be 1,654 kN and 1,170 kN respectively from FLAC3D, which is a quite close to the field test results of 1,801 kN and 1,326 kN respectively. The differences in magnitude of ultimate pile capacity obtained from the measured field test and that computed using FLAC3D for all the 3 different locations of Kolkata city where High Strain Dynamic Pile Testing was implemented is tabulated in Table 9.

8. Conclusions

The results of High Strain Dynamic Pile Testing conducted at 3 different locations of Kolkata city of India have been reported in the present study. The dynamic measurements were accurately recorded using strain transducers and accelerometers that were fixed below the pile head. Further the force versus time variation measured at the pile top using pile driving analyzer (PDA) were compared to the results computed using CAPWAP software and the pile and soil resistance model were being adjusted in an iterative procedure till a good match between the magnitude of force at the pile top was obtained. The estimated static load settlement curves were further obtained using the pile and soil models.

It was observed that the ultimate load carrying capacity of pile P1 was maximum with a magnitude of 1,800 kN, followed by pile P2 which had an ultimate load capacity of 1,553 kN. The ultimate capacity of the other piles were comparatively less with P3 having the minimum capacity of 1,116 kN. The variation of the ultimate capacity can be attributed to the difference in length and diameter of the individual piles and the soil strata in which these piles were excavated. Pile P1 had a length of 27 m and diameter of 600 mm, while the length of P3 was 25.3 m long with the diameter being 450 mm. This clearly explains that piles of longer length and larger diameter had more ultimate capacity compared to smaller piles, irrespective of the grade of concrete with which they were formed. Pile P1 was excavated through cohesive soil (composed of silty clay and clayey silt) which was medium stiff to stiff in nature. On the other hand, pile P2 was excavated through cohesionless soil composed of silty sand which varied from loose to medium. This illustrates the influence of soil strata on the ultimate capacity of a single pile. The compressive stress experienced by the pile at the pile top due to the impact of the hammer was also found to be within permissible limit for all the piles. The integrity factor of the piles was greater than 0.8, thereby signifying that no substantial damage was imparted to the piles during the process of testing. A factor of safety equivalent to 1.5 was assumed for calculating the safe load on the piles. It was found that for all the piles the safe load was more than the design load and hence the piles could be used for design purposes for the typical design loads. However, in the present study, the pile capacity only has been considered while the variation of force and velocity with time has not been studied in details. Further the curves obtained are not ideal due to manual application of impact with winch rig hammer without proper guide frame along with defects in pile casting. The average capacity obtained by output of CAPWAP analysis is presented only for comparison. This can be considered as a limitation of the present study.

The ultimate capacity and mobilized shaft resistance of pile P1 was further validated using finite difference based numerical software FLAC3D. The model of the pile for each case along with the soil profile of the particular site under consideration and appropriate boundary conditions were analyzed with the help of FLAC3D simulating appropriate field conditions. The results obtained from FLAC3D differed from field test results by only around 9%, thereby indicating a very good agreement between results of numerical analyses and field tests, for the given soil data. Thus it also illustrates the efficiency of FLAC3D as reliable software for analysis of single piles under impact loading. Hence the present case study signifies that although static load tests are a reliable means of determining the ultimate capacity of a pile, dynamic load test provides a suitable alternative since they impart additional information on the hammer – pile – soil system, pile impedance and integrity factor of the pile at the time of testing. It may, therefore, be suggested to include the test in relevant IS code of practice.

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GM

List of symbols

N	SPT value at a particular depth below the ground
N_{avg}	average field SPT N value of the particular soil stratum
V_s	shear wave velocity
$G_{\max}$	low strain shear modulus
ρ	mass density of the soil
E	Young's Modulus of the pile
ξ	damping coefficient
c	velocity of the propagating stress wave
Z	pile impedance
A	cross sectional area of the pile
t_o	duration of travel
L	pile length
T	characteristic length of the pile
R_u	ultimate capacity of the pile
R_b	mobilized pile toe resistance
D_x	maximum static displacement at the pile top
$E_{\max}$	maximum energy transferred to the pile
S	permanent penetration of pile (set) for all impacts applied
IF	pile integrity factor
R_c	safe load on the pile
k_n	normal stiffness
k_s	shear stiffness
r_o	radius of pile
μ	Poisson's ratio