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Experimental study of a modeled building frame supported by pile groups embedded in cohesionless soil

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Abstract. This paper presents the results of static vertical load tests carried out on a model building frame supported by pile groups embedded in cohesionless soil (sand). The effect of soil interaction on displacements and rotation at the column base and also the shears and bending moments in the columns of the building frame were investigated. The experimental results have been compared with those obtained from the finite element analysis and conventional method of analysis. Soil nonlinearity in the lateral direction is characterized by the *p-y* curves and in the axial direction by nonlinear vertical springs along the length of the piles (τ-z curves) at their tips (Q-z curves). The results reveal that the conventional method gives the shear force in the column by about 40-60%, the bending moment at the column top about 20-30% and at the column base about 75-100% more than those from the experimental results. The response of the frame from the experimental results is in good agreement with that obtained by the nonlinear finite element analysis.

Keywords: soil interaction; experimental analysis; pile group; building frame; cohesionless soil.

1. Introduction

The influence caused by the settlement of the supporting ground on the response of framed structures was often ignored in structural design. Soil settlement is a function of the flexural rigidity of the superstructure. The structural stiffness can have a significant influence on the distribution of the column loads and moments transmitted to the foundation of the structure. Previous studies have, however, indicated that the effect of interaction between soil and structure can be quite significant. Interaction analyses have been reported in numerous previous studies such as Meyerhof (1947, 1953), Chamecki (1956), Morris (1966), Lee and Harrison (1970), Lee and Brown (1972), and even a few studies in the recent past such as Deshmukh and Karmarkar (1991), Noorzaei *et al*. (1995), Srinivasa Rao *et al*. (1995), Dasgupta *et al*. (1998) and Mandal *et al*. (1999). The common practice of obtaining foundation loads from the structural analysis without allowance for foundation settlement may, therefore, result in extra cost that might have been avoided had the effect of soil-structure interaction been taken into account in determining the settlements. This requires that the

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engineers not only understand the properties of the ground but they also need to know how the building responds to deformation and what the consequences of such deformation will be to the function of the building. In this regard, many analytical works have been reported on the building frames founded on pile groups by Buragohain *et al*. (1977), Ingle and Chore (2007), Chore and Ingle (2008a, b) and Chore *et al*. (2009, 2010). But no significant light was thrown in the direction of experimental investigation of the effect of soil interaction on building frames founded on pile groups.

The aim of this paper is to present an experimental investigation as well as numerical analysis through the Linear Finite Element Analysis (FEA) and nonlinear FEA of a model plane frame supported by pile groups embedded in cohesionless soil (sand) under the static loads (central concentrated load, uniformly distributed load (UDL) and eccentric concentrated load). The need for consideration of soil interaction in the analysis of building frames is emphasized by the experimental investigation by comparing the behavior of the frame obtained from the experimental and numerical analysis with that by the conventional method of analysis. An attempt is made to quantify the soil interaction effect on the response of the building frame in terms of displacements, rotations, shears and bending moments through the experimental investigation.

2. Analysis programme using ANSYS

The analysis of the model plane frame is carried out using ANSYS for the following cases:

i) Frame with fixed bases to evaluate the shear force and bending moment in the column, which is the usual practice done known as the conventional method;
ii) Linear and nonlinear analyses to evaluate the lateral displacements, vertical displacements and rotations, shear forces and bending moments on the frame; and
iii) Frame with bases released by imposing the lateral displacements, vertical displacements and rotations measured from the experiments for the corresponding loading on the frame to get the back the shear forces and bending moments generated in the columns.

2.1 Validation by comparison with other numerical studies

The results of linear analysis of a typical column supported by a pile group using ANSYS were compared with results those by Won *et al*. (2006). A 2×2 pile group structure consisting of a pier, a pile cap, and four identical vertical piles, which are spaced by 3 m (i.e. 6D, where D is the pile diameter), is used for the linear analysis. The four piles have an embedded length of 10 m, a diameter of 0.5 m, and a flexural rigidity (EI) of 147,264 KN m^2. The thickness of the pile cap is 0.75 m, and the pile head condition is fixed. The pier is 10 m in length and 1 m in diameter, and has a flexural rigidity of 1,963,600 KN m^2. The soil condition at the site is modeled as linear springs in the lateral and axial directions along with tip springs. The pile group was subjected only to a lateral load of 1000 KN at the pier top. Table 1 describes that the results are identical to those obtained from the YS Group method reported by Won *et al*. (2006).

Table 1 Comparison of YS group method and ANSYS on displacement and forces in the pile group

	Check point	YS group method	ANSYS
Displ.	Lateral displ. at pier top (mm)	510.9	557.5
	Axial displ. at pier top (mm)	−8.5	−10.4
	Lateral displ. of 1, 3 pile head (mm)	52.9	62.6
	Axial displ. of 1, 3 pile head (mm)	−51.4	−46.7
	Rotation angle of 1, 3 pile head	−0.028	−0.031
Forces	Lateral force at 1, 3 pile head (kN)	250	250
	Axial force at 1, 3 pile head (kN)	−1008	−1014.6
	Moment at 1, 3 pile head (kN m)	−993	−978.03

2.2 Linear finite element analysis

A single bay single storeyed model plane frame founded on 2×2 pile groups embedded in cohesionless soil (which is used for the experimental program) is considered for the linear Finite Element Analysis (FEA). The columns, beams and piles are modeled using the 3D elastic two-nodded BEAM elements. The pile cap is modeled using the four-nodded elastic SHELL elements. The resistance of the soil in the lateral and vertical directions is modeled using the spring damper (COMBIN) elements. The frame so modeled is shown in Fig. 1. Linear constitutive soil model given by Eq. (1) is used for the present problem, which is the relation for the initial linear part of the *p-y* curves given by Reese *et al*. (1974).

$$p = (\eta_h Z)\, y \tag{1}$$

where η_h = coefficient of the lateral subgrade reaction (IS: 2911-1979), and Z is the depth.

2.3 Nonlinear finite element analysis

The nonlinear analyses were performed for the single bay single storeyed model plane frame founded on 2×2 pile groups in a sandy soil (Fig. 1). The modeling is same as that of the linear FEA except for the soil. The soil around the individual piles was modeled with nonlinear load transfer curves using the COMBIN39 elements. The nonlinear constitutive soil models given by Eqs. (2) - (4) are employed for the present problem.

The lateral load transfer curves given by Eq. (2) were used as the API model

$$p = \bar{A}_s P_s \tanh\left(\frac{kZ}{\bar{A}_s P_u} y\right) \tag{2}$$

where $\bar{A}_s$ = adjustment coefficient for the static *p-y* curves; P_s = governing ultimate soil resistance; k = initial subgrade reaction constant; Z = depth; and P_u = ultimate soil resistance.

The axial load transfer curves suggested by McVay *et al*. (1989) are used in this study. Also used are the vertical τ-Z springs along the side of the pile as described below

$$Z = \frac{r_0 \tau_0}{G_i}\left[\ln\frac{(r_m - \beta)}{(r_0 - \beta)} + \frac{\beta(r_m - r_0)}{(r_m - \beta) - (r_0 - \beta)}\right] \tag{3}$$

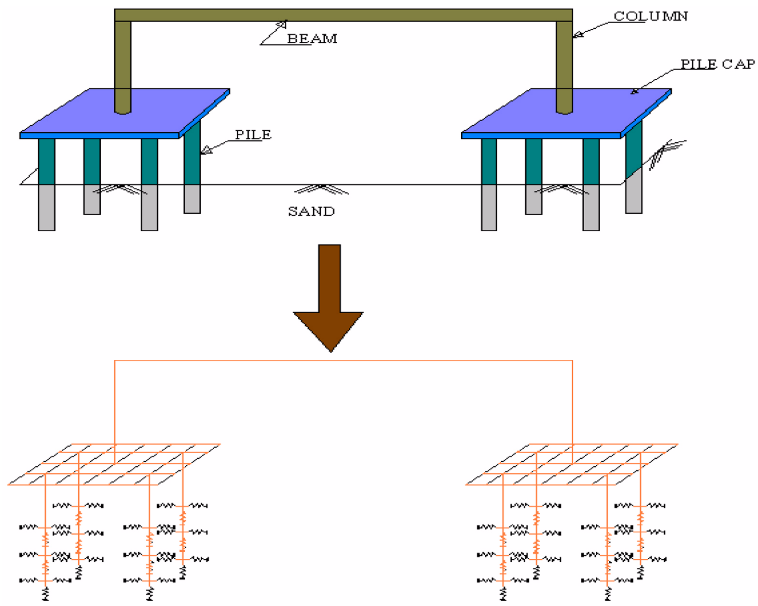


Fig. 1 modeling of the frame along with the pile groups

where $\beta = r_0\tau_0/\tau_f$; r_0 = radius of the pile; τ_0 = shear stress transferred to the soil for a given Z displacement; r_m = radius out from the pile where shear stress is negligible; G_i = initial shear modulus; τ_f = ultimate shear stress at the point of interest on the pile. As for the nonlinear tip spring (Q-Z), the following relation is used

$$Z = \frac{Q_b(1-\nu)}{4r_0 G_i\left(1-\frac{Q_b}{Q_f}\right)} \tag{4}$$

where Q_f = ultimate tip resistance; G_i = initial shear modulus; ν = Poisson's ratio of the soil; r_0 = radius of the pile; and Q_b = mobilized tip resistance for the given displacement Z.

The following soil properties are used for sand to represent its resistance in both the lateral and axial directions: angle of internal friction ϕ (evaluated from the laboratory experiments), coefficient of lateral subgrade reaction η_h, Poisson's ratio ν (a typical value of 0.3 is used), ultimate skin friction τ_f (evaluated from Tomlinson's (1971) equation), ultimate tip resistance Q_f, and shear modulus G_i (Kulhawy and Mayne 1990).

For the analysis reported herein, the following properties were employed for the loose sand: angle of internal friction ϕ of 30^0, coefficient of lateral subgrade reaction η_h of 2.551 MN/m^3, and shear modulus G_i of 9.615 MN/m^2.

The frame is loaded with a central concentrated load, UDL and eccentric concentrated load at a nominal eccentricity of 10% of length of the beam (with eccentricity measured from the center of the beam) in increments as applied in the experimental program and the response in terms of deformations, rotations, shear forces and bending moments is obtained for each load increment.

Table 2 Scaling factors used in the study

Variable	Length	Density	Stiffness	Stress	Strain	Force
Scaling factors	1/10	1	1/10	1/10	1	$1/10^3$

3. Experimental program

3.1 Frame and pile groups

Using the scaling law proposed by Wood *et al*. (2002) and reproduced in Eq. (5), the material and dimensions of the model were selected

$$\frac{E_m I_m}{E_p I_p} = \frac{1}{n^5} \tag{5}$$

where E_m is modulus of elasticity of model, E_p is modulus of elasticity of prototype, I_m is moment of inertia of model, I_p is moment of inertia of prototype, and $1/n$ is scale factor for length. An aluminum tube with an outer diameter of 16 mm and inner diameter of 12 mm was selected as the model pile with a length scaling factor of 1/10. This is used to simulate the prototype pile of 350 mm diameter solid section made of reinforced concrete with a compressive strength of 30 MPa. Columns of height 3.2 m and beam of span 5 m of the plane frame were scaled in the same manner. Aluminum plates of 13 mm thickness were used as the pile caps. In the pile group setup, pile spacing of eight diameter (8D) was adopted and the length of the piles was so selected as to maintain a length to diameter (L/D) ratio of 20 (Chandrasekaran and Boominadhan 2010). The sufficient freestanding length was maintained from the bottom of the pile cap to the top of the soil bed. Beam column junctions were made by welding for the fixed condition. Screwing of the piles and columns in the threads provided in the pile cap leads to partial fixity condition. The scaling factors used in the study are presented in Table 2.

3.2 Experimental setup and instrumentation

The schematic diagram of the test setup is shown in Fig. 2 and the photo in Fig. 3. Tests were conducted on the model pile groups with the frame embedded in sand bed in a concrete testing chamber, which was well instrumented with the dial gauges of sensitivity 0.002 to study the lateral, vertical displacements and rotations at the base of the column as shown in Fig. 4. Loads on the frame were applied through the hooks provided to the beam at required locations according to the type of loads on the beam. The model frame was placed at the centre of the testing chamber using the templates. The sand is then poured in the testing chamber gently through the pores of a steel tray in layers to attain the loose state and uniformity for the sand bed. The installation procedure simulates the bored pile condition.

3.3 Test procedure

Static vertical loads were applied on the model frame by placing weights on the hangers. The loads were applied in increments and were maintained for a minimum period to allow the deflection to stabilize. During the application of static loads, the lateral, vertical displacements at the base of

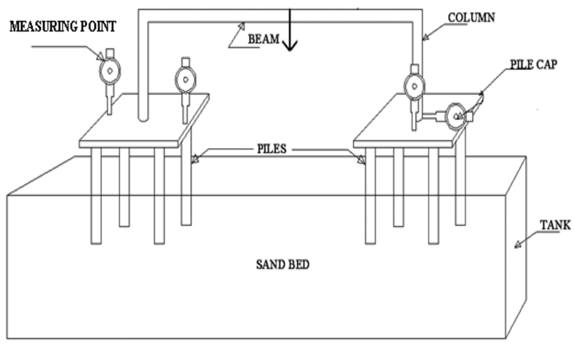


Fig. 2 Schematic diagram of the test setup

Fig. 3 Photograph of model frame and testing chamber

Fig. 4 Photograph of instrumented and loaded model frame

the column and the rotation of the pile cap were measured using the instrumentation setup as described earlier.

3.4 Testing phases

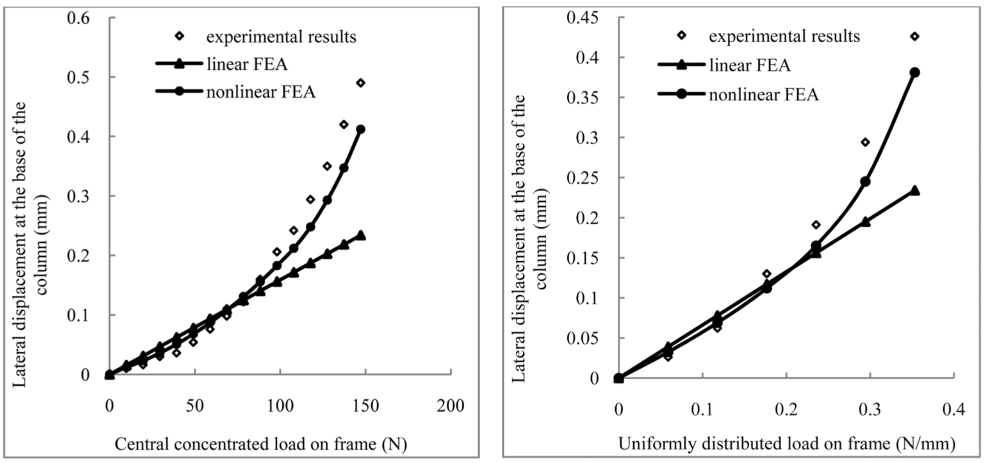
Static vertical load tests were conducted on the model frame with 2×2 pile groups embedded in the sand bed shown in Fig. 3. Tests were conducted in the following sequence:

1. Central concentrated load is applied in increments (1, 2, 3 kg etc.) at the centre of the beam.
2. The beam is loaded at third points with equal loads in increments (3, 6, 9 kg etc.) to simulate the uniformly distributed load (UDL) condition.
3. Eccentric concentrated load is applied in increments (1, 2, 3 kg etc.) at an eccentricity of 0.1 L, where L is span of the beam.

4. Results and discussion

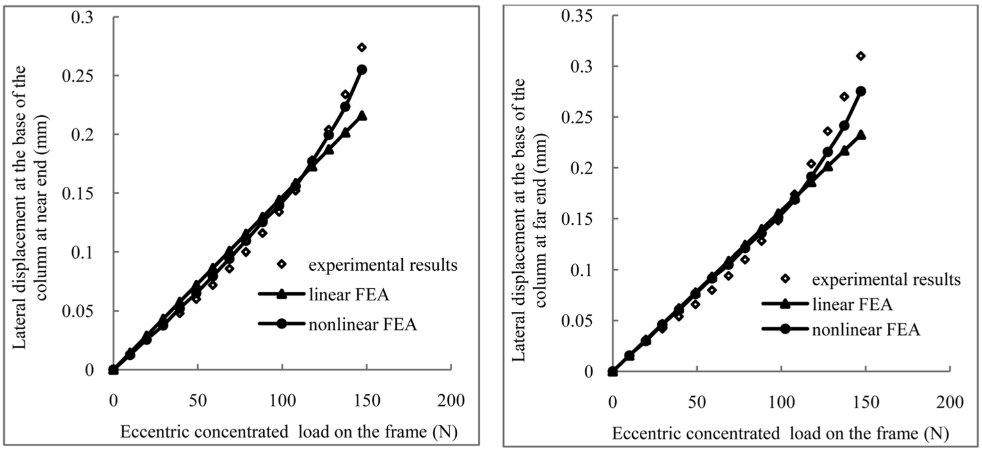
4.1 Lateral displacement, settlement and rotation at the base of the column from the experimental results, linear FEA and nonlinear FEA

Figs. 5(a) and 5(b) represent the variation of lateral displacement with the static load applied on the frame as central concentrated load and uniform distributed load. Figs. 6(a) and 6(b) are the plots showing the variation of lateral displacement with the eccentric concentrated load applied at the near end and far end, respectively. From the plots shown herein, it is observed that, for relatively lower loads on the frame, the lateral displacements predicted by all the three methods are nearly the same. For higher loads on the frame, the lateral displacements predicted by the experiment and the nonlinear FEA, deviate significantly from the one by the linear FEA. The lateral displacement from



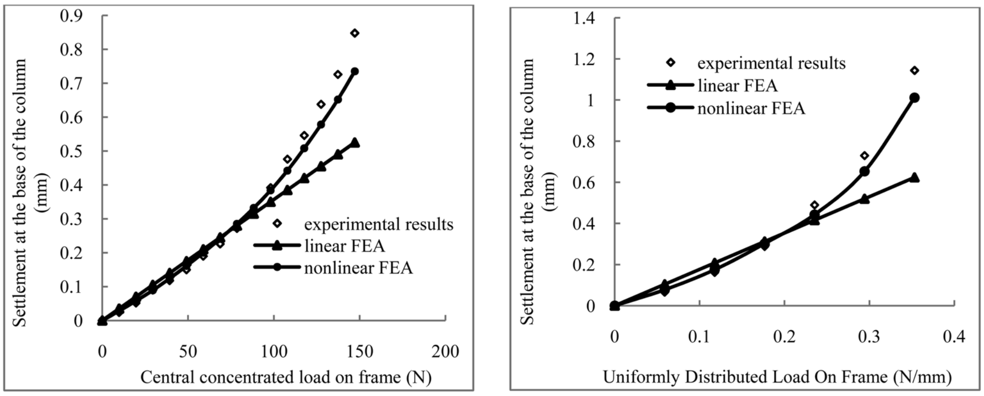
(a) (b)

Fig. 5 Lateral displacement at the base of the column : (a) central concentrated load and (b) UDL



(a) (b)

Fig. 6 Lateral displacement at the base of the column at: (a) near end and (b) far end (eccentric concentrated load)



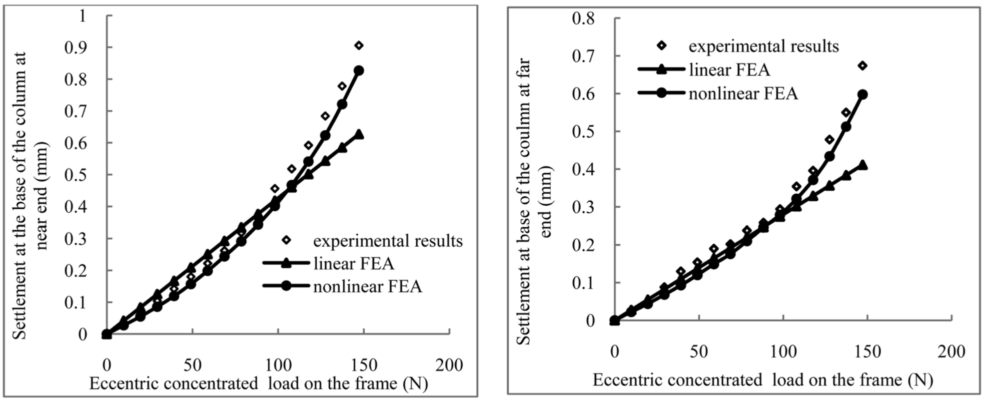
(a) (b)

Fig. 7 Settlement at the base of the column: (a) central concentrated load and (b) UDL

the experiment is 30-50% more than that by the linear FEA in the vicinity of the failure load on the frame. The displacement from the experiment shows a variation of 7-15% with respect to that from the nonlinear FEA. Hence the displacement from the experiment is in good agreement with that by the nonlinear FEA.

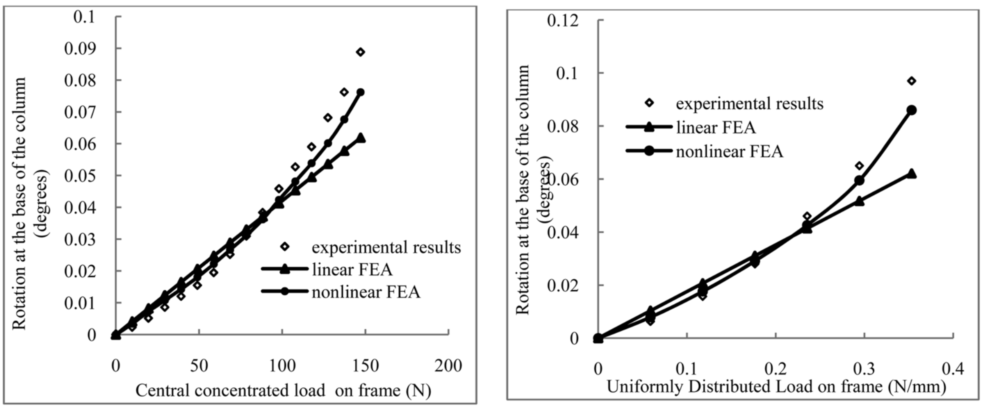
The variation of settlement at the base of the column with respect to the central concentrated load and UDL on the frame is presented in Figs. 7(a) and 7(b), respectively, and the variation of settlement at the near end and far end of the column base for the frame under the eccentric concentrated load is presented in Figs. 8(a) and 8(b), respectively.

The variation of rotation at the base of the column for the central concentrated load and UDL applied on the frame is presented in Figs. 9(a) and 9(b), respectively. Meanwhile, the variation of rotation at the column base of the near and far end, respectively, of the frame under the eccentric concentrated load is presented in Figs. 10(a) and 10(b).



(a) (b)

Fig. 8 Settlement at the base of the column at: (a) near end and (b) far end (eccentric concentrated load)

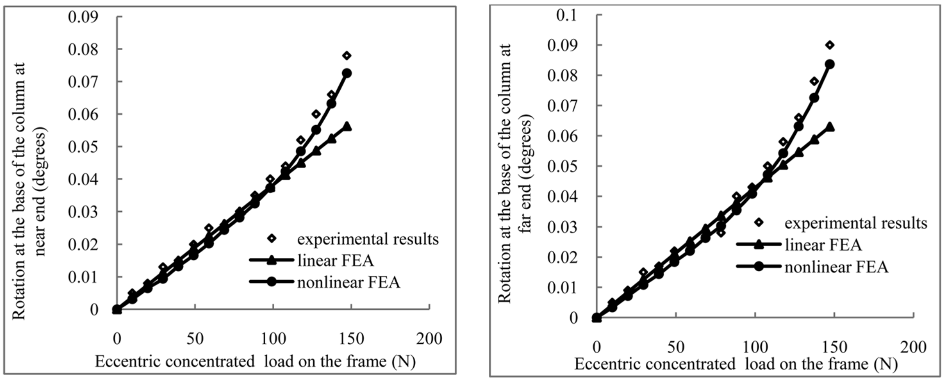


(a) (b)

Fig. 9 Rotation at the base of the column: (a) central concentrated load and (b) UDL

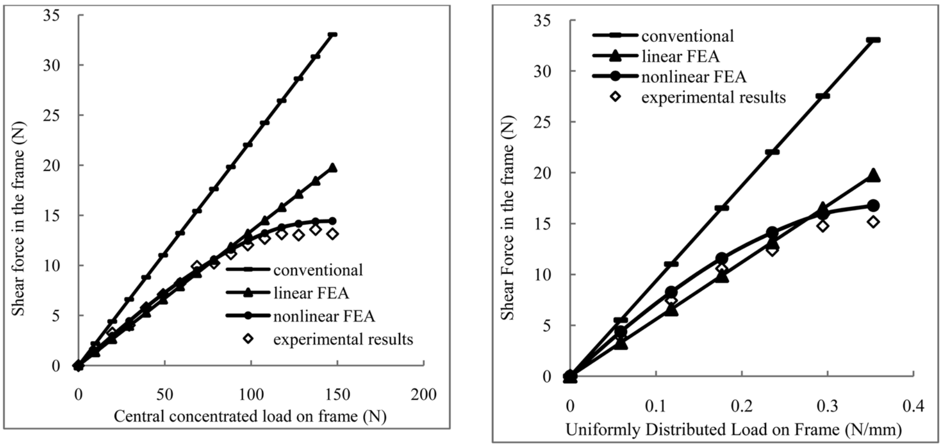
As can be seen, the plots mentioned herein also exhibit the same behavior as that of the variation of lateral displacement. Hence, the same observation holds good for both the settlement and rotation.

In all the aforementioned results, it is observed that the deflections obtained by the linear analysis are larger than those by the nonlinear analysis when the load is small. This unusual behaviour is due to the fact that the linear analysis is performed by modeling the soil as a series of unconnected linear springs, with the coefficient of the lateral subgrade reaction taken as 2.551 MN/m^3 as suggested by IS: 2911-1979 for the type of soil used in the present problem to calculate the spring stiffnesses, whereas in the nonlinear analysis, the soil is modeled using the nonlinear springs for the lateral and axial directions and the nonlinear load transfer curves are evaluated from the expressions in Eqs. (2)-(4) and the basic soil parameters involved in the expressions are found from the laboratory experiments for the soil used in the present study. It is observed that the stiffness suggested by IS: 2911-1979 for the soil used in the present study for linear analysis is less than that

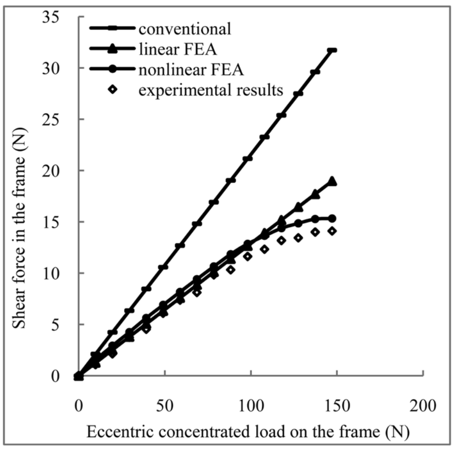


(a) (b)

Fig. 10 Rotation at the base of the column at: (a) near end and (b) far end (eccentric concentrated load)



(a) (b)



(c)

Fig. 11 Shear force: (a) central concentrated load, (b) UDL and (c) eccentric concentrated load of the initial linear portion of the load transfer curves used in the nonlinear analysis (i.e. the API model and expressions suggested by McVay *et al*. (1989)). Also evident from the literature is that the stiffness of soil suggested by Reese *et al*. (1974) for the type of soil used in the present study is 2.67 times that of the stiffness suggested by the IS: 2911-1979. The above can explain why the deflections computed from the linear analysis in the present study are higher than those by the nonlinear analysis.

4.2 Shear force in the frame by conventional method, experiments, linear FEA and non-linear FEA

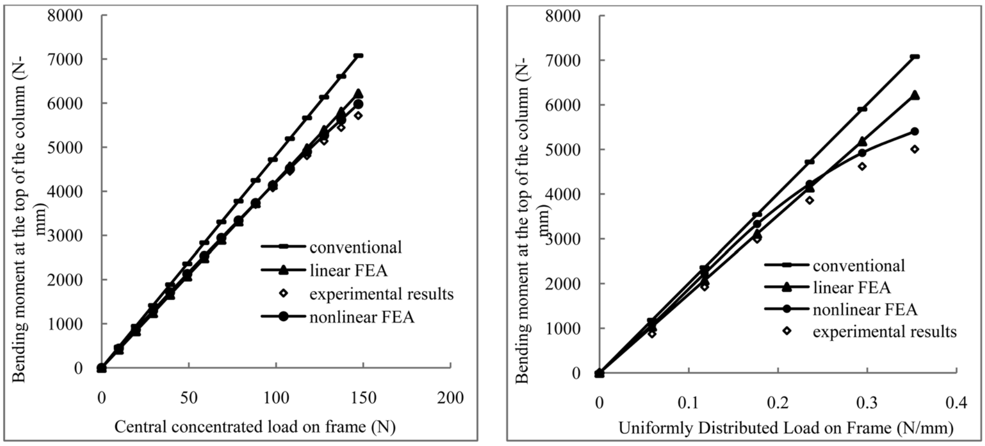
The shear force in the frame under the central concentrated load, UDL, and eccentric concentrated load have been plotted in Figs. 11(a)-11(c), respectively. From these plots, it can be observed that the shear force predicted by the conventional method is always on the higher side. For relatively lower loads on the frame, the shear force predicted by the nonlinear FEA and experiment follow closely the shear force by the linear FEA. The shear force predicted by the conventional method is 40.2% higher than that by the linear FEA for all levels of loading. The shear force obtained from the experiment deviates by 8-10% of that given by the nonlinear FEA, which indicates that the nonlinear soil model is in good agreement with the experimental results. The shear force predicted by the conventional method is 54-60% more than that of the experiment for higher loads acting on the frame.

The difference in the shear force predicted by the linear and nonlinear FEA is 15-25% only. Hence, by allowing some marginal error in calculation of the shear force, we can use the linear FEA to evaluate the shear force as a substitute of the rigorous nonlinear FEA in some preliminary designs. In general, we will be on the conservative side as the maximum shear force given by the linear FEA is higher than that by the nonlinear FEA.

4.3 Bending moment at top of the column by conventional method, experiments, linear FEA and nonlinear FEA

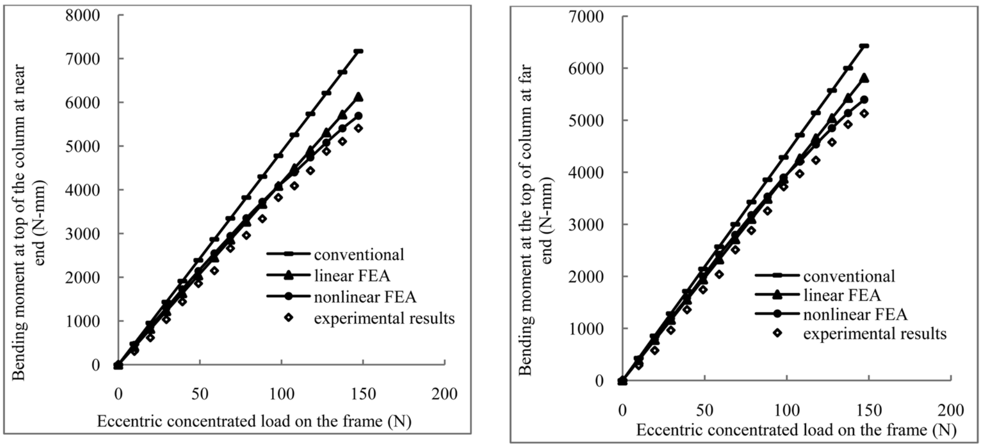
The bending moment at the top of the column of the frame under the central concentrated load and UDL is plotted in Figs. 12(a) and 12(b), respectively, and the one of the near end and far end, respectively, of the frame under the eccentric load is plotted in Figs. 13(a) and 13(b).

From the above figures, it is observed that the bending moment predicted by the conventional method is higher than that by the other methods of analysis, indicating that the conventional method of analysis for obtaining the design moment is uneconomical. Compared with the experimental result, the bending moment predicted by the conventional method is 20-30% higher. This indicates the need for consideration of soil interaction in evaluating the design parameters in a building frame. The values of bending moment predicted by the nonlinear FEA and experiments differ by 5-7% only, which indicates that the nonlinear soil model is well suited for representing the nonlinear behavior of soil. Moreover, the bending moment predicted by the conventional method is 10-15% higher than that given by the linear FEA. For the above reason, the designers may favor the use of linear analysis concerning the economy in design. The linear FEA gives the bending moment 7-14% higher than that by the nonlinear FEA, indicating that the bending moments evaluated by the linear FEA will be on conservative side. The point to be noted with respect to the bending moments at the top of the column of the frame predicted by different methods is that though the percentages of



(a) (b)

Fig. 12 Bending moment at top of the column: (a) central concentrated load and (b) UDL



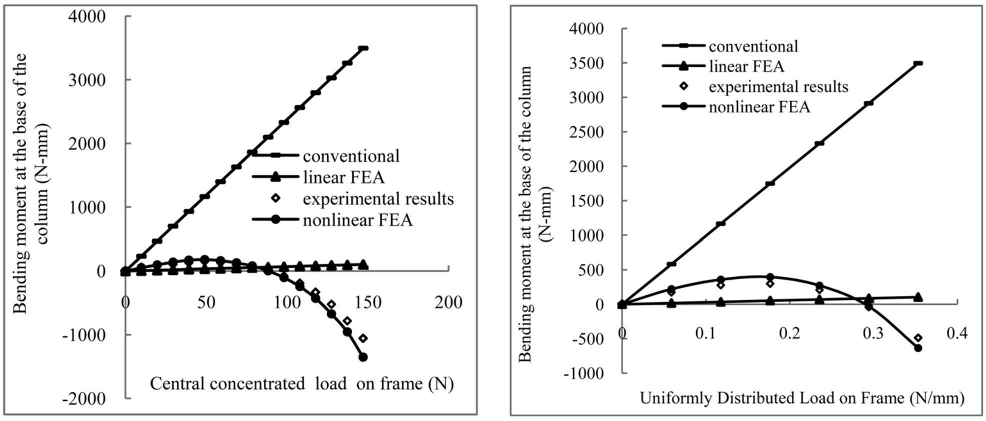
(a) (b)

Fig. 13 Bending moment at top of the column at: (a) near end and (b) far end (eccentric concentrated load)

variation may not be great, the differences are significant because the magnitudes of bending moment are of multiples of thousands.

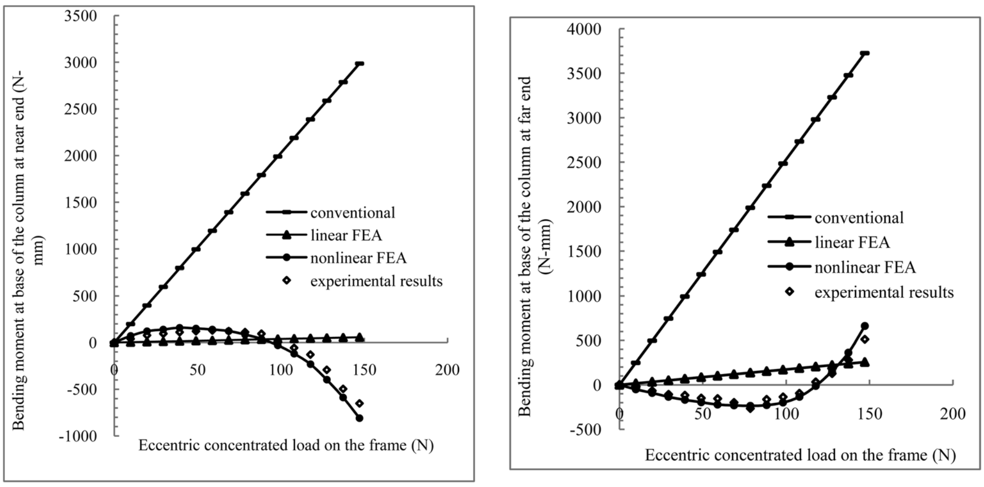
4.4 Bending moment at the base of the column by the conventional method, experiments, linear FEA, and nonlinear FEA

The variation of bending moment at the base of the column of the frame under the central concentrated load and UDL have been plotted in Fig. 14(a) and 14(b), respectively. These figures show that, for the conventional method and linear FEA, as the load increases the bending moment increases in the linear manner, as the load-displacement curves are linear. The conventional method gives a bending moment 97% higher value than that by the linear FEA irrespective of the amount



(a) (b)

Fig. 14 Bending moment at the base of the column: (a) central concentrated load and (b) UDL



(a) (b)

Fig. 15 Bending moment at the base of the column of: (a) near end and (b) far end (eccentric concentrated load)

of load on the frame. The bending moments given by the experiments agree well with those by the nonlinear FEA with a variation of 15-25%. Moreover, the bending moment at the base of the column changes its sign, when the load reaches some value. This is due to the fact that for relatively smaller loads on the frame, the column is rigidly connected to the pile cap and the soil is in its linear range hence it behaves like a frame with fixed base. As the load on frame increases, the connection between base of the column and pile cap becomes partially rigid and the behaviour of the soil will be in the nonlinear range, increase in the rotation of the pile cap will be so high hence the nature of bending of column at the base will change its sign. The conventional method gives a bending moment at the column base that is 70-100% higher than that by the experiment.

Figs. 15(a) and 15(b) show the variation of bending moment at the base of the column of the near end and far end, respectively, of the frame under the eccentric concentrated load. Clearly, based on the conventional method and linear FEA, the bending moment at the far end of the column base of the frame is higher than that of the near end, whereas the nonlinear FEA and experiment show that the near-end bending moment at the base is dominant for higher loads on the frame. The conventional method gives a bending moment 80-100% higher than that of the experimental result. The sign change of the bending moment is observed to occur at an earlier stage of loading at near end than at the far end.

5. Conclusions

Based on the results of the present experimental and numerical investigations on the model building frame resting on pile groups embedded in cohesionless soil, the following conclusions are drawn:

- As the load on the frame increases, the behavior of the frame in terms of displacement and rotation at the base of the column predicted by the linear FEA, nonlinear FEA and experiment appears to be linear for relatively smaller loads. For higher load range, the experimental results show a non-linear variation and considerable deviation from the linear FEA results.
- The displacements and rotations from the experimental results and the nonlinear FEA show a maximum difference of about 15%, indicating that the nonlinear curves used to characterize the soil behavior are generally good for representing the load-displacement response of the soil. For the frame under the eccentric concentrated load, larger lateral displacements, rotations and lesser settlements are generated at the column base of the far end from the load. This behavior of differential settlements and rotations may alter the bending moment at the column base.
- The deflections estimated by the linear analysis using the soil stiffness suggested by IS: 2911-1979 are higher than those calculated by the nonlinear analysis using the actual stiffness of the soil (initial linear portion of the load transfer curves).
- The conventional method of analysis gives a shear force of about 40% higher than that by the linear FEA irrespective of the amount of loading on the frame, and about 40-60 % higher than that from the experimental result. As the load acting on the frame increases, the percentage of variation of shear force predicted by the conventional method with respect to that of the experimental result also increases.
- The conventional method gives a bending moment at the top of the column that is 20-30% higher than that by the experimental result, but such a difference is still significant as the bending moment values are in the multiples of thousands. The bending moment at the near end of the frame is higher than that of the far end for the eccentric concentrated load case.
- The conventional method gives a bending moment at the base of the column that is 75 - 100% higher than that by the experimental result. For a nominal eccentricity given for the concentrated load (10% length of the beam), the conventional method gives a higher value of bending moment at the column base of the far end from the load than the one of the near end, but the experimental result gives a lesser bending moment at the far end from the load than the one at the near end. This means that the bending at the column base of the near end is to be considered as the governing design parameter.

The response of the frame in terms of the design parameters (i.e. shear and bending moment) from the conventional method of analysis is always on the higher side irrespective of the level of loading, which reveals the need for consideration of the interaction between the building frame, pile foundation, and soil.

References

American petroleum Institute (1987), "Recommended practice for planning, designing, and constructing fixed offshore platforms", API Recommended Practice, 2A (RP-2A), 17th edn.

Buragohain, D.N., Raghavan, N. and Chandrasekaran, V.S. (1977), "Interaction of frames with pile foundation", *Proceedings of International Symposium on Soil-Structure Interaction*, Roorkee, India, January.

Chamecki, C. (1956), "Structural rigidity in calculating settlements", *J. Soil Mech. Found. Div. - ASCE*, **82**(1), 1-19.

Chandrasekaran, S.S. and Boominadhan, A. (2010), "Group interaction effects on laterally loaded piles in clay", *J. Geotech. Geoenviron. Eng. - ASCE*, **136**, 573-582.

Chore, H.S. and Ingle, R.K. (2008a), "Interaction analysis of building frame supported on pile group", *Indian Geotech. J.*, **38**(4), 483-501.

Chore, H.S. and Ingle, R.K. (2008b), "Interactive analysis of building frame supported on pile group using a simplified F.E. model", *J. Struct. Eng. SERC*, **34**(6), 460-464.

Chore, H.S., Ingle, R.K. and Sawant, V.A. (2009), "Building frame- pile foundation- soil interactive analysis", *Interact. Multiscale Mech.*, **2**(4), 397-411.

Chore, H.S., Ingle, R.K. and Sawant, V.A. (2010), "Building frame - pile foundation - soil interaction analysis: a parametric study", *Interact. Multiscale Mech.*, **3**(1), 55-79.

Dasgupta, S., Dutta, S.C. and Bhattacharya, G. (1998), "Effect of soil- structure interaction on building frames on isolated footings", *J. Struct. Eng. SERC*, **26**(2), 129-134.

Deshmukh, A.M. and Karmarkar, S.R. (1991), "Interaction of plane frames with soil", *Proceedings of Indian Geotechnical Conference (IGC-1991)*, Surat, India.

Ingle, R.K. and Chore, H.S. (2007), "Soil- structure interaction analysis of building frames- an overview", *J. Struct. Eng. SERC*, **34**(5), 201-209.

IS: 2911-1979 (1979), "Code of practice for design and construction of pile foundation", BIS, New Delhi, India.

Kulhawy, F.H. and Mayne, P.W. (1990), "Manual on estimating soil properties for foundation design", *EPRI Rep. EL-6800, Electric Power Research Institute*, Palo Alto, Calif., 5-1–5-25.

Lee, I.K. and Harrison, H.B. (1970), "Structures and foundation interaction theory", *J. Struct. Div. - ASCE*, **96**(2), 177-198.

Lee, I.K. and Brown, P.T. (1972), "Structures and foundation interaction analysis", *J. Struct. Div. - ASCE*, **11**, 2413-2431.

Mandal, A., Moitra, D. and Dutta, S.C. (1999), "Soil- structure interaction on building frame: a small scale model study", *Int. J. Struct. Roorkee*, **18**(2), 92-107.

McVay, M.C., Townsend, F.C., Bloomquist, D.G., O'Brien, M. and Caliendo, J.A. (1989). "Numerical analysis of vertically loaded pile groups", *Proc., Found. Engrg. Current Principles and Practices*, Vol. 1, ASCE, New York, 675-690.

Meyerhof, G. (1947), "The settlement analysis of building frames", *Struct. Eng.*, **25**, 369-409.

Meyerhof, G. (1953), "Some recent foundation research and its application to design", *Struct. Eng.*, **31**(6), 151-167.

Morris, D. (1966), "Interaction of continuous frames and soil media", *J. Struct. Div. - ASCE*, **5**, 13-43.

Noorzaei, J., Viladkar, M.N. and Godbole, P.N. (1995), "Elasto-plastic analysis for soil-structure interaction in framed structures", *Comput. Struct.*, **55**(5), 797-807.

Reese, L.C., Cox, W.R. and Koop, F.D. (1974), "Analysis of laterally loaded piles in sand", *Proceedings of 6th Annual Offshore Technology Conference*, Richardson, Texas, USA.

Srinivasa, Rao P., Rambabu, K.V. and Allam, M.M. (1995), "Representation of soil support in analysis of open plane frames", *Comput. Struct.*, **56**, 917-925.

Tomlinson, M.J. (1971), "Some effects of pile driving on skin friction", *Proceedings of international conference* *on Behavior of Piles*, Institution of Civil Engineers, London.

Won, J., Ahn, S.Y. and Jeong, S. (2006), "Nonlinear three-dimensional analysis of pile group supported columns considering pile cap flexibility", *Comput. Geotech.*, **33**, 355-370.

Wood, D.M., Crew, A. and Taylor, C. (2002), "Shaking table testing of geotechinical models", *Int. J. Phys. Model. Geotech*., **1**, 1-13.