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Nonlinear interaction analysis of infilled frame-foundation beam-homogeneous soil system

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Abstract. A proper physical modeling of infilled building frame-foundation beam-soil mass interaction system is needed to predict more realistic and accurate structural behavior under static vertical loading. This is achieved via finite element method considering the superstructure, foundation and soil mass as a single integral compatible structural unit. The physical modelling is achieved via use of finite element method, which requires the use of variety of isoparametric elements with different degrees of freedom. The unbounded domain of the soil mass has been discretized with coupled finite-infinite elements to achieve computational economy. The nonlinearity of soil mass plays an important role in the redistribution of forces in the superstructure. The nonlinear behaviour of the soil mass is modeled using hyperbolic model. The incremental-iterative nonlinear solution algorithm has been adopted for carrying out the nonlinear elastic interaction analysis of a two-bay two-storey infilled building frame. The frame and the infill have been considered to behave in linear elastic manner, whereas the subsoil in nonlinear elastic manner. In this paper, the computational methodology adopted for nonlinear soil-structure interaction analysis of infilled frame-foundation-soil system has been presented.

Keywords: conventional analysis; nonlinear analysis; constitutive law; differential settlement; exponential decay; infinite elements; interaction analysis; truncation boundary

1. Introduction

In the conventional method of design, a structure is designed assuming the fixity at the base. Such analysis does not provide the realistic structural behaviour because interaction takes place between the superstructure, foundation and soil mass. The forces in the frame members get significantly altered due to differential settlement of the soil mass. Thus, it is essential to consider the superstructure, foundation and the soil mass as a single integral compatible unit for more realistic and accurate structural analysis.

2. Literature review

Several investigators studied the influence of the phenomenon of soil-structure interaction in framed structures and investigated that the forces change significantly due to interaction effect.

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Numerous studies e.g., Meyarhoff (1947), Chameski (1956), Greshoff (1957), Baker (1957), Morries (1966), Larnach (1970), Lee and Brown (1972), Nayak *et al*. (1972), Seetharamulu and Kumar (1973), King and Chandrasekaran (1974), Jain *et al*. (1977), King and Yao (1983), Subbarao (1985), Brown and Yu (1986), Salvadurai (1989), Sharda Bai *et al*. (1990), Allam et al. (1991), Viladkar and Godbole (1991), McCallen et al. (1993), Noorzaei *et al*. (1994), Fardis and Panagiostakos (1997), Dutta and Bhattacharya (1999), Junvi *et al*. (2003), Singh *et al*. (2006), Abate *et al*. (2007), Orakdoen and Girgin (2008), Puglisi *et al*. (2009) and Chore *et al*. (2010) have made to quantify the effect of soil-structure interaction on building frames. These studies have clearly indicated that force quantities are revised due to interaction phenomenon.

Desai *et al*. (1982) presented a finite element procedure for the general problem of three-dimensional soil-structure interaction involving nonlinearity caused by material behaviour, geometrical changes and interface behaviour. The formulation presented is based on the updated Lagrangian approach with appropriate provision for constitutive laws. Desai *et al*. (1985) developed hybrid finite element procedure for nonlinear elastic and elasto-plastic soil-structure interaction analysis including simulation of construction sequences.

Aljanabi *et al*. (1990) studied the interaction of plane frames with an elastic foundation of the Winkler's type, having normal and shear moduli of sub-grade reactions. An exact stiffness matrix for a beam element on an elastic foundation having only a normal modulus of sub-grade reaction was modified to include the shear modulus of sub-grade reaction of the foundation as well as the axial force in the beam. It was investigated that bending moments get considerably affected due to the type of frame and loading.

Viladkar *et al*. (1991) used coupled finite-infinite elements for modeling of superstructure-soil mass interaction and considered the soil mass to behave nonlinearly. The far-field domain (soil) was best modeled by infinite elements with different types of decay. This approach is logical, more rational and easy for computer implementation. Noorzaei *et al*. (1994) presented the influence of strain hardening on soil-structure interaction analysis of a plane frame-combined footing-soil system taking into account the elasto-plastic behaviour of the compressible sub-soil and its strain hardening characteristics.

Fardis and Panagiostakos (1997) studied the effects of masonry infill on the global seismic response of reinforced concrete structure by numerical analysis. In this study, it was investigated that response spectra of elastic SDOF frames with nonlinear infill show that despite their apparent stiffening effect on the system infill reduce spectral displacements and forces mainly through their high damping in the first large post-cracking excursion.

Mandal *et al*. (1998) proposed a computational iterative scheme for studying the effect of soil-structure interaction on axial force, column moment and finally adjusted foundation settlement of two-bay two-storey building frame. The results obtained from this computational scheme were validated from experimental study. The proposed computational scheme could be used to predict the increase in axial force and moments in structural members due to the effect of soil- structure interaction

Stavridis (2002) presented the simplified analysis of layered soil-structure interaction. The stratified soil was represented with a linear elastic half space model with specific geometrical and elastic properties for its layers. Junvi *et al*. (2003) presented a coupling procedure of finite element (FE) and scaled boundary finite element (SBFE) for three-dimensional dynamic analysis of unbounded soil-structure interaction in the time domain.

Lehman *et al*. (2004) carried out a complete analysis of soil-structure interaction problems which includes a modelling of near surrounding of the building (near field) and a special description of the wave propagation process in large distance (far field). Singh *et al*. (2006) analyzed a 2-D reinforced concrete building frame to investigate the behaviour of multi-storeyed building frames with and without soil-structure interaction effect adopting spring analogy method in which appropriate spring constants were introduced at the foundation level replacing the fixed foundation condition.

Abate *et al*. (2007) investigated the dynamic seismic response of a fire station building structure considering soil plasticity and soil-foundation plastic hinges. The sliding at the soil foundation interface, uplifting of the foundation from the soil and mobilization of bearing capacity failure was taken into account. They investigated the effects of soil elasto-plastic constitution equation and foundation uplifting on the acceleration transmission on bending moments and shear forces in the structure.

Orakdoen and Girgin (2008) presented the performance evaluations of 3-D building frame strengthen by additional shear walls as a case study by considering the foundation effects. The nonlinear soil-structure interaction analysis of the building frame-soil system was presented using FEMA-440.

Puglisi *et al*. (2009) proposed a new model to investigate the behaviour of masonry infilled frames. The model is based on the theory of plasticity and the concept of an equivalent strut. The nonlinear hyperbolic model was adopted to account for the non-linear stress-strain behaviour of the soil mass. The results revealed the significance of the nonlinearity of soil mass in the response of the structure.

Chore *et al*. (2010) examined the effect of soil-structure interaction on a single-storey, two-bay space frame resting on a pile group embedded in the cohesive soil (clay) with flexible cap. A model is worked out separately for the pile foundation by using the beam elements, plate elements and spring elements to model the pile, pile cap and soil respectively. The stiffness obtained for the foundation is used in the interaction analysis of the frame to quantify the effect of soil-structure interaction on the response of superstructure.

The literature review reveals that there is no work done in the area of soil-structure interaction of infilled frame. The previous investigators Viladkar *et al*. (1991, 1994) and Noorzaei *et al*. (1994) made investigations on nonlinear plane frame-soil interaction system considering the soil to behave in nonlinear elastic and elasto-plastic manner and investigated the forces in the frame members and settlements in the soil mass using coupled finite-infinite elements.

The present work mainly investigates:

(i) The effect of inclusion of infill walls in further redistribution of forces in the frame members. Hence, the forces in the frame members were evaluated and compared with the plane frame-soil system using coupled finite-infinite elements. The soil mass is treated as homogeneous isotropic material and to follow nonlinear stress-strain relationship given by Kondner and Zelasko (1963).

(ii) Secondly, the effect of differential settlements is investigated on the forces in the frame members and contact pressure distribution below foundation caused by various load increments.

3. Coupled finite-infinite modelling of interaction system

3.1 Superstructure and foundation beam

The finite element idealization of plane frame-foundation-soil interaction system requires use of variety of isoparametric finite and infinite elements. Three node isoparametric beam-bending elements with three degrees of freedom (u, v, φ) per node are used to represent the members of the frame and the foundation beam.

3.2 Modelling of soil media

(a) Infinite Elements

Before the development of infinite elements, the conventional finite element method was used to model the unbounded domain of soil mass extending to infinity in one or two direction. The finite element mesh was truncated at some large but finite distance. This type of approximation to infinity proved to be computationally uneconomical, expensive and sometimes inaccurate. The formulation of infinite elements is available in the literature (Bettess 1977). The modelling of unbounded domain using coupled finite-infinite elements has proved computationally economical (Viladkar *et al*. 1991). However, the location of truncation boundary between finite and infinite elements is the most important aspect, especially in case of plain strain type of problem. The truncation boundary is located by trial and error in a systematic manner. The infinite elements with different types of decay pattern are able to model the far field behaviour quite accurately.

(b) Decay Pattern and Positioning of Reference Pole for an infinite element

The formulation of the infinite element depends upon the type of decay pattern to be adopted. In general, the type, which is adopted, is ($1/r^n$) with $1/r$, $1/r^2$ and $1/\sqrt{r}$ as the special cases of decay, where 'r' is the distance from the pole to a general point within an element (Kumar 1985). The geometry and the unknown variable expansions (displacements) involved in the mapped infinite element technique are both referred to the same point or a set of points formed as a pole(s). Therefore, the geometry and the physical characteristics of the problem must both be taken into account, while positioning the pole(s). This reference pole must be exterior to the infinite element.

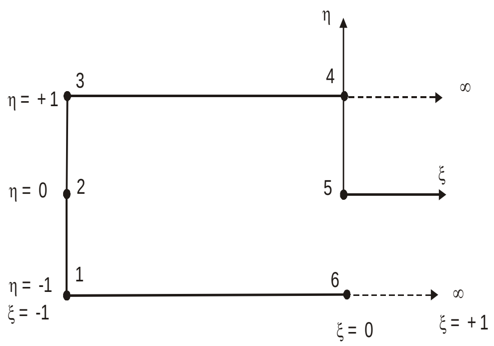
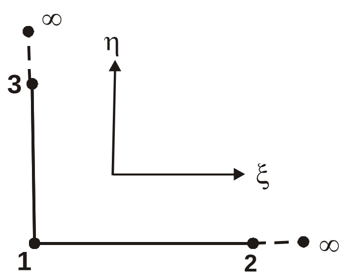
The unbounded domain of the soil mass is represented by conventional eight node plane strain finite elements with two degrees of freedom per node (u, v) coupled with six node infinite elements with 1/r type decay having two degrees of freedom per node (u, v). A three node doubly infinite element is used as a corner element in the finite-infinite element mesh (Viladkar *et al*. 1991). Table 1 shows various finite elements used and their shape functions.

4. Nonlinear elastic hyperbolic soil model

Mainly, there are two types of materials involved in the present problem: reinforced concrete and the soil. The stiffness of the reinforced concrete is much higher in comparison to that of soil. Therefore, in this study, material non-linearity of the soil mass has been considered while the reinforced concrete has been assumed to follow the linear stress-strain relationship.

The nonlinearity of soil mass is represented using the hyperbolic model proposed by Kondner and Zelasko (1963). The model is used in the literature by Duncan and Chang (1970) for nonlinear stress analysis of soil. Here, the tangent modulus (E_T), of the soil mass at any stress level is represented as:

Table 1 Shape functions for isoparametric finite and infinite elements

Element Type	Element Figure	Shape Functions
Three node beam element 3 d.o.f. (u, v and θ) per node		$N_1 = \xi(1-\xi)/2$ $N_2 = (1-\xi^2)$ $N_3 = \xi(1+\xi)/2$
Six node infinite element with 1/r type decay* 2 d.o.f. (u,v) per node		$N_1 = \frac{\xi\eta(1-\eta)}{(1-\xi)}$ $N_2 = \frac{-2\xi(1-\eta^2)}{(1-\xi)}$ $N_3 = \frac{-\xi\eta(1+\xi)}{(1-\xi)}$ $N_4 = \frac{(1+\xi)\eta(1+\eta)}{2(1-\xi)}$ $N_5 = \frac{(1+\xi)(1-\eta^2)}{(1-\xi)}$ $N_6 = \frac{-(1+\xi)\eta(1-\eta)}{2(1-\xi)}$
Three node doubly infinite element with 1/ r type decay* 2 d.o.f. (u,v) per node		$N_1 = \frac{(\xi\eta+3)(-1-\xi-\eta)}{(1-\xi)(1-\eta)}$ $N_2 = \frac{2(1+\xi)}{(1-\xi)(1-\eta)}$ $N_3 = \frac{2(1+\eta)}{(1-\xi)(1-\eta)}$

* The distance ‘r’ is measured from a reference pole to a general point within an element

$$E_T = \left[1 - \frac{R_f \left(1 - \sin\phi\right)\left(\sigma_1 - \sigma_3\right)}{2\left(c\cos\phi + \sigma_3 \sin\phi\right)} \right]^2 E_i \tag{1}$$

Where,

$$E_i = K\, P_a \left(\frac{\sigma_3}{P_a} \right)^n \tag{2}$$

Various parameters representing the non-linearity of soil mass are:
E_i = initial tangent modulus
c = cohesion
P_a = atmospheric pressure
σ_1, σ_3 = major and the minor principal stresses
ϕ = angle of internal friction
K = modulus number
n = exponent determining the variation of initial tangent modulus E_i, with confining pressure σ_3.

R_f = failure ratio = $\dfrac{(\sigma_1 - \sigma_3)_f}{(\sigma_1 - \sigma_3)_{ult}}$

Where,
$(\sigma_1 - \sigma_3)_f$ = compressive strength
$(\sigma_1 - \sigma_3)_{ult}$ = asymptotic value of deviatoric stress

These parameters have been taken from the literature (Noorzaei *et al*. 1994) and indicated in Fig. 1. Poisson's ratio has been kept constant in the analysis. A load, at which yielding just starts in a soil element is determined. Beyond this load value, the results obtained would not be reliable because the soil mass exhibits elasto-plastic behaviour. The model has been incorporated into the computer code developed for the nonlinear interaction analysis (NLIA).

5. Computational algorithm

The mixed (incremental-iterative) technique is adopted for the nonlinear elastic analysis of the present problem. The vertical load is applied in increments. The stiffness matrix of the soil mass is regenerated at the beginning of the first iteration of every load increment. The computational steps involved are provided here.

First Load Increment:

Let $\{\Delta P\}$ and [K] denote the incremental force vector and the stiffness matrix of the system and $\{\Delta\delta\}$, $\{\Delta\varepsilon\}$ and $\{\Delta\sigma\}$ denote the incremental deformations, strains and stresses respectively.

(i) First iteration: Evaluate incremental deformations as

$$\{\Delta P\}=[K]_1^1\{\Delta\delta\}_1^1 \tag{3}$$

(ii) Solve (Eq. (3)) for {Δδ} and evaluate the incremental strains and stresses as

$$\{\Delta\varepsilon\}_1^1=[B]\{\Delta\delta\}_1^1$$

$$\{\Delta\sigma\}_1^1=[D]_1^1\{\Delta\varepsilon\}_1^1 \tag{4}$$

(iii) Accumulate the current incremental stresses and converged stresses upto previous iteration into temporary stresses as

$$\{\sigma\}_{1,temp}^1=\{\sigma\}_{0,acc}^1+\{\Delta\sigma\}_1^1 \tag{5}$$

(iv) Evaluate principal stresses σ_1 and σ_3 using above temporary stresses.

(v) Evaluate tangent modulus of soil mass (E_T) for the current stress level using (Eq. (1)).

(vi) Modify [D] matrix on the basis of tangent modulus and evaluate modified stresses as

$$\{\Delta\sigma\}_{1,temp}^1=[D]_{1,\mathrm{mod}}^1\{\Delta\varepsilon\}_1^1 \tag{6}$$

(vii) Accumulate stresses as

$$\{\sigma\}_{1,acc}^1=\{\sigma\}_{0,1cc}^1+\{\Delta\sigma\}_{1,\mathrm{mod}}^1 \tag{7}$$

(viii) Evaluate residual force {Ψ} as

$$\{\psi\}_1^1=-\int[B]\{\sigma\}_{1,acc}^1\,dV+\{\Delta P\}_1 \tag{8}$$

Solve the set of equations with these residual forces to achieve equilibrium.

(ix) Accumulate the displacements

$$\{\delta\}_{1,acc}^1=\{\delta\}_{0,acc}^1+\{\Delta\delta\}_1^1 \tag{9}$$

(x) Check for convergence. In nonlinear analysis, the norm of displacements or norm of residual forces is selected for convergence. The present analysis considers the norm of residual forces. A tolerance limit of 1% is selected for the residual force. The number of iterations for each load increment was fixed as 20 where the iterations must stop, if the solution does not converge. When the solution converges for a load increment, switch over to next load increment, otherwise go to next iteration and repeat the steps (i) to (vii). For subsequent load increments, the stiffness matrix is modified on the basis of the stresses accumulated at the end of previous load increment and the above process is repeated till convergence takes place. The total vertical load was applied in seven load increments. The convergence took place after 7 to 10 iterations for each load increment.

6. Nonlinear elastic interaction analysis software

6.1 Features of the developed software

The computer programme has been developed in FORTRAN-90 for nonlinear elastic interaction analysis of plane frame-foundation beam-soil and infilled frame-foundation beam-soil systems under static vertical loading. It includes variety of elements needed for the discretization of the domain of the interaction system. Thus, the programme has multi-element and multiple degrees of freedom features. The beam element included in the programme is the modified form of the beam-bending element (Hinton and Owen 1977), which includes one additional degree of freedom to take care of axial deformation in the frame members. The discretization of infill panels uses conventional eight noded isoparametric elements whereas the coupled finite-infinite elements are used to discretize the soil mass (Viladkar *et al*. 1991).

The programme takes into account the nonlinearity of soil mass using mixed incremental-iterative nonlinear solution algorithm. The gauss-Legendre scheme has been employed for the evaluation of element stiffness of finite and infinite elements both. The flow chart for nonlinear elastic interaction analysis is depicted in Fig. 1.

6.2 Validation of software

The validation of a developed computer program is a necessary step in the process of software development. The linear and nonlinear soil-structure interaction analyses software have been validated by solving problem of (i) two dimensional portal frame of Weaver and Gere (1986)[45] and (ii) two-bay five-storey plane frame-foundation beam-soil system already available in the literature (Noorzaei *et al*. 1994)[44].

(i) Validation Problem of linear analysis

The problem of two dimensional portal frame of Weaver and Gere (1986)[45] is analyzed to validate linear analysis program. The geometry and loading on the portal frame is shown in Fig. 2. The geometrical and material properties of are provided in Table 2. The axial force, shear force and bending moments in the portal frame are evaluated and provided in Table 3. The results are found to be in close agreement.

(ii) Validation problem of nonlinear analysis

The geometrical and material properties of the frame and soil are provided in Table 4. The beams and the columns are discretized by three node beam bending element and soil is discretized by eight node plane strain element coupled with six node infinite element. The linear and nonlinear interaction analyses of plane frame-foundation beam-soil system have been carried out for only vertical loading. A uniformly distributed loading of intensity 15 kN/m is applied on floor and foundation beams as shown in the Fig. 3. The axial force in the columns has been evaluated and results are provided in Table 5. The results of analyses are compared with (Noorzaie *et al*. 1994)[44] and are found to be in close agreement.

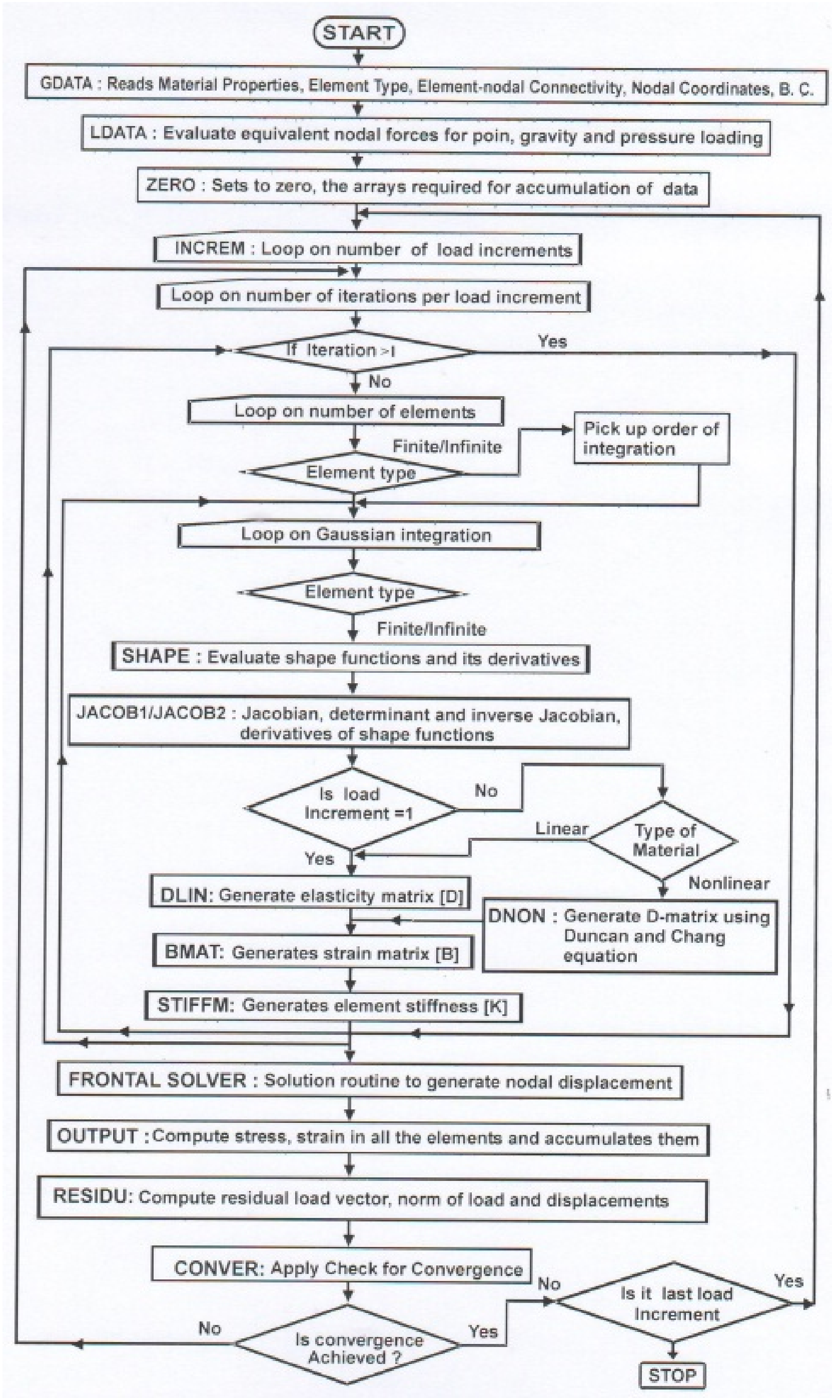


Fig. 1 Software for nonlinear elastic soil-structure interaction analysis

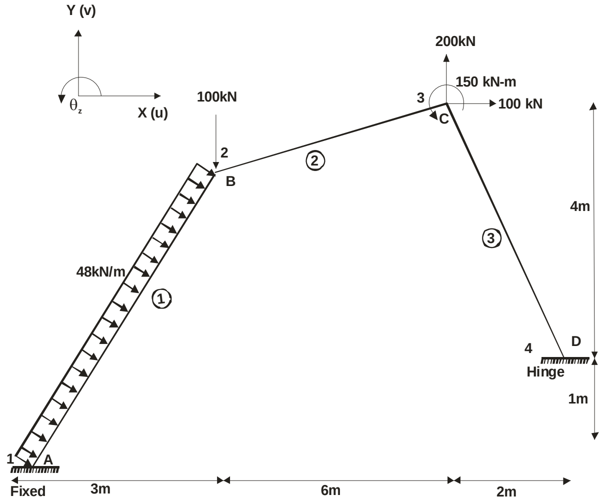


Fig. 2 Portal frame of Weaver and Gere (1986)[45]

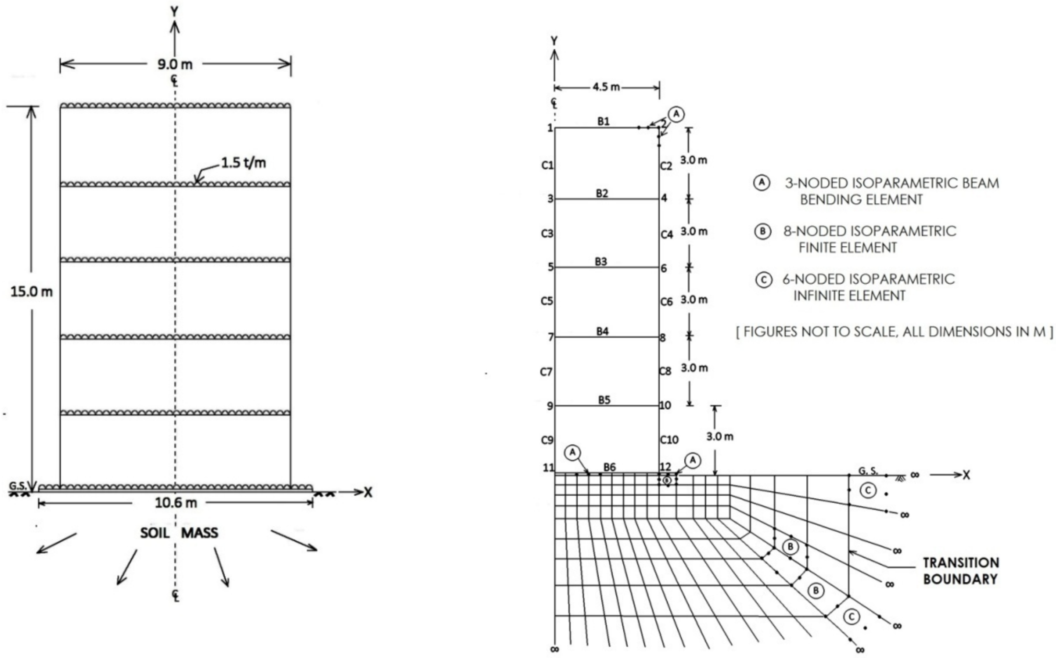
Table 2 Geometrical and material properties portal frame of Weaver and Gere (1986)[45]

Property	Value
Young's modulus (E)	2×10^8 kN/m^2
Cross-sectional area (A)	0.02 m^2
Moment of Inertia (I)	3×10^{-3} m^4
Modulus of Rigidity (G)	0.875×10^8 kN/m^2

Table 3 Comparison between results of Weaver and Gere frame and present study

Member	End –1			End – 2		
	Axial force kN	Shear force kN	Bending moment kN-m	Axial force kN	Shear force kN	Bending moment kN-m
1	-74.80*	+294.90*	+542.86*	+74.80*	-54.90*	+331.65*
	(-74.60)	(+294.79)	(+542.96)	(+74.60)	(-55.20)	(+331.96)
2	-108.46*	-110.58*	-331.65*	+108.46*	+110.58*	-340.97*
	(-108.30)	(-110.36)	(-331.96)	(+108.30)	(+110.36)	(-340.67)
3	-60.38*	+42.70*	+196.97*	+60.38*	-42.70*	0.00*
	(-60.42)	(+42.42)	(+196.77)	(+60.42)	(-42.42)	(0.00)

***** Gere and Weaver (1986)[45] Figures in brackets – Present study



(a) (b)

Fig. 3 (a) Structure-combined footing-soil system (b) Finite-infinite element idealization of plane frame-soil system

Table 4 Geometrical and material properties of frame and soil (Noorzaei et al. 1994)[44]

S. No.	Structural components/parameters	Properties/size of component
1	All floor beams	0.25 m x 0.40 m
2	Columns of storeys I and II	0.40 m x 0.40 m
3	All other columns	0.35 m x 0.35 m
4	Foundation beam	0.65 m x 0.35 m
5	Number of storeys	5
6	Number of bays	2
7	Storey height	3.0 m
8	Bay width	4.5 m
9	Modulus of elasticity of concrete	2.1×10^7 kN/m^2
10	Poisson's ratio of concrete	0.20
	Soil Properties	
11	Initial tangent modulus (E_i)	15000.0 kN/m^2
12	Poison's ratio (ν)	0.35
13	Cohesion (c)	0.0 kN/m^2
14	Angle of internal friction (Φ)	37.5^0
15	Modulus number (k)	500.0
16	Exponent (n)	0.92
17	Failure ratio (R_f)	0.85
18	Atmospheric pressure (P_a)	100.0 kN/m^2

Table 5 Comparison of axial forces (kN) in columns of two-bay five-storey plane frame soil system under vertical loading

Storey level	Member	Noorzaei *et al.* (1994)			Present study		
		NIA	LIA	NLIA	NIA	LIA	NLIA
V	C_1	3.54	2.61	2.33	3.48	2.54	2.29
	C_2	3.34	4.14	4.42	3.29	4.12	4.45
IV	C_3	6.97	4.79	4.17	7.02	4.73	4.05
	C_4	6.63	8.71	9.33	6.60	8.68	9.38
III	C_5	10.28	7.07	6.11	10.34	6.98	6.17
	C_6	9.97	13.19	14.15	10.05	13.23	14.10
II	C_7	13.67	9.24	7.93	13.51	9.84	7.90
	C_8	13.33	17.76	19.07	13.42	17.12	19.02
I	C_9	17.10	11.39	9.65	17.23	11.45	9.58
	C_{10}	16.65	22.36	24.10	16.75	23.75	23.95

7. Interaction analysis

7.1 Problem under investigation

In the present investigation, the linear and nonlinear interaction analyses of two-bay two-storey plane frame-foundation beam-soil system (FS) and infilled frame-foundation beam-soil system (FSP) have been carried out. The geometrical details of the frame and the infill are given in Fig. 4.

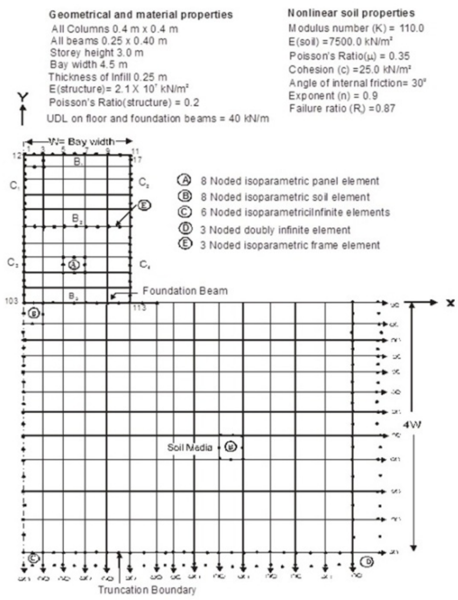


Fig. 4 Finit-infinite element discretization of infilled frame-soil system

7.2 Location of truncation boundary

In any coupled finite-infinite element formulation, the most important aspect is the location of truncation boundary (the common junction between the finite and infinite element layer), which is found by trial and error (Viladkar *et al*. 1994). To start with, the soil mass is discretized with 2-3 layers of finite elements and one layer of infinite layer. Thereafter, each trial involves shifting of the position of infinite layer by including an additional finite element layer above it. The central point deflection below the inner column is compared with the result provided by finite element discretization of the whole domain to access the correct location of the truncation boundary. In this analysis, eighteen layers of finite elements were required for finite element analysis extending to depth of nine times the bay width (w) whereas coupled analysis required eleven layers of finite elements extending to depth of about four times the bay width, thereafter one layer of infinite elements was attached below this as shown in Fig. 2. Moreover, the displacements of the free nodes of the infinite elements were found to be almost negligible. For location of truncation boundary, the behaviour of soil mass is treated as linear elastic.

7.3 Nonlinear analysis

The nonlinear interaction analysis has been carried out for the problem under investigation. The vertical load of 40 kN/m is applied on floor beams of the frame and the foundation beam. The mixed incremental-iterative nonlinear solution algorithm was used to account for non-linearity of soil mass. The total load was applied in seven load increments. The load increments are chosen depending upon the nature of the stress-strain curve, material properties etc. of the soil mass and this requires trial and error. In the present analysis, the total vertical load P of 612 kN is applied in seven load increments. Initially the behaviour of the interaction system is linear elastic up to certain load value corresponding to the first load increment of 30% of total load (P). Thereafter, the curve is nonlinear and therefore the remaining load increments are smaller (15, 15,10,10,10, 10% of total load P) as compared to initial linear elastic segment of the curve.

7.3.1 Settlements below the foundation beam

The variation of vertical settlements below foundation beam of plane frame-soil system is depicted in Fig. 5(a) in the non-dimensional form for the load increments 1, 3, 5 and 7 of nonlinear interaction analysis (NLIA). These profiles are compared with linear interaction analysis (LIA). The maximum settlement occurs below the central column and it decreases marginally towards the outer column. This causes differential settlement of small value. The total settlement below the central column due to NLIA is nearly 2.25 times as compared to LIA. It is found that the value of tangent modulus of soil (E_T) increases with load increments. The stiffness of the soil will be low because of lower values of E_T compared with the initial tangent modulus of soil (E_i).

The inclusion of infill in the system causes almost uniform settlements below entire length of the foundation beam as depicted in Fig. 5(b) for LIA and NLIA except for the seventh load increment. This is because of assumed absolute rigid connection between the infill and the foundation beam. A marginal increase of nearly 7% in vertical settlements is observed as compared to plane frame-foundation beam-soil system due to increase in dead load of infill panels.

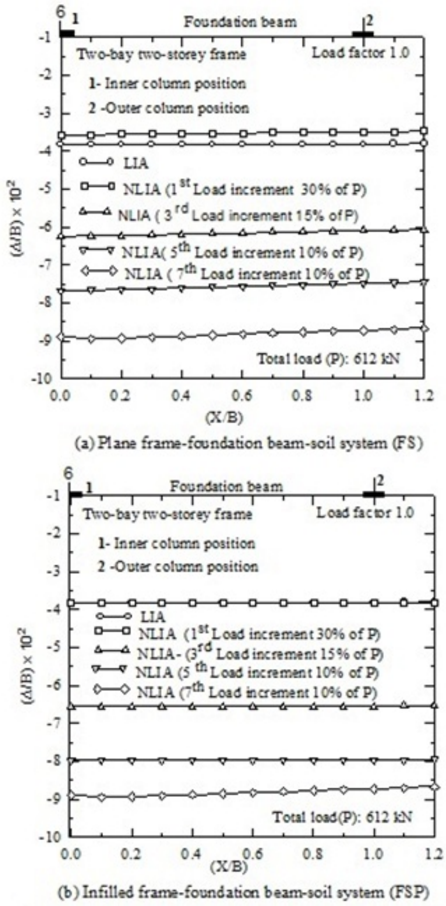


Fig. 5 Variation of vertical settlements below foundation beam

7.3.2 Contact pressures below foundation beam

The contact pressure distribution below foundation beam of the plane frame-soil system is shown in Fig. 6(a) in the non-dimensional form. The contact pressures below the central column due to linear soil behaviour and nonlinear soil behaviour for the seventh load increment are almost same. NLIA provides marginally higher contact pressures (nearly 12%) at the edge of the beam. Due to the inclusion of infill in the system, the contact pressure is relieved at the center of the beam and marginal increase is found at the edge as depicted in Fig. 6(b). A decrease of nearly 25% is found at the center and increase of nearly 6% is observed due to LIA and NLIA both.

7.3.3 Axial force in the columns

Table 6 shows the value of axial force in the columns due to various analyses. The bare frame analysis (BFA) is the conventional frame analysis carried out considering the column fixed at their bases.

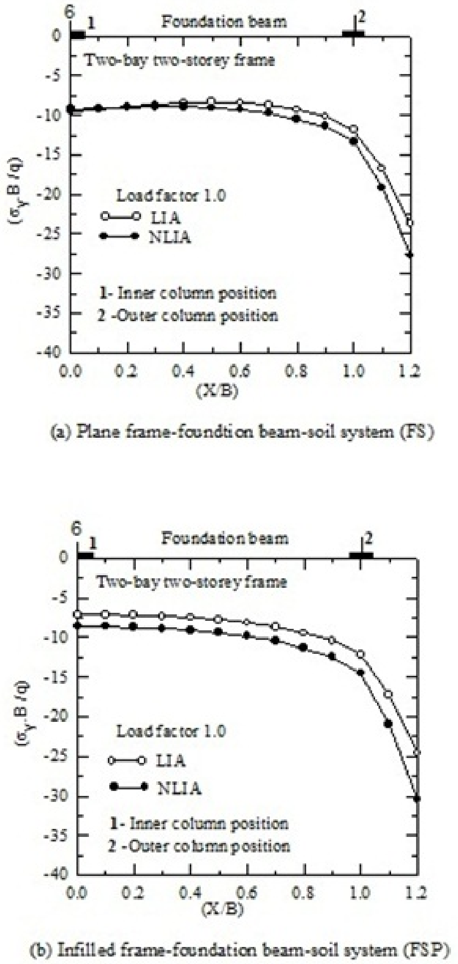


Fig. 6 Contact pressures distribution foundation beam

Table 6 Axial force (kN) in the columns of frame-soil system for various analyses

Storey Level	Member	BFA (1)	LIA (FS) (2)	% Diff. (1)-(2) (3)	LIA (FSP) (4)	NLIA (FS) (6)	% Diff. (2)-(6) (7)	NLIA (FSP) (8)	% Diff. (6)–(8) (9)
II	C_1	95.21	69.87	-26.62	15.75	67.54	-3.33	15.36	-77.25
	C_2	84.79	110.55	+30.38	78.08	113.65	+2.80	80.79	-28.91
I	C_3	187.4	129.13	-31.10	23.76	123.42	-4.42	24.47	-80.17
	C_4	172.58	230.96	+33.83	247.23	237.82	+2.97	260.75	+9.64

The comparison of axial forces due to BFA and LIA reveals that the interaction effect causes redistribution of the forces in the column members. The inner columns are relieved of the forces and corresponding increase is found in the outer columns due to differential settlements. The axial force due to LIA varies in the range of about –31 to +34%. The nonlinear interaction analysis (NLIA) provides marginally higher values of axial forces in the outer columns and marginally lower values in the inner columns.

The inclusion of infill in the system causes further redistribution of axial forces. The axial forces in the inner and outer columns decrease significantly. Table 2 shows that the decrease of nearly 30 to 82% is found due to LIA in the columns except the outer column of the first storey where a marginal increase of nearly 7% is observed. NLIA also provides almost similar variations.

Table 7 shows the variation of axial forces in the columns with differential settlements (difference between settlements of point below inner column of first storey and point below the outer column of the first storey) due to nonlinear interaction analysis.

It is observed that the increase in the differential settlement due to load increments causes increase in the axial force in the columns, which initially vary linearly up first load increment value (30% of total load) and thereafter vary nonlinearly for remaining load increments.

Table 7 Axial force (kN) in columns due to differential settlements of soil mass

Storey Level	Member	Plane frame-soil system (NLIA-FS)				**Infilled frame-soil system (NLIA-FSP)**			
		1st LF 0.3	3rd LF 0.6	5th LF 0.8	7th LF 1.0	1st LF 0.3	3rd LF 0.6	5th LF 0.8	7th LF 1.0
		3.18 mm	6.36 mm	8.27 mm	10.27 mm	0.10 mm	0.15 mm	0.20 mm	0.30 mm
II	C_1	19.95	40.00	53.49	67.54	4.59	9.17	12.36	15.36
	C_2	34.32	68.47	90.96	113.65	16.23	48.74	64.84	80.79
I	C_3	36.30	72.85	98.15	123.42	6.78	13.65	18.29	24.47
	C_4	72.00	143.69	190.63	237.82	78.64	156.63	208.68	260.75

LF- Load factor

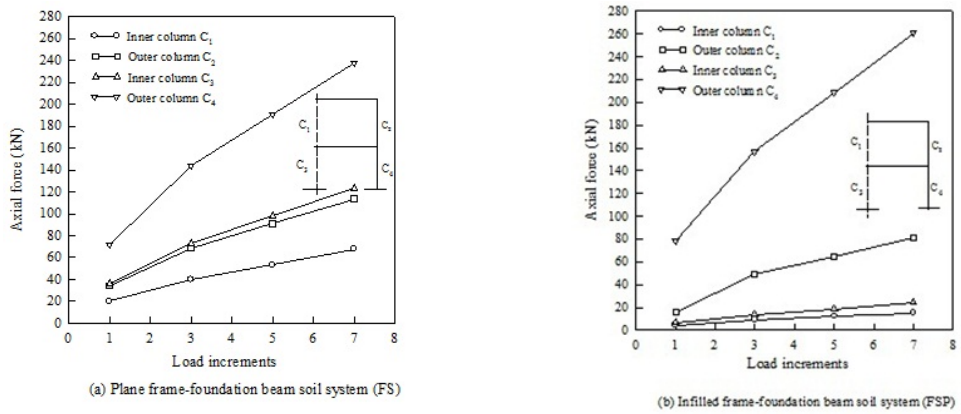


Fig. 7 Variation of axial forces in columns load increments (NLIA)

Figs. 7(a) and 7(b) shows the variation of axial force in the columns with the load increments for plane frame-foundation beam-soil system and infilled frame-foundation beam soil system due to NLIA.

7.3.4 Bending moments in the outer columns

Table 8 shows the values of bending moment in outer columns of plane frame-foundation beam-soil system.

The interaction effect causes significant increase in bending moments in the outer columns. This is because of the transfer of moments from the interior columns to the outer columns due to differential settlements. The increase of nearly 230% is found due to LIA at the roof level of the outer column of the first storey and nearly 101% for top storey. NLIA provides marginally higher values (nearly 4 to 11%) compared to LIA.

Table 8 Bending moments (kN-m) in outer columns for various analyses

Storey Level	Member	BFA (1)	LIA (FS) (2)	% Diff. (1)-(2) (3)	LIA (FSP) (4)	NLIA (FS) (6)	% Diff. (2)-(6) (7)	NLIA (FSP) (8)	% Diff. (6)-(8) (9)
II	C_2	51.03	102.75	101.35	2.40	108.18	5.29	2.44	**
		38.77	63.64	64.14	2.11	66.07	3.81	2.19	**
I	C_4	21.56	70.99	229.26	2.51	76.51	7.77	2.66	**
		11.04	114.25	**	61.05	126.55	10.76	64.85	-48.75

** Very high difference in values

Table 9 Bending moments (kN-m) in outer columns due to differential settlement

Storey Level	Member	Plane frame-soil system (NLIA-FS)				**Infilled frame-soil system (NLIA-FSP)**			
		1^{st} LF 0.3	3^{rd} LF 0.6	5^{th} LF 0.8	7^{th} LF 1.0	1^{st} LF 0.3	3^{rd} LF 0.6	5^{th} LF 0.8	7^{th} LF 1.0
		3.18 mm	6.36 mm	8.27 mm	10.27 mm	0.10 mm	0.15 mm	0.20 mm	0.30 mm
II	C_2	33.00	65.72	86.84	108.18	0.73	1.47	1.95	2.44
		20.07	40.01	52.99	66.07	0.66	1.31	1.75	2.19
I	C_4	23.51	46.73	61.51	76.51	0.80	1.60	2.13	2.66
		39.21	77.77	101.91	126.55	19.62	39.03	51.94	64.85

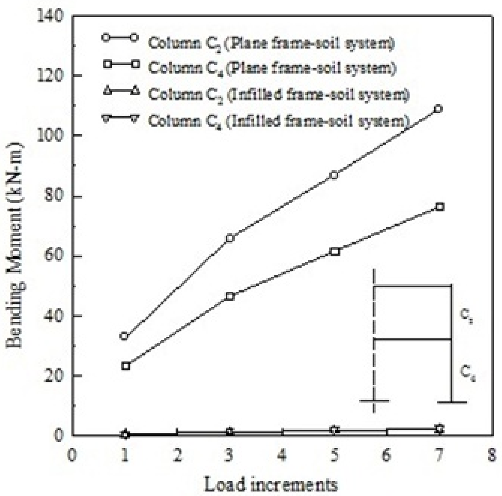


Fig. 8 Variation of B.M's at roof level of outer columns with load increments

The bending moments in the outer columns of infilled frame-foundation beam-soil system significantly reduce to very low values due to inclusion of the infill, which causes increase in the stiffness of the system**.**

Table 9 shows the variation of bending moments in the columns with differential settlements due to nonlinear interaction analysis.

It is observed that the increase in the differential settlements due to load increments causes linear increase in the bending moments in the outer column up to a particular load value and nonlinear variation is observed for further load increments.

Fig. 8 shows the variation of bending moments in the outer columns with the load increments for plane frame-foundation beam-soil system and infilled frame-foundation beam soil system due to NLIA.

7.3.5 Bending moments in the floor beams

Table 10 shows the value of bending moment at the inner and outer end of the floor beams of plane frame-foundation-soil and infilled frame-foundation beam-soil systems. The interaction effect suggests that there is transfer of bending moments from the inner end of the beam to the outer end at all floor levels due to differential settlements, which increase nearly by 101 to 123%. The reversal in sign of bending moment is observed at the junction between the beams of first storey with interior column. A significant increase of nearly 123% is found due to LIA at the outer end of first floor beam and nearly 101% in the top floor beam. NLIA provides higher values of the bending moment at the outer end of the beams.

The values of bending moments in the entire length of all floor beams significantly reduce to very low values due to inclusion of infill in the system. The reversal in the sign of bending moments, as observed in the case of plane frame-foundation beam-soil system, is reverted back. Thus, the resulting signs are same as that due to bare frame analysis.

Table 10 Bending moments (kN-m) in floor beams for various analyses

Storey Level	Member	BFA (1)	LIA (FS) (2)	% Diff. (1)-(2) (3)	LIA (FSP) (4)	NLIA (FS) (6)	% Diff. (2)-(6) (7)	NLIA (FSP) (8)	% Diff. (6)–(8) (9)
II	B_1	74.49	11.66	**	1.92	11.26	-3.43	1.90	**
		-51.03	-102.74	101.34	-2.41	-108.17	5.28	-2.43	**
I	B_2	70.29	-10.96	*	0.66	-11.85	8.12	0.61	*
		-60.33	-134.64	123.17	-4.62	-142.58	5.89	-4.83	**

* Reversal in sign ** Very high difference in values

Table 11 Bending moments (kN-m) in floor beams due to differential settlements

Storey Level	Member	Plane frame-soil system (NLIA-FS)				**Infilled frame-soil system (NLIA-FSP)**			
		1st LF 0.3	3rd LF 0.6	5th LF 0.8	7th LF 1.0	1st LF 0.3	3rd LF 0.6	5th LF 0.8	7th LF 1.0
		3.18 mm	6.36 mm	8.27 mm	10.27 mm	0.10 mm	0.15 mm	0.20 mm	0.30 mm
II	B_1	3.45	6.92	9.06	11.26	0.56	1.14	1.53	1.90
		-33.0	-65.73	-86.82	-108.17	-0.73	-1.44	-1.92	-2.43
I	B_2	-4.36	-8.29	-9.90	-11.85	0.19	0.37	0.48	0.61
		-43.58	-86.75	-114.50	-142.58	-1.47	-2.91	-3.88	-4.83

Table 11 shows variation of bending moments in the floor beams with differential settlements due to nonlinear interaction analysis. The bending moments in the floor beams also increase in linear manner with the increase in differential settlements due to certain load increments, thereafter, nonlinear variation is found for remaining load increments.

Fig. 9 shows the variation of bending moments in the floor beams with the load increments for plane frame-foundation beam-soil system due to NLIA.

7.3.6 Bending moments in the foundation beam

Fig. 10(a) exhibits the distribution of bending moments along the foundation beam of plane frame-foundation beam-soil system for linear and nonlinear interaction analyses. The variation resembles the behaviour of the beam subjected to column loads from top and upward soil pressure beneath.

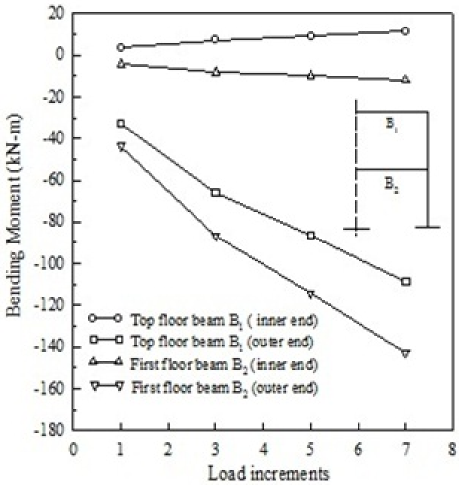


Fig. 9 Variation of bending moments in floor beams for plane frame-soil system (FS)

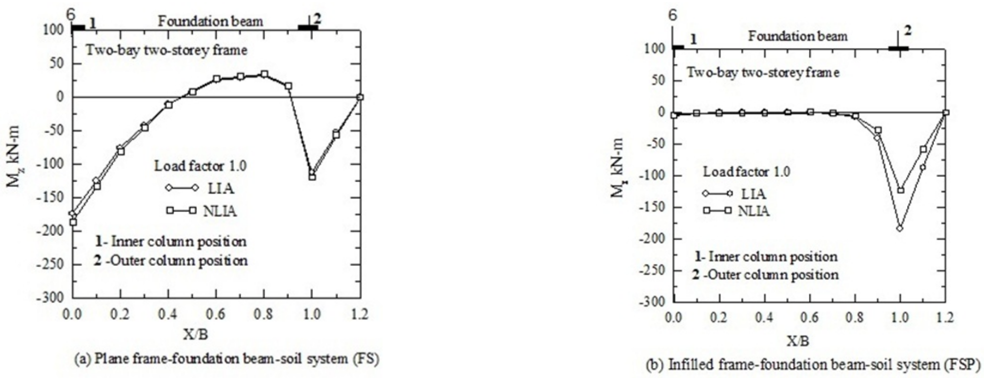


Fig. 10 Variation of Bending moments in foundation beam

Fig. 10(b) depicts the variation of bending moments along the foundation beam of infilled frame-foundation beam-soil system. Almost negligible bending moments are found in the region of direct contact with the infill and in the remaining portion, negative bending moments of the same order are observed as that in case of plane frame-foundation beam-soil system. In the present analysis, it has been assumed that the infill and the bounding frame will always remain in contact and that is why common nodes have been selected. However, this will not apply at the interface i.e., the contact surface between the three nodes of panel elements of infill panel and the three node beam elements of building frame members. It is likely that separation may occur between them. This condition of assumed rigid connection can be satisfied more closely with a finer mesh.

8. Conclusions

In reality, soil mass behaves in nonlinear manner. The proposed methodology for nonlinear interaction analysis considers the nonlinearity of the soil mass, which yields more realistic structural behaviour and more accurate results as compared to linear interaction analysis. From the interaction studies of plane frame-foundation beam-soil system and the infilled frame-foundation beam-soil system, the following conclusions are drawn:

(i) The forces in the various frame members due to interaction analysis are considerably different from the conventional frame analysis

(ii) The differential settlements increase with the increase in load increments and bilinear variation is found.

(iii) The axial forces in the columns, bending moments in the outer columns, bending moments in the floor and foundation beam increase with the increase in differential settlements due to load increments of nonlinear analysis and the bilinear variation is found.

(iv) The inclusion of infill in the system causes further redistribution of forces in the frame members and significant decrease is found in the forces.

(v) The vertical settlements below the foundation beam of infilled frame-soil system are found almost uniform and are marginally higher compared to plane frame-foundation beam–soil system. The vertical settlements due to non-linearity of the soil mass are almost 2.25 times to that of linear behaviour. It is found that the value of tangent modulus of soil (E_T) increases with load increments. The stiffness of the soil is low because of lower values of E_T compared with the initial tangent modulus of soil (E_i).

(vi) The contact pressure at the center of the foundation beam is relieved due to inclusion of infill and marginal increase is found at the edge. The contact pressures increase with the increase in differential settlements of the soil mass.

The proposed research work will lead to a more rational approach for accurate analysis and design of building frames. The conclusions and formulations will prove useful for designing building frames considering the effect of soil-structure interaction together with infill wall-frame interaction in comparison to conventional approach of building frame design.

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