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EFFECT OF SLOPE ON *P-Y* CURVES DUE TO SURCHARGE LOAD

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ABSTRACT

An extensive program of laboratory model tests was undertaken to study the effect of slope on *p-y* curves due to surcharge load in dry sand. The paper concerns the method developed in a series of laboratory model tests to experimentally determine *p-y* curves. Bending moment curves are differentiated by using curve fitting method of cubic polynomial function. The study includes effect of slope angle and relative density on bending moment, lateral soil resistance, lateral deflection and non-dimensional *p-y* curves. The non-dimensional *p-y* curves for piles on sloping ground under surcharge load are developed modifying API RP 2A (2000) method by including a Reduction Factor (*R*) using the experimental results.

Key words: bending moment, *p-y* curves, sloping ground, soil resistance, surcharge load (**IGC**: E/E12/E14)

INTRODUCTION

Piles are frequently used to support structures subjected to lateral forces and moments such as offshore structures, harbour structures, high rise buildings and bridge abutments. The governing criterion in designing pile foundations to resist lateral loads in most cases is the maximum deflection of the foundation rather than its ultimate capacity. The maximum deflection at the pile head is important to satisfy the serviceability requirements of the superstructure while the bending moment is required for the structural sizing of piles.

Early research on single pile was directed mainly towards estimating the ultimate capacity, assuming that the deformations would be acceptable if an adequate factor of safety against ultimate failure was used to determine the allowable load capacity. Broms (1964) developed solutions for the ultimate lateral resistance of a pile assuming distribution of lateral pile-soil pressures and considering the statics of the problem. Two modes of failure yielding of the soil along the length of the pile (short-pile failure), and yielding of the pile itself at the point of maximum moment (long-pile failure) are consider. Narashimha Rao et al. (1998) investigated the lateral load capacity of pile groups in soft marine clay and they found that the lateral load capacity mainly depends on rigidity of pile soil system.

Many analytical approaches have been developed in recent years for the response analysis of laterally loaded piles. The approaches assume either the theory of subgrade reaction (e.g., Matlock and Ripperger, 1956) or the theory of elasticity (e.g., Poulos, 1971; Pise, 1984: Randolph, 1981). However, the load deflection behaviour of laterally loaded piles is highly nonlinear and hence requires a nonlinear analysis. Poulos and Davies (1980) and Budhu and Davies (1987) modified the elastic solutions to account for nonlinearity using yield factors, the modulus of sub-grade reaction approach was extended to account for the soil nonlinearity. This was done by introducing *p-y* curves (Matlock, 1970; Reese and Welch, 1975).

Experimental studies were performed to examine the performance of piles under lateral loads. Alizadeh and Davison (1970) described a pile testing program conducted to determine the lateral load–deflection behaviour for individual vertical and batter piles and the effect of sand density on the pile response. The results showed the significant effect of the relative density of sand on pile behaviour. Prakash and Kumar (1996) developed a method to predict the load deflection relationship for single piles embedded in sand and subjected to lateral load, considering soil nonlinearity based on the results of 14 full-scale lateral pile load tests. However all these studies have been directed towards the response of individual piles or pile groups subjected to lateral load at pile head (direct lateral load) on horizontal ground. Very few research works have been carried out on piles subjected to lateral load on sloping ground (Mezazigh and Levacher, 1998; Clarles and Zhang, 2001).

When piles situated in sloping ground are subjected to horizontal movement (passive loading), horizontal pressures are developed between the pile and the soil with consequent development of bending moments and deflections in the piles. This phenomenon is analogous as to the phenomenon of negative friction developed in piles by

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vertical movement of the surrounding soil. The analysis of piles in soil undergoing lateral movement is studied by Poulos (1973). Ito and Matsui (1975) proposed an empirical equation for the estimation of lateral force acting on stabilizing piles. Stewart et al. (1993), Bransby and Springman (1996), Kim and Barker (2002) and Cai and Ugai (2003) have also studied the effect of lateral soil movement on pile behaviour. However, the studies on behaviour of piles on sloping ground under surcharge load are limited. Hence, the need for new research is necessary in this area. A study was accordingly undertaken to determine the effect of slope and surcharge load on *p-y* curves. The objective was to estimate a Reduction Factor that can be applied on the *p-y* curves for single piles in horizontal ground.

The paper describes the details of the experimental studies and the response of piles under surcharge load.

METHOD

Soil-pile interaction problems are three dimensional and are presently very complex to be solved by theoretical or numerical methods. Tests at full scale are impractical or very expensive due to large number of tests required.

Laboratory model test makes it possible to investigate this kind of problems by instrumenting the model piles with strain gauges, to measure the bending moments. The *p-y* reaction curves are obtained by double differentiation and double integration of experimental bending moments.

TEST PROGRAM

The objective of this paper is to study the effect of slope on *p-y* curves under surcharge load for flexible long piles installed at the crest of the slopes in dry river sand. Three slopes were tested ($1V$:$1.5H$, $1V$:$1.75H$ & $1V$:$2H$) with three different relative densities of 30%, 45% and 70% of the sand.

MODEL STUDY

The dimensions of the model pile testing are determined by a dimensional analysis (Buckingham Pi theorem). There are five variables, which are displacement (y), pile diameter (d), area (A), lateral force (F) and pile lateral stiffness (EI). In order to comply with complete similitude between the model and the prototype, the scaling factors are $1/N$, $1/N$, N^2, N^2, N^4. Accordingly, the model pile was 25 mm in outer diameter, 23 mm in inner diameter, and 700 mm in length. The model pile is modeled as an equivalent prototype pile (1:30 scale) with an outer diameter (d) of 750 mm, bending stiffness (EI_p) of 330 MN-m^2, and equivalent diameter to thickness ratio (d/t) of 50.

EXPERIMENTAL SET-UP

The experimental set-up is shown in Fig 1. The test

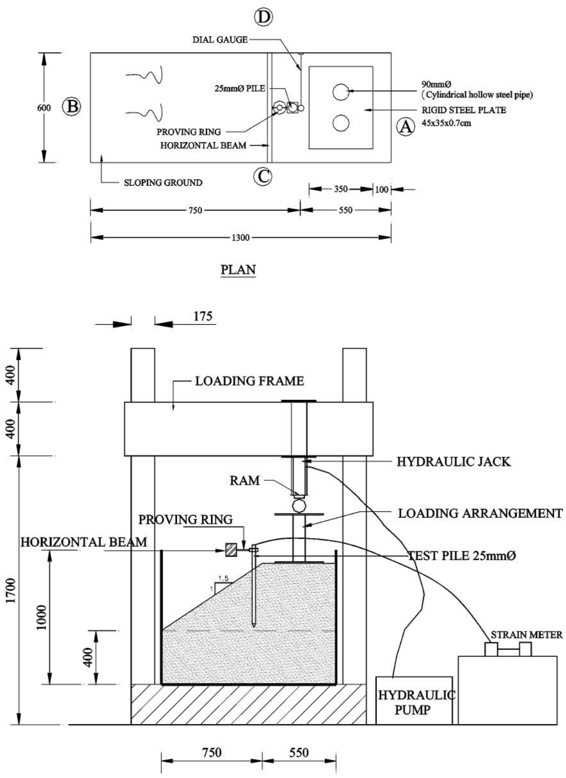

Fig. 1. Experimental set up

tank size is $1.3 \times 0.6 \times 1$ m deep. To avoid any side friction between the tank wall and soil, two layers of plastic sheets are coated with silicon grease bonded to the inside of the tank wall. The surcharge load is applied through hydraulic jack, which is fixed to the loading frame. The capacity of the jack is 250 kN and ram diameter is 75 mm. The test pile is placed at the top edge of the sloping ground (slope crest) for all slope angles under consideration. In order to distribute the surcharge load as uniformly as possible, a $450 \times 350 \times 10$ mm thick steel plate is used to transfer the load from the jack to the soil.

TEST PILE

An aluminium pipe pile having an outer diameter of 25 mm with 1mm wall thickness is used as a test pile. The total length of the model pile is 700 mm and the embedment depth is 550 mm. The flexural stiffness of the pile is determined by considering a simply supported beam test. The flexural stiffness of the pile is 416×10^6 N-mm^2. The model pile is instrumented with electrical resisting type strain gauges of 3 mm in length, 120 ohms resistance (R) and a gauge factor (K) of 2. A total of 24 strain gauges are used each at 50 mm spacing in both compression and tension side of the pile. The instrumentation details are shown in Fig. 2. The strain gauges used are calibrated by conducting a bending test. The strain response is linear with the bending moment and the calibration constant is obtained from the slope of the straight line, which is 14.2 N-mm per micro strain. (1×10^{-6} mm/mm).

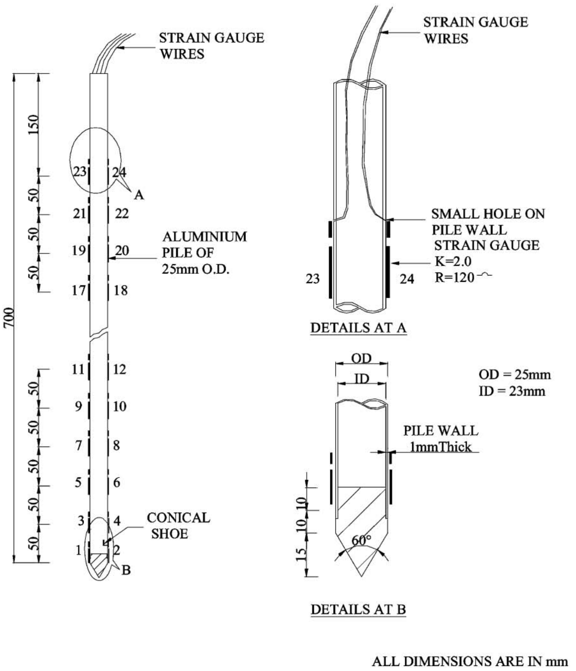

Fig. 2. Details of instrumented pile

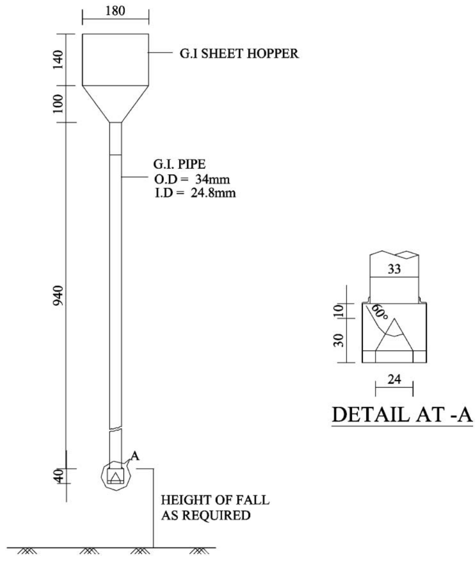

Fig. 3. Details of sand raining device

PLACEMENT OF SAND

The test is conducted in dry river sand. The properties of the sand are; Effective particle size (D_{10}) is 0.26 mm, Average particle size (D_{50}) is 0.54 mm, Coefficient of Uniformity (C_u) is 2.4, Coefficient of Curvature (C_c) is 1.1, Specific Gravity (G_s) is 2.65, Maximum Dry Density (γ_{max}) is 17.9 kN/m^3 and Minimum Dry Density (γ_{min}) is 15.3 kN/m^3. To achieve uniform density in the tank, a pipe and cone arrangement called sand raining device is fabricated. This arrangement contains a hopper connected to a 940 mm long pipe and an inverted cone at the bottom. The hopper has a holding capacity of about 80 N of sand. The sand passes through a 25 mm internal diameter pipe and is dispersed by 60° due to the inverted cone placed at the bottom. The height of fall is measured from the bottom of the pipe using an adjustable length pointer fixed at the bottom. The sand raining device is shown in Fig. 3. This arrangement is calibrated by a number of trials to get the height of fall corresponding to 30%, 45% and 70% of relative density.

TEST PROCEDURE

The test pile is placed in position and then the soil is filled to the required depth by sand raining method. The relative density of 30%, 45% and 70% is selected such that, the relative density is covered in the filed condition from loose state to dense state. The slope is varied as 1*V*:1.5*H*, 1*V*:1.75*H* and 1*V*:2*H*. These slopes are likely to be unstable due to external load. The surcharge load is applied up to 50 kN with 10 kN increments. The piles used in berthing structure generally have tie beam at the top. The tie beam stiffness is modeled by providing rigid link between the pile and the horizontal beam facing CD (Fig. 1) fixed in the side wall of the tank. The horizontal displacement of the pile head and the strain at various levels in the piles are measured at each load increments. Vertical settlements are also measured for few tests and it is observed to be negligible. The mechanical dial gauges are used to measure the horizontal displacement. A manual compensating strain meter with full bridge circuit is used to measure bending strain along pile length at various elevations. The lateral deflection and strain readings are recorded for each increment of load.

RESULTS AND DISCUSSIONS

Effect of Slope on Bending Moment

Typical bending moment variations are shown in Figs. 4 to 6 for 30%, 45% and 70% relative density with 1*V*:1.5*H* and 1*V*:2*H* slope respectively. From all the figures it can be seen that the increase in surcharge load increases the bending moment. This is due to increasing lateral soil movement. The maximum bending moment of 24000 Nmm is observed in 30% relative density with 1*V*:1.5*H* slope and minimum bending moment of 13200 Nmm is observed in 70% relative density with 1*V*:2*H* slope. The depth of maximum bending moment is observed at 12*D* and 14*D* for slope angle of 1*V*:1.5*H* and 1*V*:2*H* respectively. However the change in relative density does not have significant influence on the depth of maximum bending moment for flatter slope of 1*V*:2*H*.

Effect of Steepness of Slope and Relative Density on Maximum Bending Moment

Figure 7 shows the effect of slope on maximum bending moment for 30%, 45% and 70% relative density with 50 kN surcharge load. The increase in steepness of slope increases the maximum bending moment for all three relative densities. The increase in steepness of slope from 1*V*:2*H* to 1*V*:1.5*H* increases the maximum bending mo-

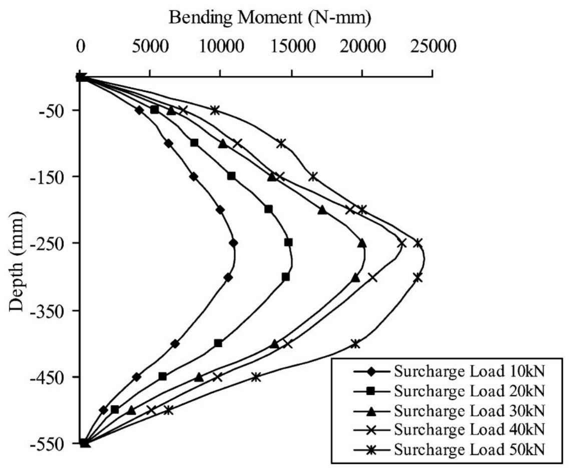

Fig. 4. Bending moment vs depth for 1*V*:1.5*H* slope with 30% relative density

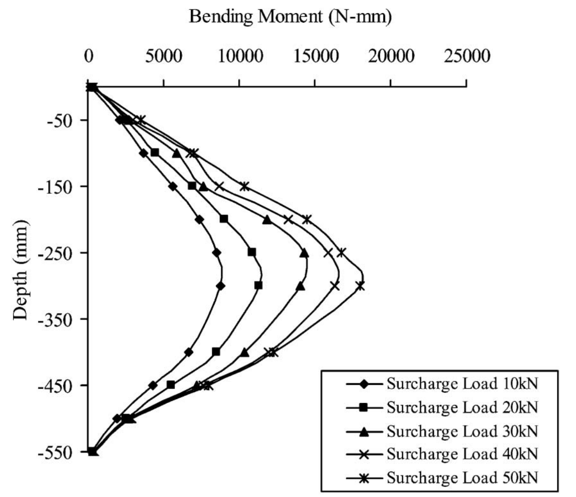

Fig. 5. Bending moment vs depth for 1*V*:2*H* slope with 45% relative density

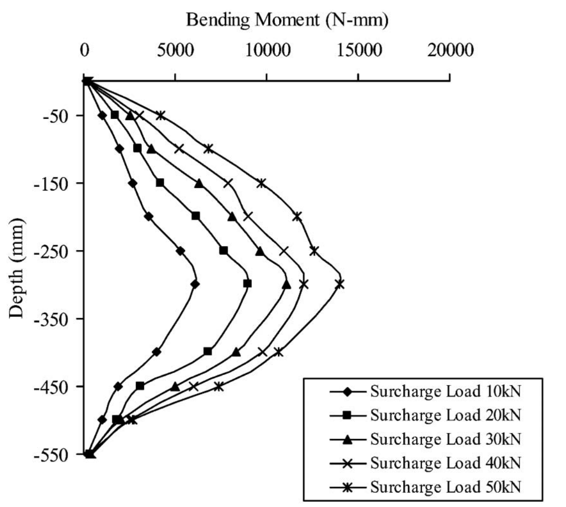

Fig. 6. Bending moment vs depth for 1*V*:2*H* slope with 70% relative density

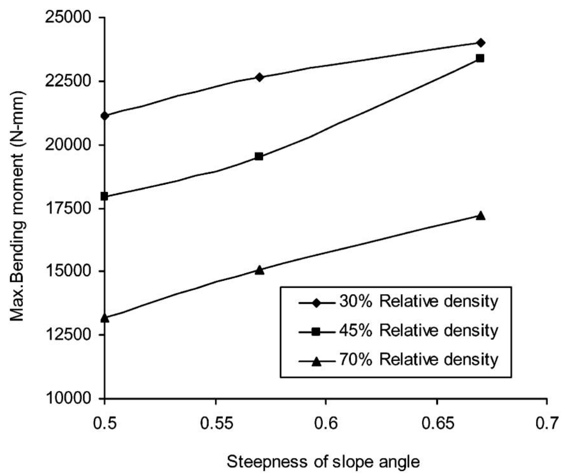

Fig. 7. Effect of steepness of slope on maximum bending moment for 50 kN surcharge load

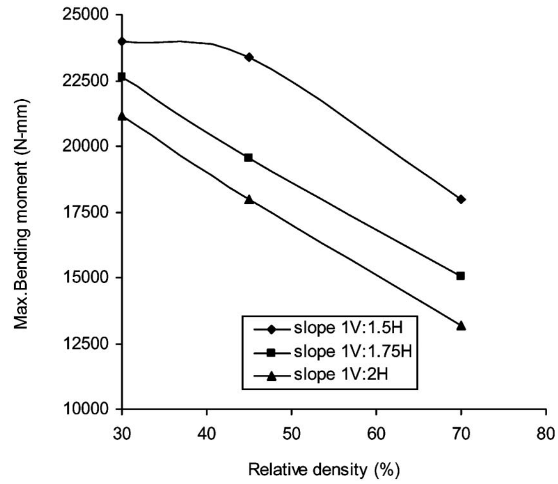

Fig. 8. Effect of relative density on maximum bending moment for 50 kN surcharge load

ment by 11%, 23% and 23.5% for relative density of 30%, 45% and 70% respectively. The maximum reduction is observed in flatter slope of 1*V*:2*H* with 70% relative density. Figure 8 shows the effect of relative density on maximum bending moment for 1*V*:1.5*H*, 1*V*:1.75*H* and 1*V*:2*H* slope with 50 kN surcharge load. The increase in relative density reduces the maximum bending moment for all three slope angles. The increase in relative density from 30% to 70% reduces the maximum bending moment by 25%, 20% and 37% for slope angle of 1*V*:1.5*H*, 1*V*:1.75*H* and 1*V*:2*H* respectively.

Estimation of Soil Resistance and Deflection

The soil resistance (p) and lateral deflection (y) along the pile shaft are obtained from the measured bending moment in the experiments using an approach similar to that presented by Matlock and Ripperger (1956) and Naggar and Wei (1999).

The distribution of the bending moment along the pile shaft is curve fitted by a cubic polynomial function, i.e.,

$$M(x) = ax^3 + bx^2 + cx + d \quad (1)$$

Where x is the depth below the sand surface, and a, b, c, and d are constants obtained from the curve-fitting process. The distribution of the soil resistance along the pile shaft is obtained by double differentiating the bending moment, i.e.,

$$P(x) = \frac{d^2M}{dx^2} = 6ax + 2b \quad (2)$$

The deflection of the pile along its shaft is obtained by double integrating the bending-moment function, i.e.,

$$y(x) = \frac{1}{EI}\left\{\int\int [M(x)dx]dx\right\} \quad (3)$$

which yielded

$$y(x) = \frac{1}{EI}\left\{\frac{a}{20}x^5 + \frac{b}{12}x^4 + \frac{c}{6}x^3 + \frac{d}{2}x^2 + Fx + G\right\} \quad (4)$$

In Eq. (4), a, b, c, and d are the curve-fitting constants and EI is the flexural rigidity of the pile. F and G are integrating constants which are obtained from the boundary conditions. Two boundary conditions are used to obtain the integral constants of F and G. First boundary condition is, slope is zero at the maximum bending moment occurring depth (i.e., when x = depth of maximum bending moment, $dy/dx = 0$). Second boundary condition is, deflection is zero at the pile tip (i.e., $x = l$, $y = 0$. where l is the depth of embedment). The curve-fitting procedure is introduced to smoothen the bending moment diagram in order to reduce the scatter of experimental errors.

The obtained soil resistance along the depth of pile for 45% relative density with slope angles of $1V{:}1.5H$, $1V{:}1.75H$ and $1V{:}2H$ are shown in Figs. 9 to 11 and the obtained corresponding deflections along the depth of pile are shown in Figs. 12 to 14. From all the figures, it is observed that the increase in surcharge load increases the soil resistance and deflection. The negative soil resistance

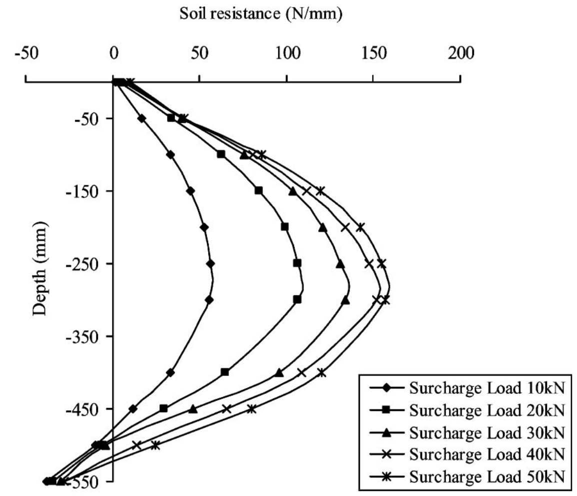

Fig. 10. **Soil resistance vs depth for 45% relative density with $1V{:}1.75H$**

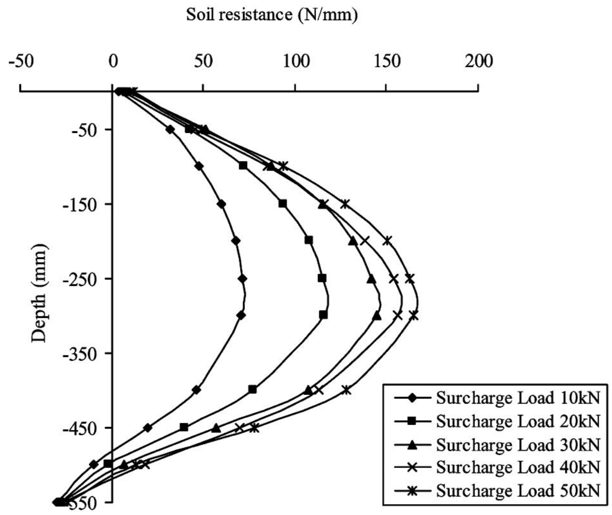

Fig. 11. **Soil resistance vs depth for 45% relative density with $1V{:}2H$**

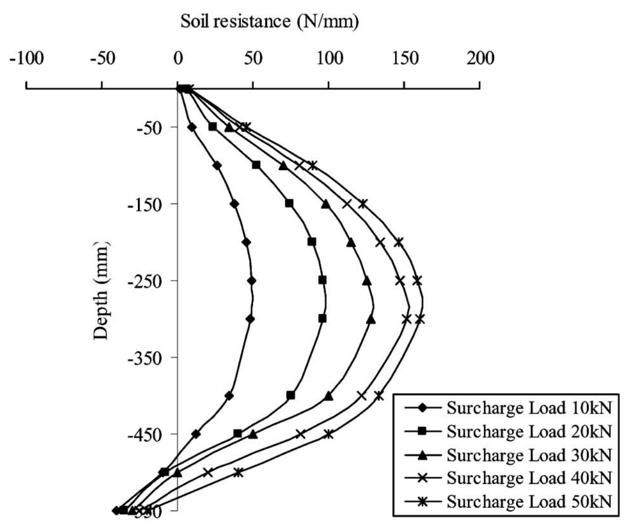

Fig. 9. **Soil resistance vs depth for 45% relative density with $1V{:}1.5H$**

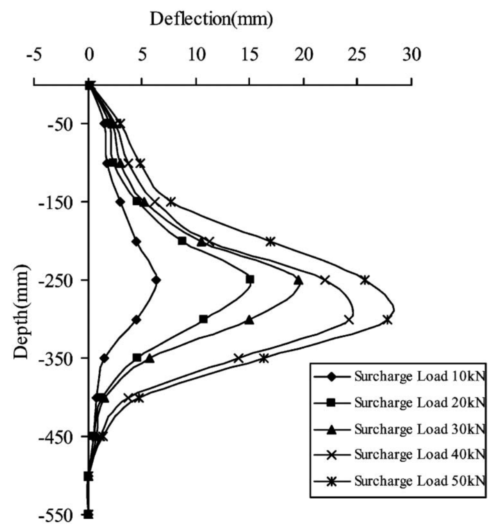

Fig. 12. **Deflection vs depth for 45% relative density with $1V{:}1.5H$ slope**

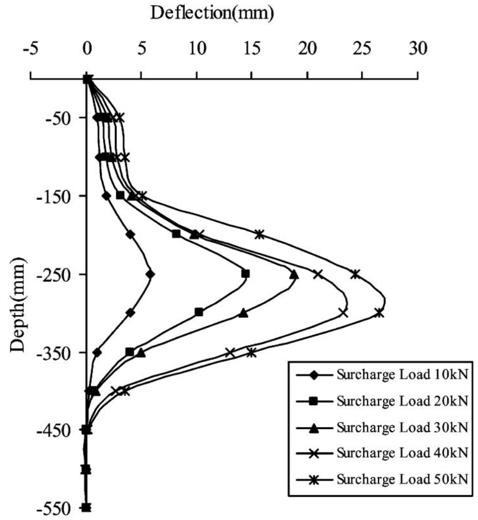

Fig. 13. Deflection vs depth for 45% relative density with 1*V*:1.75*H* slope

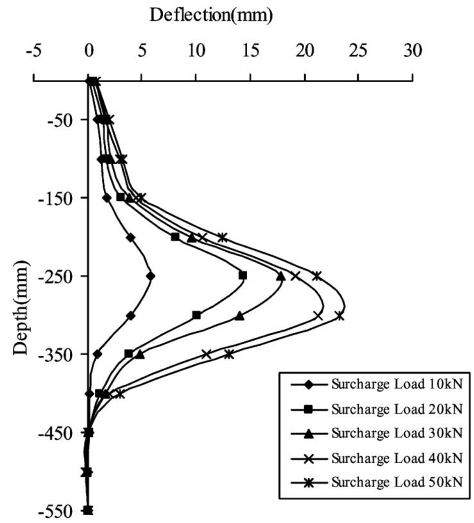

Fig. 14. Deflection vs depth for 45% relative density with 1*V*:2*H* slope

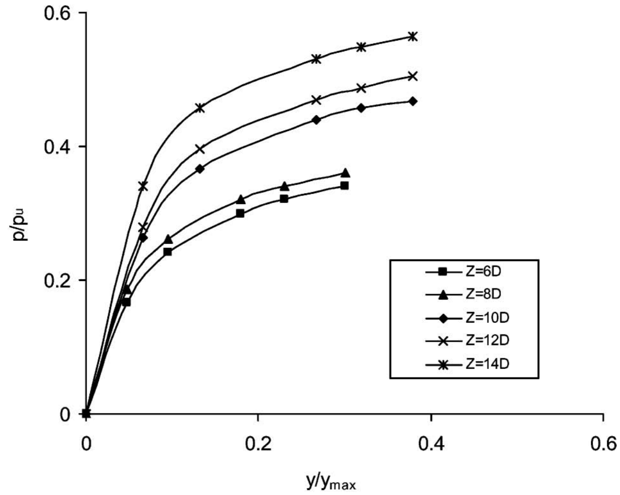

Fig. 15. Non dimensional *p-y* curve for 30% relative density with 1*V*:1.5*H* slope

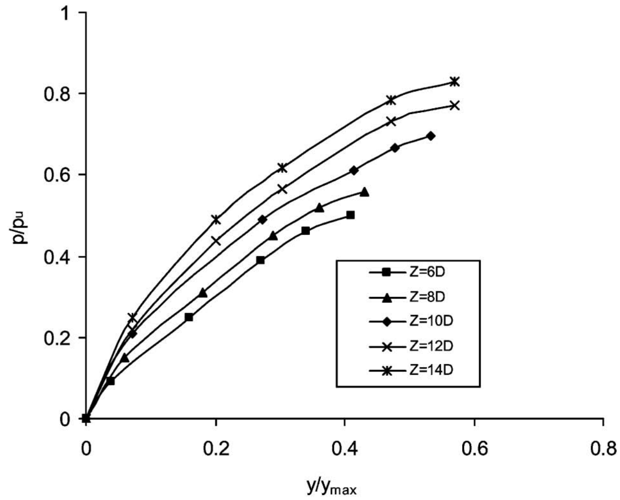

Fig. 16. Non dimensional *p-y* curve for 45% relative density with 1*V*:1.5*H* slope

below -500 mm depth indicates that the mobilization of passive resistance is in the embankment side. The deflection obtained below -450 mm is almost equal to zero for 45% relative density with 1*V*:1.5*H* slope.

Effect of Slope on Non Dimensional p-y Curves

The ultimate soil resistance (p_u) is calculated using the equation given in API RP 2A (2000). These two equations (Eqs. (5) and (6)) are used to calculate the ultimate soil resistance of piles in horizontal ground and these equations can not be used for piles located on sloping ground.

$$p_{us} = (C_1 Z + C_2 D)\gamma Z \tag{5}$$

$$p_{ud} = C_3 D \gamma Z \tag{6}$$

where,

p_u–ultimate resistance (force/unit length) (kN/m) (*s*-shallow, *d*-deep)
γ–effective soil unit weight (kN/m^3)
Z–depth in (m)
ϕ–angle of internal friction of sand. (degrees)
C_1, C_2, C_3–Coefficients determine based on angle of internal friction
D–average pile diameter (m)

Equation (5) is used for shallow depths and Eq. (6) is used for deeper depths. The coefficients C_1, C_2 and C_3 are obtained based on the angle of internal friction. The maximum lateral deflection (y_{max}) is taken as the maximum deflection observed from the experimental results. p_u and y_{max} are used to obtain the non-dimensional parameters for p and y respectively.

The non dimensional *p-y* curves for 30%, 45% and 70% relative densities with 1*V*:1.5*H* slope are shown in Figs. 15 to 17. From these figures, it is observed that the increase in depth increases the (p/p_u). This is due to the increase in passive resistance for increase in overburden

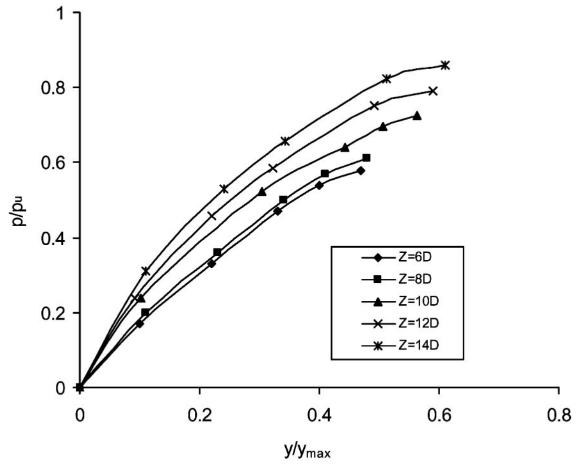

Fig. 17. **Non dimensional *p-y* curve for 70% relative density with 1*V*:1.5*H* slope**

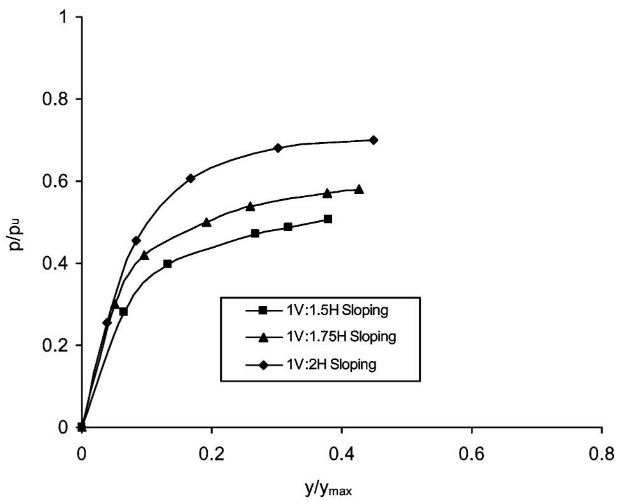

Fig. 18. **Effect of slope on non dimensional *p-y* curve for 30% relative density at $Z=12D$**

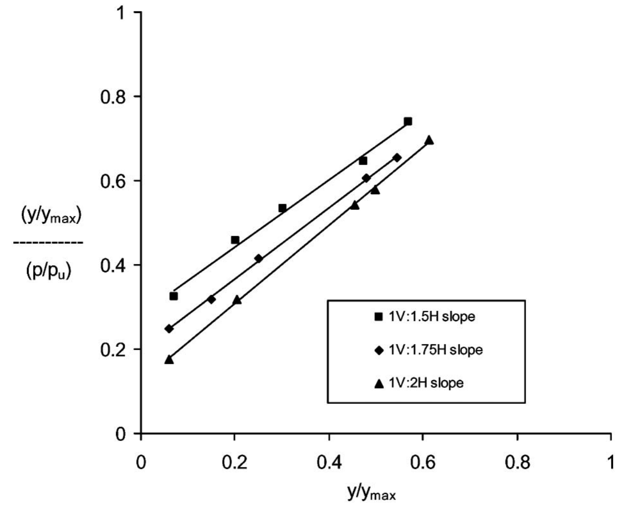

Fig. 19. **$(y/y_{max})/(p/p_u)$ vs (y/y_{max}) for 45% relative density at $Z=12D$**

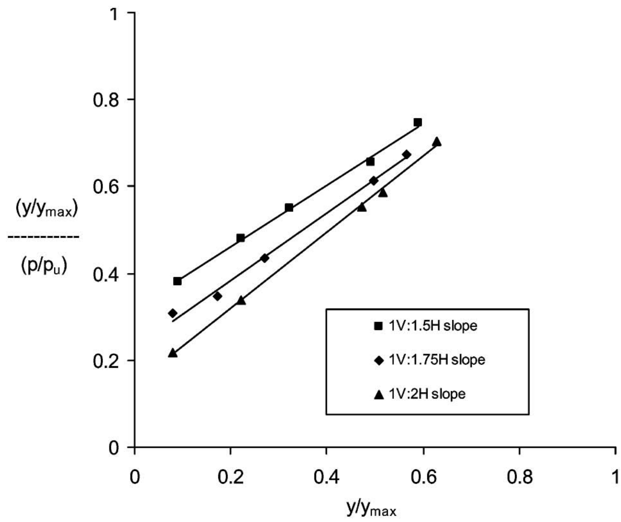

Fig. 20. **$(y/y_{max})/(p/p_u)$ vs (y/y_{max}) for 70% relative density $Z=12D$**

pressure of the soil mass as depth increases. The increment is more in 1 *V*:2*H* slope than 1:1.5*H* slope which can be seen from Fig. 18.

The normalized *p-y* curves are further normalized to obtain the (p/p_u) for all slopes and relative densities. These types of normalized plots are presented by Wu et al. (1998). The values of y/y_{max} are further divided by p/p_u and then the plot is drawn between y/y_{max} and $(y/y_{max})/(p/p_u)$. In horizontal ground the value of p/p_u is taken as 1, when the soil resistance reaches to ultimate level (the mobilization of passive resistance at the failure stage is equal to the ultimate passive resistance of the soil). Figures 19 and 20 show the effect of slope on $(y/y_{max})/(p/p_u)$ vs (y/y_{max}) plot for 45% and 70% relative density at a depth of $Z=12D$. The values of p/p_u are the slopes of $(y/y_{max})/(p/p_u)$ vs (y/y_{max}) curve are taken from the plot for different relative density with different slope angle. The values of p/p_u obtained from experiments are presented in Table 1.

Construction of Lateral Load–Deflection (p-y) Curve for Sloping Ground

The API RP 2A (2000) method for construction of *p-y* curves in horizontal ground is modified for piles in sloping ground under surcharge load using the reduction factor (R) as given in Eq. (7)

$$p = A \times R \times p_u \times \tan h\left[\frac{k \times Z}{A \times R \times p_u} \times y\right] \tag{7}$$

A–factor to account for cyclic or static loading condition. Evaluated by;
$A = 0.9$ for cyclic loading
$A = (3.0\text{–}0.8\ Z/D) \geq 0.9$ for static loading
R = factor to account the slope of the ground surface
p_u–ultimate bearing capacity at depth Z (kN/m)
k–initial modulus of subgrade reaction (kN/m^3)
Z–depth in meter
y–lateral deflection in meter

Table 1. Values of p/p_u obtained from experiments

Relative density	Slope	Z/D	p/p_u
30%	$1V$:$1.5H$	8	0.47
30%	$1V$:$1.5H$	10	0.60
30%	$1V$:$1.5H$	12	0.69
30%	$1V$:$1.75H$	8	0.64
30%	$1V$:$1.75H$	10	0.66
30%	$1V$:$1.75H$	12	0.70
30%	$1V$:$2H$	8	0.64
30%	$1V$:$2H$	10	0.74
30%	$1V$:$2H$	12	0.80
45%	$1V$:$1.5H$	8	0.50
45%	$1V$:$1.5H$	10	0.62
45%	$1V$:$1.5H$	12	0.72
45%	$1V$:$1.75H$	8	0.66
45%	$1V$:$1.75H$	10	0.74
45%	$1V$:$1.75H$	12	0.80
45%	$1V$:$2H$	8	0.69
45%	$1V$:$2H$	10	0.78
45%	$1V$:$2H$	12	0.83
70%	$1V$:$1.5H$	8	0.54
70%	$1V$:$1.5H$	10	0.69
70%	$1V$:$1.5H$	12	0.75
70%	$1V$:$1.75H$	8	0.68
70%	$1V$:$1.75H$	10	0.75
70%	$1V$:$1.75H$	12	0.83
70%	$1V$:$2H$	8	0.71
70%	$1V$:$2H$	10	0.80
70%	$1V$:$2H$	12	0.85

p/p_u value obtained from the experimental results is used to predict R using the multiple regression analysis. In the multiple regression analysis, R is used as dependent variable and slope (S) and Z/D as independent variables. The following Eq. (8) is obtained based on the multiple regression analysis.

$$R = 0.74 + 0.0378(Z/D) - 0.6315(S);\ R \leq 1 \qquad (8)$$

Z–depth in meter
D–diameter of pile in meter
S–slope angle in radians (applicable in the range of 0.66 to 0.50)

The scatter plot of reduction factor (R) is shown in Fig. 21. The value of R^2 is equal to 0.9639 and the maximum error is 0.06725 and hence the fit is very good. The variation of R as a function of Z/D for $1V$:$1.5H$, $1V$:$1.75H$ and $1V$:$2H$ slope are shown in Fig. 22. The increase in Z/D increases R and the value of $R = 1$ is observed at $Z/D = 18$, 16.5 and 16 for $1V$:$1.5H$, $1V$:$1.75H$ and $1V$:$2H$ slopes respectively.

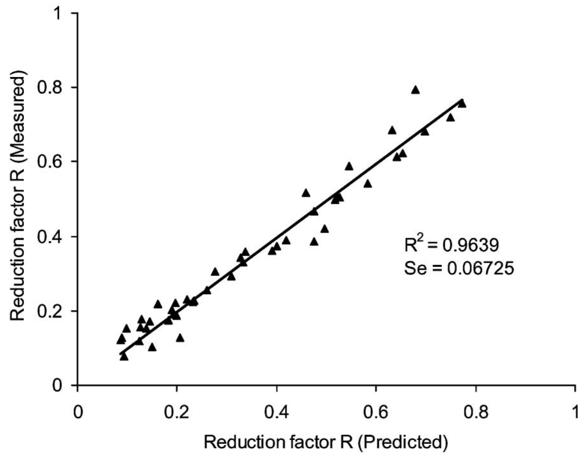

Fig. 21. Scatter plot for reduction factor (R)

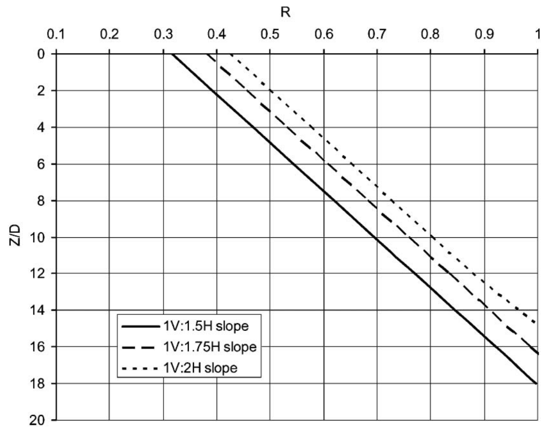

Fig. 22. Variation of reduction factor (R)

CONCLUSIONS

1. The results of the model study are used to modify the API method for construction of p-y curves for piles in horizontal ground under surcharge load for the analysis and design of piles in sloping ground. The effect of sloping ground is included using a reduction factor 'R' in the modified approach. The value of the R is observed to increase from 0.31 to 0.42 at $Z/D = 0$ as slope is varied from $1V$:$1.5H$ to $1V$:$2H$. As Z/D increases, R increases and the limiting value of 1 for R is observed at $Z/D = 18$, 16.5 and 16 for $1V$:$1.5H$, $1V$:$1.75H$ and $1V$:$2H$ slopes respectively.
2. The reduction factor R is obtained based on the results of a single pile located at the slope crest. Therefore, the application of the reduction factor R is limited to single pile located at the slope crest.
3. The increase in steepness of slope from $1V$:$2H$ to $1V$:$1.5H$ increases the maximum bending moment by 11%, 23% and 23.5% for relative density of 30%, 45% and 70% respectively. The increase in relative density from 30% to 70% reduces the maximum bending moment by 25%, 20% and 37% for slope angle of $1V$:$1.5H$, $1V$:$1.75H$ and $1V$:$2H$ respectively.
4. The increase in steepness of slope from $1V$:$2H$ to $1V$:$1.5H$ reduces the maximum soil resistance by 3%, 6% and 8% for relative density of 30%, 45% and 70% respectively. The increase in relative density from 30% to 70% increases the maximum soil resistance by 15%, 19.8% and 17.5% for slope of $1V$:$1.5H$, $1V$:$1.75H$ and $1V$:$2H$ respectively.

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