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Elasto-Plastic Soil-Structure Interaction Analysis of Building Frame-Soil System

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ABSTRACT

The conventional building frame analysis is carried out assuming unyielding supports and ignoring the interactive response of supporting soil media. In fact, the building frame and its foundation along with the soil mass on which it rests together constitute a single integral compatible structural unit to resist the loads. A proper idealization of the building frame-foundation-soil system is thus needed for accessing the realistic and more accurate analysis. The interaction system analyzed in this way yields a rational structural behavior as the shear forces and bending moments get significantly altered due to the resulting differential settlements of soil mass. In addition to this, the constitutive relationship of the soil mass also plays an important role.

The paper presents the elasto-plastic interaction analysis of two-bay two-storey plane frame-foundation beam-soil system using the finite element method. The superstructure is considered to behave in linear elastic manner whereas the soil mass to behave in elasto-plastic manner and to yield according to various yields criteria. The settlements in soil mass, contact pressure below foundation beam, forces in the frame members and the foundation beam are evaluated and collapse load is determined considering various yield criteria. The results of the elasto-plastic analysis are compared with conventional analysis.

INTRODUCTION

In the conventional method of analysis, a structure is analyzed assuming fixity at the base of the foundation and ignoring the effect of supporting soil media. The structure analyzed in this way does not provide the realistic behaviour. In reality, the structure is generally supported on soil mass and there exists, the interaction between structure, foundation and soil mass. The flexibility of the foundation, the compressibility of the soil mass and other factors cause redistribution of bending moments and shear forces in the superstructure due to differential settlement of soil mass.

The superstructure, foundation and soil mass can be considered as a single integral compatible unit for the interaction analysis. The numerical analysis of the plane frame-foundation beam-soil interaction system is carried out using the finite element method. The discretization of the domain of the interaction system involves the use of variety of isoparametric elements with different degrees of freedom. The coupled finite-infinite elements discretization of soil mass with proper location of truncation boundary provides accurate and computationally economical solutions. A computer soil-structure interaction including simulation of construction sequences. Aljanabi et al.[6] studied the interaction of plane program has been developed for the elasto-plastic interaction analysis of frame-foundation beam-soil system.

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 et al.[4] proposed an economical iterative procedure for building frames and found significant reduction in differential settlements and consequent additional moments. Desai and Sargand[5] developed hybrid finite element procedure for nonlinear elastic and elasto-plastic analysis of

frames with an elastic foundation, of Wrinkler's type, having normal and shear modulli of subgrade reaction.

Viladkar et al.[7] employed a coupled finite-infinite element formulation to highlight the advantage of using the infinite elements to study the interaction analysis of framed structures. Noorzaei et al.[8] considered the nonlinearity aspect of the subsoil and thoroughly investigated its influence on the interaction behaviour of the framed structures. Noorzaei et al.[9] considered the elasto-plastic behaviour of soil mass and carried out the interaction analysis of plane frame-combined footing-soil system to study the various aspects of the interaction behaviour.

Dasgupta et al.[10] studied the effect of three influencing parameters on the column axial force and column moment of three-dimensional building frames. These parameters are namely, relative flexural stiffness of columns with respect to beams, number of bays and number of storeys. Stavridis[11] presented the simplified interaction analysis of layered soil-structure interaction. The stratified soil was represented by linear elastic half space model having specific geometrical and elastic properties for its layer.

The modeling of unbounded domain using coupled finite-infinite elements has proved computationally economical (Viladkar et al.[7]) in comparison to fully finite element analysis. 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 infinite elements with different types of decay pattern are able to model the far field behaviour quite accurately. In this paper, an attempt has been made to investigate elasto-plastic the interaction behaviour of the plane frame-foundation beam-soil system considering the superstructure to behave in linear elastic manner whereas the compressible subsoil to behave in elasto-plastic manner and to yield according to various yields criteria.

COUPLED FINITE-INFINITE MODELLING OF INTERACTION SYSTEM

The finite element idealization of plane frame-foundation beam-soil interaction system requires use of variety of isoparametric finite and infinite elements. Three noded 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. The unbounded domain of the soil mass is represented by conventional eight noded plane strain finite elements with two degrees of freedom per node (u, v) coupled with six noded infinite elements with 1/r type decay (Viladkar et al.[7, 20]) having two degrees of freedom per node (u, v). The distance 'r' is measured from a reference pole to a general point within an element. A three noded doubly infinite element is used as a corner element in the finite-infinite element mesh. Table 1 depicts various finite elements and their shape functions.

ELASTO-PLASTIC CONTITUTIVE MODELLING OF SOIL MEDIA

Zienkiewicz[12] established the incremental elasto-plastic stress-strain relationship and the elasto-plastic $[D]_{ep}$ matrix for the associated flow rule, which is expressed as:

$$[D]_{ep} = [D] - \left[\frac{[D]\{a\}\{a\}^T [D]}{A + \{a\}^T [D]\{a\}} \right] \tag{1}$$

where,

[D] = Elasticity constitutive matrix
$\{a\}$ = plastic flow vector

$$a = C_1\, a_1 + C_2\, a_2 + C_3\, a_3 \tag{2a}$$

where,

$$a_1 = \frac{\partial J_1}{\partial \sigma} \; ; \; a_2 = \frac{\partial J_2'^{\,1/2}}{\partial \sigma} \; ; \; a_3 = \frac{\partial \theta}{\partial \sigma} \tag{2b}$$

Thus, the evaluation of elasto-plastic matrix requires the determination of plastic flow vector and the constants for the flow rule for a yield criterion under consideration. The plastic flow vector (a) can be expressed as:

$$\{a\} = \left\{ \frac{\partial F}{\partial \sigma} \right\} = \left\{ \frac{\partial F}{\partial \sigma_x}, \quad \frac{\partial F}{\partial \sigma_Y}, \quad \frac{\partial F}{\partial \sigma_Z}, \quad \frac{\partial F}{\partial \tau_{xy}}, \quad \frac{\partial F}{\partial \tau_{yz}} \quad \frac{\partial F}{\partial \tau_{xz}} \right\} \tag{2c}$$

where,

$$\sigma = \{ \sigma_x , \quad \sigma_y , \quad \sigma_z , \quad \tau_{xy} , \quad \tau_{yz} , \quad \tau_{zx} \}$$

The yield function is expressed as:

$$F = Yield\ Function = F\left(J_1 , J_2'^{\,1/2} , \theta \right) = 0 \tag{2d}$$

and the constants for plastic flow vectors are evaluated as:

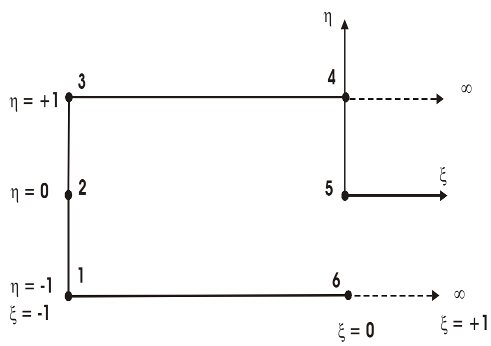
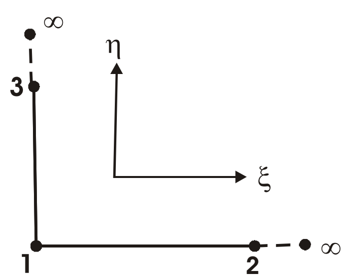
$$C_1 = \frac{\partial F}{\partial J_1} \tag{2e}$$

$$C_2 = \frac{\partial F}{\partial J_2'^{\,1/2}} - \frac{\tan 3\theta}{\left(J_2'\right)^{1/2}} \frac{\partial F}{\partial \theta} \tag{2f}$$

$$C_3 = -\left[\sqrt{\frac{3}{2\cos 3\theta}} \right] \frac{1}{\left(J_2'\right)^{3/2}} \frac{\partial F}{\partial \theta} \tag{2g}$$

The computer coding of the yield function and the flow rule can be easily done using this formulation. It requires the

Table 1. Shape Functions For Isoparametric Finite And Infinite Elements

<table>
<tr><th>Element Type</th><th>Element Figure</th><th>Shape Functions</th></tr>
<tr><td>Three noded Beam element</td><td>
</td><td>$N_1 = \xi(1-\xi)/2$
$N_2 = (1-\xi^2)$
$N_3 = \xi(1+\xi)/2$</td></tr>
<tr><td>Six noded infinite element with 1/r type decay∗</td><td>
</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>Three noded doubly infinite element with 1/ r type decay∗</td><td>
</td><td>$N_1 = \frac{(\xi\eta+3)(-1-\xi-\eta)}{(1-\xi)(1-\eta)}$
$N_2 = \frac{2(1+\xi)}{(1-\xi)(1-\eta)}$
$N_3 = \frac{2(1+\eta)}{(1-\xi)(1-\eta)}$</td></tr>
</table>

specification of the constants C_1, C_2 and C_3 for inclusion of any individual yield function. This is achieved by converting these yield criteria into convenient forms (Nayak and Zienkiewicz[13]) in terms of the variants, namely J_1, $(J'_2)^{1/2}$ and θ and then using eqns. (2e-2g). The various yield criteria have been converted into convenient form for their easy implementation in the finite element code. The implementation of a yield function requires the effective stress level, the equivalent yield stress and the constants needed for the evaluation of flow vector. The effective stress level and the equivalent yield stress for the various isotropic elasto-plastic soil models and constants for flow vectors (Noorzaei et. al. [9]) have been used.

ELASTO-PLASTIC ANALYSIS SOFTWARE

A computer code in FORTRAN-77 has been developed for elasto-plastic interaction analysis of frame-foundation beam-soil system. It includes library-containing variety of elements needed for the discretization of the domain of the interaction system. The beam element included in the program is the modified form of the beam-bending element (Hinton and Owen[14]), which includes one additional degree of freedom to take care of axial deformation in the frame members. The software takes into account the elasto-plastic behaviour of soil mass by considering various yield criteria. Different yield criteria for the soil mass have been transformed into convenient form for their easy implementation in the finite element code. The interaction analysis is carried out using mixed incremental-iterative method. A frontal solver (Godbole et al.[15]) is made compatible to the existing problem to solve a set of linearized simultaneous equations arising from a discretization of the domain with variety of elements.

CONCEPTUAL ASPECT OF ELASTO-PLASTIC ANALYSIS

When an increment of load is applied on the structure-foundation-soil system beyond the elastic limit, some of the soil elements may yield fully and some partially. Hence, it is required to monitor the stress and strain at each gaussian point to see whether or not the plastic deformation has taken place at these gaussian points. There can also be a situation where an element may behave partly elastically and partly elasto-plastically. Therefore, for any load increment, it is necessary to determine what portion of the element is elastic and what part produces plastic deformation and then adjust the stresses and strains until the yield criterion and the constitutive laws are satisfied. The computational algorithm adopted for elasto-plastic analysis is quite identical to those of Owen and Hinton[16].

ELASTO-PLASTIC INTERACTION ANALYSIS

In the present investigation, the linear elastic interaction analysis (LIA) and elasto-plastic interaction analysis (EPIA) of two-bay two-storey plane frame-foundation beam-soil system (FS) has been carried out considering the superstructure to behave in linear elastic manner whereas the subsoil in elasto-plastic manner. The floor beams and the foundation beam carry uniformly distributed load of 40 kN/m, which includes dead load and live load. Fig. 1 shows the discretization of the interaction system along with the geometrical properties of the frame and soil parameters for

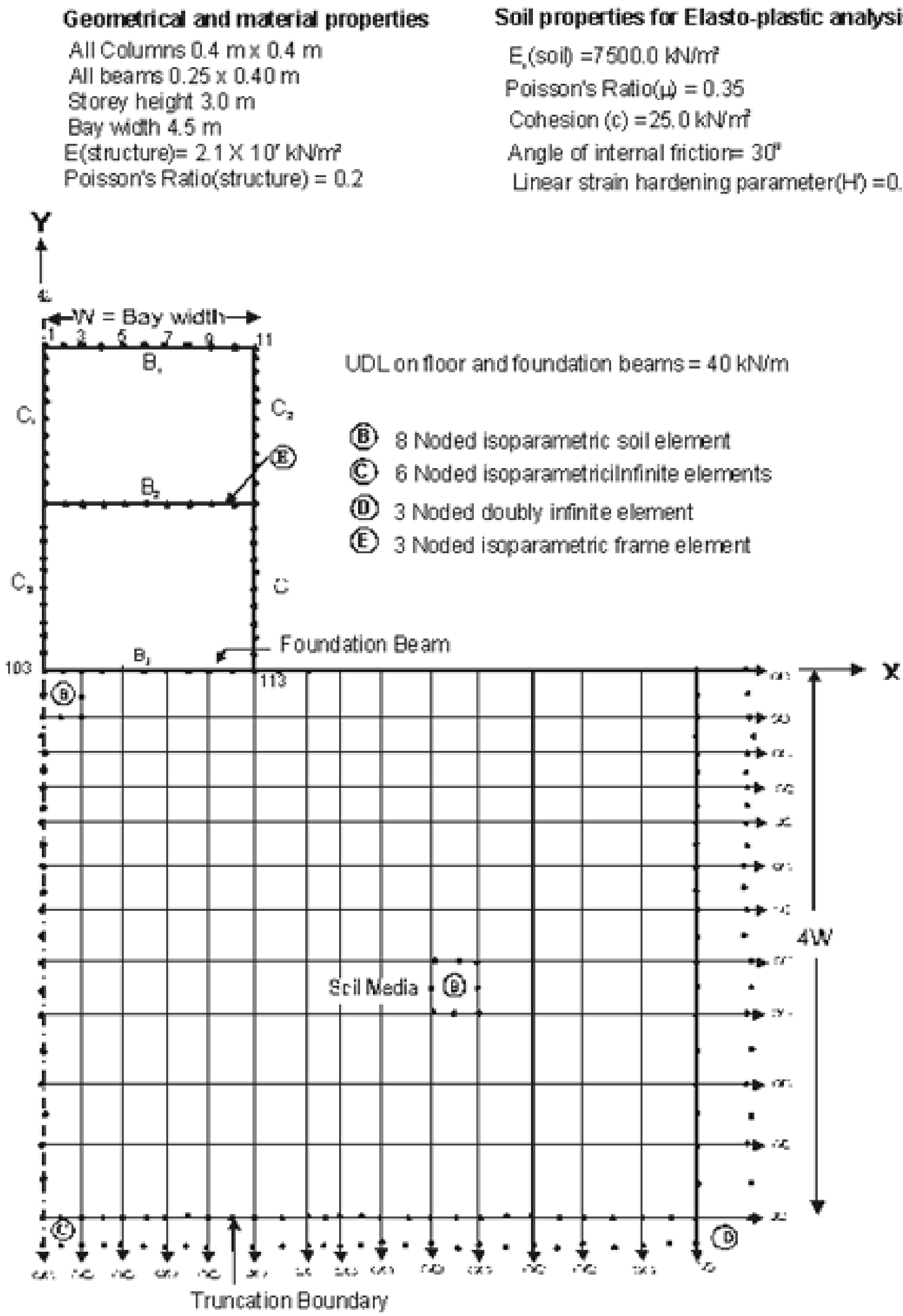


Fig. 1 Finite – Infinite Discretization Of Plane Frame – Soil System

elasto-plastic analysis (Noorzaei et al.[9]). 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. 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. For location of truncation boundary, the behavior of soil mass is treated as linear elastic. The infinite elements with 1/r type of decay pattern were used for analysis. The coupled analysis required eleven layers of finite elements extending to depth of about four times the bay width (w), thereafter one layer of infinite elements was attached below this. The results of linear elastic interaction analysis and elasto-plastic interaction analysis have also been compared with those of non-interaction analysis (columns are treated as fixed at their bases). The elasto-plastic analysis of the interaction system is carried out using mixed incremental iterative method. In this analysis, the load was limited to a value which causes local failure in some finite elements of the soil mass (i.e. load factor of unity which corresponds to 40 kN/m). The vertical load is applied in thirteen increments (50, 10, 10, 10, 10, 10, 10, 10, 10, 10, 10, 5, 5% of 40 kN/m). 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. The first load increment of 50% is applied because the stress-strain curve is linear elastic initially; thereafter-smaller load increments are applied in the elasto-plastic region of the stress-strain curve. The collapse load varies marginally for various yield criteria and hence number of load increments is different. The norm of residual force for convergence is adopted for nonlinear elasto-plastic interaction analysis. A tolerance limit of 1% is selected for residual forces. A load at which computer program fails to converge during iterative process of convergence is defined as collapse load for the system.

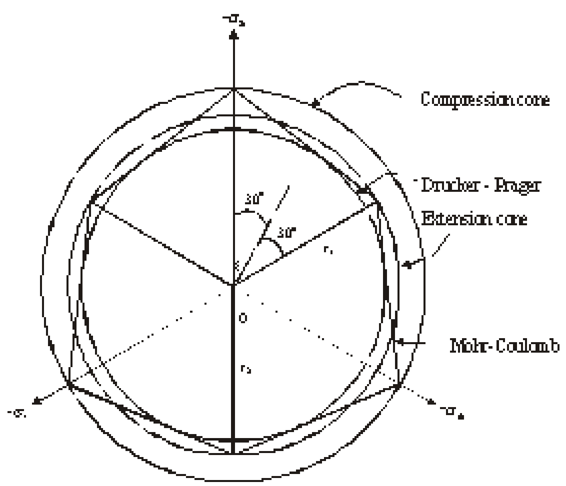


Fig .2 Two Dimensional Representation Of Yield Criteria

The elasto-plastic interaction analysis is carried out considering the subsoil to yield according to the following yield criteria (Naylor-Zienkiewicz models[17-19]):

(i) Mohr-Coulomb yield criterion (MC)

(ii) Drucker-Prager yield criterion-I (DP-I)

(iii) Compromise cone yield criterion (CC)

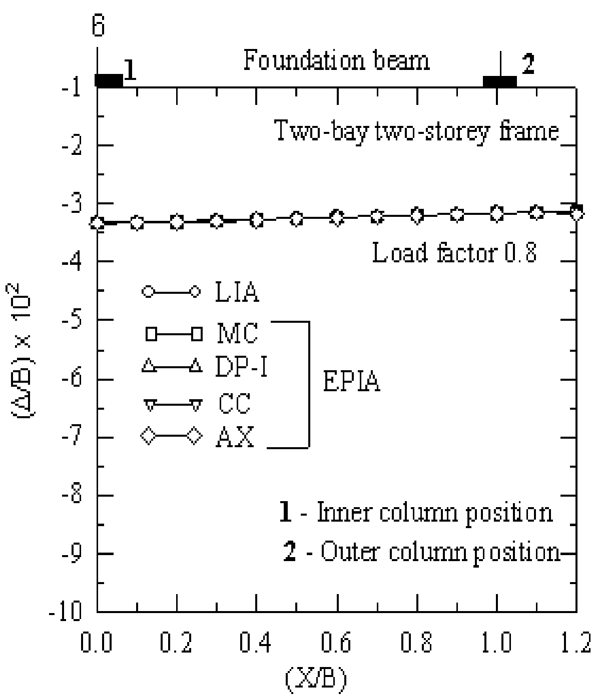
(iv) Axial extended cone yield criterion (AX)

The vertical displacements, contact pressure beneath the foundation beam, the variation of bending moments in the frame members as well as in the foundation beam and the variation of axial forces and bending moments in the column members are presented in the following discussion and compared for different analyses and various yield criteria. The collapse loads are also determined considering different yield criteria. Fig. 2 shows two-dimensional graphical representation of various yield criteria.

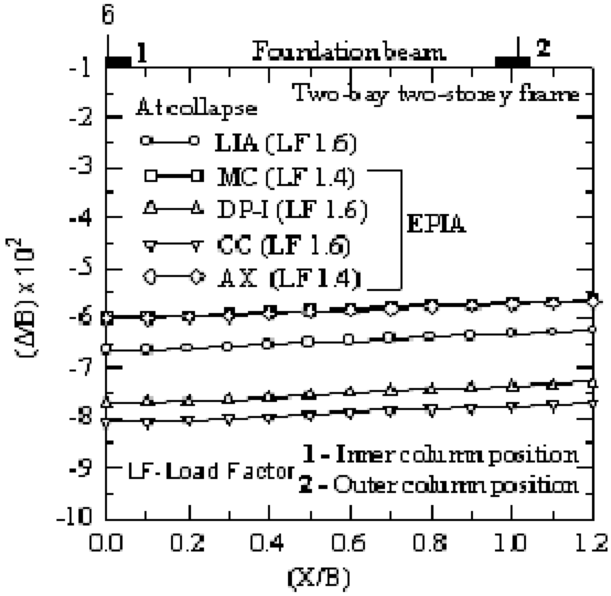
Vertical Settlements below Foundation Beam

Fig. 3 (a-b) shows the variation of vertical settlements below the foundation beam of plane frame-soil system in the non-dimensional form for LIA and EPIA. It is observed that the maximum settlement occurs below the central column and it decreases marginally towards the outer column. This causes differential settlement of small value.

At lower load factor (0.8), the total vertical settlements obtained from LIA and EPIA are almost the same. The various yield criteria also give almost identical values of vertical settlements. Fig. 3(b) shows that at load factor of 1.6 (corresponding to collapse), there is both a qualitative and quantitative departure in the settlements. It is because most of the soil elements start yielding at higher load factors and there is progressive spread of the plastic zone in the soil mass.



(a) Load Factor 0.8



(b) At collapse

Fig. 3 Vertical Settlements Below Foundation Beam

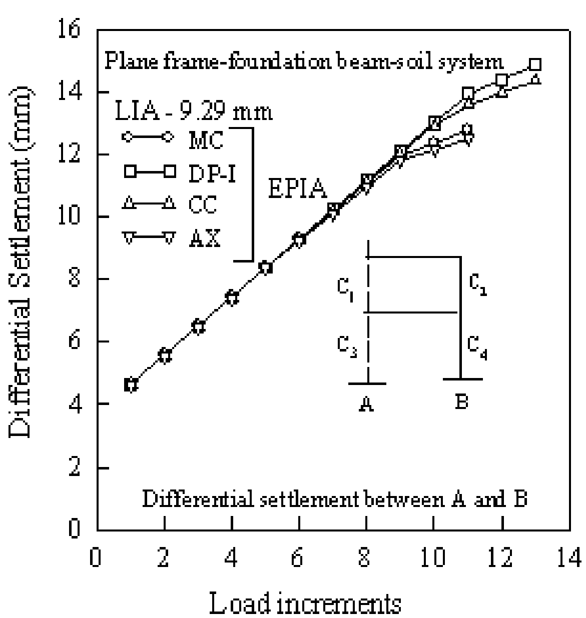


Fig. 4 Variation Of Differential Settlements With Load Increments

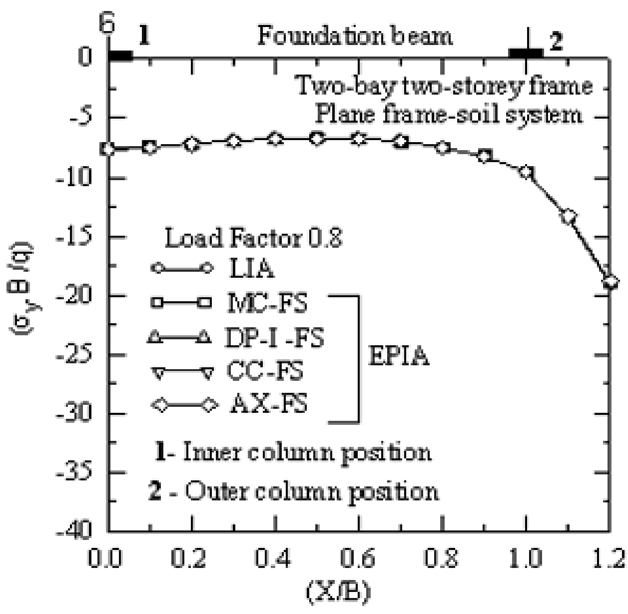
The compromise cone yield criterion gives maximum settlements at collapse. The settlements due to Mohr-Columb and Axial extension cone yield criterion are almost same but values are less as compared to other yield criteria.

Fig. 4 depicts that the differential settlement increase with load increments for plane frame-soil system considering EPIA. The linear variation is observed initially up to 9th load increment; thereafter for further load increments, it follows the elasto-plastic nature of the curve. All yield criteria provide almost same type of variation but the Druckur-Prager yield criterion provides comparatively higher value of differential settlement at higher load increments (9th load increment onwards). The differential settlement has a maximum value of 14.5 mm. The value of differential settlement is 9.29 mm due to LIA.

Contact Pressures Distribution below Foundation Beam

The contact pressure distribution below the foundation beam of plane frame-soil system is shown in Fig. 5 in non-dimensional form due to LIA and EPIA for plane frame-soil system. It is found that the minimum pressure exists at the center of the foundation beam whereas the maximum pressure is found at the edge. This is because the central column is relieved of the moments and only the end columns transfer the moments to the foundation.

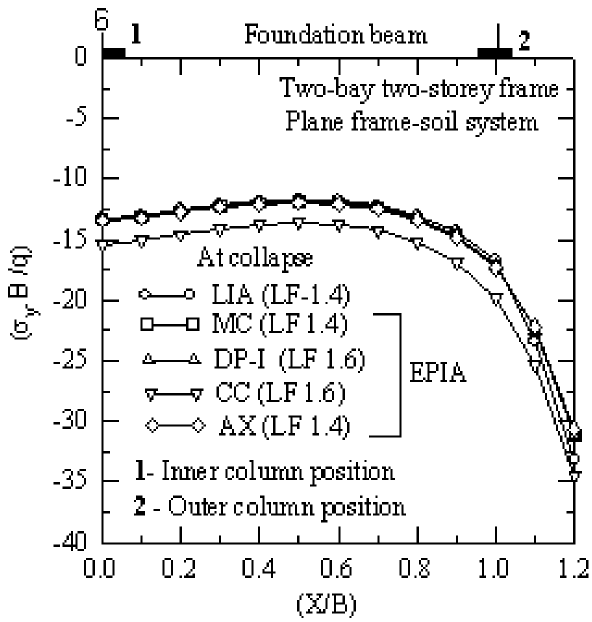
Fig. 5 (a-b) depicts that the contact pressures due to LIA and EPIA are almost same at lower load factor of 0.8 All yield criteria provide almost the same contact pressures at lower load factors. At load factor of 1.6 corresponding to collapse



(a) Load factor 0.8

(when sufficient number of finite elements of the soil yield to cause collapse or the iterative procedure diverges for a particular load value), the contact pressure at the center as well as at the edge due to Mohr's-Columb and Axial extension cone yield criterion, is lower compared to other yield criteria (collapse load factor 1.6) as depicted in Fig 5(b).

The contact pressures at collapse are almost 1.75 times to that of load factor 0.8 at which local failure in some of the finite elements is observed. The compromise cone yield criterion provides maximum contact pressures.



(b) at collapse

Fig. 5 Contact Pressures Below Foundation Beam

Axial Forces in Columns

Table 2 shows the value of axial force in the columns due to various analyses for plane frame-soil system. 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 in the column due to EPIA considering compromise yield criterion are evaluated. The axial forces at lower load factors (0.8, 1.0 and 1.2) due to EPIA and LIA are almost the same. The axial forces due to all yield criteria are also found almost same. The axial forces at the collapse are almost 1.6 times to that at load factor of 0.8 at which local failure in some of the finite elements is observed.

Table 3 shows the variation of axial force in the columns due to differential settlement of soil mass for plane frame-soil system considering EPIA. The axial forces increase with load increments. Fig. 6 shows the variation of axial force with load increments for EPIA considering compromise cone yield criterion. The linear variation in axial force is observed initially up to 9th load increment; thereafter for further load increments, it follows the elasto-plastic nature of the curve. All yield criteria provide almost same variation.

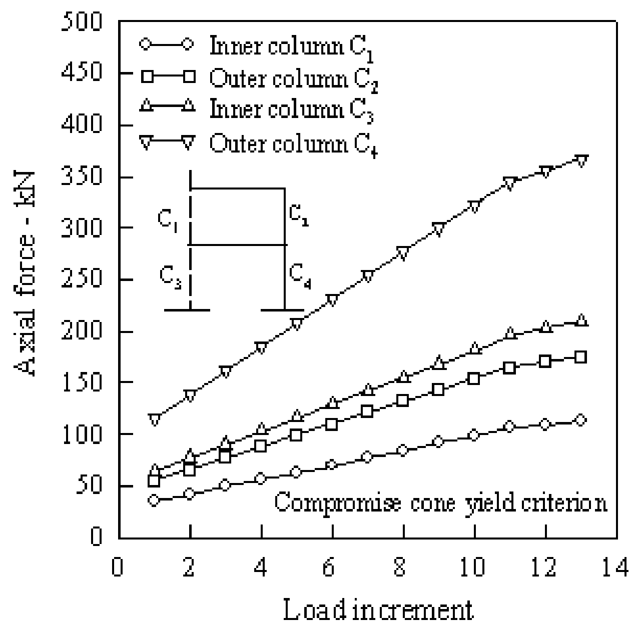


Fig. 6 Variation of Axial Force in Columns

Table 2. Axial Force (kN) in Columns of Plane Frame-Soil System

Load Factor (1)	Storey Level (2)	Member (3)	BFA (4)	LIA (FS) (5)	EPIA-FS (CC) (6)	% Diff. (4 - 6) (7)
0.8	II	C_1	76.16	55.89	55.78	-26.75
		C_2	67.83	88.44	88.28	+30.16
	I	C_3	149.94	103.30	103.14	-31.14
		C_4	138.06	184.16	184.99	+34.00
1.0	II	C_1	95.21	69.87	69.86	-26.62
		C_2	84.79	110.55	110.37	+30.16
	I	C_3	187.43	129.13	129.06	-31.14
		C_4	172.58	230.96	231.35	+34.05
1.6 Collapse	II	C_1	152.33	111.79	113.08	-26.62
		C_2	135.66	176.88	175.54	+30.16
	I	C_3	299.88	206.60	209.91	-31.14
		C_4	276.13	369.53	366.86	+34.05

Table 3. Axial Force (kN) in Columns due to Differential Settlements of Soil Mass

Storey Level (1)	Member (2)	Plane Frame-Foundation Beam-Soil System (FS) -CC (3)						
		First LF 0.5	Third LF 0.7	Fifth LF 0.9	Seventh LF 1.1	Ninth LF 1.3	Eleventh LF 1.5	Thirteenth LF 1.6
		4.64 mm	6.51 mm	8.37 mm	10.21 mm	12.02 mm	13.58 mm	14.36 mm
II	C_1	34.99	48.86	62.85	76.86	90.94	105.75	113.08
	C_2	55.34	77.25	99.38	121.47	143.56	164.84	175.54
I	C_3	64.52	90.26	116.11	142.07	168.14	195.92	209.91
	C_4	115.52	161.86	208.17	254.36	300.15	344.41	366.86

Table 4. Bending Moments (kN-m) in Outer Columns of Plane Frame-Soil System

Load Factor (1)	Storey Level (2)	Member (3)	BFA (4)	LIA-FS (5)	EPIA-FS (CC) (6)	% Diff. (4 - 6) (7)
0.8	II	C_2	40.82	82.20	82.26	+101.51
			31.01	50.91	50.94	+64.22
	I	C_4	17.24	56.72	56.86	+229.24
			8.83	91.4	91.57	**
1.0	II	C_2	51.03	102.75	102.76	+101.30
			38.77	63.64	63.66	+64.22
	I	C_4	21.56	70.90	70.98	+229.24
			11.04	114.25	114.16	**
1.6 Collapse	II	C_2	81.65	164.40	161.72	+98.06
			62.03	101.82	101.44	+63.53
	I	C_4	34.50	113.44	109.40	+217.10
			17.66	182.80	172.01	**

** Very high difference in values

Bending Moment in Outer Columns

Table 4 depicts the values of bending moment in outer columns of plane frame-soil system due to various interaction analyses. 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 significant 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.

Table 5 shows the bending moments in the outer columns as roof level for various load increments of EPIA considering compromise cone yield criterion. At lower load factors the results provided by LIA and EPIA are nearly same. At load factor of 1.6 (corresponding to collapse), the bending moments in the outer columns are nearly 1.6 times to that at lower load factor of 0.8.

Fig. 7 shows the variation of bending moments in the outer columns with load increments for EPIA The linear variation in bending moment is observed initially up to 9th load increment; thereafter for further load increments, it follows the elasto-plastic nature of the curve. All yield criteria provide almost same type of variation.

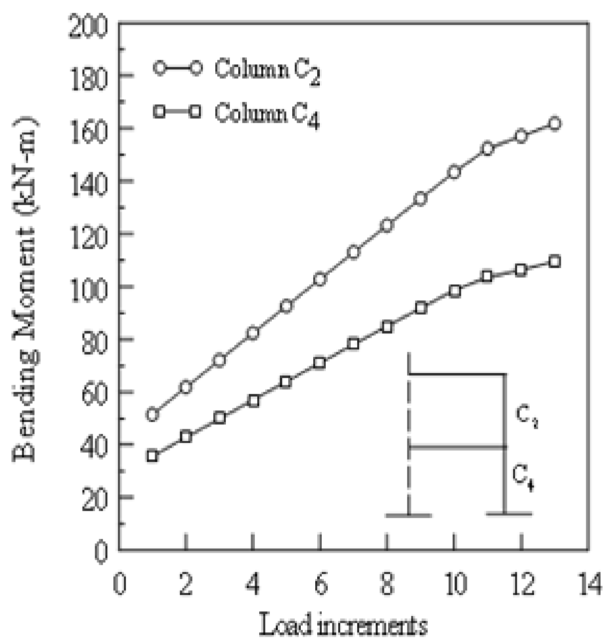


Fig. 7 Variation Of B.M.'S At Roof Level In Outer Columns

Bending Moments in Floor Beams

Table 6 depicts the values of bending moments in the floor beams due to various analyses. The comparison of BFA and LIA suggests that the interaction effect causes transfer of bending moments from the inner end of the beam to the outer end at all floor levels due to differential settlement of soil mass. The reversal in the sign of the bending moment is observed at the junction between the beams of first storey with interior column. The interaction effect also causes shifting of location of maximum positive bending moment in all floor beams, which shifts towards the outer end. A significant increase of nearly 123% is found at the outer end of first floor beam and nearly 101% in the top floor beam due to LIA.

Table 7 shows the bending moments in the floor beams for various load increments of EPIA considering compromise cone yield criterion. At lower load factors the results provided by LIA and EPIA are nearly same. At load factor of 1.6, the bending moments in the outer columns are nearly 1.6 times to that at lower load factor of 0.8.

Fig. 8 shows the variation of bending moments in the floor beams with load increments for EPIA. The linear variation in bending moment is observed initially up to 9th load increment; thereafter for further load increments, it follows the elasto-plastic nature of the curve. All yield criteria provide almost same type of variation.

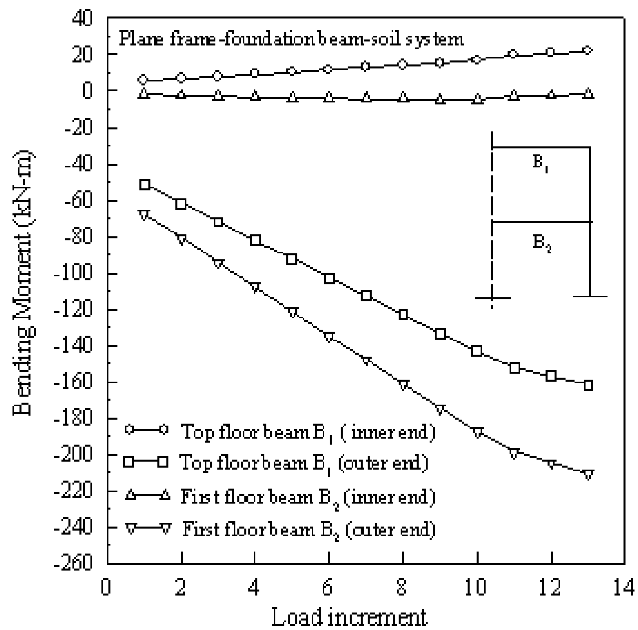


Fig. 8 Variation of Bending Moments in Floor Beams

Table 5. Bending Moments (kN-m) in Outer Columns due to Differential Settlement of Soil Mass

Storey Level (1)	Member (2)	Plane Frame-Foundation Beam-Soil System (FS) –CC (3)						
		First LF 0.5	Third LF 0.7	Fifth LF 0.9	Seventh LF 1.1	Ninth LF 1.3	Eleventh LF 1.5	Thirteenth LF 1.6
		4.64 mm	6.51 mm	8.37 mm	10.21 mm	12.02 mm	13.58 mm	14.36 mm
II	C_2	51.37	71.98	92.52	112.96	133.26	152.24	161.62
		31.82	44.57	57.36	51.97	82.72	95.24	101.44
I	C_4	35.50	49.76	63.94	77.96	91.76	103.46	109.40
		57.12	80.12	102.91	125.26	147.02	163.45	172.01

LF - Load factor   CC - Compromise yield criteria

Table 6. Bending Moments (kN-m) in Floor Beams of Plane Frame-Soil System

Load Factor (1)	Storey Level (2)	Member (3)	BFA (4)	LIA-FS (5)	EPIA-FS (CC) (6)	% Diff. (4 - 6) (7)
0.8	II	B_1	59.59	9.32	9.24	**
			-40.82	-82.19	-82.26	101.34
	I	B_2	56.23	-8.76	-9.82	*
			-48.26	-107.71	-107.80	123.17
	II	B_1	74.49	11.66	11.62	**
1.0			-51.03	-102.74	-102.77	101.34
	I	B_2	70.29	-10.96	-10.94	*
			-60.33	-134.64	-134.63	123.17
	II	B_1	119.18	18.65	+22.0	**
1.6			-81.65	-164.38	-161.73	98.07
Collapse	I	B_2	112.46	-17.53	-17.34	*
			-96.52	-215.42	-210.84	-118.44

CC-Compromise cone yield criterion ** Very high difference in values * Reversal in sign

Table 7. Bending Moments (kN-m) in Floor Beams due to Differential Settlement of soil mass

Storey Level (1)	Member (2)	Plane Frame-Foundation Beam-Soil System (FS) – CC (3)						
		First LF 0.5	Third LF 0.7	Fifth LF 0.9	Seventh LF 1.1	Ninth LF 1.3	Eleventh LF 1.5	Thirteenth LF 1.6
		4.64 mm	6.51 mm	8.37 mm	10.21 mm	12.02 mm	13.58 mm	14.36 mm
II	B_1	+5.83	+8.09	+10.41	+12.88	+15.52	+19.82	+22.00
		-51.37	-71.98	-92.53	-112.97	-123.27	-152.24	-161.73
I	B_2	-1.98	-2.86	-3.64	-4.25	-4.63	-2.90	-2.01
		-67.32	-94.33	-121.25	-147.97	-174.48	-198.70	-210.84

LF- Load factor CC-Compromise yield criteria

Bending Moments in Foundation Beam

Fig. 9 exhibits the distribution of bending moments along the foundation beam of plane frame-soil system due to LIA and EPIA for load factor of unity. The variation resembles the behavior of the beam subjected to column loads from top and upward soil pressure beneath. Both interaction analyses almost depict same behavior and the values are also almost same. Different yield criteria also provide almost same type of variation of bending moments.

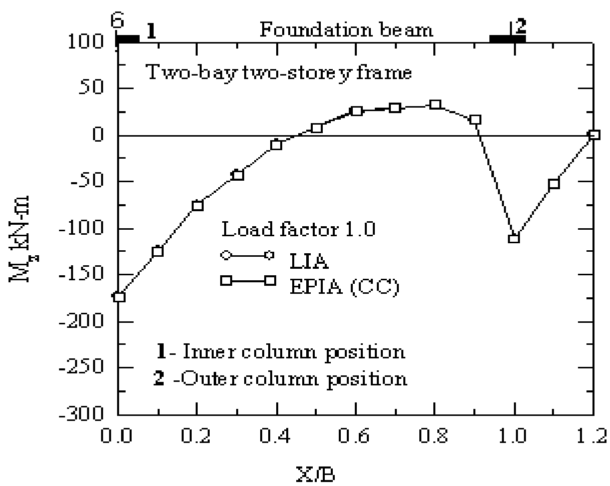


Fig. 9 Variation of Bending Moments in Foundation Beam

Collapse Load due to Various Yield Criteria

In the elasto-plastic analysis, the collapse load is deemed to have occurred if the iterative process diverges for a particular load increment. Table 8 shows the collapse loads due to various yield criteria for plane frame-foundation beam-soil system. Different yield criteria provide different collapse loads, which precisely depend upon the physical properties and the type of material under consideration. The axial extension cone and the Mohr-Coulomb yield criteria give the marginally lower value of collapse load as compared to other yield criteria.

Table 8. Collapse Load for Plane Frame-Soil System

Yield Model	Collapse Load
Mohr-Coulomb (MC)	1.40 q
Drucker-Prager- I (DP)	1.60 q
Compromise cone (CC)	1.60 q
Axial extension cone (AX)	1.40 q

q = 40 kN/m

SUMMARY AND CONCLUSIONS

In the present study, an attempt has been made to investigate the interaction behaviour of plane frame-soil interaction system considering the soil mass to behave in elasto-plastic manner. The forces in the various frame members due to interaction analysis are considerably different from those due to conventional frame analysis. The elasto-plastic interaction analysis suggests that, in general, there takes a transfer of forces and moments from the exterior columns towards the interior ones, below which the soil remains in an elastic state, although the soil mass below the outer edges has fully yielded. This has been observed, in particular, at higher load factors only. The collapse load for the plane frame-foundation beam-soil system is marginally different for different yield criteria. 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 in comparison to conventional approach of building frame design.

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