

Comparative Study of Lateral Pile Behaviour in Cohesionless Soil under Inclined Compressive Loading with P-y curve Method

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ABSTRACT: Pile foundations are extensively used to support various structures built on loose/ soft soils where shallow foundations would undergo excessive settlements or have low bearing capacity. Piles are slender, having high length to width ratio, and are mainly designed to resist axial loads. However, some structures such as high rise buildings, offshore structures, tall chimneys, earth retaining walls are subjected to horizontal or lateral pressure caused by wind force, wave force, traffic movement, earthquake etc. These loads cause lateral and vertical displacements and rotation of the pile cap. These overall behaviour of the piles can be estimated from the available conventional theoretical approaches. A widely popular method for analysing lateral displacement of the piles under working loads is the p-y method, where the soil is modeled as disjointed springs that offer lateral resistance to pile movement. This paper is an attempt to study the behaviour of single pile in cohesionless soil, subjected to varying inclined load until failure with the angle of applied load varying from 0° to 90° from the vertical axis of the pile, through an experimental model study on model mild steel and concrete piles driven into dry river sand. Lateral load carrying capacities of both piles of various slenderness ratios (10, 15, and 20) are found through the load – settlement diagrams and are compared. The effects of lateral components of inclined loads on horizontal and vertical displacement of the pile head are discussed. Also, the effects of pile material on the lateral load capacity of piles are studied. The results obtained are assessed by comparing with the theoretical p – y curve analysis method.

KEYWORDS: Bearing Capacity, Cohesionless soil, Inclined Loads, Lateral loads, Model tests, Pile foundation.

I. INTRODUCTION

Pile foundations are generally adopted, when the soil conditions are poor. The purpose of a pile foundation is to transmit the loads of the superstructure to the deep underlying firm soil while preventing excessive structural deformations. Pile foundations of offshore structures, transmission towers, wind energy converters, are often subjected to inclined loads and moment from usual structural dead load and horizontal loads generated by wind, waves, earth pressure etc. For proper functioning of such structures, two criteria must be satisfied: (1) A pile should be safe against ultimate failure, (2) Normal deflection at working loads should be within the permissible limit. For design of such piles, a quantitative estimate of allowable inclined compressive load is needed. If a vertical pile is subjected to an inclined and eccentric load, the ultimate bearing capacity in the direction of the applied load is intermediate between that of ultimate lateral load and ultimate vertical load. When load acts vertical, it is resisted by pile through skin friction and base resistance but when load acts with inclination, the deficiency in the base resistance and to some extent in the shaft friction leads to decrease in carrying capacity of single pile. Analysis of piles subjected to inclined loading action is complicated due to large number of variables involved. Further, experimental studies on behaviour of single pile subjected to inclined compressive loads are limited. The effect of angle of inclination of compressive load on horizontal deflection and vertical displacement of vertical piles of different length to diameter ratio are also rarely tested. The behaviour of a single pile under the action of lateral load components induced by various means of inclined compressive loads are studied in this paper.

A number of model tests have been conducted by Sharma and Pise (1994) and some other methods of analysis have been proposed by Meyerhof and Adams (1968), Chattopadhyay and Pise (1986), Rao and Kumar (1994), for the piles

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or anchors separately and that too in homogeneous media. The comparative study by LianyangZang et al. (2005) has shown that these studies produced significantly different ultimate resistance values of single as well as pile group. Behaviour of vertical and batter piles subjected to inclined oblique loads have been reported by researchers like Madhusudan Reddy. K et al. (2011), Muthukkumaran et al. (2015) etc. Salini U. and Girish M. S (2009), Alice Johny and C. Prabha (2014), JerinWiba, V. JeyanthiVineetha (2014) studied the lateral load analysis results of single piles in cohesionless soils for range of soils. Their studies captured both the short pile and long pile behaviour observed for loose sands from dry state to submerged condition.

The present work involves an experimental analysis of lateral pile capacities of single piles of different materials under the action of inclined loads with varying angles of inclination, in cohesionless soil. Also, the results are validated with the conventional p-y curve analysis method. An attempt is made here to study the behavior of single model steel and concrete piles in cohesionless soil, subjected to varying inclined load until failure with the angle of applied load varying from 0° to 90° from the vertical axis of the pile, through an experimental model study.

Paper is organized as follows. Section II describes the Experimental Investigation in which the Foundation medium chosen, Materials used and the Experimental Setup of the study are explained. Section III explains the p-y curve method of finding the Lateral pile load capacity. The results of the study and the supporting discussion on the results obtained are given in Section IV. Finally, the Section V presents conclusion.

II. EXPERIMENTAL INVESTIGATION

2.1 FOUNDATION MEDIUM

The foundation medium used for the study is dry sand obtained from the river Bharathapuzha in Palakkad district, Kerala, India. The physical properties of sand were determined using various laboratory experiments such as specific gravity test, density test, sieve analysis and direct shear test. The results obtained are shown in Table 1.

TABLE 1: PROPERTIES OF SAND

Property	Value
Specific Gravity (G)	2.66
Maximum Dry Density (γ_{max})	1.8 gm/cc
Minimum Dry Density (γ_{min})	1.6 gm/cc
Particle size at 10% finer, D_{10}	0.450 mm
Coefficient of curvature (Cc)	1.08
Uniformity coefficient (Cu)	2.55
Cohesion (C)	0
Angle of internal friction (ϕ)	39°

2.2 MODEL PILES

A steel pile and a concrete pile are used as the model piles, as shown in Fig 1. The range of prototype dimensions represented by the model piles for different scale factors is calculated using the following formula (Wood et al. 2002; Wood 2004)

$$(EI)_p / (EI)_m = n^{4.5}$$

Where, n = scale factor, $(EI)_p$ = Flexural rigidity of prototype pile and $(EI)_m$ = Flexural rigidity of model pile. L/D ratios of 10, 15, and 20 are used in the present investigation for simulating piles in the model experiments.

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Fig 1: Steel Piles and Concrete Piles

Rigid rough Mild Steel (MS) solid tubes and precast concrete tubes having diameter 20 mm with lengths of 200mm, 300mm and 400mm to achieve desired slenderness ratios of 10, 15 and 20 respectively were used as model piles. The Mild steel piles were slightly roughed to ensure better interaction with the sand particles. The concrete piles were casted in a PVC pipe, using cement and sand in the ratio of 1:1.5 and 0.5 water-cement ratio, providing a reinforcement of 6 mm diameter steel bar at the center. The casted piles were cured for 28 days and were taken out of the casing for testing. The model piles used in the present study are classified in terms of long and short piles respectively according to clause C-3 of IS 2911 (Part 1/Sec 1): 2010. The stiffness factor T , for mild steel piles and concrete piles used in the study for densely deposited soil are found to be 133 for 200mm, 139 for 300mm and 128 for 400mm piles respectively. Therefore, the Piles chosen for the study are long piles.

2.3 EXPERIMENTAL SETUP

The dimensions of the model test tank are decided based on the effective stressed zone of soil mass from the edge of the foundation. For a laterally/ inclined loaded single pile, it is inferred from the literature that the boundary effect is more predominant within 10 times the pile diameter from the pile periphery (Matlock 1970; Rao et al. 1996, 1998). In the present study, the model pile diameter is 20 mm and hence the size of the tank in the plan should be larger than 200 mm for a single pile. Accordingly, the dimensions of the model steel tank are fixed as $1.0 \times 1.0 \times 0.75$ m (depth) to avoid the boundary effects while testing the model pile under axial, lateral and inclined loads.

Model tests were carried out using model Mild Steel solid piles and model concrete piles, embedded in sand medium. The test tank of size 100 cm x 100 cm x 75 cm was fabricated as water tight using mild steel materials. The tank is provided with a detachable frame with the provision of applying loads at various inclinations of 0° , 15° , 30° , 45° , 60° and 90° . The vertical and horizontal settlement measuring dial gauges were also dully attached to the loading frame. The tests were conducted in dry soil conditions. A schematic view of the experimental assembly is shown in Fig.2. Placement density of the sand during testing was 1.4 g/cc. Sand was poured in the tank evenly keeping the height of fall constant throughout the time of fall. Further sand pouring was continued to top level surface and then compacted to achieve the placement density. Pile was then fully embedded in sand in vertical position in model tank was subjected to inclined.

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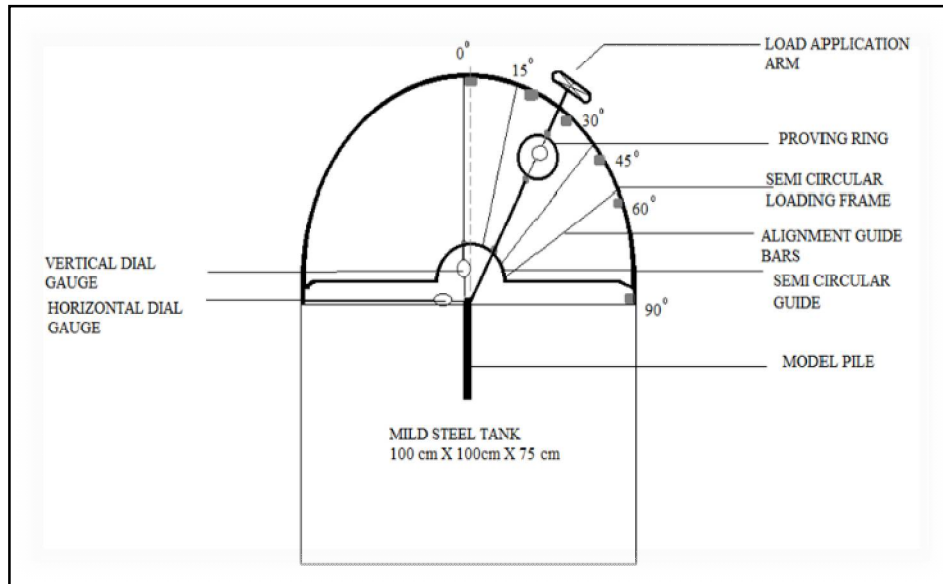


Fig.2: Schematic view of the experimental assembly

The Fig.3 shows the pictorial representation of this test setup. Loading was applied at the pile head by wheel axle arrangement for load inclination angle ranging from 0° to an angle of 60° with the vertical axis of the pile. Vertical and the horizontal displacements of the pile top were measured with dial gauges of sensitivity 0.001 mm. Loads were applied in small increments till the ultimate capacity of the pile was reached with little subsequent increment of the applied load.



Fig.3: Pictorial view of the experimental assembly

III. LATERAL PILE LOAD CAPACITY USING P-Y CURVE METHOD

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Ultimate capacity of pile for a central inclined load at the head of a fully embedded pile, the limits of vertical and horizontal components, P_v and P_h respectively, of ultimate capacity can be approximated by a semi-empirical relationship expressed as

$$\left(\frac{P_v}{P_U}\right)^2 + \left(\frac{P_h}{P_H}\right)^2 = 1$$

where, P_U and P_H are the ultimate pile capacities under axial loading and horizontal loading respectively.

Reese *et al* (1974) proposed criteria for cohesionless soils for analysing the behaviour of piles subjected to lateral loadings under static conditions. The following procedures were developed for constructing $p-y$ curves under static conditions.

1. Obtain the values of the angle of internal friction ϕ , the effective unit weight γ , and the pile diameter d .
2. Obtain the ultimate resistances of soil per unit length of pile using following equations.

For soil close to ground surface:

$$P_{st} = \gamma x \left[\frac{K_o x \tan \phi \sin \beta}{\tan(\beta - \phi) \cos \alpha} + \frac{\tan \beta}{\tan(\beta - \phi)} (d + x \tan \beta \tan \alpha) + K_o x \tan \beta (\tan \phi \sin \beta - \tan \alpha) - K_A d \right] \quad (1)$$

For soil well below ground surface:

$$P_{sd} = K_A d \gamma x (\tan^8 \beta - 1) + K_o d \gamma x \tan \phi \tan^4 \beta \quad (2)$$

Where, $\alpha = \phi/2$, $\beta = 45^\circ + \phi/2$, $K_o = 0.4$, $K_A = \tan^2(45^\circ - \phi/2)$

3. Find the depth at which there is an intersection of equations (1) and (2). Above this depth use equation (1) and below this depth, use equation (2).
4. Select a depth x at which $p-y$ curve is required.
5. Establish $y_u = 3d/80$, compute P_u by the equation,

$$P_u = A_s P_s$$

Use the appropriate value of A_s for the particular non-dimensional depth x/d .

6. Establish $y_m = d/60$, compute P_m by the equation,

$$P_m = B_s P_s$$

Use the appropriate value of B_s for the particular non-dimensional depth x/d .

7. The two straight line portions of the curve beyond the point where $y = d/60$ can now be established as shown in Fig.4.

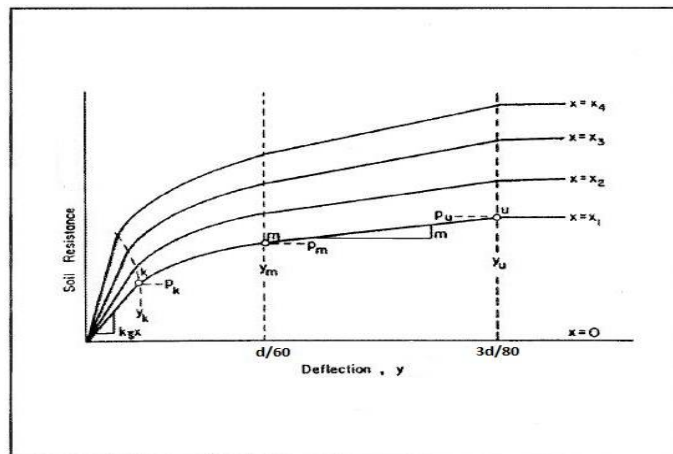


Fig 4: Characteristic shape of $p-y$ curves for static and cyclic loading in sand.

8. Establish the initial portion of the $p-y$ curve by the equation,

$$P = (n_h x) y$$

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9. Establish the parabolic section of the curve by the relation,

$$P = Cy^{1/n}$$

- (a). Fit the parabola between the points k and m as follows:

$$m = \frac{P_u - P_m}{y_u - y_m}$$

- (b). Obtain the power of the parabolic section by,

$$n = \frac{P_m}{my_m}$$

- (c). Obtain the coefficient C as,

$$C = \frac{P_m}{y_m^{1/n}}$$

- (d). Determine point k as,

$$y_k = \left(\frac{C}{n_h x} \right)^{\frac{n}{n-1}}$$

$$P_k = Cy_k^{1/n}$$

10. Compute appropriate number of points on the parabola using the equation on step 9.

For design of these piles, a quantitative estimate of allowable inclined compressive load is needed.

IV. RESULTS AND DISCUSSIONS

Due to application of oblique load P_θ on pile head, it is subjected to a vertical component, P_v , and a horizontal component, P_h . From geometry, these components can be expressed as:

$$P_v = P_\theta \cos \theta$$

$$P_h = P_\theta \sin \theta$$

It can be seen that, as θ increases, P_v decreases and P_h increases. Thus, with the increase of tilt of load with the vertical direction the horizontal component dominates the vertical load component. Previous analytical and experimental approaches propose that depending on the relative magnitudes of the ultimate axial capacity and lateral ultimate resistance of a vertical pile, the axial component may attain critical value equal to ultimate load carrying capacity of pile axially. In this case, the horizontal component remains smaller than the ultimate horizontal load carrying capacity causing axial failure of the pile. However, if the horizontal component reaches the critical value equal to the ultimate horizontal capacity, while the axial component is less than the ultimate axial load capacity, plastic hinge formation takes place and bending failure will occur in long pile and in case of short pile translation failure in pile occurs.

In this study, the results of the lateral load components versus axial and lateral movement of the pile head are plotted. The failure load is taken to be that load at which the load vs. pile head displacement curve passes in to steep and fairly straight tangent. Comparisons of obtained experimental results with the available $p-y$ curve analysis are carried out.

4.1 EXPERIMENTAL ULTIMATE LATERAL CAPACITY OF THE PILES

In order to obtain the lateral load bearing capacity of the Steel Piles and Concrete Piles subjected to various load inclinations, plots of horizontal component of the loads versus the lateral settlement of the pile head are drawn for all pile slenderness ratio at load inclinations of 15° , 30° , 45° and 60° as shown in Fig.5 (a), (b) and (c) and Fig.6 (a), (b) and (c). All loads applied at inclinations have horizontal components, P_h , whose values are different from that under purely lateral conditions (at $\theta = 90^\circ$) i.e. when no vertical components are present. Further it is observed that the presence of axial component of the applied load, when the inclination of loading with the vertical axis of pile approaches an angle around 60° to 90° caused an increase in lateral load carrying capacity of the pile and at 15° a decrease in the lateral load carrying capacity.

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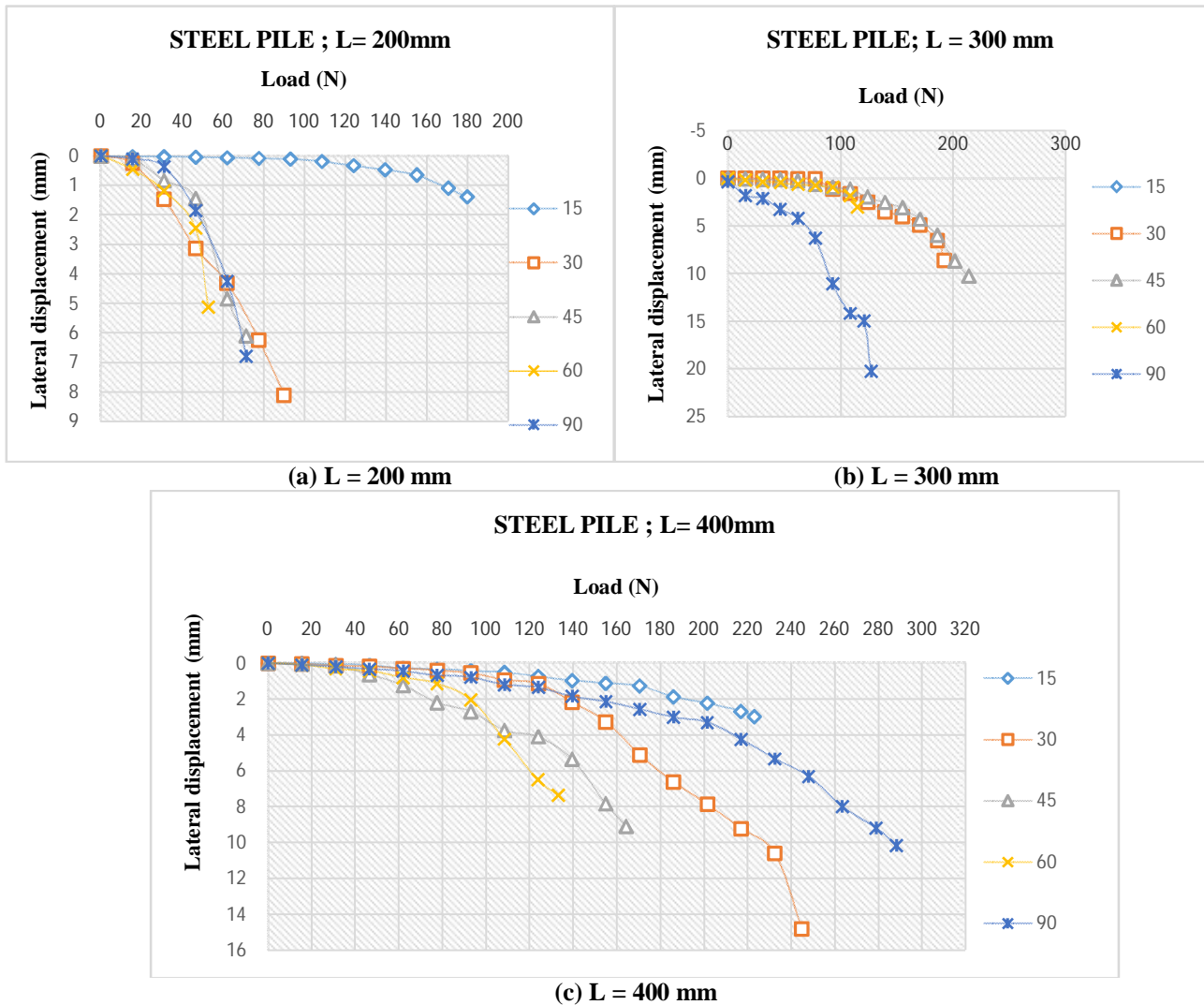


Fig.5: Horizontal component the load vsLateral of displacement of steel pile.

TABLE 2: LATERAL CAPACITIES OF STEEL PILES AT VARYING INCLINATIONS IN (N)

Length of pile in mm	Load inclinations	15°	30°	45°	60°	90°
	200		39	45	40	41
300		62	93	109	100	120
400		56	117	116	94	200

From the above table, it can be seen that lateral load capacity increases with increase in length of the pile. This is because the passive resistance is mobilized on increased embedment of pile, i.e. when the embedment length of pile increases. It is inferred that the *Ultimate lateral capacities of the steel piles* having lengths 200mm, 300mm and 400mm are found to be 72 N, 120 N and 200 N respectively.

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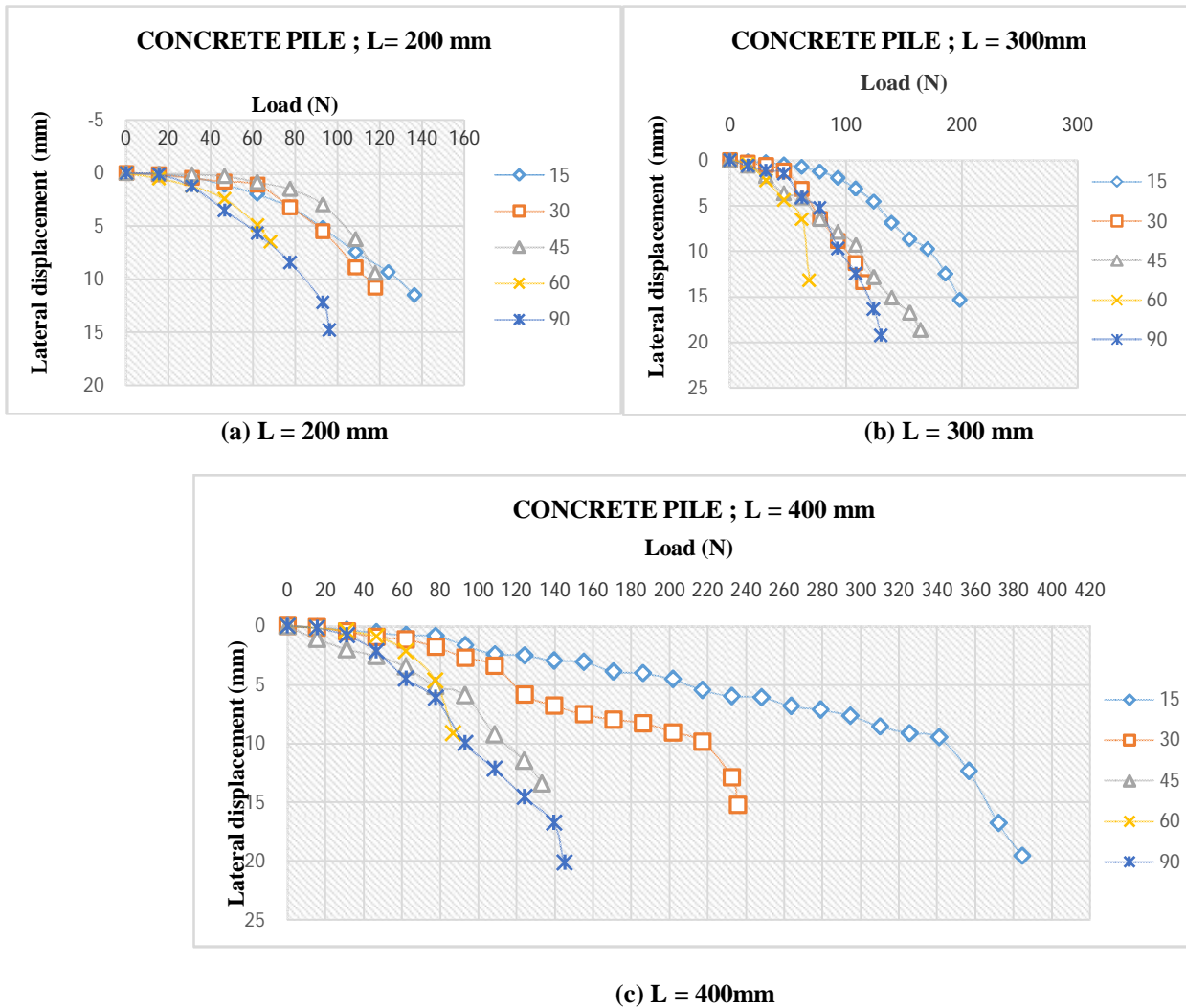


Fig.6: Horizontal component of the load vs Lateral displacement of concrete pile

TABLE 3: LATERAL CAPACITIES OF CONCRETE PILES AT VARYING INCLINATIONS IN (N)

Length of pile in mm	Load inclinations	15°	30°	45°	60°	90°
	200		24	30	37	40
300		43	55	75	54	110
400		85	108	94	67	140

From the above table, the lateral load capacity is found to increase with increase in length of the pile because the passive resistance is mobilized on increased embedment of pile, i.e. when the embedment length of pile increases. Also, it can be inferred that the *Ultimate lateral capacities of the concrete piles* having lengths 200mm, 300mm and 400mm are found to be 62 N, 110 N and 140 N respectively.

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TABLE 4: COMPARISON OF LATERAL CAPACITIES OF PILES (N)

LENGTH OF PILE (mm)	STEEL PILES (N)	CONCRETE PILES (N)
200	72	62
300	120	110
400	200	140

From the above table, it is observed that the lateral load capacity of mild steel pile is more than that of concrete pile. Therefore, it is evident that with increase in the slenderness ratio, the lateral load capacity in case of Steel pile is found to increase more than that in concrete due to the increase in the weight and stiffness (EI) of the pile.

4.2. THEORETICAL ULTIMATE LATERAL LOAD CARRYING CAPACITY

The experimental values of the ultimate lateral resistance of piles have been compared with the theoretical values predicted by plotting *p-y* curve. Sample calculations for P- y curve analysis for 200mm long Steel Pile is depicted as below.

a) *P- y curve for 200 mm long pile*

- i. $\phi = 39^\circ$
 $\gamma = 17 \text{ kN/m}^3$
 $d = 0.02 \text{ m}$
- ii. ultimate soil resistance per unit length of pile

Soil close to ground surface,

$$P_{st} = \gamma x \left[\frac{K_o x \tan \phi \sin \beta}{\tan(\beta - \phi) \cos \alpha} + \frac{\tan \beta}{\tan(\beta - \phi)} (d + x \tan \beta \tan \alpha) + K_o x \tan \beta (\tan \phi \sin \beta - \tan \alpha) - K_A d \right]$$

$\alpha = \phi/2 = 19.5^\circ$
 $\beta = 45^\circ + \phi/2 = 64.5^\circ$
 $K_o = 0.4$
 $K_A = \tan^2 (45 - \phi/2) = 0.227$

$$= 17x \left[\frac{0.4 x \tan 39 \sin 64.5}{\tan(64.5 - 39) \cos 19.5} + \frac{\tan 64.5}{\tan(64.5 - 39)} (0.02 + x \tan 64.5 \tan 19.5) + 0.4 x \tan 64.5 (\tan 39 \sin 64.5 - \tan 19.5) - 0.227 \times 0.02 \right]$$

$$= 71.905 x^2 + 1.4188 x \tag{1}$$

Soil well below the ground surface,

$$P_{sd} = K_A d \gamma x (\tan^3 \beta - 1) + K_o d \gamma x \tan \phi \tan^4 \beta$$

$$= 0.227 \times 0.02 \times 17 \times x (\tan^3 64.5 - 1) + 0.4 \times 0.02 \times 17 \times x \tan 39 \tan^4 64.5$$

$$= 30.85 x \tag{2}$$

- iii. Equating (1) and (2), $x = 0.409 \text{ m}$, therefore, use equation (1).

- iv. Deflections,
 $y_u = 3d/80 = 3 \times 0.02 / 80 = 7.5 \times 10^{-4} \text{ m}$
 $y_m = d/60 = 0.02 / 60 = 3.33 \times 10^{-4} \text{ m}$

- v. Compute P_u, P_m and m ,

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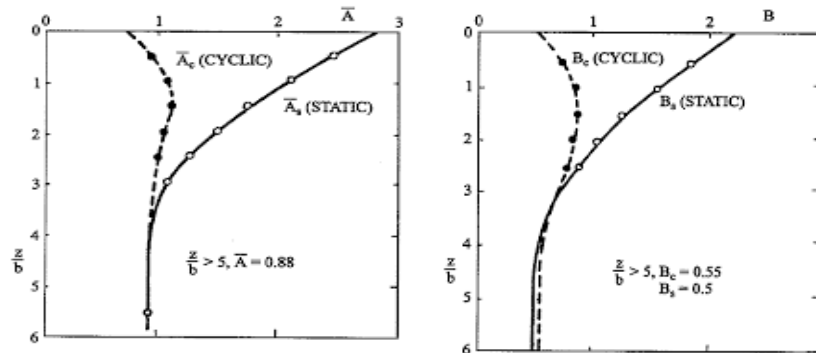
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$$P_u = A_s P_{st}$$

$$P_m = B_s P_{st}$$

$$m = \frac{P_u - P_m}{y_u - y_m} = \frac{P_u - P_m}{4.17 \times 10^{-4}}$$

Get values of A_s and B_s from the graphs shown below, for various values of x/d .



(a) Values of A_s

(b) Values of B_s

Fig.7 :Non dimensional coefficients for the development of P-y curves.

Depth x (m)	x/d	A_s	B_s	P_{st} (kN/m)	P_u (kN/m)	P_m (kN/m)	m
0.02	1	2.1	1.6	0.057138	0.1199	0.0914	68.34
0.04	2	1.45	1.1	0.1718	0.24911	0.18898	144.20
0.06	3	1.05	0.75	0.3439	0.36109	0.25792	247.41
0.08	4	0.9	0.6	0.57369	0.516321	0.34421	412.73
0.1	5	0.88	0.5	0.86093	0.75761	0.4304	784.67
0.12	6	0.88	0.5	1.20568	1.06099	0.60284	1098
0.14	7	0.88	0.5	1.60797	1.4150	0.8039	1465
0.16	8	0.88	0.5	2.06777	1.8195	1.03388	1884
0.18	9	0.88	0.5	2.5851	2.2748	1.29255	2355
0.20	10	0.88	0.5	3.1560	2.7772	1.578	2875

From the values of A_s and B_s obtained from the graphs shown in Fig 7, P_{st} , P_u , P_m , and then m are calculated for various depths, x as shown in above table.

vi. Compute n , C , y_k , P_k using the formulae given, as shown in the table below for various depths.

$$n = \frac{P_m}{m y_m}, \quad C = \frac{P_m}{y_m^{1/n}}$$

$$y_k = \left(\frac{C}{n_h x} \right)^{n-1}, \quad P_k = C y_k^{1/n}$$

For dense sand, $n_h = 50 \times 10^3 \text{ kN/m}^3$ (After Reese, 1986)

Depth x (m)	P_m (kN/m)	m	n	$1/n$	$n/n-1$	C	$n_h x$ (kN/m ²)	y_k (m)	P_k (kN/m)
0.02	0.0914	68.34	4.01	0.249	1.33	0.67	1000	6.02×10^{-5}	0.0596
0.04	0.18898	144.20	3.93	0.254	1.34	1.44	2000	6.14×10^{-5}	0.1226
0.06	0.25792	247.41	3.13	0.319	1.46	3.317	3000	4.83×10^{-5}	0.139
0.08	0.34421	412.73	2.50	0.4	1.66	8.469	4000	3.63×10^{-5}	0.141
0.1	0.4304	784.67	1.649	0.60	2.54	52.52	5000	0.94×10^{-5}	0.050
0.12	0.60284	1098	1.648	0.60	2.54	73.57	6000	1.39×10^{-5}	0.089
0.14	0.8039	1465	1.647	0.60	2.54	98.11	7000	1.96×10^{-5}	0.146

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0.16	1.03388	1884	1.647	0.60	2.54	126	8000	2.63×10^{-5}	0.225
0.18	1.29255	2355	1.647	0.60	2.54	157	9000	3.41×10^{-5}	0.327
0.20	1.578	2875	1.647	0.60	2.54	192	10000	4.36×10^{-5}	0.464

vii. Construct the p–y curve as shown below, so as to obtain the solution for the lateral loading in piles.

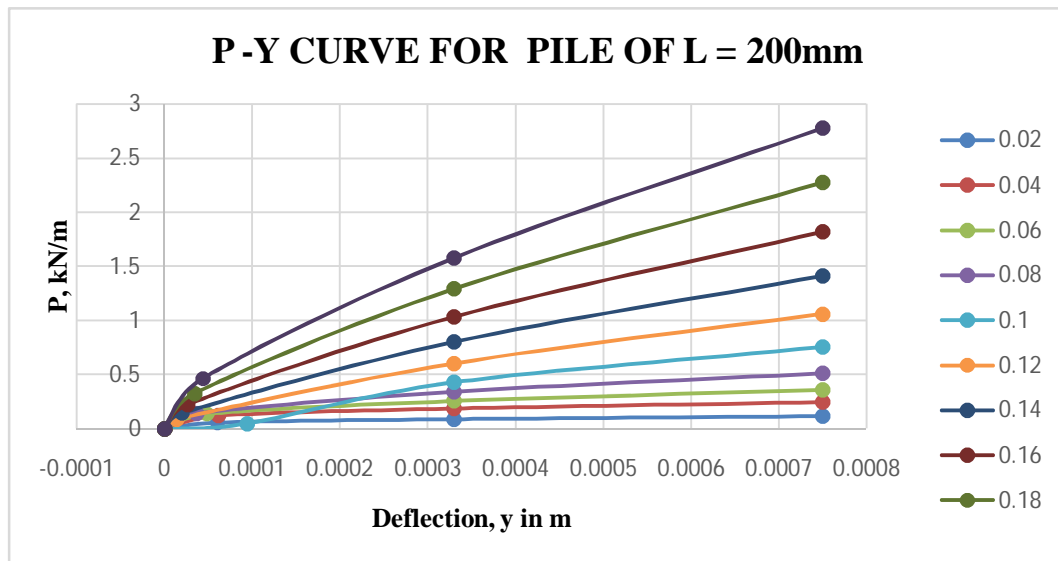


Fig. 8: P-y curve for 200mm long Pile in sand at various depths

Solution for steel pile:

- i. As a first trial, assume $n_h = 50 \times 10^3 \text{ kN/m}^3$
- ii. $EI = 1.56 \text{ kNm}^2$
- iii. $T = \left(\frac{EI}{n_h}\right)^{1/5} = \left(\frac{1.56}{50000}\right)^{1/5} = 0.125 \text{ m}$
- iv. Compute deflection y at various depths as shown in table below using the equation,

$$y = \frac{P_t T^3}{EI} A_y$$

A_y is obtained from the table given by Reese and Matlock.

P_t for steel pile is taken as 0.072 kN

Depth x (m)	$Z = x/T$	A_y	y (m)
0.02	0.16	2.176	1.96×10^{-4}
0.04	0.32	1.920	1.728×10^{-4}
0.06	0.48	1.674	1.50×10^{-4}
0.08	0.64	1.438	1.29×10^{-4}
0.1	0.8	1.216	1.09×10^{-4}
0.12	0.96	1.0116	0.91×10^{-4}
0.14	1.12	0.827	0.744×10^{-4}
0.16	1.28	0.66	0.594×10^{-4}
0.18	1.44	0.511	0.46×10^{-4}
0.20	1.6	0.381	0.343×10^{-4}

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- v. Obtain p from the p - y curve shown in Fig 8, for various values of y obtained above and compute E_s by,

$$E_s = \frac{P}{y} \text{ as shown in the table below.}$$

Depth x (m)	y (m)	P (kN/m)	E_s (kg/cm ²)
0.02	1.96×10^{-4}	0.12	6.1224
0.04	1.728×10^{-4}	0.18	10.4166
0.06	1.50×10^{-4}	0.22	14.66
0.08	1.29×10^{-4}	0.26	20.15
0.1	1.09×10^{-4}	0.1	9.17
0.12	0.91×10^{-4}	0.28	30.76
0.14	0.744×10^{-4}	0.3	40.32
0.16	0.594×10^{-4}	0.34	57.23
0.18	0.46×10^{-4}	0.4	86.95
0.20	0.343×10^{-4}	0.48	140

- vi. Plot E_s against depth, shown in Fig 9, to determine n_h by drawing an average line. The value of n_h is as $n_h = 42.57 \times 10^3 \text{ kN/m}^3$.

- vii. The value of T is obtained as

$$T = \left(\frac{EI}{n_h}\right)^{1/5} = \left(\frac{1.56}{42570}\right)^{1/5} = 0.129 \text{ m}$$

P_i for steel pile is taken as 0.072 kN

Find using the formula in step iv and are tabulated as shown below

Depth x (m)	$Z = x/T$	A_y	y (m)
0.02	0.155	2.184	2.163×10^{-4}
0.04	0.31	1.93	1.91×10^{-4}
0.06	0.465	1.69	1.67×10^{-4}
0.08	0.620	1.483	1.468×10^{-4}
0.1	0.775	1.34	1.326×10^{-4}
0.12	0.93	1.033	1.022×10^{-4}
0.14	1.08	0.85	0.8415×10^{-4}
0.16	1.24	0.74	0.7326×10^{-4}
0.18	1.39	0.56	0.554×10^{-4}
0.20	1.55	0.42	0.4158×10^{-4}

- viii. Obtain p from the p - y curve shown in Fig 8, for various values of y obtained above and compute E_s by,

$$E_s = \frac{P}{y} \text{ as shown in the table below.}$$

Depth x (m)	y (m)	P (kN/m)	E_s (kg/cm ²)
0.02	2.163×10^{-4}	0.14	6.47
0.04	1.91×10^{-4}	0.19	9.94
0.06	1.67×10^{-4}	0.24	14.37
0.08	1.468×10^{-4}	0.29	19.75
0.1	1.326×10^{-4}	0.12	11.75
0.12	1.022×10^{-4}	0.32	31.31

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0.14	0.8415×10^{-4}	0.34	40.40
0.16	0.7326×10^{-4}	0.38	51.87
0.18	0.554×10^{-4}	0.42	75.81
0.20	0.4158×10^{-4}	0.5	120.25

- ix. Plot E_s against depth as shown in Fig 9, to determine n_h by drawing an average line. The value of n_h is obtained as $n_h = 38.26 \times 10^3 \text{ kN/m}^3$.

- x. The value of T is obtained as

$$T = \left(\frac{EI}{n_h} \right)^{1/5} = \left(\frac{1.56}{41260} \right)^{1/5} = 0.132 \text{ m}$$

P_i for steel pile is taken as 0.072 kN

Find using the formula in step iv and are tabulated as shown below

Depth x(m)	$Z = x/T$	A_y	y (m)
0.02	0.150	2.1925	2.327×10^{-4}
0.04	0.30	1.952	2.071×10^{-4}
0.06	0.45	1.72	1.824×10^{-4}
0.08	0.60	1.496	1.587×10^{-4}
0.1	0.75	1.284	1.362×10^{-4}
0.12	0.90	1.086	1.152×10^{-4}
0.14	1.06	0.988	1.048×10^{-4}
0.16	1.21	0.75	0.795×10^{-4}
0.18	1.36	0.588	0.623×10^{-4}
0.20	1.51	0.4625	0.49×10^{-4}

- xi. Obtain p from the p - y curve shown in Fig 8, for various values of y obtained above and compute E_s by,

$$E_s = \frac{P}{y} \text{ as shown in the table below.}$$

Depth x(m)	y (m)	P (kN/m)	E_s (kg/cm ²)
0.02	2.327×10^{-4}	0.14	6.00
0.04	2.071×10^{-4}	0.19	9.17
0.06	1.824×10^{-4}	0.25	13.70
0.08	1.587×10^{-4}	0.30	18.90
0.1	1.362×10^{-4}	0.18	13.21
0.12	1.152×10^{-4}	0.32	27.77
0.14	1.048×10^{-4}	0.36	34.35
0.16	0.795×10^{-4}	0.39	49
0.18	0.623×10^{-4}	0.43	69
0.20	0.49×10^{-4}	0.56	114.28

- xii. Plot E_s against depth as shown in Fig 9, to determine n_h by drawing an average line and n_h is obtained from the third trial as $n_h = 37 \times 10^3 \text{ kN/m}^3$.

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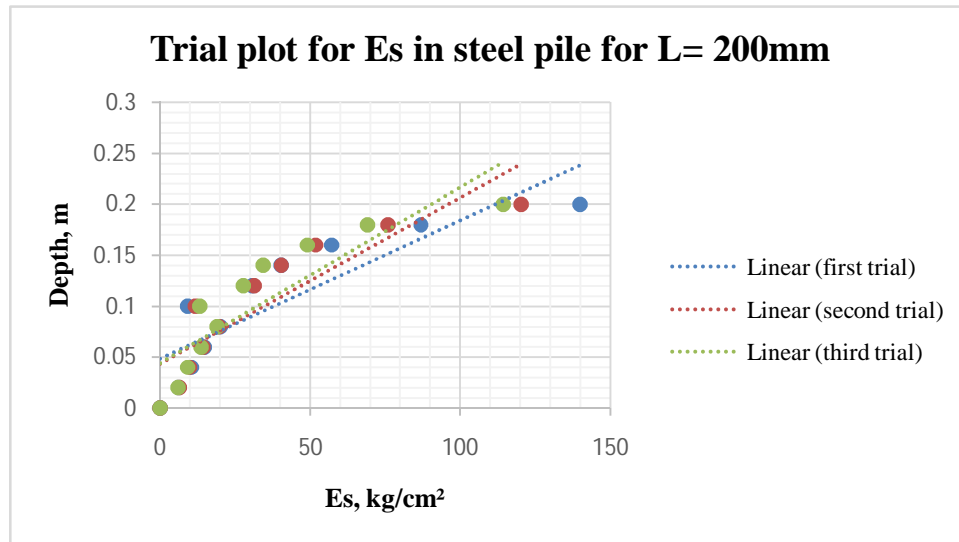


Fig.9: Trial plots for E_s for 200 mm long steel pile

- xiii. The value of ultimate lateral load for 200mm long steel pile, obtained from the equation $P = E_s y$ corresponding to $n_h = 37 \times 10^3 \text{ kN/m}^3$ is,
 $P_U = 172 \text{ N}$.

Similarly, the ultimate lateral capacities of other Piles are obtained simultaneously by plotting $P - y$ curves and they are compared with that of its experimental values, as tabulated in the Table 5.

TABLE 5: EXPERIMENTAL AND THEORETICAL VALUES OF LATERAL LOAD CAPACITY (N)

Pile type	Length of Pile in mm	L/d	Theoretical ultimate Lateral capacity using $P - Y$ curve in (N)	Experimental values (N)
Steel Pile	200	10	172	72
	300	15	196	120
	400	20	211	200
Concrete Piles	200	10	110	62
	300	15	160	110
	400	20	190	140

From the above table, it can be seen that the experimental values of lateral load carrying capacities of steel and concrete piles are found to be comparable to the predicted values for $P - y$ curve method. Also it can be seen that for shorter piles, the theoretical values have overestimated the experimental values by more than 100%, whereas, for longer piles the percentage variations are found to be lesser, i.e, from 5% to 45 %.

V. CONCLUSION

- The lateral capacities of the piles are found to increase with increase in the L/d ratio. This could be due to increase of the passive resistance with the increase in pile length. Also, on applying loads in various inclinations, increase in the inclined load carrying capacity of the vertical pile was found at lesser values of θ , mainly at 15° . The lateral load capacities of steel piles were found to be more than that of concrete piles.

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- The curves of horizontal component of the load versus lateral settlement of the steel piles and concrete piles at 15°, 30°, 45° and 60° are different from pile's vertical settlement under lateral loading condition (at $\alpha = 90^\circ$) i.e. when no vertical components are present. Further it is observed that the presence of axial component of the applied load, when the inclination of loading with the vertical axis of pile approaches an angle around 60° to 90° caused an increase in lateral load carrying capacity of the pile and at 15° a decrease in the lateral load carrying capacity. The Ultimate lateral capacities of the steel piles having lengths 200mm, 300mm and 400mm are found to be 72 N, 120 N and 200 N respectively. The Ultimate lateral capacities of the concrete piles having lengths 200mm, 300mm and 400mm are found to be 62 N, 110 N and 140 N respectively.
- The lateral load capacity in case of Steel piles is found to be more than that in concrete piles due to the increase in the weight and stiffness (EI) of the pile. The modulus of elasticity is also more for steel when compared to concrete.
- Experimental values of lateral load carrying capacities of steel and concrete piles are comparable to the values from $P-y$ curve method. For shorter piles, the theoretical values have overestimated the experimental values by more than 100%, whereas, for longer piles the percentage variations are found to be lesser, i.e, from 5% to 45%.

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