Effect of Rigidity on Geotextile as Plinth beam on Soil Interaction of Model Building Frame Supported on Pile Groups Using FEA

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Abstract - This paper presents the nonlinear FEA for static vertical loads on a model building frame without plinth beam, with conventional plinth beam and geotextile as plinth beam supported by pile groups embedded in cohesion less soil (sand). The effect of soil interaction, conventional plinth beam and geotextile as plinth beam 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 from the literature 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 (-z curves) at their tips (Q-z curves). The results reveal that the conventional method of analysis gives a shear force of about 57% higher than that by the nonlinear FEA for the frame without plinth beam, about 15% higher than that for the frame with conventional plinth beam and about 53% higher than that for the frame with geotextile as plinth beam. Hence, when the geotextile is used as plinth beam instead of conventional one the reduction in shear force is about 35-45%. The conventional method gives a bending moment at the top of the column that is about 3-5% higher than that by the nonlinear FEA for the frame with conventional plinth beam, it is about 20% higher than that for the frame without plinth beam, it is about 17% higher than that for the frame with geotextile as plinth beam. Hence the use of geotextile as plinth beam instead of conventional one reduced the bending moment at top by about 15% 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 60 - 100% higher than that by the nonlinear FEA for the frame without plinth beam, it is about 50-98% higher than that for the frame with geotextile as plinth beam and it is about 10-20% higher than that for the frame with conventional plinth beam. Hence the use of geotextile as plinth beam instead of conventional one reduced the bending moment at bottom by about 40-91%. The response of the frame from the experimental results is in good agreement with that obtained by the nonlinear finite element analysis. ***Keywords:* Finite element analysis, geotextile, nonlinear, p-y curves, lateral direction, bending moment**

1. INTRODUCTION

Pile foundations are commonly used to resist axial and lateral loads applied to structures. A number of different approaches are available to determine the behavior of laterally loaded piles. One of the most popular approaches is the load–transfer approach, often referred to as the ‘‘p – y curve method’’. The p – y curve method models the pile as an elastic member and the soil as a series of nonlinear springs. The nonlinear soil spring describes the local variation of lateral soil–pile interacting resistance with lateral displacement. A number of p – y curves were developed for sands (O’Neill and Murchinson 1983, Reese et al. 1974). The traditional p – y curves are semi-empirical models and do not account for pile properties such as pile bending stiffness, pile cross-sectional shape, pile head restraint, pile installation method (Ashour and Norris 2000), or drilling method (Hameed et al. 2000). It is known that p – y curves can be employed in comprehensive numerical soil–structure interaction analysis (finite element method) to model the soil–pile response of a structural problem involving the superstructure along with the substructure. Soil-Pile interaction behaviour also depends on the constitutive behaviour of soil model. Therefore, selection of a proper constitutive model leads to better results in FE analysis. Constitutive models are widely used in numerical analysis of geomaterials. They can be modelled to behave linear elastically, nonlinear elastically or elsastoplastically. Structure-soil interaction is a subject currently receiving close attention from researchers in a wide range of research centres, mainly concerned with practical applications. Piles in foundations are often submitted to strong horizontal forces, as for example in the case of piles in the foundations of bridges, high buildings, off-shore structures and support-walls, among others.

Objective of study:

The aim of this thesis is to present the numerical analysis nonlinear FEA of a model plane frame without plinth beam, frame with conventional plinth beam and frame with geotextile as 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 frames and the use of geotextile as plinth beam instead of conventional one is emphasized by comparing the behaviour of the frame obtained by the experimental results from literature and numerical analysis with that by the conventional method of analysis. An attempt is made to quantify the soil interaction effect and the sue of geotextile as plinth beam instead of conventional one on the response of the building frame in terms of displacements, rotations, shears and bending moments through the nonlinear FEA.

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 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.

Finite Element Models:

The finite element method has been a very popular method for analysis of problems in soil mechanics. One of the primary advantages of the finite element method is that it gives a reasonably accurate solution with proper discretization. Also, it can be easily extended to a stratified soil medium as well as material nonlinearity. One of the main disadvantages to the finite element method is that it is computationally expensive and the development of the system equations can be very cumbersome. There have also been variations of the finite element models developed by researchers. Hybrid techniques using finite element models in conjunction with the boundary element method have been developed. There are two approaches in which the problem of laterally loaded pile can be handled in the finite element method. The first method is to consider the pile soil system as a 3-dimensional system. Three-dimensional brick elements are used to model the system. This approach has been adopted by Vallabhan and Sivakumar , Rahman , etc. One of the disadvantages of this method is that the brick element has a large number of degrees of freedom and can be computationally expensive. Also, the brick element does not accurately model the behaviour of the pile as the pile exhibits the behaviour of a beam. In other words to accurately model the behaviour of the pile a large number of divisions need to be employed. The second and more popular approach towards the analysis of this system is to use axisymmetric elements on which a non axisymmetric load acts. This method has been used by researchers such as Wilson et al. For axisymmetric structures, Extension of this method to the problem of laterally loaded piles has been done by Chandrasekharan

2.3 Models based on Winkler Concept:

The Winkler's subgrade reaction concept of modelling soil behaviour is widely used when deriving a model for laterally loaded piles. In the Winkler's subgrade reaction approach the soil is modelled as a set of independent elastic springs. The spring coefficients of these springs reflect the material properties of the soil and are known as coefficients of subgrade reaction. In these models the pile is modelled as an elastic beam resting on these springs. The springs are attached to the beam at discrete points, so the displacement of the pile-soil system depends on the soil at discrete points.

In any basic model that uses the Winkler's subgrade reaction concept, the pressure applied on the pile and the deflections of the soil are related by the following equation.

$$u$$

The pile is assumed to act as a prismatic long beam. Hence the governing equation is given by

$$+ \quad u = 0$$

In the above equations

= Modulus of elasticity of the pile,

= Moment of inertia of the pile,

= Coefficient of horizontal soil resistance,

u = Lateral displacement at the pile-soil boundary, and

z = Coordinate in the axial direction of the pile.

Loads acting on the beam are then applied as boundary conditions to obtain the complete system equations. These equations can be solved using analytical or numerical methods.

Analysis Programme Using ANSYS

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

a) Without plinth beam

b) Geotextile as plinth beam

c) Conventional plinth beam

The above three problems are solved 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) 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 figured shear forces and bending moments generated in the columns.

3.3 Nonlinear Finite Element Analysis

The nonlinear analyses were performed for the single bay single storeyed model plane frame founded on 2 x 2 pile groups in a sandy soil (Fig. 3.1). The columns, beams and piles are modeled using the 3D elastic two-nodded BEAM4elements. The pile cap is modeled using the four-nodded elastic SHELL63 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 Eqs. (1) - (3) are employed for the present problem.

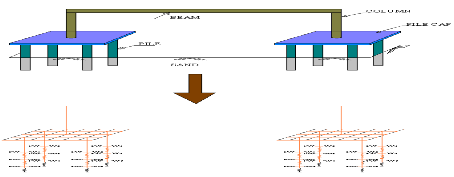

Fig. 3.1: Modelling of the frame along with the pile groups (without plinth beam)

BEAM 4: Element Description

BEAM4 is a uniaxial element with tension, compression, torsion, and bending capabilities.
The element has six degrees of freedom at each node: translations in the nodal x, y, and z directions and rotations about the nodal x, y, and z axes. Stress stiffening and large deflection capabilities are included. A consistent tangent stiffness matrix option is available for use in large deflection (finite rotation) analyses

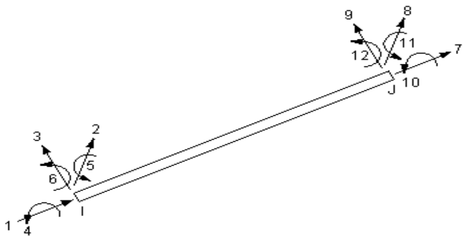

Fig. 3.2: Order of Degrees of Freedom for BEAM 4 element

3.3.2 SHELL 63: Element Description

SHELL63 has both bending and membrane capabilities. Both in−plane and normal loads are permitted. The element has six degrees of freedom at each node: translations in the nodal x, y, and z directions and rotations about the nodal x, y, and z−axes. Stress stiffening and large deflection capabilities are included. A consistent tangent stiffness matrix option is available for use in large deflection (finite rotation) analyses.

The user explicitly defines the force-deflection curve for COMBIN39 by the input of discrete points of force versus deflection. Up to 20 points on the curve may be defined, and are entered as real constants. The input curve must pass through the origin and must lie within the unshaded regions, if KEYOPT(1) = 1. COMBIN39 is a non linear spring element, used to represent tip resistance, axial resistance and lateral resistance of soil. The following will explain the formulation of above said properties and will give a typical example

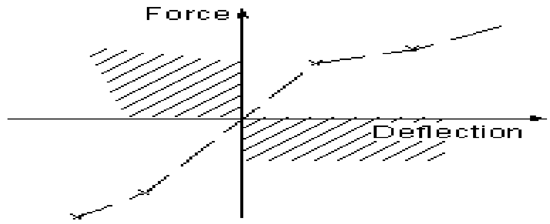

Fig. 3.5: Force-Deflection curve for COMBIN39 element

Soil nonlinearity in the lateral direction characterized by the p-y curves and in the axial direction by nonlinear vertical springs along the length of the piles (□-z curves) at their tips (Q-z curves) is explained as follows

3.3.3.1 TIP RESITANCE:

As for the nonlinear tip spring (Q-Z), the following relation is used:

(1)

Where Q_f = Ultimate tip resistance (force);
G_i = Initial shear modulus;
□ = Poisson's ratio of the soil;
r_0 = Radius of the pile;
Q_b = Mobilized tip resistance for the given displacement Z.
Determination of Q_f
= *
= ϒ* *
Where ϒ = Unit weight of soil
=Depth of pile embedded in the soil

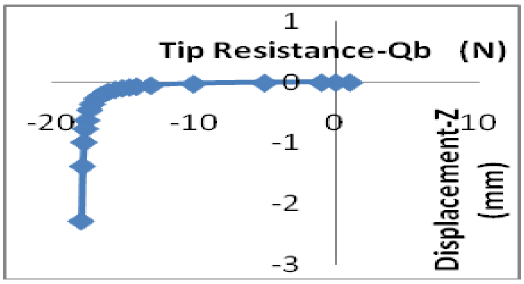

Fig. 3.6: Non linear pile tip spring for soil model

=Base area of the pile

3.3.3.2 AXIAL RESISTANCE:

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

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

Where □□= r_0□$_0$/□$_f$

r_0 = radius of the pile

□$_0$ = shear stress transferred to the soil for a given Z displacement

r_m = radius out from the pile where shear stress is negligible

=).L. (1-ν)

L= length of pile

ν = poison's ratio

G_i = initial shear modulus

□$_f$ = ultimate shear stress at the point of interest on the pile

Determination of

= (α c+ q ktanδ) (from Tomlinson's eq.-1971)

= q ktanδ (for sands cohesion=0)

Where q= ϒZ =vertical stress

K=Co-efficient of lateral Earth pressure

δ =Effective friction angle between Soil and Pile material

ϒ= unit weight of soil

Ktanδ = 0.18+0.0065 (from Bhushan 1982)

= Relative density

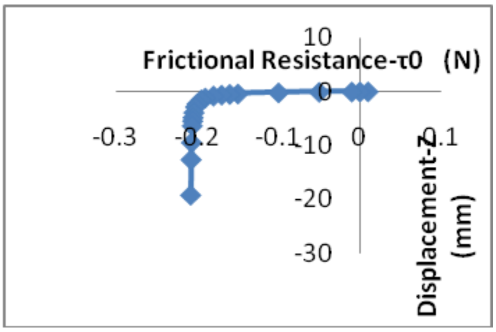

Fig. 3.7: Non linear pile axial spring for soil model

3.3.3.3 LATERAL RESISTANCE:

The lateral load transfer curves given by Eq. were used as the API model,

$$p = \bar{A}$$

Where = adjustment coefficient for the static p-y curves

P_s = governing ultimate soil resistance

k = initial subgrade reaction constant

Z = depth

P_u = ultimate soil resistance

P =Soil resistance per unit length of pile

= +

Ultimate tip resistance = $Q_f = A_b * q_f$ = $\frac{\pi}{4}.D^2$ γ.L.N_q

Ultimate Frictional resistance= = *ϒ*Z *Ktanδ (2)

= Smaller of or

Theoretical ultimate soil resistance due to wedge failure= = (Z+ D) ϒZ

Theoretical ultimate soil resistance due to flow failure = = DϒD

= (3.0-0.8) 0.9 (for static loading)

, , and K values are taken from following graphs

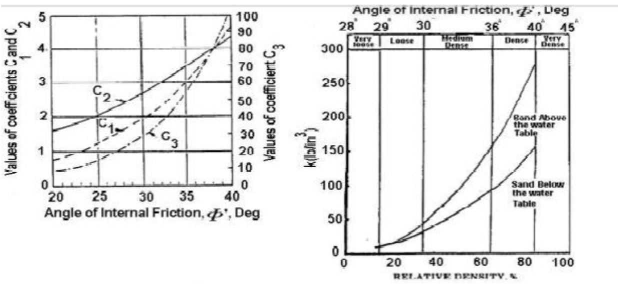

Fig. 3.8: Coefficients as function of ϕ and Initial modulus of subgrade reaction for API sand

3.4 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:

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 aluminium 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. Aluminium plates of 13 mm thickness were used as the pile caps. For plinth beam case, An aluminium bar of 10x10 mm size is used as plinth beam model, to simulate the prototype of 230x230 mm size of plinth beam. 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 (3) (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.

3.5 Geotextile:

The wide-width tensile strength test is a popular method to evaluate properties of various geosynthetics. In order to investigate the contribution of single filaments to the wide-width tensile stress-strain properties of the selected geotextiles tests were performed using the procedure described in ASTM D 4595. Various studies have been conducted by many researchers about the effects of sample preparation on the test results. However, it is known that there is no universal relationship between specimen sizes and test results (Koerner, 1998). Featuring high tensile strengths and low elongations, woven geotextiles have a remarkable capacity for filtering soils, distributing loads, reducing rutting and extending the life of paved and unpaved roadways. Made from individual yarns woven together to provide dimensionally stable geotextiles, they are resistant to ultraviolet (UV) degradation and to biological and chemical environments normally found in soils. All of our woven geotextiles are backed by decades of in-field performance in everything from separation and filtration to erosion control and waste containment applications.

Features & benefits

- Tensile strength ranges from 135 to 370 lbs (600 to 1645 N) for a wide variety of soil stabilization and filtration applications
- Higher strengths available in our line of soil reinforcement woven geotextiles
- Made from polypropylene resin for superior chemical resistance in even the most aggressive environmental applications
- Yarns are woven together to form a strong fabric capable of withstanding construction installation stresses
- Contains additives for maximum UV resistance

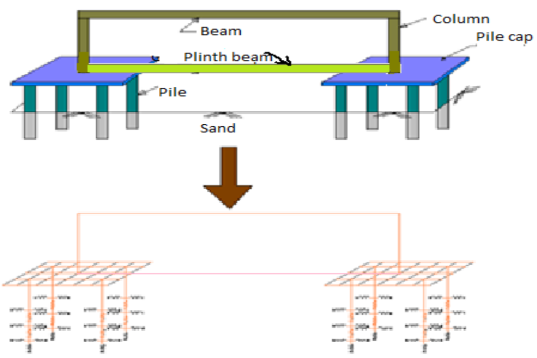

Fig. 3.10: Frame model with conventional plinth beam or (Geotextile as plinth beam)

RESULTS AND DISCUSSIONS

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

Figures 4.1 and 4.2 represent the variation of the lateral displacement with the static load applied on the frame 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 without plinth beam for central concentrated load on the frame is reduced by 33-61% when the geotextile is used as plinth beam. Whereas for uniformly distributed load on the frame it is reduced by 25-63%. In both the cases the lateral displacement at the base of the column for frame with conventional plinth beam is negligibly small. The displacement from the experiment shows a variation of 5-15% with respect to that from the nonlinear FEA for central concentrated load on the frame. It is 10-15% for uniformly distributed load on the frame. Hence the displacement from the experiment is in good agreement with that by the nonlinear FEA.

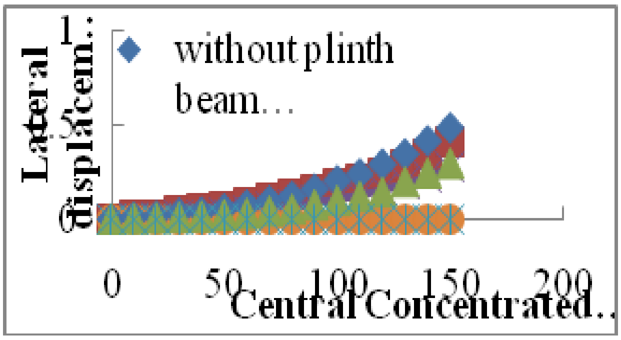

Fig.4.1: Lateral displacement at the base of the column for central point load

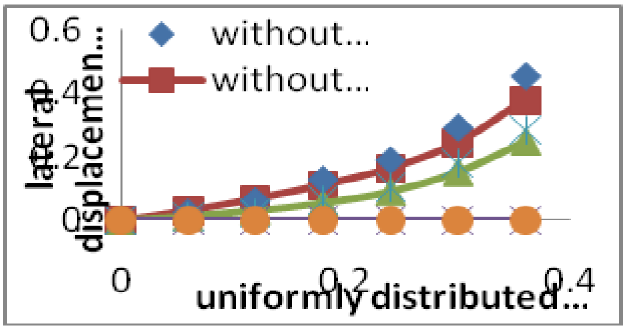

Fig.4.2: lateral displacement at the base of the column for UDL

Figures 4.3 and 4.4 represent 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 behaviour of frame with plinth beam is different from that of the frame without plinth beam. Whereas the behaviour of the frame with geotextile as plinth beam is similar to that of the frame without plinth beam except at far end after certain level of loading the increase in the lateral displacement is decreased. In case of frame without plinth beam and frame with geotextile as plinth beam the base of the column at near end and far end moves outward when the load is applied on the frame, but in case of frame with plinth beam the base of column at near end and far end moves in the same direction with nearly same amount of displacement (5% difference) and it is towards eccentricity. The lateral displacement at the base of the column at near end of frame without plinth beam for eccentric concentrated load on the frame is reduced by 18-21% when the geotextile is used as plinth beam. It is 10-44% at far end. The displacement from the experiment shows a variation of 3-14% with respect to that from the nonlinear FEA for eccentric concentrated load on the frame at near end. It is 6-14% at far end. Hence the displacement from the experiment is in good agreement with that by the nonlinear FEA

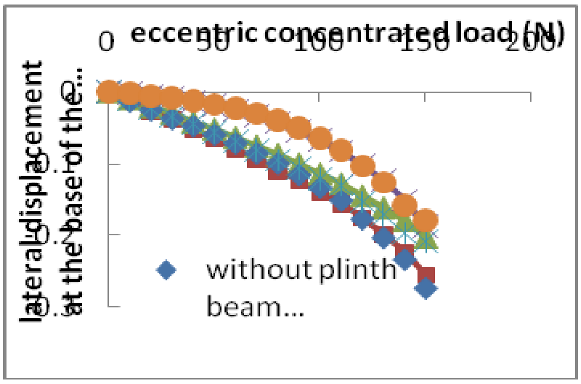

Fig.4.3: Lateral displacement at the base of the column at near end for eccentric point load

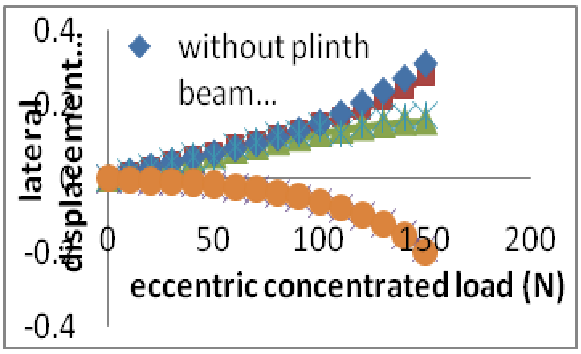

Fig.4.4: Lateral displacement at the base of the column at far end for eccentric point load

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. 4.5 and 4.6, 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. 4.7 and 4.8, respectively. From the plots mentioned herein, it is to be noted that the settlement at the base of the column of frame with geotextile as plinth beam and with plinth beam is more than that of the frame without plinth beam. The settlement at the base of the column for frame with conventional plinth beam is 15-20% more than that of the frame without plinth beam for central concentrated load and uniformly distributed load on the frame. For eccentric loading at near end the settlement at the base of the column for frame with conventional plinth beam is 20-27% more than that of the frame without plinth beam. It is 10-18% at the far end. Settlement at the base of the column is not much affected by replacing the geotextile with conventional plinth beam. The maximum difference in settlements for both the cases is 5%. 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.

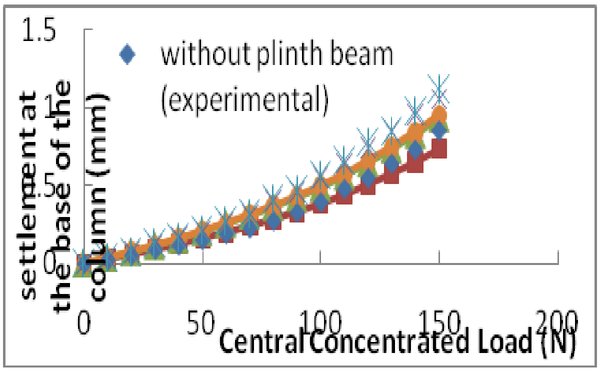

Fig.4.5: Settlement at the base of the column for central point load

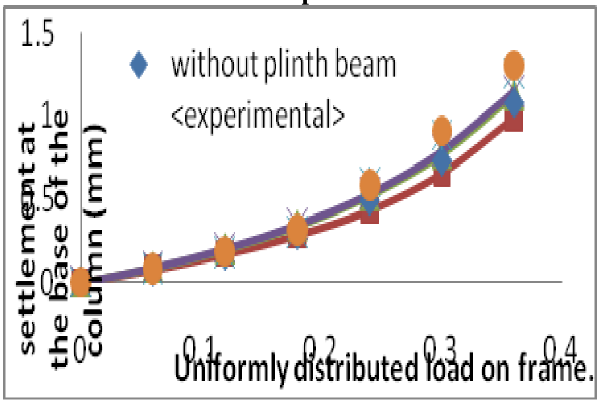

Fig.4.6: Settlement at the base of the column for UDL

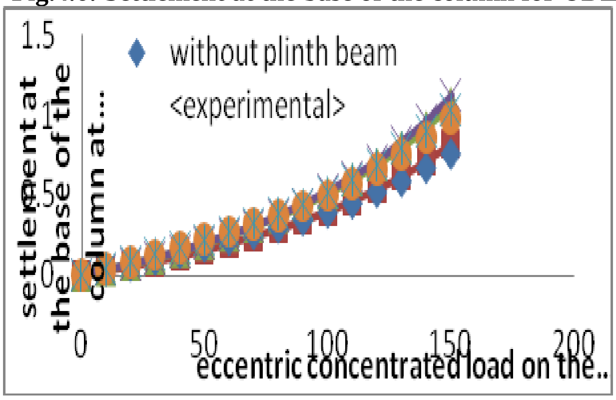

Fig.4.7: Settlement at the base of the column at near end for eccentric point load

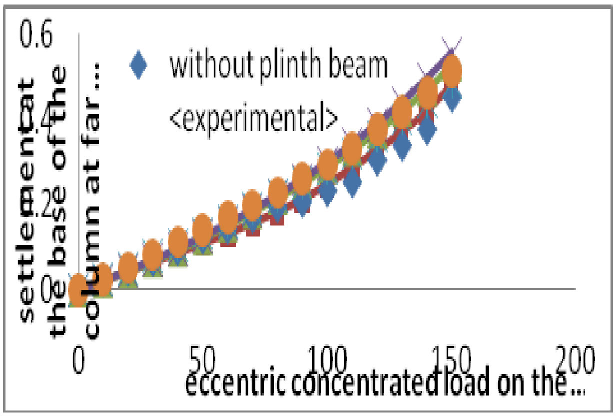

Fig.4.8: Settlement at the base of the column at far end for eccentric point load

Figure 3.3: Photograph of Model plane frame at base setp

Figure 3.4: Photograph of Model plane frame at central concentrated load

Figure 3.6: Photograph of Model plane frame at arrangement of dial gauge

Figure 3.11: Photograph of Model plane frame at base (3mm)

Figure 3.16: Photograph of Model plane frame at base (5mm)

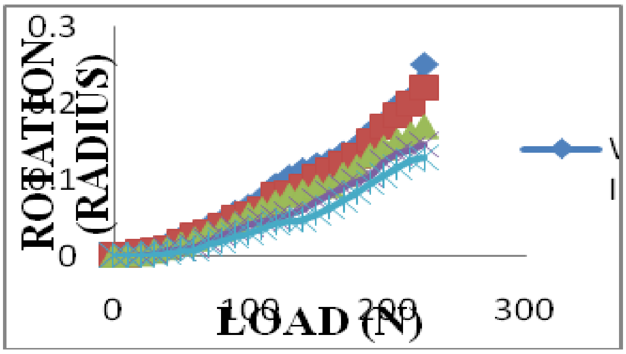

Figure 5.2: variation of load vs rotation for central concentrated load

- shows the variation of rotation at column bases with central concentrated load .
- The above graph is drawn between load vs rotation.
- The plot shows that for the lower load on the frame **load vs rotation** is follows liner relation for higher loads on the frame it is non liner relation.
- It is found as the axial regidity various from **0 to 4480 KN** ,the ratation decreases by **63.1%.**

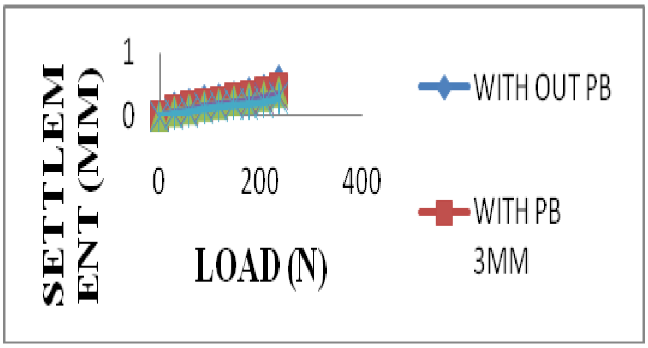

Figure 5.6: variation of load vs settelement for uniform distributatted load

- shows the variation of settelement at column bases with uniform distributatted load .
- The above graph is drawn between load vs settelment**.**
- The plot shows that for the lower load on the frame **load vs settelment** is follows liner relation for higher loads on the frame it is non liner relation.
- It is found as the axial regidity various from **0 to 4480 KN ,**the settelment decreases by **38.58%.**

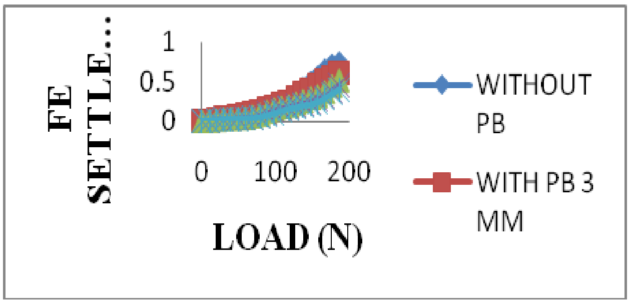

Figure 5.11: variation of load vs farend settelment for eccentricity load

Conclusions

The experimental results shows the variation of load vs. displacement is nearly linear of loading for higher load on the frame it is showing nonlinear variation. As the axial rigidity of plinth beam increases from 0 to 4480 KN, the lateral displacement decreases by 55.88%. As the axial rigidity of plinth beam increases from 0 to 4480 KN, the rotation decreases by 64.12%. As the axial rigidity of plinth beam increases from 0 to 4480 KN, the settlement decreases by 54.45%. The results show that the lateral displacement, rotation and settlement as the base of the column of a building frame deepens as the axial rigidity of the plinth beam increases. As the axial rigidity of plinth beam increases from 0 to 4480 KN ,the shear force increases by13.7% As the axial rigidity of plinth beam increases from 0 to 4480 KN, the bending moment top increases by 14.19% As the axial rigidity of plinth beam increases from 0 to 4480 KN, the bending moment bottom increases by 19.77% Hence the shear force and bending moment in the frame increases. So to reduce the effect of rigidity of plinth beam on design parameters. it is suggested that any element which will have less axial rigidity such as geotextiles can be used as plinth beam.

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