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EFFECT OF DIFFERENTIAL SETTLEMENTS ON NONLINEAR INTERACTION BEHAVIOUR OF PLANE FRAME-SOIL SYSTEM

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ABSTRACT

The building frame, foundation and soil mass form a complete structural system to resist the external loads. The mechanics of soil-structure interaction takes place between these components. The superstructure, foundation and soil mass can be considered as a single integral compatible structural unit for carrying out the interaction analysis to predict more realistic behaviour. The stress-strain characteristics of the supporting soil play a vital role in the interaction analysis. The resulting differential settlements of the soil mass are responsible for the redistribution of forces in the superstructure. In the present work, the nonlinear interaction analysis of a two-bay two-storey plane building frame-soil system has been carried out using the coupled finite-infinite elements. The nonlinear constitutive hyperbolic soil model available in the literature is adopted to model the nonlinear behaviour of the soil mass. The structural behaviour of the interaction system is investigated as the shear forces and bending moments in superstructure get significantly altered due to differential settlements of soil mass.

Keywords: model, soil-structure interaction, nonlinear analysis, differential settlement, plane frame, shear forces, bending moments.

1. INTRODUCTION

The interaction behaviour of frame-soil system is mainly governed by the mechanical response of the compressible subsoil. In reality, the stress-strain response of soil mass is nonlinear and needs a numerical technique to model its behaviour. The differential settlement of the soil mass is responsible for redistribution of forces in the frame members. The structural behaviour of such an interaction system needs to be investigated as the shear forces and bending moments in superstructure get significantly altered due to differential settlements.

The finite element method is a powerful numerical tool for numerical analysis of any soil-structure interaction problem. 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 uneconomical and expensive. The coupled finite-infinite isoparametric elements are numerically very powerful and computationally economical to model the far field behaviour of the unbounded extent of the soil mass with proper location of truncation boundary (the common junction between the finite and infinite elements). The infinite element with exponential decay pattern is adopted to model the far field behaviour of the soil mass.

2. REVIEW OF LITERATURE

Several investigators have studied the influence of the phenomenon of soil-structure interaction in framed structures and investigated that the force quantities are revised due to interaction. Lee and Brown [1] presented an interaction analysis of a seven-storey; three-bay framed structure in which the soil mass was treated as a Wrinkler's or elastic half space medium. King and Chandrasekaran [2] provided the solution for a rafted plane frame, in which the frame and the combined footing were discretized into beam bending elements and the soil mass into plane rectangular elements.

Brown [3] examined the effect of sequence of construction on the interaction behaviour and found that the effective stiffness of a building during construction is about half the stiffness of the completed structure. Jain [4] proposed an economical iterative procedure for building frames and found significant reduction in differential settlements and consequent additional moments. Desai [5] presented a finite element procedure for the general problem of three-dimensional soil-structure interaction involving non-linearity caused by material behaviour, geometrical changes and interface behaviour. Desai and Sargand [6] developed hybrid finite element procedure for nonlinear elastic and elasto-plastic soil-structure interaction analysis including simulation of construction sequences. Desai and Lightner [7] used this approach for carrying out nonlinear elastic and elasto-plastic interaction analysis of some engineering problems using Von-Mises and Drucker-Prager's yield criteria.

Aljanabi [8] studied the interaction of plane frames with an elastic foundation, of Wrinkler's type, having normal and shear modulli of subgrade reaction. Lai and Booker [9] presented an iterative process based upon a hybrid 'residual force' method for solving elasto-plastic soil-structure interaction analysis. In this approach the soil and the structure were treated as separate bodies and related only by compatibility of displacements and equilibrium of forces at soil-structure interface.

Viladkar [10] employed a coupled finite-infinite element formulation to highlight the advantage of using the infinite elements to study the interaction analysis of framed structures. Chandrashekhara [11] used photo-elastic approach to study the behaviour of frame structure-foundation-soil interaction system. A parametric study of the framed structure-foundation beam-soil interaction problem was undertaken by considering the stiffness of the superstructure, the stiffness of the foundation beam and the stiffness of the soil.

Noorzaei [12] presented nonlinear soil-structure interaction analysis of two-bay five-storey plane frame. The soil was considered to behave in nonlinear elastic manner and well-known hyperbolic soil model was adopted. Noorzaei [13] presented 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. Dasgupta [14] and Dutta [15] investigated the effect of three influencing parameters namely relative flexural stiffness of columns with respect to beams, number of storeys and number of bays on the column axial force and column moment of three-dimensional building frames.

Cremer [16] presented nonlinear soil-structure interaction analysis of building frame resting on cohesive soil using macro-element for shallow foundation. The element describes the behaviour in the near field of the foundation under cyclic loading, reproducing the material nonlinearity of the soil under the foundation as well as the geometrical nonlinearity at the soil-structure interface. Roy and Dutta [17] investigated the effect of differential settlements on the forces in the frame members. It was investigated that the differential settlements among various footings result in a redistribution of the column loads, the amount of which depends on the rigidity of the structure and the load-settlement characteristics of the soil.

Stavridis [18] 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. The proposed procedure is based on a purely analytical treatment of the underlying soil models as well as on the use of a structural model with fictitious supports inserted at the contact nodes of the foundation elements with the soil surface. Javier [19] evaluated the effect of soil-structure interaction in yielding systems including both kinematical and inertial interaction. The concepts developed previously for interacting elastic systems were extended to include the nonlinear behaviour of the structure. A simple soil-structure system representative of code-designed buildings is proposed. Dutta [20] studied the behaviour of low-rise building frames resting on shallow foundations, viz. isolated and grid foundation. The influence of soil-structure interaction on elastic and inelastic range responses of such building frames due to seismic excitations was examined.

Junvi [21] 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. The interaction forces between the unbounded soil and the structure are evaluated by a system of linear equations instead of time-consuming convolution integrals. Lehman [22] 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).

Hora [23] investigated the interaction behaviour of infilled building frame-soil system under static loading using finite-infinite isoparametric elements. The coupled formulation was used to model the far field behaviour of unbounded domain of soil mass. The soil mass was considered to behave in elasto-plastic manner and to yield according to various yield criteria. Singh [24] analyzed a two-dimensional 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.

Orakdoen [25] presented the performance evaluations of a three-dimensional 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 carried out and the performance evaluations were predicted according to FEMA-440. The tensionless elasto-plastic Wrinkler's soil model was used for soil behaviour in this study.

Puglisi [26] 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 conventional strut model has been modified by the inclusion of a new concept of "plastic concentrator". The use of plastic concentrators leads to a more realistic representation of the behaviour than the conventional models. The proposed model can be used in a simplified way to study the behaviour of any infilled frame (i.e. reinforced concrete or steel frame).

In the present study, the nonlinear stress-strain characteristic of the soil mass is modeled with well known hyperbolic model [27, 28]. The effect of differential settlements on the forces in the frame members, bending moments in the foundation beam and the contact pressure distribution below the foundation beam is investigated.

3. COUPLED FINITE-INFINITE ELEMENTS MODELING OF INTERACTION SYSTEM

The idealization of plane frame-foundation-soil interaction system is achieved with isoparametric finite and infinite elements. The floor beams, the columns and the foundation beam are discretized using three node beam bending elements with three degrees of freedom per node (u, v, θ). The unbounded domain of the soil mass is represented by eight node conventional plane strain finite elements coupled with six node infinite elements with exponential type decay with two degrees of freedom per node (u, v) [10]. A doubly infinite element is used as corner element in the finite-infinite element mesh. Table-1 provides the shape functions for various elements.

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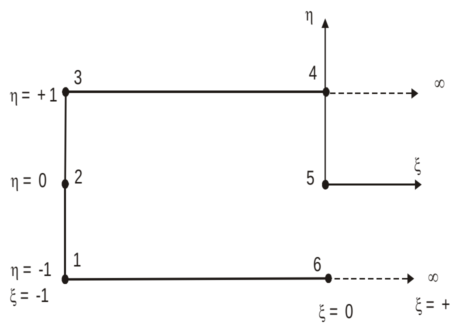
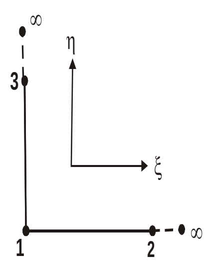
Table-1. Shape functions for isoparametric finite and infinite elements.

<table>
<tr><th>Element</th><th>Shape functions</th></tr>
<tr><td>6 node isoparametric infinite element with exponential decay pattern</td><td>$N_1 = \frac{\xi\eta(1-\eta)}{(1-\xi)}$
$N_2 = \frac{-2\xi(1-\eta^2)}{(1-\xi)}$
$N_3 = \frac{-\xi\eta(1+\xi)}{(1-\xi)}$
$N_4 = \frac{(1+\xi)\eta(1+\eta)}{2(1-\xi)}$
$N_5 = \frac{(1+\xi)(1-\eta^2)}{(1-\xi)}$
$N_6 = \frac{-(1+\xi)\eta(1-\eta)}{2(1-\xi)}$</td></tr>
<tr><td>3 node doubly infinite isoparametric element with exponential decay</td><td>$N_1 = \frac{(\xi\eta+3)(-1-\xi-\eta)}{(1-\xi)(1-\mu)}$
$N_2 = \frac{2(1+\xi)}{(1-\xi)(1-\mu)}$
$N_3 = \frac{2(1+\eta)}{(1-\xi)(1-\mu)}$</td></tr>
<tr><td>3 node isoparametric beam bending element</td><td>$N_1 = \xi(1-\xi)/2$
$N_2 = (1-\xi^2)$
$N_3 = \xi(1+\xi)/2$</td></tr>
</table>

4. NONLINEAR ELASTIC HYPERBOLIC SOIL MODEL

In the present problem, there are mainly two types of materials involved: 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 nonlinearity of the soil mass is considered while the reinforced concrete is assumed to follow the linear stress-strain relationship. The non-linearity of soil mass has been represented using the hyperbolic model proposed by Kondner [27]. The model is used in the literature by Duncan and Chang [28] for nonlinear stress analysis of soil. The tangent modulus (E_T), of the soil mass at any deviatoric stress level is represented as:

$$E_T = \left[1 - \frac{R_f(1-\sin\phi)(\sigma_1-\sigma_3)}{2(c\cos\phi + \sigma_3\sin\phi)}\right]^2 E_i \qquad (4.1)$$

Where,

$$E_i = K P_a \left(\frac{\sigma_3}{P_a}\right)^n \qquad (4.2)$$

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 = $\frac{(\sigma_1-\sigma_3)_f}{(\sigma_1-\sigma_3)_{ult}}$

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

VOL. 5, NO. 7, JULY 2010 ISSN 1819-6608
ARPN Journal of Engineering and Applied Sciences

The values of these parameters is taken from the literature (Noorzaei [12]) and indicated in Figure-2. Poisson's ratio has been kept constant in the analysis. This hyperbolic model has been incorporated into the computer code developed for the nonlinear interaction analysis.

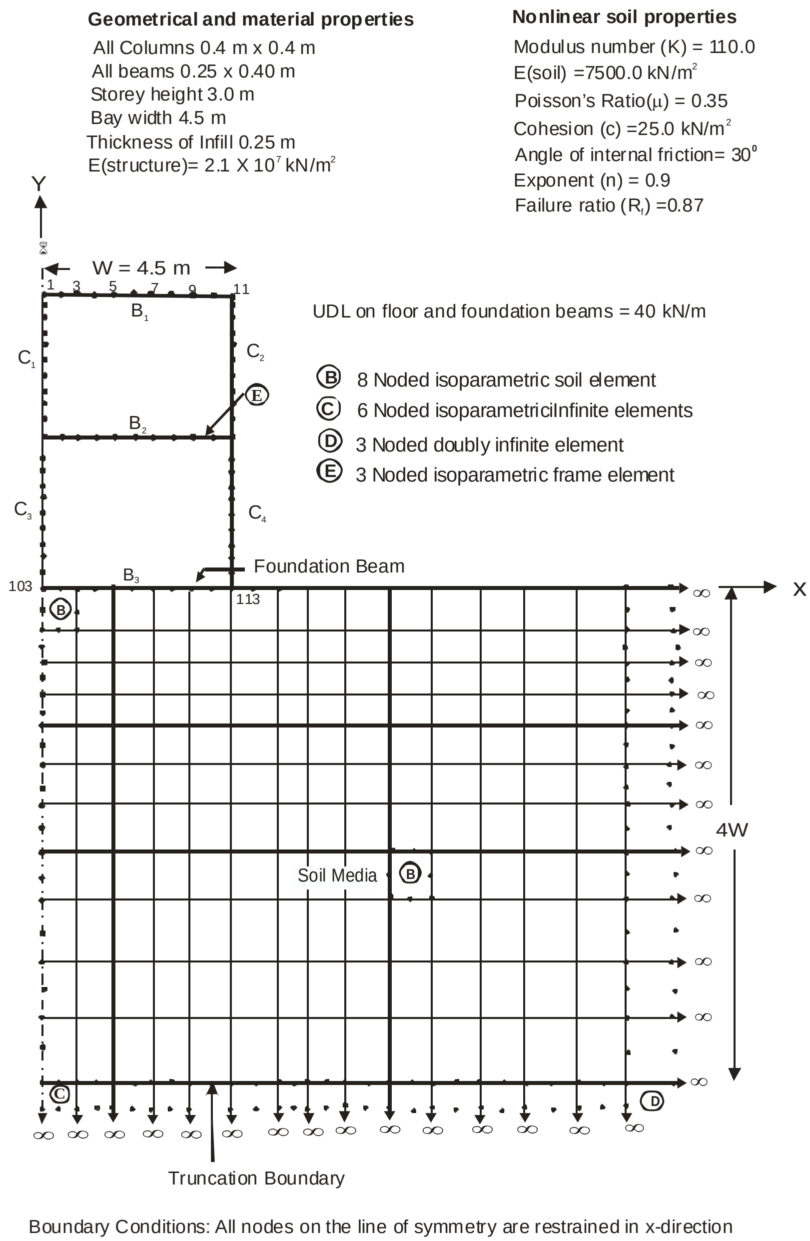


Figure-2. Coupled finite-infinite idealization of plane frame-soil system.

5. COMPUTATIONAL ALGORITHM

The mixed (incremental-iterative) technique has been adopted for the nonlinear elastic analysis of the present problem (Noorzaei [12]). 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:

$$\{\Delta P\}_1^1 = [K]_1^1 \{\Delta\delta\}_1^1 \tag{5.1}$$

ARPN Journal of Engineering and Applied Sciences

(ii) Solve (Eq. 5.1) for $\{\Delta\delta\}$ and evaluate the incremental strains and stresses as:

$$\{\Delta\varepsilon\}_1^1 = [B]\{\Delta\delta\}_1^1$$
$$\{\Delta\sigma\}_1^1 = [D]_1^1\{\Delta\varepsilon\}_1^1 \qquad (5.2)$$

Where [B] and [D] are strain-displacement and elasticity matrices respectively.

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

$$\{\sigma\}_{1,temp}^1 = \{\sigma\}_{0,acc}^1 + \{\Delta\sigma\}_1^1 \qquad (5.3)$$

Where $\{\sigma\}_{0,acc}^1$ is the accumulated stress and is initially zero at the beginning of first iteration of first load increment.

(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. 4.1).

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

$$\{\Delta\sigma\}_{1,mod}^1 = [D]_{1,mod}^1\{\Delta\varepsilon\}_1^1 \qquad (5.4)$$

(vii) Accumulate stresses as:

$$\{\sigma\}_{1,acc}^1 = \{\sigma\}_{0,1cc}^1 + \{\Delta\sigma\}_{1,mod}^1 \qquad (5.5)$$

(viii) Evaluate residual force $\{\Psi\}$ as:

$$\{\psi\}_1^1 = -\int [B]\{\sigma\}_{1,acc}^1\, dV + \{\Delta P\}_1^1 \qquad (5.6)$$

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

(ix) Finally, accumulate the displacements:

$$\{\delta\}_{1,acc}^1 = \{\delta\}_{0,acc}^1 + \{\Delta\delta\}_1^1 \qquad (5.7)$$

(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 maximum number of iterations for each load increment was fixed as 30 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 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.

6. NONLINEAR INTERACTION ANALYSIS SOFTWARE

The computer programme has been developed in FORTRAN-90 for nonlinear interaction analysis of frame-foundation beam-soil system. It includes a library of variety of elements needed for the discretization of domain of the interaction system. The beam element included in the programme is the modified form of the beam-bending element (Hinton and Owen [29]), which includes one additional degree of freedom to take care of axial deformation in the frame members. The mixed incremental-iterative nonlinear algorithm is implemented in the programme to take care of nonlinear soil behaviour. The gauss-Legendre scheme is employed for the evaluation of element stiffness of finite and infinite elements both. In the present study, a frontal equation solver proposed by Godbole [30] has been further modified and made compatible to the present problem. The flow chart for nonlinear interaction analysis is depicted in Figure-1.

VOL. 5, NO. 7, JULY 2010 ISSN 1819-6608
ARPN Journal of Engineering and Applied Sciences

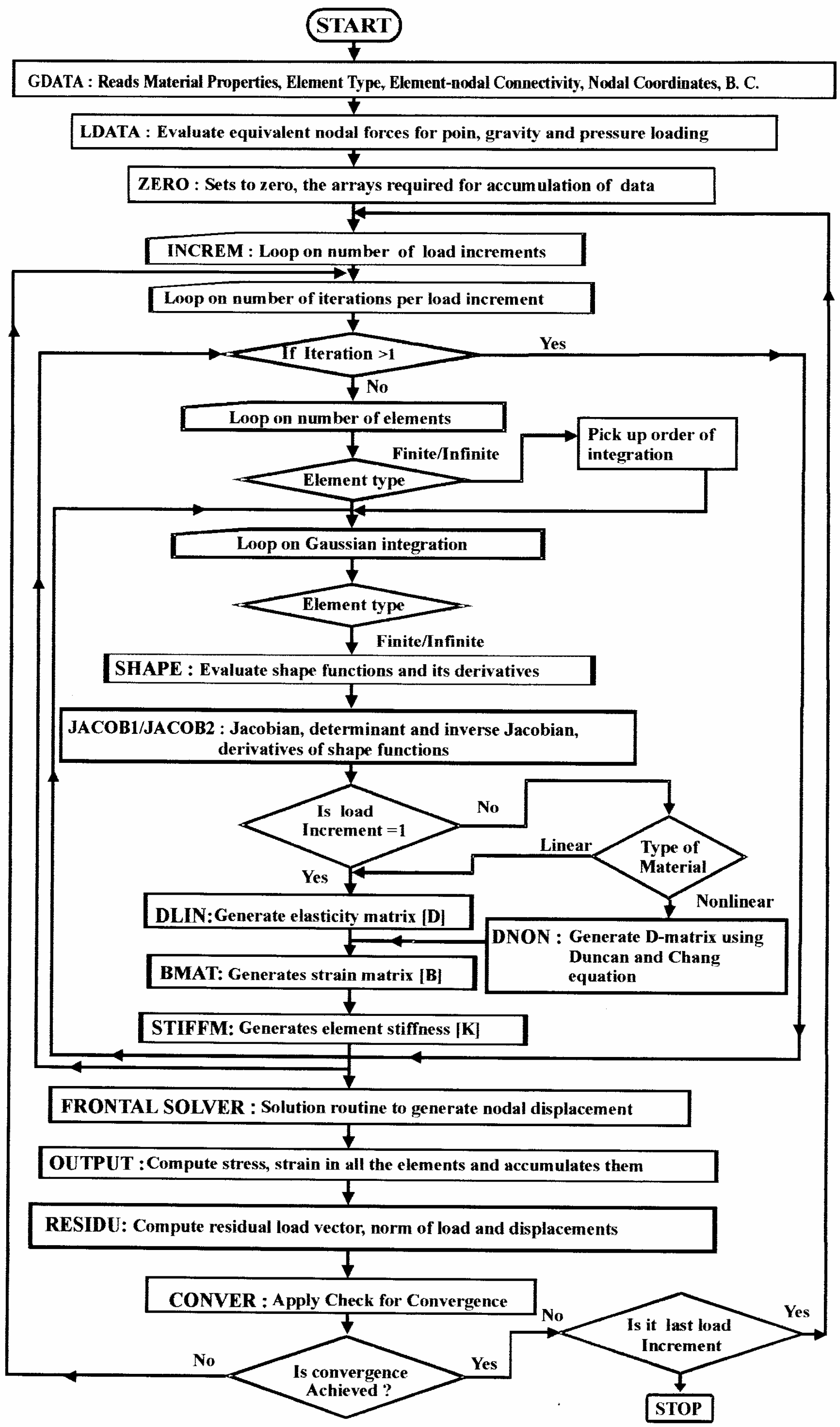


Fig. 1. Flow chart for non-linear elastic interactive analysis

VOL. 5, NO. 7, JULY 2010 ISSN 1819-6608
ARPN Journal of Engineering and Applied Sciences

7. INTERACTION ANALYSIS

7.1 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 requires trial and error. To locate a truncation boundary, firstly 2-3 layers of finite elements are taken and one layer of infinite layer is attached below. Thereafter, each trial involves shifting its position by including an additional finite element layer above it. The deflection below any selected nodal point is compared with the result provided by fully finite element discretization of the problem to access the correct location of the truncation boundary.

In the present analysis, twenty layers of finite elements were required which extended to a depth of nine times of the bay width whereas coupled analysis required only eleven layers of finite elements and one layer of infinite elements extending to depth of about four times the bay width as shown in Figure-2. Moreover, the displacements of the free nodes of the infinite elements were found to be almost negligible which justifies the location of the truncation boundary. For location of truncation boundary, the behaviour of soil mass treated is as linear elastic.

7.2 Nonlinear analysis

In the present investigation, the linear elastic interaction analysis (LIA) and nonlinear interaction analysis (NLIA) of two-bay two-storey plane frame-foundation beam-soil system has been carried out considering the frame to behave in linear elastic manner whereas the subsoil in nonlinear elastic manner. The building frame has a bay width of 4.5 m and total height of 6.0 m. The floor beams and the foundation beam carry total uniformly distributed load of intensity 40 kN/m (dead load and live load). Figure-2 shows the discretization of the interaction system. Since the system is symmetrical with respect to geometry and loading, only half of the structural-foundation-soil system is considered and meshed for carrying out the interaction analysis.

The nonlinear interaction analysis is carried out using mixed incremental-iterative algorithm. The total vertical load (P) of 612 kN is applied in seven load increments (i.e. 30, 15, 15, 10, 10, 10, 10% of P). The load increments depend upon the nature of the stress-strain curve, material properties and are chosen accordingly. The first load increment of 30% of total load is applied because the stress-strain curve is linear elastic initially; thereafter-smaller load increments are applied to follow the nonlinear stress-strain portion of the curve. The norm of residual force for convergence is adopted and a tolerance limit of 1% is selected for residual forces.

The interaction behaviour is studied with respect to differential settlements caused due to incrementally applied loads of the nonlinear analysis. The variation of axial forces in the columns, bending moments in the columns, floor beams and foundation beam have been investigated due to increase in the differential settlements. The results of the nonlinear interaction analysis are compared with linear interaction analysis.

7.2.1 Load versus differential settlement

Figure-3 depicts the variation of differential settlement for plane frame-soil system for various load increments of NLIA. The differential settlement varies from 3.18 mm (first load increment) to 10.30 mm (seventh load increment). The variations below and above 3rd load increment (load factor 0.60) are linear i.e. bilinear variation is found. The value of differential settlement provided by LIA is 9.00 mm. The convergence took place after certain number of iterations for each load increment of the nonlinear analysis as shown in Figure-4.

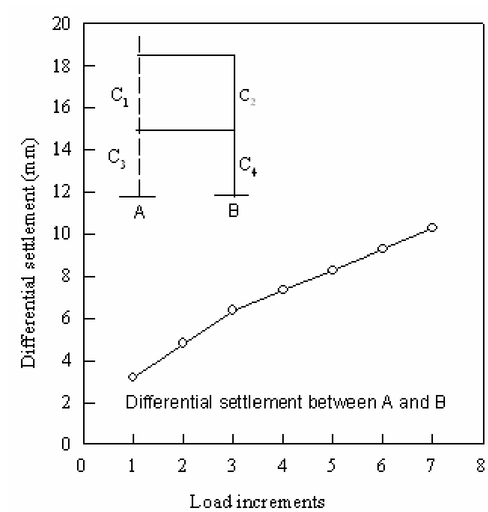


Fig. 3 Variation of differential settlement with load increments for nonlinear inteaction analysis

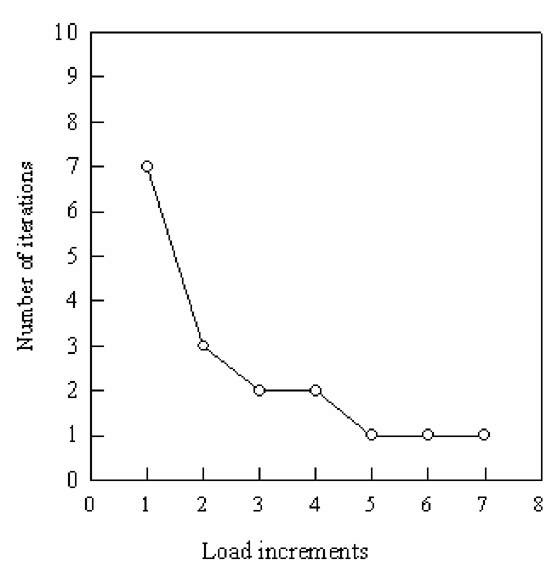


Fig.4 Plot between load increments verses number of iterations

VOL. 5, NO. 7, JULY 2010 ISSN 1819-6608
ARPN Journal of Engineering and Applied Sciences

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7.2.2 Vertical settlements below the foundation beam

The variation of vertical settlements below foundation beam of plane frame-soil system is depicted in Figure-5 in the non-dimensional form for the load increments 1, 3, 5 and 7 of NLIA. 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 due to the fact 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).

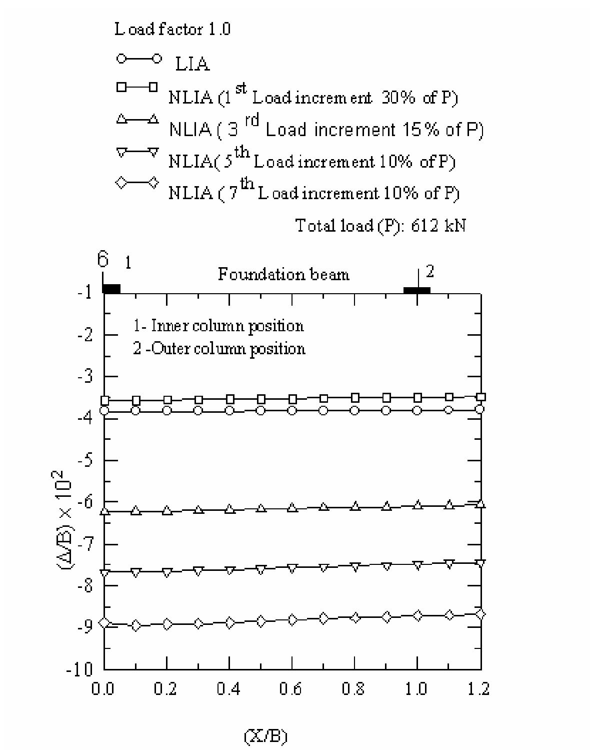

Fig. 5 Variation of vertical settlements below foundation beam with load increments of nonlinear analysis

7.2.3 Contact pressures below foundation beam

Figure-6 shows the variation of contact pressure with various load increments of nonlinear analysis in the non-dimensional form. The contact pressure increase with increase in load increments but the nature of variation is found to be the same. Figure-7 shows the comparison of variation of contact pressures between LIA and NLIA at load factor of unity. The contact pressures below the central column due to linear soil behaviour and nonlinear soil behaviour for the seventh load increment (load factor 1.0) are almost same at the centre of the foundation beam. At the edge of the foundation beam, NLIA provides marginally nearly 12% higher contact pressures.

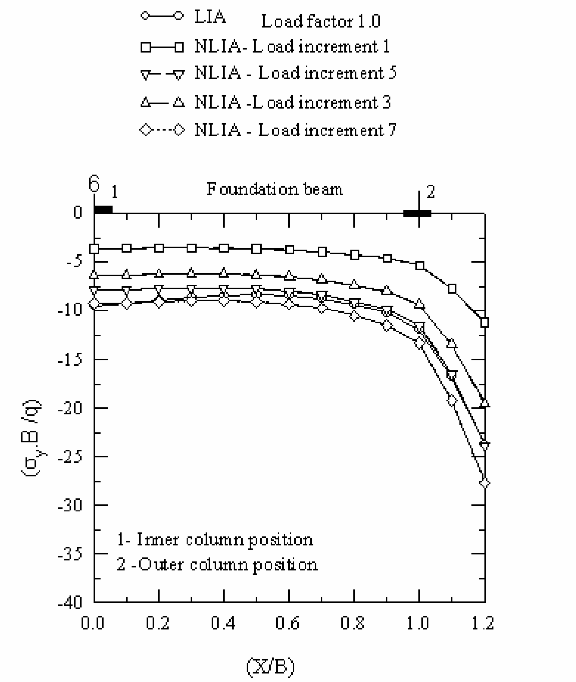

Fig. 6 Contact pressures distribution below foundation beam for various load increments of nonlinear analysis

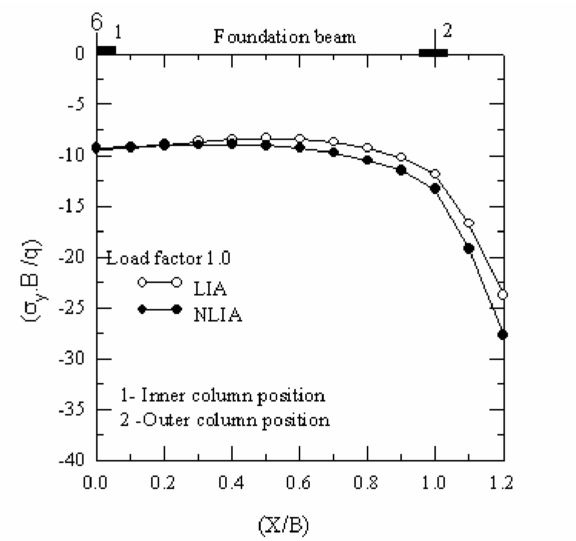

Fig. 7 Comparison of contact pressures distribution below foundation beam for linear and nonlinear analysis

7.2.4 Axial force in the columns

Table-2 shows the value of axial force in the columns due to various analyses. The conventional frame analysis (CFA) is carried out considering the column fixed at their bases. 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.

Table-2. Axial force (kN) in columns of plane frame-soil interaction system.

Storey level	Member	CFA	LIA	% Diff. (3-4)	NLIA	% Diff. (4-5)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
II	C_1	95.21	69.87	-26.62	67.54	-3.33
	C_2	84.79	110.55	+30.38	113.65	+2.80
I	C_3	187.43	129.13	-31.10	123.42	-4.42
	C_4	172.58	230.96	+33.83	237.82	+2.97

BFA - Bare frame analysis; LIA - Linear interaction analysis; NLIA - Nonlinear analysis

Table-3 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 NLIA. It is observed that the increase in the differential settlements due to load increments causes increase in the axial force in the columns. The bilinear variation in axial force is found above and below load factor of 0.60. Figure-8 shows the variation of axial force in the columns of plane frame-foundation beam-soil system with various load increments of NLIA.

Table-3. Axial force (kN) in columns due to differential settlements for nonlinear interaction analysis.

Storey level	Member	Plane frame-soil system (3)						
(1)	(2)	1st LF 0.30	2nd LF 0.45	3rd LF 0.60	4th LF 0.70	5th LF 0.80	6th LF 0.90	7th LF 1.00
		3.18 mm	4.77 mm	6.36 mm	7.33 mm	8.27 mm	9.28 mm	10.30 mm
II	C_1	19.95	29.97	40.00	46.70	53.49	60.60	67.54
	C_2	34.32	51.48	68.47	80.07	90.96	102.35	113.65
I	C_3	36.30	54.51	72.85	85.25	98.15	110.70	123.42
	C_4	72.00	108.06	143.69	167.39	190.63	214.16	237.82

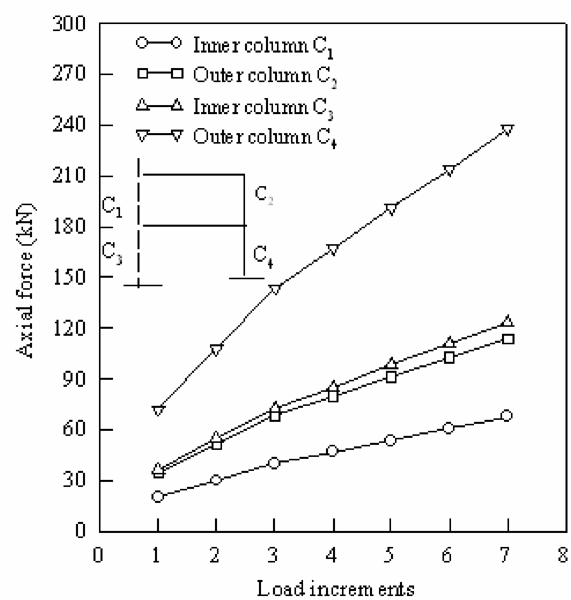

Fig. 8 Variation of axial force in columns with load increments of nonlinear interaction analysis

7.2.5 Bending moments in the outer columns

Table-4 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 the top storey. NLIA provides marginally higher values (nearly 4 to 11%) as compared LIA.

Table-5 shows the variation of bending moments in the columns with differential settlements due to NLIA. It is observed that the increase in the differential settlements due to load increments causes increase in the bending moments in the outer columns. The bilinear variation in bending moments is found above and below load factor of 0.60. Figure-9 shows the variation of bending moments in the outer columns with the load increments for plane frame-foundation beam-soil system due to NLIA.

Table-4. Bending moments (kN-m) in outer columns of plane frame-soil interaction system.

Storey level	Members	CFA	LIA	% Diff. (3-4)	NLIA	% Diff. (4-6)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
II	C_2	+51.03 +38.77	+102.75 +63.64	+101.35 +64.14	+108.18 +66.07	+5.29 +3.81
I	C_4	+21.56 +11.04	+70.99 +114.25	+229.26 **	+76.51 +126.55	+7.77 +10.76

Table-5. Bending moments (kN-m) in outer columns due to differential settlement for load increments of nonlinear analysis.

Storey level	Member	Plane frame-soil system (3)						
(1)	(2)	1st LF 0.30	2nd LF 0.45	3rd LF 0.60	4th LF 0.70	5th LF 0.80	6th LF 0.90	7th LF 1.00
		3.18 mm	4.77 mm	6.36 mm	7.33 mm	8.27 mm	9.28 mm	10.30 mm
II	C_2	33.00 20.07	49.49 30.09	65.72 40.01	76.52 46.61	86.84 52.99	97.49 59.53	108.18 66.07
I	C_4	23.51 39.21	35.25 58.78	46.73 77.77	54.37 90.39	61.51 101.91	69.00 114.20	76.51 126.55

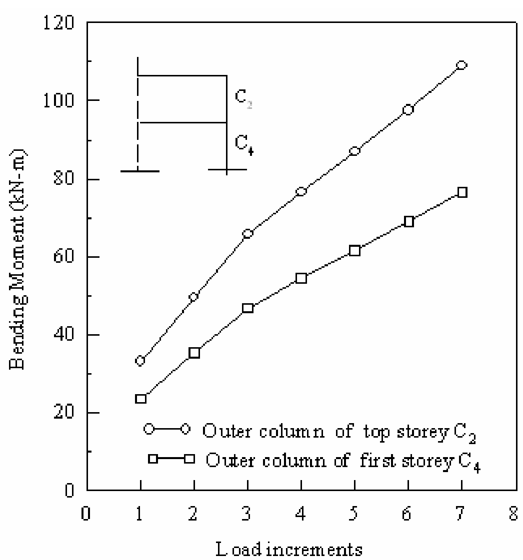

Fig. 9 Variation of B.M.'s at roof level of outer columns with with load increments of nonlinear analysis

7.2.6 Bending moments in the floor beams

Table-6 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.

Table-7 shows the 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 with the increase in differential settlements. The bilinear variation in bending moments in floor beams is found above and below load factor of 0.60. Figure-10 shows the variation of bending moments in the floor beams of plane-frame soil system for various load increments of NLIA.

Table-6. Bending moments (kN-m) in floor beams of plane frame-soil interaction system.

Storey level	Members	CFA	LIA	% Diff. (3-4)	NLIA	% Diff. (4-6)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
II	B_1	+74.49 -51.03	+11.66 -102.74	** +101.34	+11.26 -108.17	-3.43 +5.28
I	B_2	+70.29 -60.33	-10.96 -134.64	* +123.17	-11.85 -142.58	+8.12 +5.89

* Reversal in sign; ** Very high difference in values

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Table-7. Bending moments (kN-m) in floor beams due to differential settlement for nonlinear analysis.

Storey level (1)	Member (2)	Plane frame-soil system (3)						
		1st LF 0.30	2nd LF 0.45	3rd LF 0.60	4th LF 0.70	5th LF 0.80	6th LF 0.90	7th LF 1.00
		3.18 mm	4.77 mm	6.36 mm	7.33 mm	8.27 mm	9.28 mm	10.30 mm
II	B_1	+ 0.83 -33.00	+1.05 -49.46	+2.03 -65.73	+2.73 -76.54	+3.70 -86.82	+4.35 -97.49	+5.05 -108.17
I	B_2	-4.36 -43.58	-6.52 -65.34	-8.29 -86.75	-9.44 -100.96	-9.90 -114.50	-10.85 -128.52	-11.85 -142.58

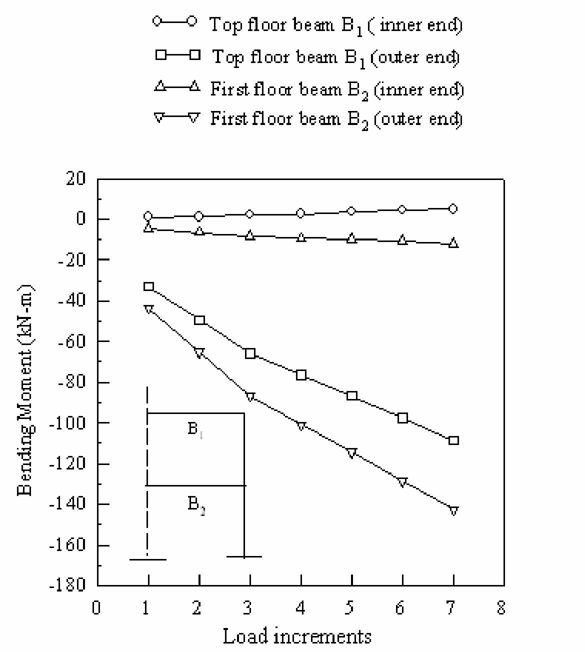

Fig.10 Variation of bending moments in floor beams with load increments of nonlinear interaction analysis

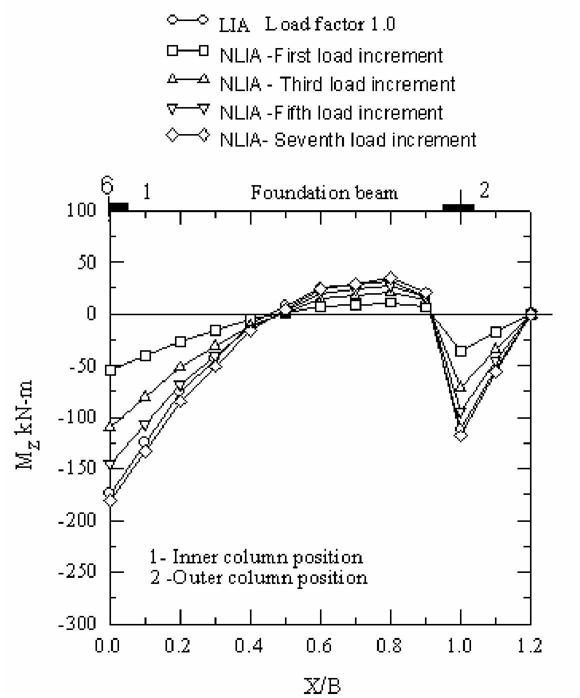

Fig. 11 Variation of bending moments in foundation beam with load increments of nonlinear analysis

7.2.7 Bending moments in the foundation beam

Figure-11 depicts the variation of bending moments along the foundation beam of plane frame-foundation beam-soil system for various load increments of nonlinear analysis. The variation resembles the behaviour of the beam subjected to column loads from top and upward soil pressure beneath. It is found that both the positive and negative bending moments increase with increase in differential settlements. Figure-12 shows the comparison of variation in bending moments in the foundation beam due to LIA and NLIA. It is found that both the analyses provide almost identical values at load factor of unity.

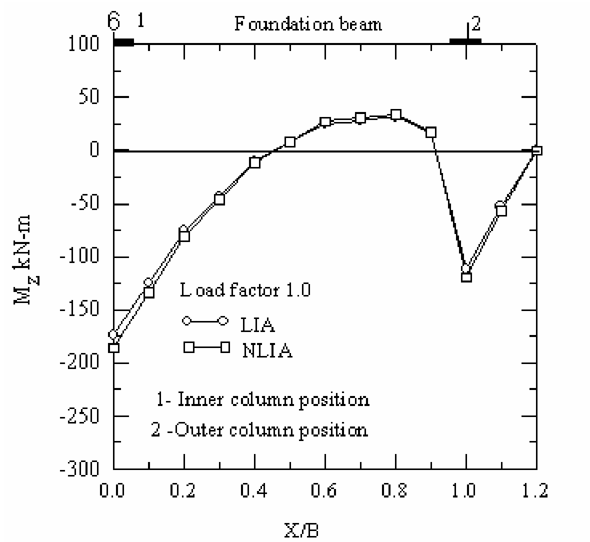

Fig.12 Comparison of variation of bending moments in foundation beam for linear and nonlinear analyses

8. CONCLUSIONS

The forces in the various frame members due to interaction analysis are considerably different from the conventional methods of frame analysis. The vertical settlements due to non-linearity of the soil mass are almost 2.25 times to that of linear behaviour. The differential settlement causes significant increase in the forces in the frame members and the nature of variation is found to be bilinear.

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