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Study of soil interaction in a model building frame with plinth beam supported by pile group

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Abstract

This paper presents the results of static vertical load tests carried out on a model building frame with plinth beam 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 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 axial direction is characterized by nonlinear vertical springs along the length of the pile (τ-z curves) and at the tip of the pile (Q-z curves) and in the lateral direction by the p-y curves. The results reveal that the conventional method gives the shear force in the column by about 20%, the bending moment at the column top about 10%, and at the column base about 20% to 30%, 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: Nonlinear analysis, Soil structure interaction, Experimental analysis, Conventional method, Pile group, Building frame, Cohesionless soil, Plinth beam, Model

Introduction

Soil settlement is a function of the flexural rigidity of the superstructure. The influence caused by the settlement of the supporting ground on the response of framed structures was often ignored in a structural design. 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. (1994, 1995), 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 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), Chore et al. (2009, 2010) and the experimental work by Reddy and Rao (2011). But no significant light was thrown in the direction of the effect of soil interaction on building frames with plinth beam founded on pile groups.

The aim of this paper is to present an experimental investigation as well as numerical analysis through the nonlinear finite element analysis (FEA) of a model plane frame with plinth beam 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 frame with plinth beam is emphasized by the experimental investigation by comparing the behavior of the frame obtained from the experimental and numerical

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

	Check point	YS group method	ANSYS
Displacement	Lateral at pier top (mm)	510.9	557.5
	Axial at pier top (mm)	−8.5	−10.4
	Lateral of 1, 3 pile head (mm)	52.9	62.6
	Axial of 1, 3 pile head (mm)	−51.4	−46.7
	Rotation angle of 1, 3 pile head	−0.028	−0.031
Forces	Lateral at 1, 3 pile head (kN)	250	250
	Axial at 1, 3 pile head (kN)	−1,008	−1,014.6
	Moment at 1, 3 pile head (kNm)	−993	−978.03

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 experimental investigation.

Methods

Analysis program using ANSYS

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

1. 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;
2. Nonlinear analyses to evaluate the lateral displacements, vertical displacements and rotations, shear forces, and bending moments in the frame; and
3. 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

figured shear forces and bending moments generated in the columns.

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., 6*D*, 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 kNm^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 kNm^2. The soil condition at the site is modeled as linear springs in the lateral and axial directions along with the tip springs. The pile group was subjected only to a lateral load of 1,000 kN at the pier top. Table 1 describes that the results are identical to those obtained from the YS group method as reported by Won et al. (2006).

Nonlinear finite element analysis (nonlinear FEA)

The nonlinear analyses were performed for the single bay, single storeyed model plane frame with plinth beam founded on 2 × 2 pile groups in a sandy soil (Figure 1). The columns, beams, and piles are modeled using the 3D elastic two-noded beam elements. The pile cap is modeled using the four-noded elastic shell elements. The soil around the individual piles was modeled with nonlinear load transfer curves using the COMBIN39 elements.

The nonlinear constitutive soil models given by Equations 1, 2, and 3 are employed for the present problem. The lateral load transfer curves given by Equation 1
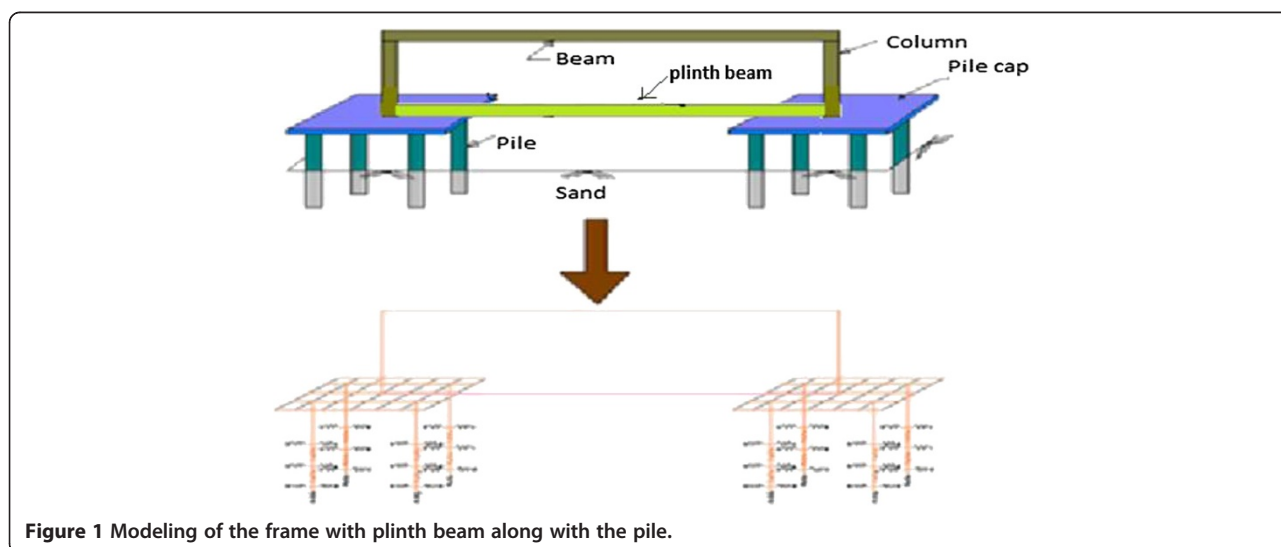
Figure 1 Modeling of the frame with plinth beam along with the pile.

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	$1/10^3$

were used as the API model (American Petroleum Institute 1987),

$$p = \bar{A}_s P_s \tanh\left(\frac{kZ}{\bar{A}_s P_u} y\right), \tag{1}$$

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 given by the Equation 2,

$$Z = \frac{r_0 \tau_0}{G_i}\left[\ln\frac{(r_m - \beta)}{(r_0 - \beta)} + \frac{\beta(r_m - r_0)}{(r_m - \beta)*(r_0 - \beta)}\right], \tag{2}$$

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; and τ_f = ultimate shear stress at the point of interest on the pile. As for the nonlinear tip spring (Q-Z), the following relation given by the Equation 3 is used:

$$Z = \frac{Q_b(1-\nu)}{4r_0 G_i\left(1 - \frac{Q_b}{Q_f}\right)}, \tag{3}$$

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), Poisson's ratio ν (a typical value of 0.3 is used), ultimate skin friction τ_f (evaluated from Tomlinson's equation (Tomlinson 1971)), 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°, shear modulus G_i of 9.615 MN/m², unit weight of soil of 17kN/m³ and relative density of 35%.

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

## Experimental program
Frame and pile groups
Using the scaling law proposed by Wood et al. (2002) and reproduced in Equation 1, the material and dimensions of the model were selected:

$$\frac{E_m I_m}{E_p I_p} = \frac{1}{n^5}, \tag{4}$$

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
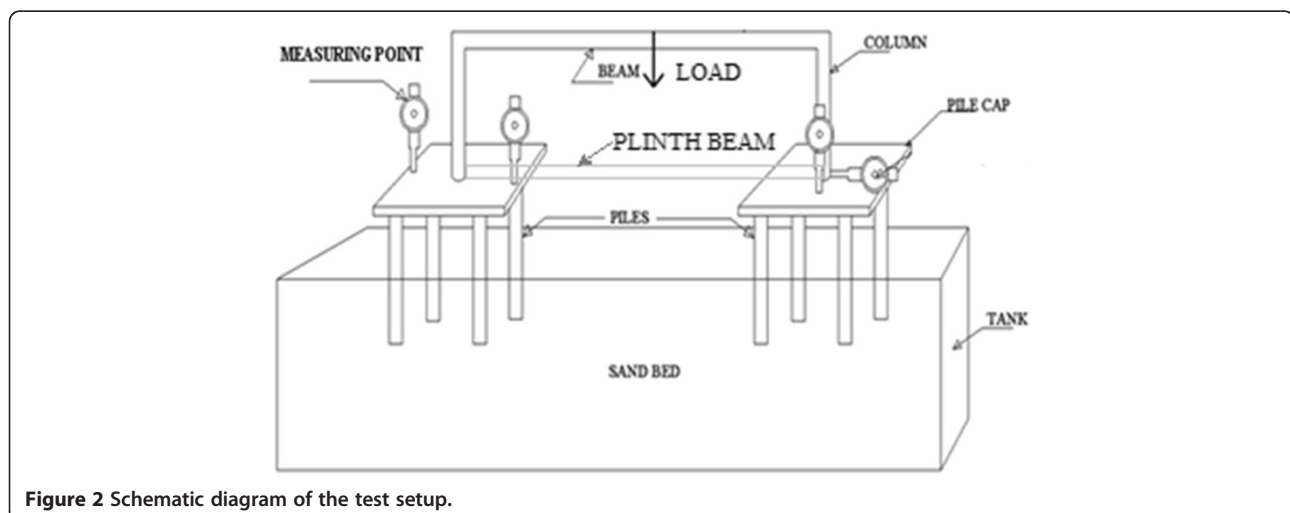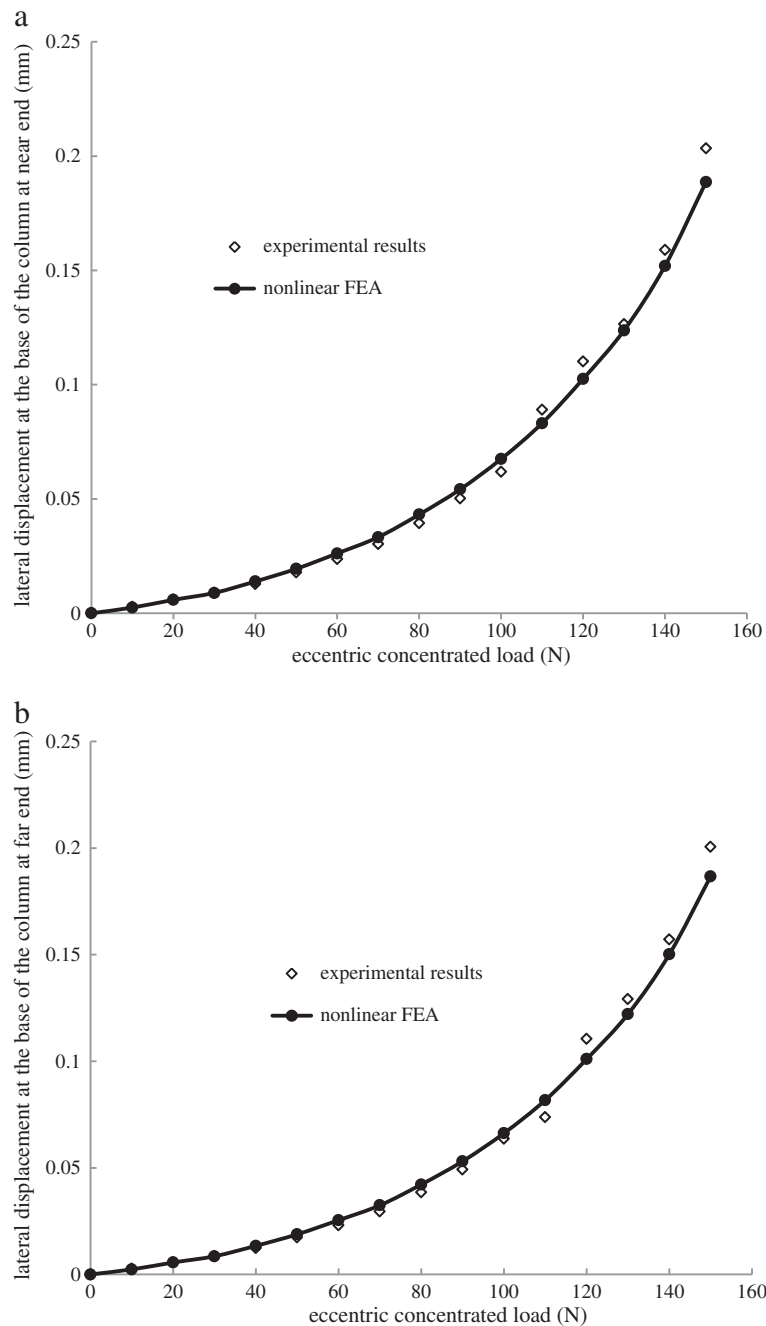

Figure 2 Schematic diagram of the test setup.
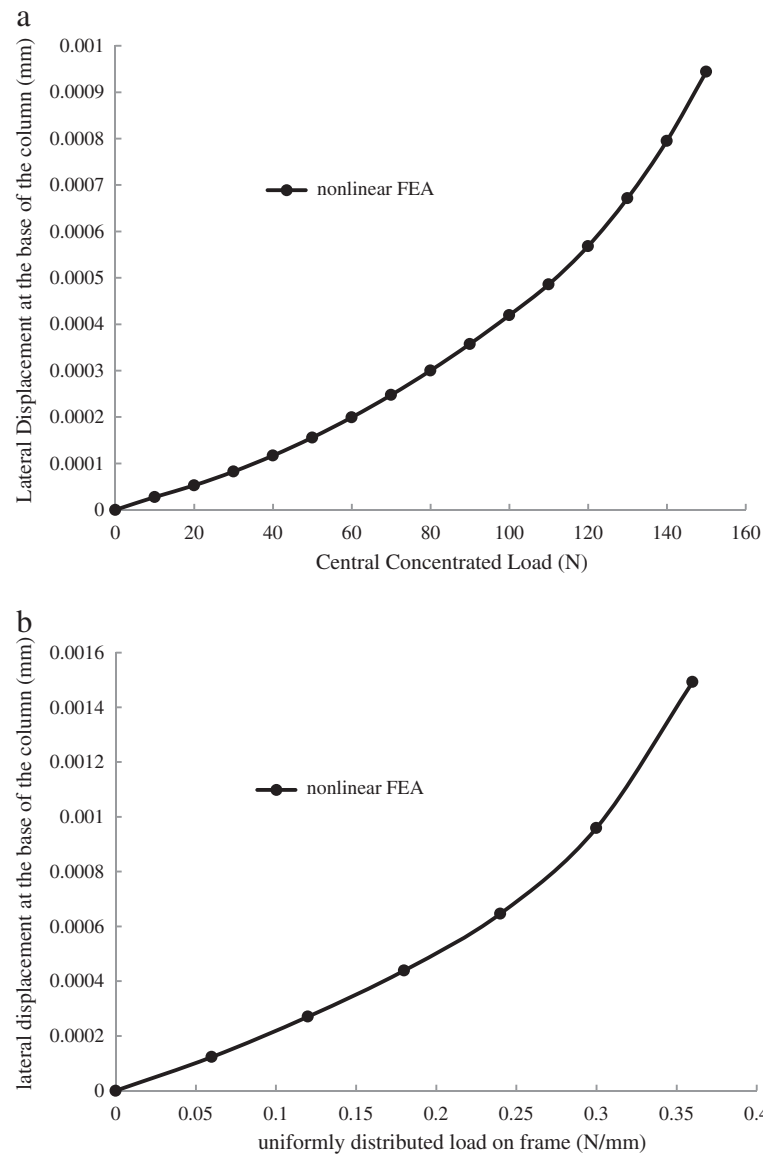
Figure 3 Variation of the lateral displacement with the static load. (**a**) Lateral displacement at the base of the column (central concentrated load). (**b**) Lateral displacement at the base of the column (uniformly distributed load).

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. Columns with height of 3.2 m, beam with span of 5 m, and plinth beam 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 (8*D*) 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 at a fixed condition. Screwing

of the piles and columns in the threads which are provided in the pile cap leads to partial fixity condition. The scaling factors used in the study are presented in Table 2.

Experimental setup and instrumentation

The schematic diagram of the test setup is shown in Figure 2. Tests were conducted on the model pile groups with the frame embedded in sand bed in a testing chamber, which was well instrumented with the dial gauges of sensitivity 0.002 to study the lateral and vertical displacements and rotations at the base of the column. Loads on the frame were applied through the hooks provided at the beam in required locations

Figure 4 Variation of the lateral displacement with static load applied on frame as eccentric concentrated load. (**a**) Lateral displacement at the base of the column at near end. (**b**) Lateral displacement at the base of the column at far end.

according to the type of loads on the beam. The model frame was placed at the center 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.

Test procedure

Static vertical loads were applied on the model frame with plinth beam 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 the column
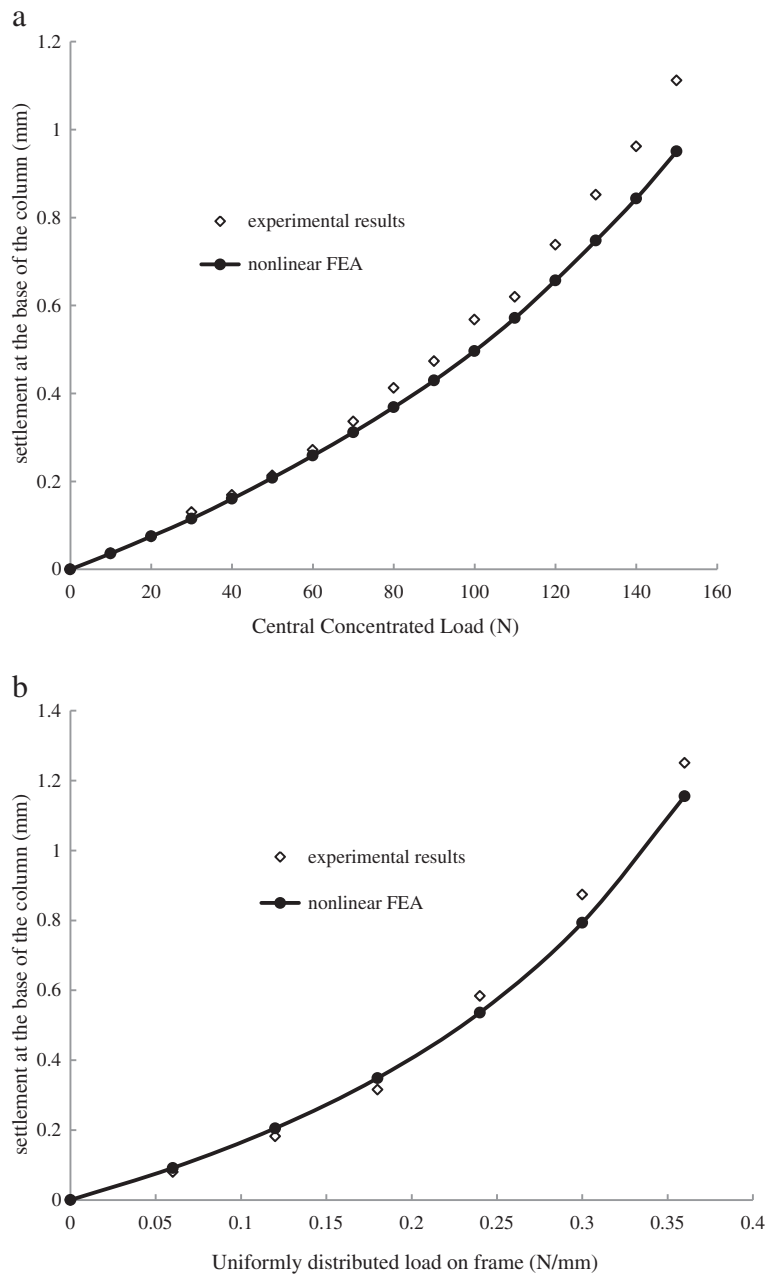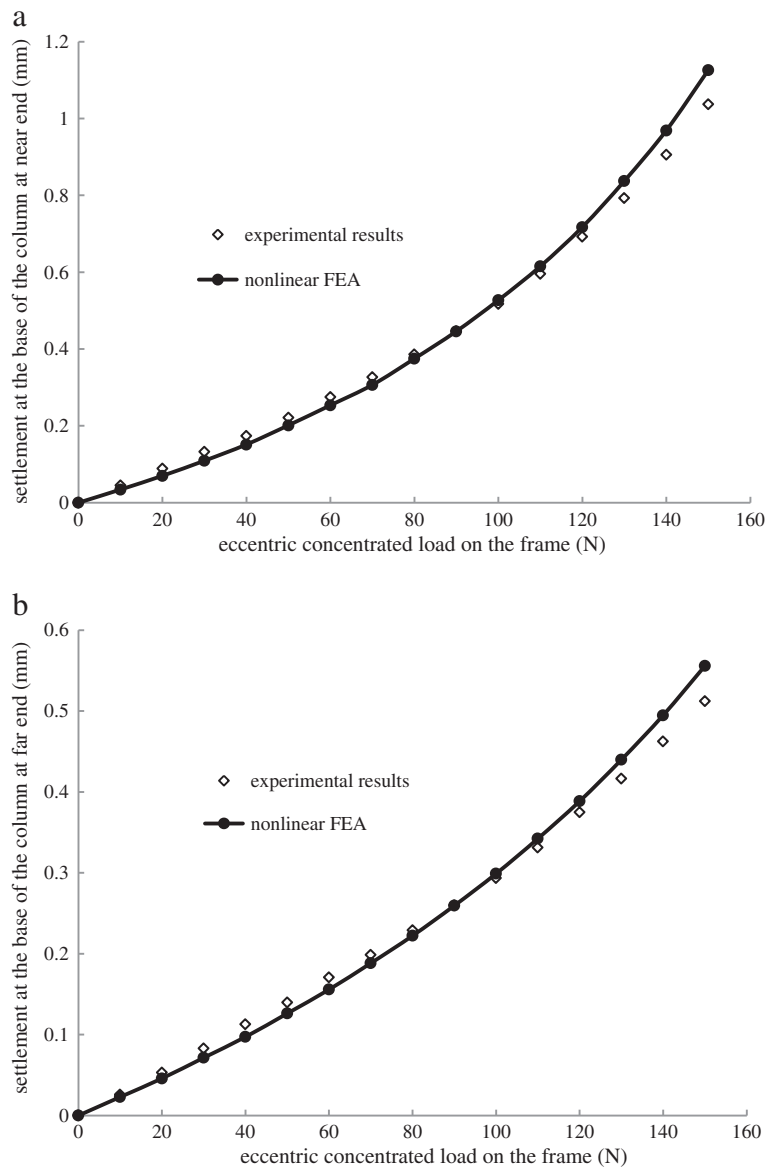
Figure 5 Variation of settlement at the base of the column. (**a**) Settlement at the base of the column (central concentrated load). (**b**) Settlement at the base of the column (UDL).

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

Testing phases

Static vertical load tests were conducted on the model frame with plinth beam supported on pile groups embedded in the sand bed as shown in Figure 2. Tests were conducted in the following sequence:

1. Central concentrated load is applied in increments (1, 2, 3 kg, etc.) at the center of the beam.
2. The beam is loaded at the third points with equal loads in increments (3, 6, 9 kg, etc.) to simulate the UDL condition.
3. Eccentric concentrated load is applied in increments (1, 2, 3 kg, etc.) at a nominal eccentricity of 10% span of the beam.

Figure 6 Variation of settlement at the near end and far end of the column base. (**a**) Settlement at the base of the column at near end (eccentric concentrated load). (**b**) Settlement at the base of the column at far end (eccentric concentrated load).

Results and discussion

Lateral displacement, settlement, and rotation at the base of the column

Figure 3a,b represents the variation of the lateral displacement with the static load applied on the frame with plinth beam as central concentrated load and uniformly distributed load. From the plots shown herein, it is observed that the lateral displacement at the base of the column of frame with plinth beam in both cases is negligibly small.

Figure 4a,b represents the variation of the lateral displacement with the static load applied on the frame as eccentric concentrated load. From the plots shown

herein, it is observed that the behavior of the frame with eccentric concentrated load is different from that of the frame with central concentrated load and uniformly distributed load. In the case of the frame with central concentrated load and uniformly distributed load, the base of the column at near end and far end moves outward when the load is applied on the frame; but in the case of the frame with eccentric concentrated load, the base of column at near and far ends moves in the same direction with nearly same amount of displacement (5% difference) and towards the eccentricity. The displacement from the experiment shows a variation of 3% to 14% with respect to that from the nonlinear FEA for
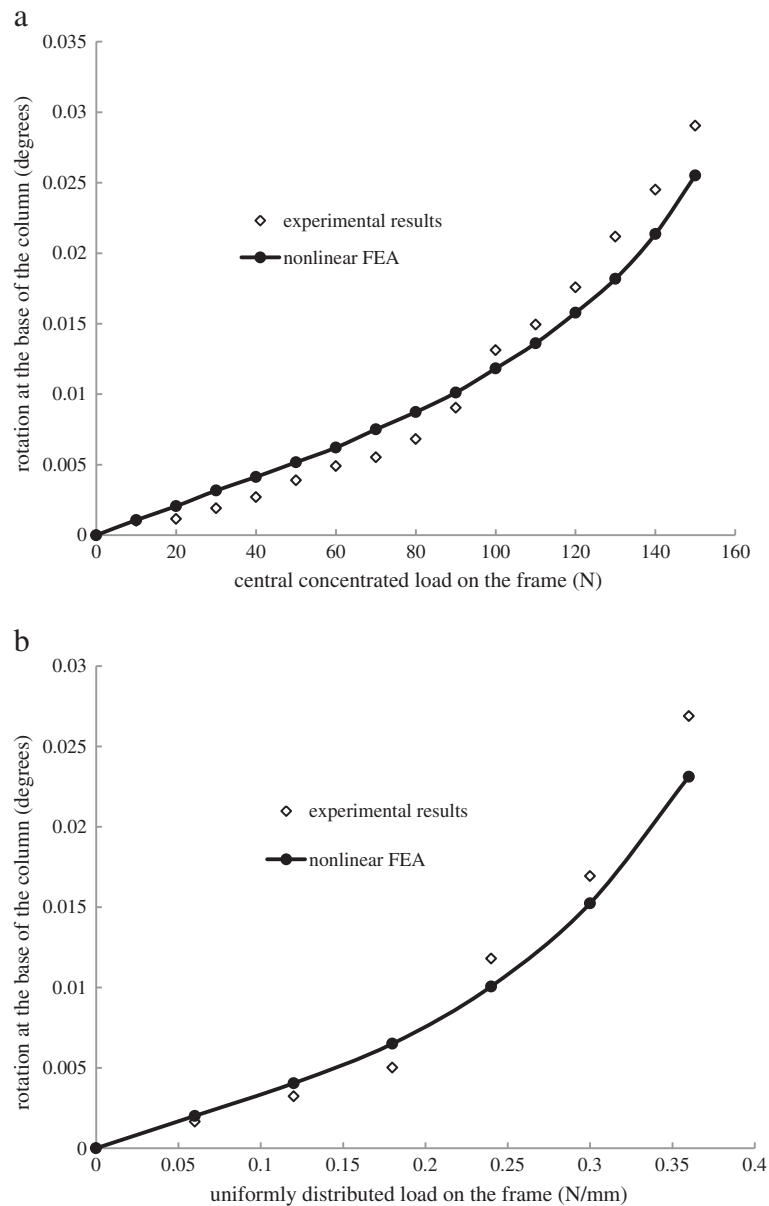
Figure 7 Variation of rotation at the base of the column. (**a**) Rotation at the base of the column (central concentrated load). (**b**) Rotation at the base of the column (UDL).
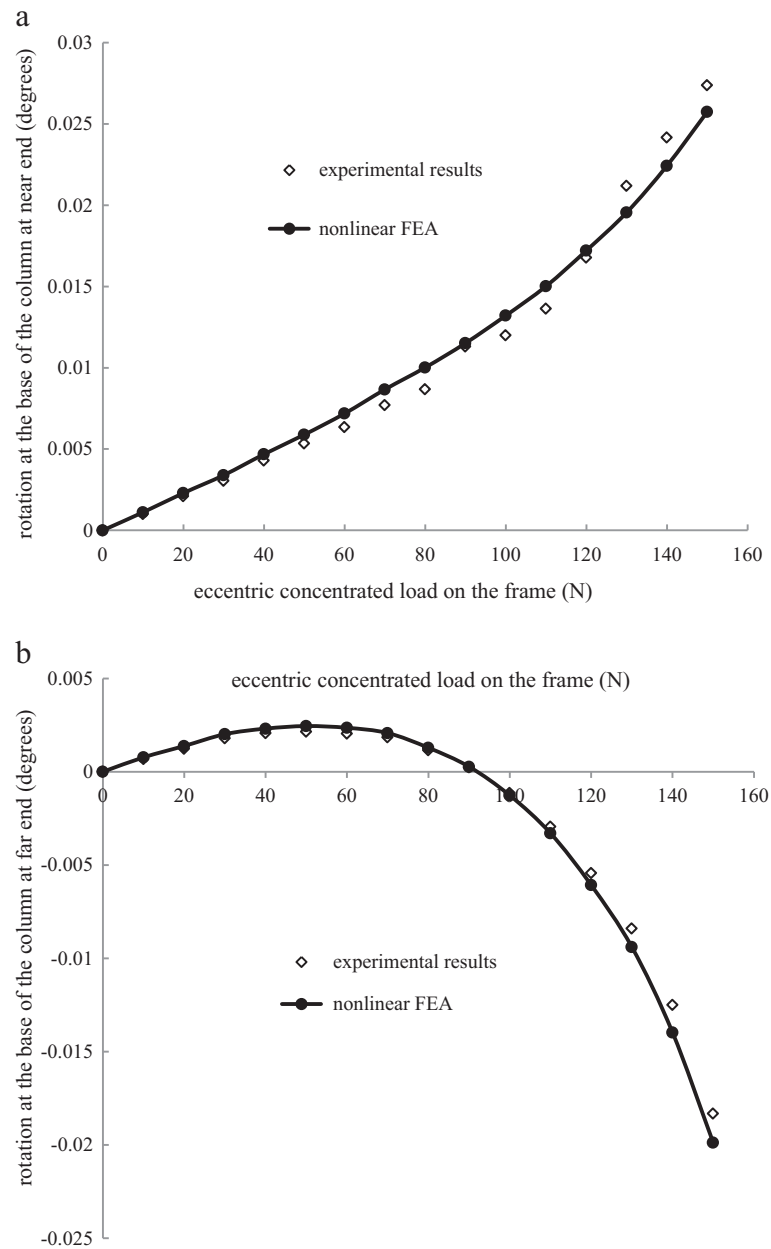
eccentric concentrated load on the frame at near end, which is 6% to 14% at the far end. Hence, the displacement from the experiment is in good agreement with that by the nonlinear FEA.

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 Figure 5a,b; 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 Figure 6a,b. The settlement from the experiment shows a variation of not more than 15% with respect to that from the nonlinear FEA for central

concentrated load and uniformly distributed load on the frame. For eccentrically loaded frame at near end the variation is not more than 13%, at far end it is not more than 14%. Hence, the displacement from the experiment is in good agreement with that by the nonlinear FEA.

The variation of rotation at the base of the column for the central concentrated load and UDL applied on the frame is presented in Figure 7a,b. 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 Figure 8a,b. In case of eccentric concentrated load on the frame, after certain

Figure 8 Variation of rotation at the column base of the near and far end. (**a**) Rotation at the base of the column at near end (eccentric concentrated load). (**b**) Rotation at the base of the column at far end (eccentric concentrated load).

level of loading, rotation at the far end is changed from clockwise to anti-clockwise. This is expected because of the lateral movement of the near and far ends are in the same direction which causes the far end to rotate in the reverse manner. The rotations from the experiment show a variation of 7% to 14% with respect to that from the nonlinear FEA. Hence, the displacement from the experiment is in good agreement with that by the non-linear FEA.

In all the aforementioned results, it is observed that, for relatively lower loads on the frame, the values
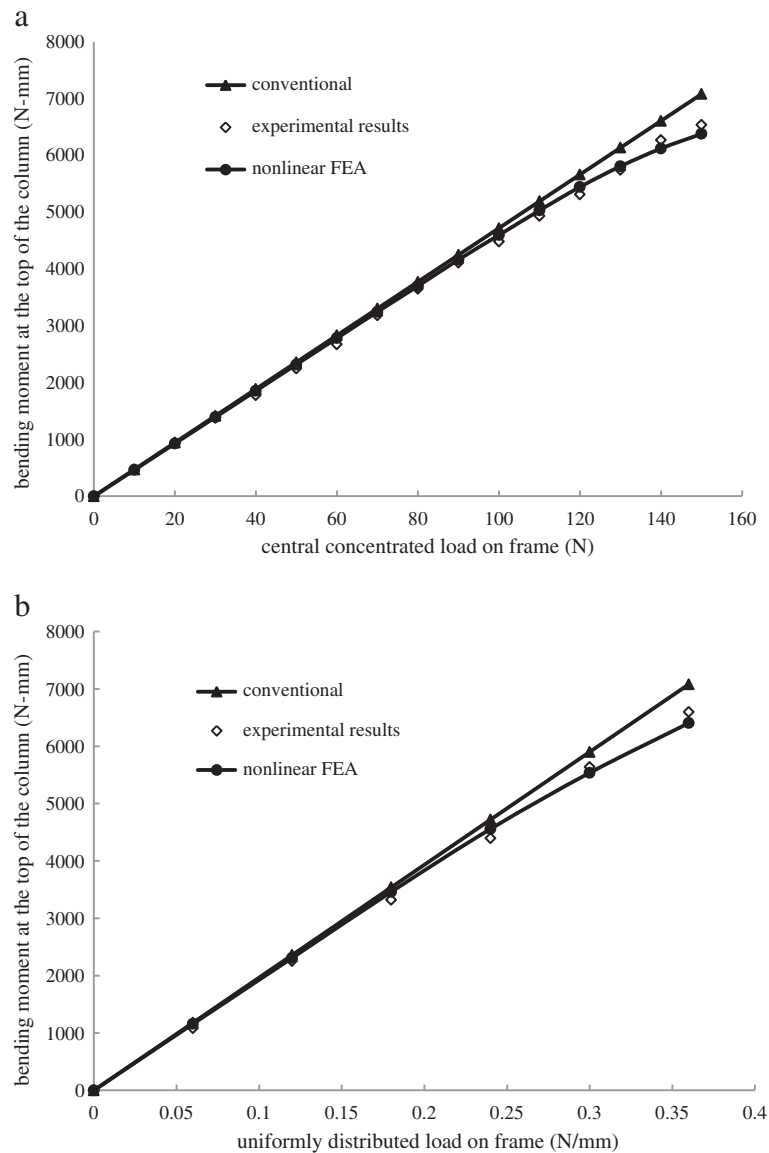
predicted by the nonlinear FEA and experiment are nearly linear. For higher loads on the frame, the results deviate significantly from the linearity.

Shear force in the frame by conventional method, experiments, and nonlinear FEA

Shear forces in the frame under the central concentrated load, UDL, and eccentric concentrated load have been plotted in Figure 9a,b,c. From these plots, it can be observed that the shear force predicted by the conventional method is always on the higher side. For relatively
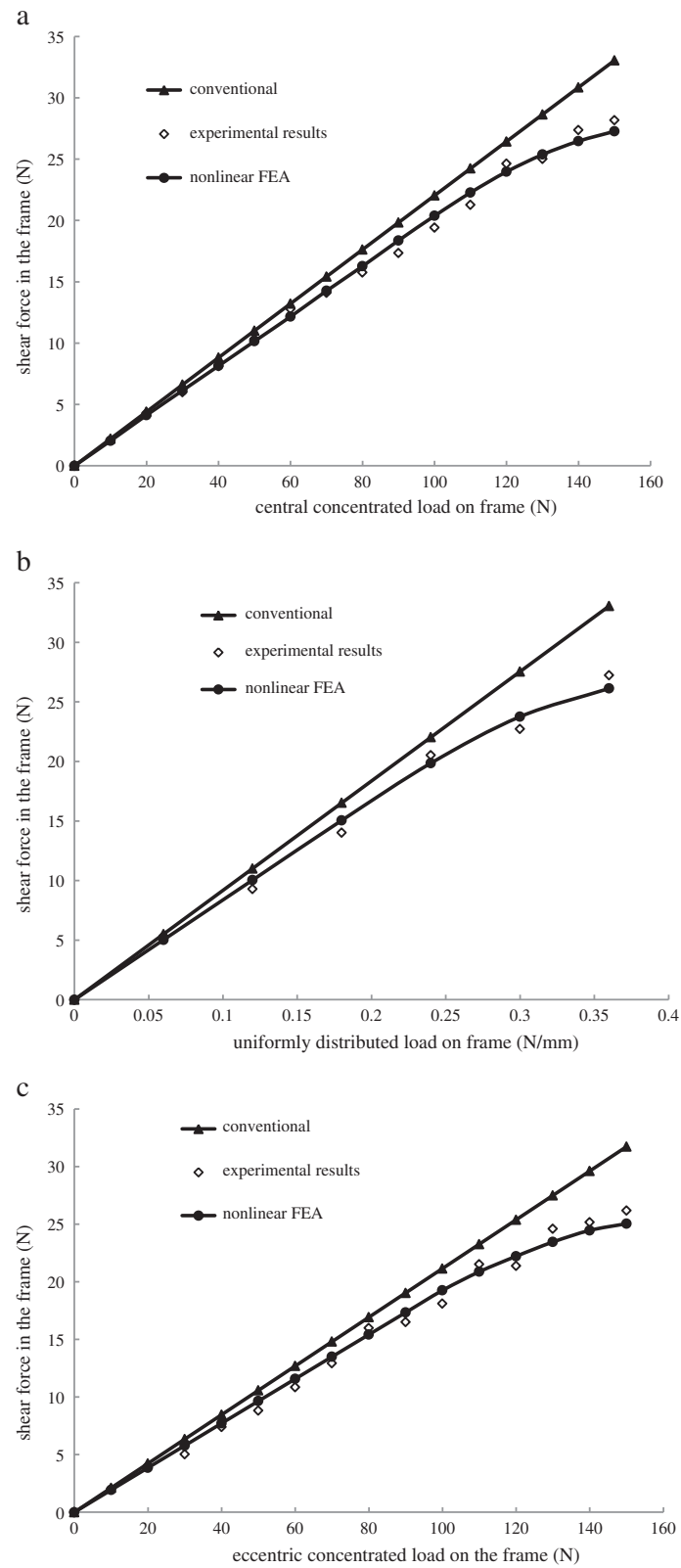
Figure 9 Shear forces in the frame. (**a**) Shear force (central concentrated load). (**b**) Shear force (UDL). (**c**) Shear force (eccentric concentrated load).

Figure 10 Bending moment at the top of the column of the frame. (a) Bending moment at top of the column (central concentrated load).
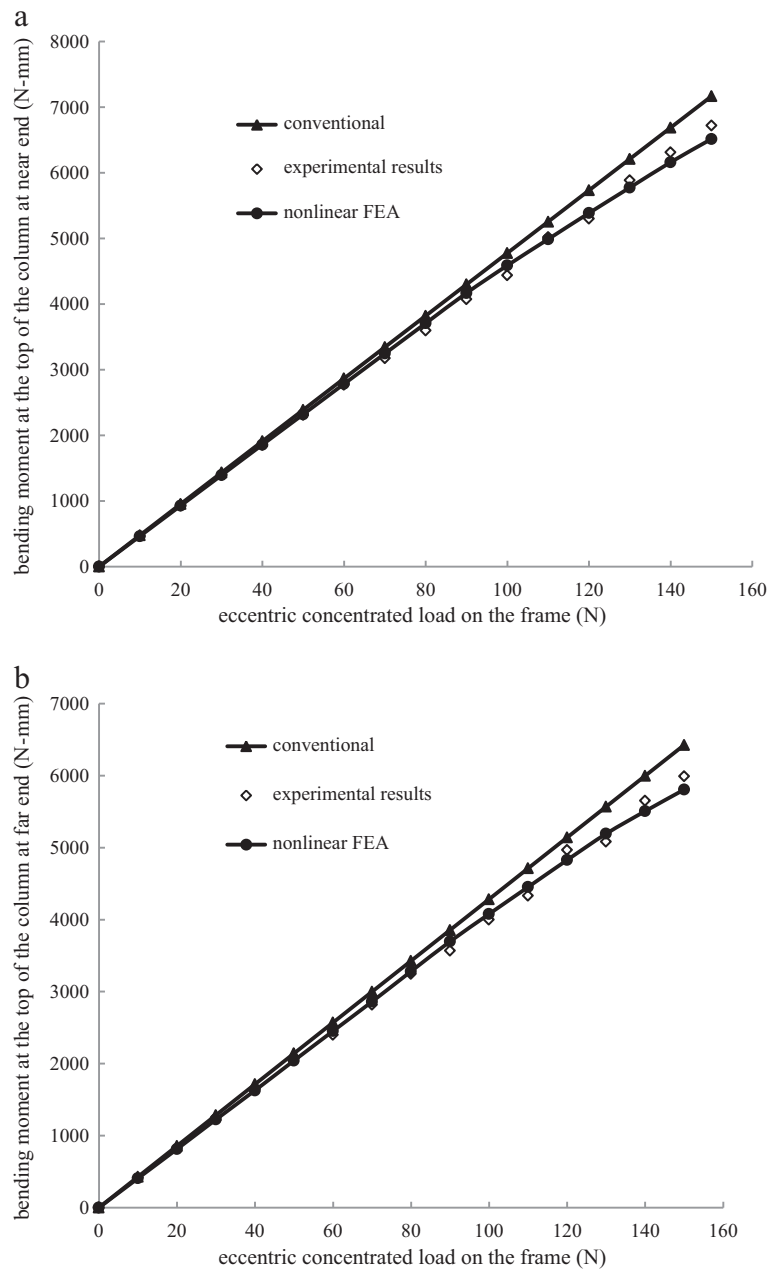(**b**) Bending moment at top of the column (UDL).

lower loads on the frame, the shear force predicted by the nonlinear FEA and experiment follow closely the shear force by the conventional method. The maximum difference in shear force predicted by the conventional method and the nonlinear FEA for frame with plinth beam is about 20%. The shear force obtained from the experiment deviates by about 5% of that given by the nonlinear FEA, which indicates that the nonlinear soil model is in good agreement with the experimental results.

Bending moment at the top of the column by conventional method, experiments, and nonlinear FEA
Bending moment at the top of the column of the frame under the central concentrated load and UDL

is plotted in Figure 10a,b; the one of the near and far ends of the frame under the eccentric load is plotted in Figure 11a,b. From the above figures, it is observed that the bending moment predicted by the conventional method is higher than that by the nonlinear FEA, indicating that the conventional method of analysis for obtaining the design moment is uneconomical. Compared with the bending moment predicted by the nonlinear FEA, the reduction in bending moment of the frame with plinth beam is about 10% of the bending moment from the conventional method.

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

Figure 11 Bending moment of the near end and far end of the frame under the eccentric load. (**a**) Bending moment at top of the column at near end (eccentric concentrated load). (**b**) Bending moment at top of the column at far end (eccentric concentrated load).

percentages of variation may not be great, the differences are significant because the magnitudes of bending moment are of multiples of thousands. 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 2% to 3% only, which indicates that the nonlinear soil model is well suited for representing the nonlinear behavior of the soil.

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

The variation of bending moments at the base of the column of the frame under the central concentrated load and UDL have been plotted in Figure 12a,b. Figure 13a,b shows 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. These figures show that the conventional method gives a bending moment 20% to 30% higher value than that of the
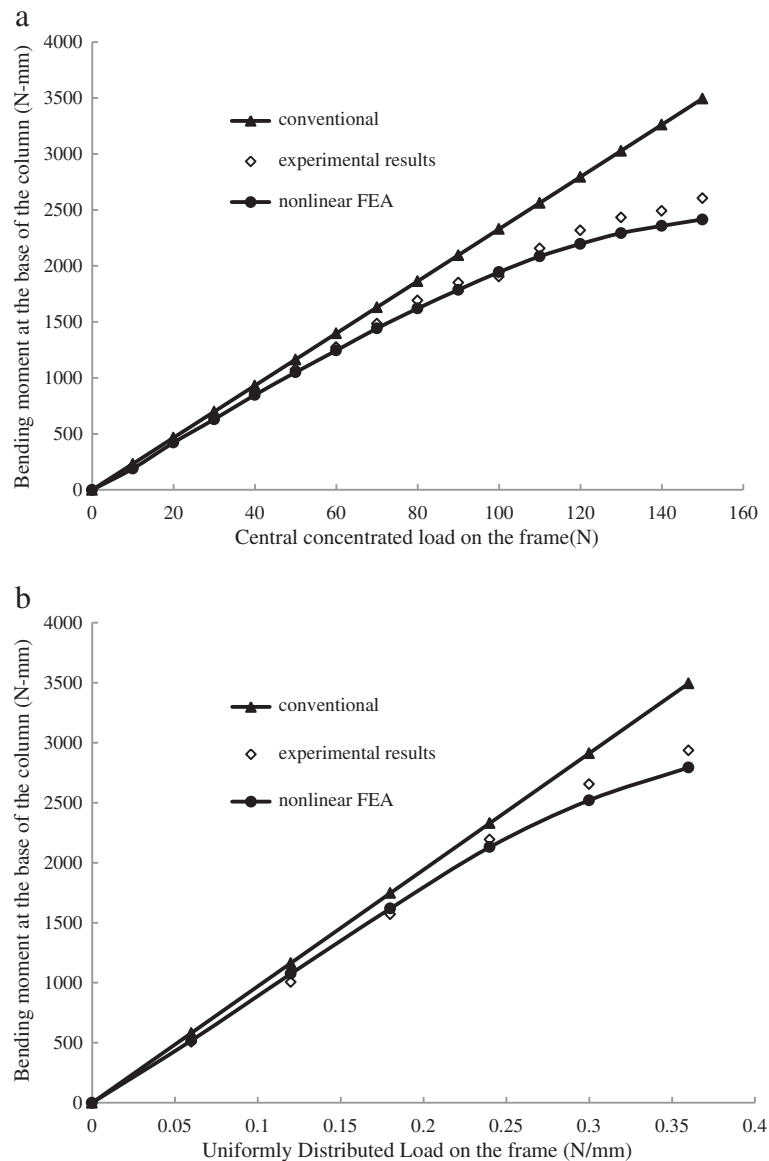
Figure 12 Variation of bending moments at the base of the column of the frame. (**a**) Bending moment at the base of the column (central concentrated load). (**b**) Bending moment at the base of the column (UDL).

bending moment from nonlinear FEA. Hence, from the above results, it is to be noted that the consideration of soil interaction reduced the bending moment considerably when compared to the frame analysis with fixed bases. The bending moments given by the experiments agree well with those by the nonlinear FEA with a variation of 5% to 7%, indicating that the soil nonlinearity is well represented by the constitutive relations used for the soil.

Conclusions

Based on the results of the present experimental and numerical investigations on the model building frame with plinth beam 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 nonlinear FEA and experiment appears to be linear for relatively smaller loads, and for higher load range they show a nonlinear variation. For eccentric concentrated load on the frame after certain level of loading, rotation at the far end is changing its sign as the lateral displacements at near end and far end are in the same direction. The displacements and rotations from the experimental
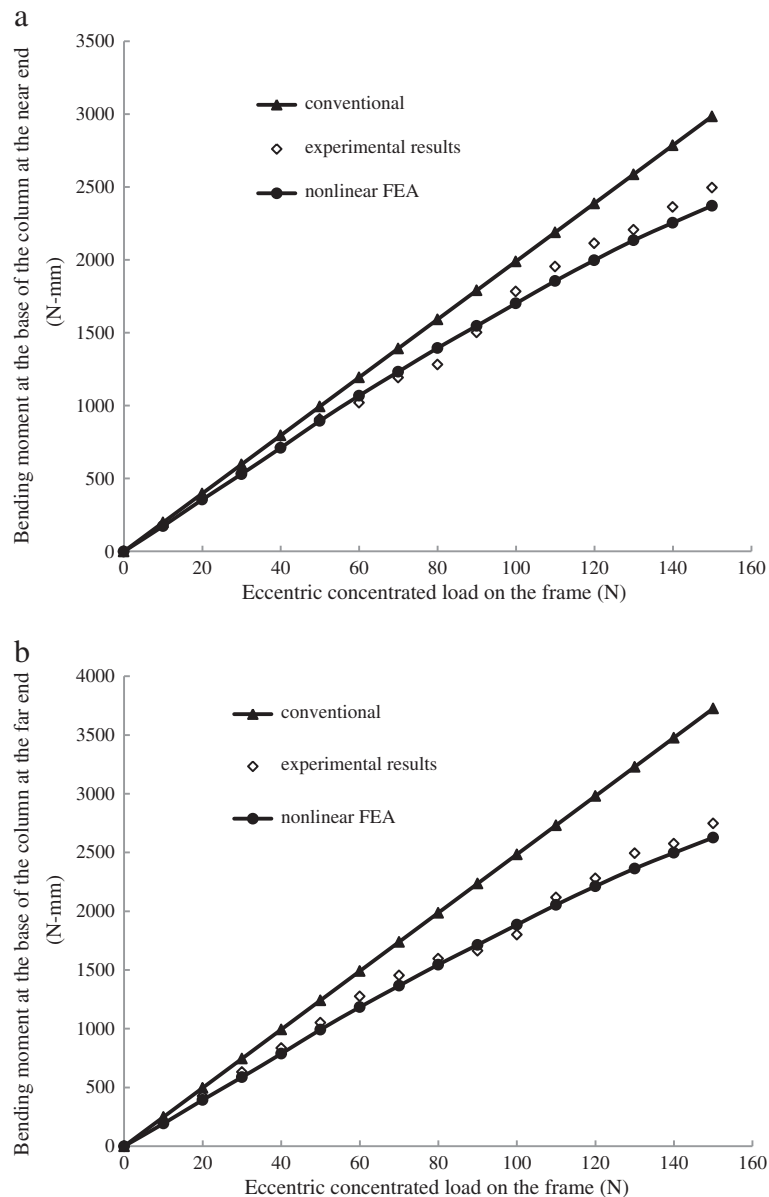
Figure 13 Variation of bending moments at the base of the column of the near end and far end of the frame. (**a**) Bending moment at the base of the column of near end (eccentric concentrated load). (**b**) Bending moment at the base of the column of far end (eccentric concentrated load).

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.

• The conventional method of analysis gives a shear force of about 20% higher than that by the nonlinear FEA. 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 nonlinear FEA also increases.

• The conventional method gives a bending moment at the top of the column that is about 10% higher than that by the nonlinear FEA for the frame with plinth beam, but such a difference is still significant as the bending moment values are in the multiples of thousands.

• The conventional method gives a bending moment at the base of the column that is 20% to 30% higher than that by the nonlinear FEA for the frame with plinth beam. For a nominal eccentricity given for the concentrated load (10% length of the beam), the conventional method and nonlinear FEA for the frame

with plinth beam 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. The reason behind this behavior is that the displacements and rotations at far end are lower than the near end.

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. The shear force and bending moments from nonlinear FEA of the frame with plinth beam were reduced considerably when compared to the conventional method of analysis which reveals the need for consideration of the interaction between the building frame with plinth beam, pile foundation, and soil.

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