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Analysis for foundation moments in space frame-shear wall-nonlinear soil system

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Abstract. The soil-structure interaction effect significantly influences the design of multi-storey buildings subjected to lateral seismic loads. The shear walls are often provided in such buildings to increase the lateral stability to resist seismic loads. In the present work, the nonlinear soil-structure analysis of a G+5 storey RC shear wall building frame having isolated column footings and founded on deformable soil is presented. The nonlinear seismic FE analysis is carried out using ANSYS software for the building with and without shear walls to investigate the effect of inclusion of shear wall on the moments in the footings due to differential settlement of soil mass. The frame is considered to behave in linear elastic manner, whereas, soil mass to behave in nonlinear manner. It is found that the interaction effect causes significant variation in the moments in the footings. The comparison of non-interaction and interaction analyses suggests that the presence of shear wall causes significant decrease in bending moments in most of the footings but the interaction effect causes restoration of the bending moments to a great extent. A comparison is made between linear and nonlinear analyses to draw some important conclusions.

Keywords: soil-structure interaction; ANSYS; space frame; shear wall; nonlinear analysis; foundation moments; isolated footings; seismic forces

1. Introduction

RC space frames are conventionally designed assuming the foundations to be resting on unyielding supports. The analysis is carried out by considering bottom end of the columns fixed and neglecting the effect of soil deformations. In reality, any building frame resting on deformable soil results in redistribution of forces and moments due to soil-structure interaction. Thus, conventional analysis is unrealistic and may be unsafe. The interaction effect is more pronounced in case of multi-storeyed buildings due to their heavy loads. When such buildings are subjected to lateral seismic loads, the effect may become further intensified. The shear walls are usually provided in such situation to increase the lateral stability of the building. The study of role of shear walls in the space frame under the influence of soil-structure interaction is a matter of great importance.

In the present work, a G+5 storey RC building frame with isolated footings is considered for 3-

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dimensional nonlinear soil-structure interaction analysis under normal as well as seismic loads using ANSYS software. The analysis is carried out considering the space frame with and without shear walls oriented along the direction of seismic load. Analysis is carried out for various combinations of dead, live and seismic loads, as per IS-1893 (Part-1): 2002. Full 3-dimensional space frame is considered for analysis, which makes the model easily extendable to any configuration of space frame and shear wall. The frame is considered to behave in linear elastic manner, whereas, soil mass to behave in nonlinear manner. The results of non-interaction and interaction analyses are compared for the space frame with and without shear walls to investigate the effect of interaction on the moments in the footings. The results show that there is considerable redistribution of moments in the space frame due to the interaction effect. The provision of shear walls causes significant reduction in the bending moments in most of the column footings especially when structure is resting on deformable soil, but the interaction effect causes restoration of the bending moments to a great extent. A comparison is made between linear and nonlinear analyses to draw some important conclusions.

2. Literature review

Investigation of effects of soil-structure interaction on building frames and foundations have been a subject matter of keen interest for the researchers. Several studies have been carried out in the past and important conclusions are drawn. It is well established that the soil-structure interaction effect causes redistribution of forces in the superstructure. In most of these research works, the building frames and soils were approximated or idealised in various ways. Earlier research considered 2-D idealisation of structure and soil. Research gained momentum with the advent of more powerful tools like the finite element method. With the availability of increasing computing power and sophisticated modelling techniques, three dimensional soil-structure interaction analysis with more realistic idealisation has been witnessed during recent few years. Still a lot of work need to be done to bring the studies from research to actual implementation stage in design offices, overcoming various modelling and analysis complexities.

A double storey, two bay plane frame, having combined footing supported on deformable soil, was considered by Noorzaei *et al.* (1995) for linear and non-linear soil-structure analyses. An analytical study on moment resisting RC frame building having soft first storey and brick infill in the upper storey, with isolated column footings resting on medium soil, was conducted by Arlekar *et al.* (1997). Mylonakis and Gazetas (2000) re-explored the role of soil-structure interaction in the seismic response of the structures using recorded motions and theoretical considerations and pointed out that the effect of soil-structure interaction may not always be beneficial.

Stavridis (2002) considered an arbitrary structure to present a simplified analysis approach for layered soil-structure analysis based on a purely analytical treatment of the underlying soil models. A fifty storey symmetrical building was approximated by a 2-dimensional model by Edgers *et al.* (2005) to study the soil -structure interaction effects using ANSYS software. Hora (2006) proposed a computational methodology for nonlinear interaction analysis of infilled frame-foundation-soil system using coupled finite-infinite elements for the soil mass. Yahyai *et al.* (2008) carried out 2-D analysis of two adjacent 32 storey buildings, with variable distance, subjected to seismic loading to investigate the effect of soil -structure interaction between them. Natarajan and Vidivelli (2009) carried out nonlinear analysis using ANSYS software to examine the influence of column spacing on behaviour of a space frame raft foundation soil system under static loading. Interaction and non-interaction analyses for the space frame-raft foundation-soil system, with varying soil and raft stiffness, were compared by Thangaraj and Illamparuthi (2010) using ANSYS software.

Effect of differential settlement of foundations on nonlinear interaction behavior of a 2-bay 10-storey plane frame-soil system was studied by Agrawal and Hora (2010) using coupled finite-infinite elements. Shakib and Atefatdoost (2011) studied the effect of soil structure interaction on torsional response of an asymmetrical wall type system having a slab supported on two asymmetrical walls with foundation resting over soil. Garg and Hora (2012) evaluated the effect of soil-structure interaction on a 3-bay, 3-storey RCC space frame-footing-strap beam-soil system using ANSYS software. Renzi *et al.* (2013) presented a simplified empirical method for assessing seismic soil-structure interaction effects on ordinary shear-type buildings. Tabatabaiefar *et al.* (2013) implemented finite difference method to investigate the effects of dynamic soil-structure interaction on seismic behaviour and lateral structural response of mid-rise moment resisting building frames. Hokmabadi *et al.* (2014) evaluated seismic-soil-pile-structure interaction of mid-rise buildings by carrying out experimental and numerical studies. Jain and Hora (2015) investigated the effect of inclusion of shear walls on the total and differential settlements in the footings of a space frame due to deformations in the soil mass.

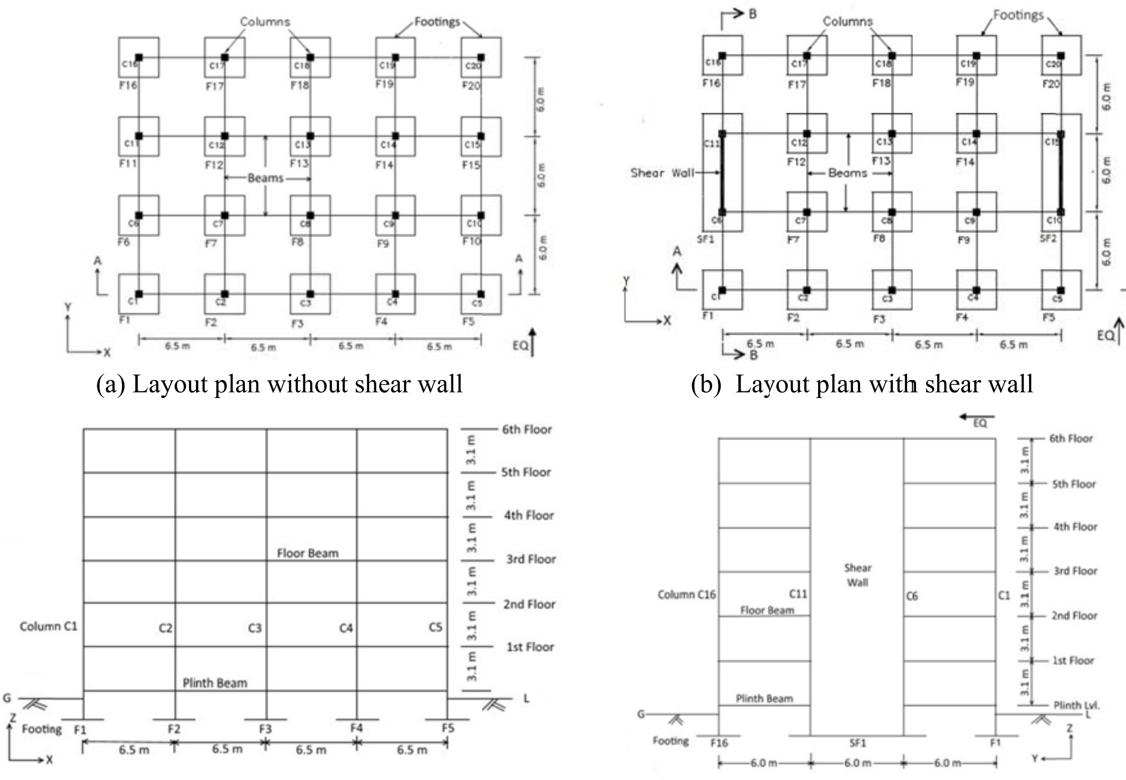
The present situation demands 3-D soil-structure-interaction analysis of important structures for more realistic and accurate analysis. The present study is an effort in that direction.

3. Problem for investigation

A six storey RC framed building having isolated column footings and resting on homogeneous soil mass is considered in this study as shown in Figs. 1(a)-(d). To resist lateral forces, a dual system, comprising of special moment resisting frames (SMRF) and reinforced concrete shear walls, is considered. The shear walls are provided on the outer frames along *Y*-direction i.e., the assumed direction of lateral seismic forces. Such buildings are very common in urban areas. The space frame, shear walls and soil mass are considered to act as a single compatible structural unit for the interaction analysis. The interaction analyses are carried out with and without shear walls.

Table 1 Geometric parameters of the space frame-shear wall-soil system

Description of parameter	Value/size
Number of storeys	6
Storey height	3.1 m
Slab/ shear wall thickness	200 mm
Column sizes:	
(i) Foundation to 3rd floor	500 mm×500 mm
(ii) 3rd floor to 6th floor	400 mm×400 mm
Beam size	300 mm×500 mm
Depth of foundation below G.L.	1.5 m
Height of Plinth above G.L.	0.6 m
Footing size below column	3 m×3 m×0.5 m
Footing size below shear wall	3 m×9 m×0.5 m
Semi-infinite extent of soil mass	100 m×100 m×25 m



(a) Layout plan without shear wall (b) Layout plan with shear wall

(c) Front sectional elevation (section A-A) (d) Side sectional elevation (section B-B)

Fig. 1 Geometry of space frame with and without shear wall

The building is considered to be situated in seismic zone V of India. For the linear interaction analysis, super-structure, foundation, as well as soil are considered to behave in linear elastic manner. Nonlinear stress-strain behaviour of soil is considered in the nonlinear interaction analysis.

The geometrical properties of the space frame-shear wall-soil system are provided in Table 1.

It is a general practice to reduce the column sizes in upper floors due to lesser loads. Though, this is not going to make any major difference in the results of soil-structure interaction analysis, the column sizes are reduced from 3rd floor to 6th floor to make the analysis even more realistic.

The nonlinear stress-strain curve of soil (Bishop and Henkel 1962), shown in Fig. 2, is considered for nonlinear analysis. For linear interaction analysis, the initial tangent modulus of soil is taken as 14.78 N/mm^2 considering the same curve. The material properties of concrete and soil are provided in Table 2.

The building is considered to be an institutional building. The live loads are considered as per IS 875 (Part 2):1987. The live loads of 4 KN/m^2 on floors and 1.5 KN/m^2 on roof are considered. The brick masonry wall on outer periphery of the building and parapet wall on roof are also considered. The details of various loads considered are given in Table 3. These are in addition to the self-weight of the structure.

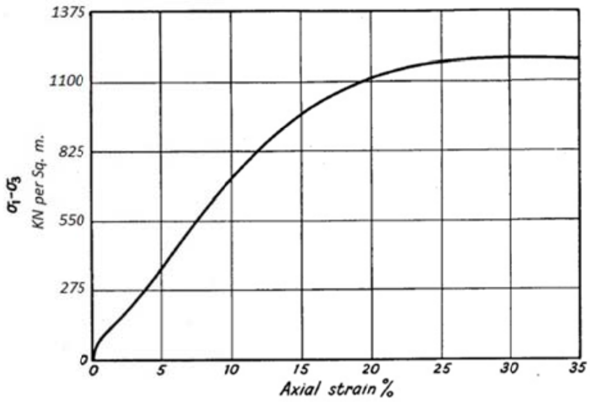


Fig. 2 Nonlinear stress-strain curve of the soil

Table 2 Material properties of concrete and soil

Property	Value
Grade of concrete for all structural components	M25
Modulus of elasticity of concrete	25000 N/mm^2
Poisson's ratio of concrete	0.15
Density of concrete	25000 N/m^3
Elastic Modulus of soil	14.78 N/mm^2
Poisson's ratio of soil	0.35

Table 3 Dead load and live load on structure

Description	Value
Dead load of floor finish	1 KN/m^2
Dead load of finishing and water proofing on roof	2.5 KN/m^2
Live load on floors	4 KN/m^2
Live load on roof	1.5 KN/m^2
Brick walls (only on outer periphery of floor/plinth)	11.362 KN/m
Parapet wall on roof periphery	4.37 KN/m

4. Seismic load calculations

The equivalent static lateral force method [IS 1893 (Part 1): 2002] is adopted for evaluation of seismic forces:

4.1 Calculation of lumped masses to various floor levels

The seismic loads are calculated for full dead load plus the percentage of imposed load as given Table 8 of IS 1893 (Part 1): 2002. Accordingly, 50% of live load on floors and 25% of live load on roof is considered.

The lumped mass of each floor is worked out by adding mass of slab, mass of reduced live load

on slabs, mass of beams in longitudinal as well as transverse directions at that floor, mass of column for half column height above and below floor, mass of wall for half height above and below beams (wall is considered only on outer periphery), mass of parapet wall on outer periphery beams on roof.
Seismic weight of floor= lumped masses of floors *x* g
g= Acceleration due to gravity
W=Seismic weight of building (sum of seismic weights of all floors)

4.2 Determination of fundamental natural period of the shear wall-space frame

The approximate fundamental natural period of vibration (T_a) of the space frame-shear wall structure is estimated as per the empirical expression given in the clause 7.6.1 of IS 1893 (Part 1): 2002:
T_a=0.075 $h^{0.75}$
Where *h*=height of building, in m.

No provision exists in IS 1893 (Part 1):2002 to account for the change in fundamental natural period of vibration due to soil-structure interaction effect. Any such change in natural period and consequent change in design base shear (up or down, depending on spectral shape) is beyond the scope of this study.

4.3 Determination of design base shear

The design base shear is calculated as per clause 7.5.3 of IS 1893 (Part 1): 2002:
The design seismic base shear is V_B=A_h W
A_h=Design horizontal acceleration spectrum coefficient, as per clause 6.4.2 of IS 1893 (Part 1): 2002.
W=Seismic weight of the building
A_h=(Z/2)×(*I*/*R*)×(S_a/*g*)
Z=Zone factor [Table 2 of IS 1893 (Part 1): 2002].
I=Importance factor [Table 6 of IS 1893 (Part 1): 2002].
R=Response reduction factor, depending on the perceived seismic damage performance of the building [Table 7 of IS 1893 (Part 1): 2002].
S_a/*g*=Average response acceleration coefficient for soil for 5% damping [Fig. 2 of IS 1893 (Part 1): 2002] for the natural period as worked out above.

4.4 Determination of vertical distribution of base shear to different floor levels

The design seismic base shear, V_B is distributed to different floor levels along the height of the building as per the clause 7.7.1 of IS 1893 (Part 1): 2002

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$

Where,
Q_i=Design lateral force at floor '*i*',
W_i=Seismic weight of floor '*i*',
h_i=Height of floor i measured from base, and

Table 4 Parameters for Lateral Seismic Load calculations on the structure

Parameter	Value
Earthquake zone	V
Zone factor 'Z' (Table 2 of IS 1893 (Part 1): 2002)	0.36
Importance factor 'I' (Table 6 of IS 1893 (Part 1): 2002)	1.5
Response reduction factor 'R' (Table 7 of IS 1893 (Part 1): 2002) (Ductile shear wall with SMRF)	5.0
Approximate fundamental natural period of vibration (T_a) T_a=0.075 h$^{0.75}$=0.075 (20.7)$^{0.75}$=0.728 (as per clause 7.6.1 of IS 1893 (Part 1): 2002)	0.728 sec
Average response acceleration coefficient (S_a/g) S_a/g=1.36/T_a (for soil for 5% damping, as given in Fig. 2 of IS 1893 (Part 1): 2002, for the natural period T_a of 0.728 sec)	1.868

Table 5 Lateral seismic loads at various floor levels

Floor level	Intensity of seismic load (KN)	
	Space frame without shear wall	Space frame with shear wall
6	1181.5	1185.4
5	899.6	906.6
4	610.6	615.4
3	382.4	385.4
2	205.4	206.9
1	80.6	81.2
0	3.9	4.1

n=Number of storeys in the building is the number of levels at which masses are located

4.5 Distribution of design lateral force at floor level to different frames of the structure

The design lateral force at floor level is distributed amongst the frames in the direction considered for seismic load (i.e., Y-direction in present analysis) in proportion to their stiffness [clause 7.7.2.1 of IS 1893 (Part 1): 2002].

The parameters used for the calculation of seismic load are shown in Table 4.

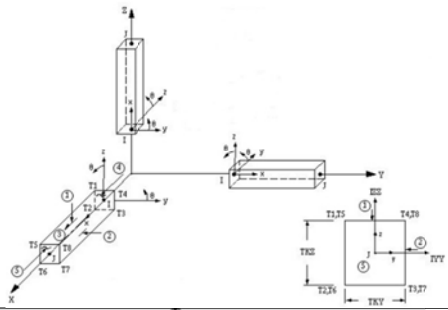
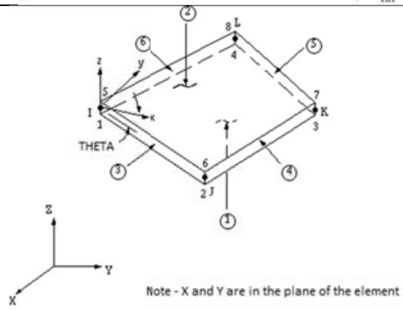
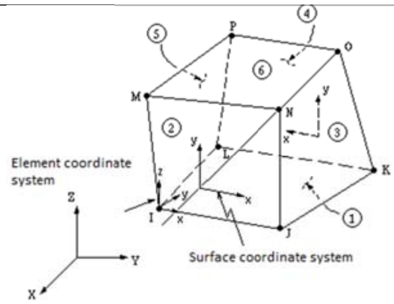
The calculated values of design lateral seismic loads are as shown in Table 5.

5. Finite element idealization

The finite element modelling is carried out using ANSYS software. It has wide range of inbuilt elements and material models suited for the problem under consideration.

Table 6 shows the elements adopted for modelling various structural components and soil mass.

Table 6 Details of elements

Structural component	Element designation and type	No. of nodes/DOF	Sketch
Columns and beams	BEAM 4 3D elastic beam	2 nodes (Six DOF at each node) (translations in the nodal x, y and z directions and rotations about the nodal x, y and z axes)	
Floor slabs and shear walls	SHELL63 Elastic shell	4 nodes (6 DOF at each node) (translations in the nodal x, y and z directions and rotations about the nodal x, y and z axes)	
Footings and Soil mass	SOLID 45 3D structural solid	8 nodes (3 DOF at each node) (translations in the nodal x, y, and z directions)	

Element designation in ANSYS, its type, number of nodes and degrees of freedom (DOF) at each node are described in the table.

To establish contact/ interface between foundation and soil, surface to surface contact elements namely CONTA174 and TARGE170 are used. Contact behavior is set as 'standard' to allow sliding and lifting of footings. Coefficient of friction between concrete and soil is taken as 0.5.

5.1 Semi-infinite extent of soil mass

Homogeneous isotropic soil mass with nonlinear elastic stress-strain relationship is considered in the study. Bed rock is very often encountered at many sites at varying depth. It is assumed that bed rock is available 25 m below top of the soil in the present case.

To decide the horizontal extent of soil in the X and Y directions, numerous trial analyses were carried out with different soil extents considering linear interaction analysis. It is found that the displacements in the soil gets reduced to less than 5% of the maximum value at a distance of nearly 25 m from the building line under worst combination of vertical loads and about 30 m in case of worst combination with lateral loads. These values of displacements become negligible as we move further away from the building line by nearly 10 to 15 m. Therefore, horizontal extent of 100 m in *X* as well as *Y* direction is adopted by considering the soil participating volume of 100 m×100 m×25 m, which is sufficient enough to capture the dominant effect of soil-structure-interaction for the problem under consideration.

5.2 Mesh optimization and boundary conditions

The finite element discretization is achieved through the mesh tool of ANSYS. The extensive mesh refinement has been carried out for the structure as well as for the soil mass through several trial runs to achieve the most efficient mesh giving converged results with least processing time.

Several mesh sizes were tried for the structural components to observe the effect of increasing number of mesh division on maximum displacement and maximum stresses. No appreciable change in results is observed beyond 10 divisions. To capture reasonably accurate results about 8 to 10 divisions are found sufficient. The finer meshing consumes considerable execution time for the interaction analysis. Finally, the mesh size of 500 mm is adopted for beams, slabs, shear wall and footings resulting in 12 to 13 divisions. For the columns 12 mesh divisions are found suitable with a mesh size of about 300 mm. The discretization of the structure is shown in Fig. 3.

It is found that keeping equal mesh size for entire soil domain, results in inefficient model, consuming excessive processing time even for preliminary results. The graded meshing is, therefore, adopted, keeping finer mesh for soil in nearby region of building (in view of deformations and stresses being of higher order) and increasing mesh size for soil mass away from the building line. Several different graded mesh sizes were tried and it is found that mesh size of 500 mm near building region, though consumes somewhat more processing time but provides efficient and better converged results. The finite element discretization is shown in Fig. 4.

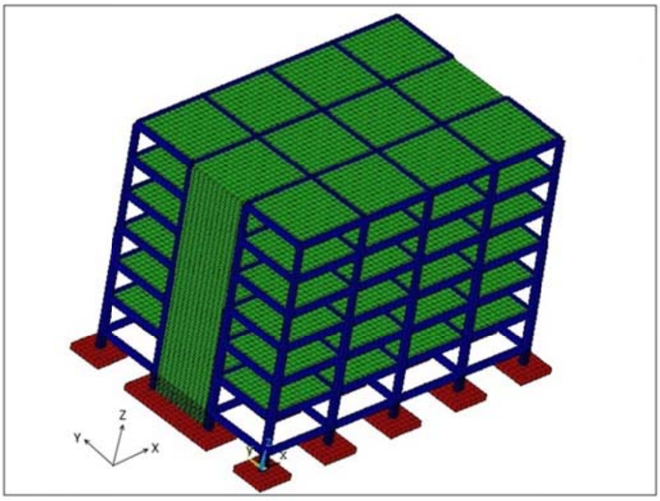


Fig. 3 Finite element discretization of structural components of the interaction system

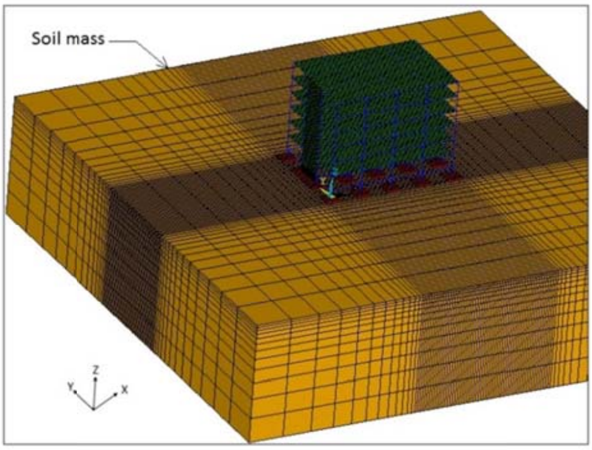


Fig. 4 Finite element discretization of space frame-shear wall-soil system

The vertical displacement at the bottom and horizontal displacements at the side boundaries of the participating volume of soil mass (100 m×100 m×25 m), are restrained.

6. Nonlinear interaction analysis

The linear and nonlinear interaction analyses of the space frame-soil system and space frame-shear wall-soil system are carried out assuming the structure, shear wall and soil to act as a single compatible structural unit. The linear analysis is carried out assuming that all components of the interaction system behave in linear elastic manner, whereas, in the nonlinear analysis, the soil mass is considered to behave in nonlinear elastic manner and stress-strain curve (Bishop and Henkel 1962) is adopted. The non-interaction analysis (with and without shear wall) are also carried out for comparison of the results with the interaction analyses.

The following analyses are carried out for the structural system;

Case 1: The conventional (non-interaction) analysis of space frame without shear wall **(NIA-SW)** considering columns fixed at their bases.

Case 2: The conventional (non-interaction) analysis of space frame with shear wall **(NIA+SW)** considering columns fixed at their bases.

Case 3: The linear interaction analysis of the space frame-soil system without shear wall **(LIA-SW)** considering the columns supported on isolated footings and resting on deformable soil.

Case 4: The linear interaction analysis of space frame-shear wall-soil system **(LIA+SW)**, considering the columns supported on isolated footings and resting on deformable soil.

Case 5: The nonlinear interaction analysis of space frame-soil system without shear wall **(NLIA-SW)** considering the columns supported on isolated footings and resting on deformable soil.

Case 6: The nonlinear interaction analysis of space frame-shear wall-soil system **(NLIA+SW)**, considering the columns supported on isolated footings and resting on deformable soil.

For each of the these analyses, the following combinations of dead load (DL), live load (LL) and seismic load (EL) are considered as per Clause 6.3.1.2 of IS 1893 (Part 1): 2002, as shown in Table 7.

Table 7 Load combinations

S. No.	Load case	Load combination
1	LC1	1.5(DL+LL)
2	LC2	1.2(DL+LL+EL)
3	LC3	1.2(DL+LL-EL)
4	LC4	1.5(DL+EL)
5	LC5	1.5(DL-EL)
6	LC6	0.9DL+1.5EL
7	LC7	0.9DL-1.5EL

One of the purposes of this study is to compare the effect of inclusion of shear walls in the space frame in the structurally weaker direction (i.e., *Y*-direction) in resisting seismic forces. To accomplish the same, the analyses are carried out for the seismic forces assumed to act in the *Y*-direction only.

The positive sign of seismic load shows that it is applied from front whereas negative sign shows that it is applied from back i.e., from the opposite direction.

7. Results and discussion

The results of non-interaction, linear and nonlinear interaction analyses are compared to investigate the following;

- Bending moment in the footings about *x*-axis (M_x)
- Bending moment on the footings about *y*-axis (M_y)

The results are discussed to highlight the effect of shear wall. The results are tabulated taking advantage of symmetry and hence only quarter portion of the problem is considered for tabulation of results. Thus, the bending moments are tabulated for the footings F1, F2, F3, F6, F7, F8 and SF1, owing to the symmetry.

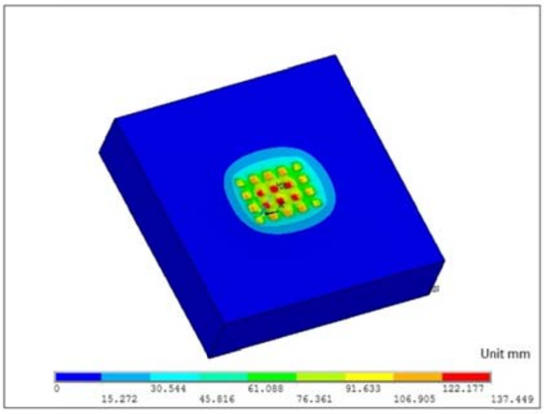
Due to interaction effect, differential settlements take place in the footings, which results in redistribution of moments in the footings. Figs. 5(a)-(d) show the settlements in the footings for NLIA-SW and NLIA+SW systems, under vertical and seismic loads.

7.1 Bending moment M_x in footings

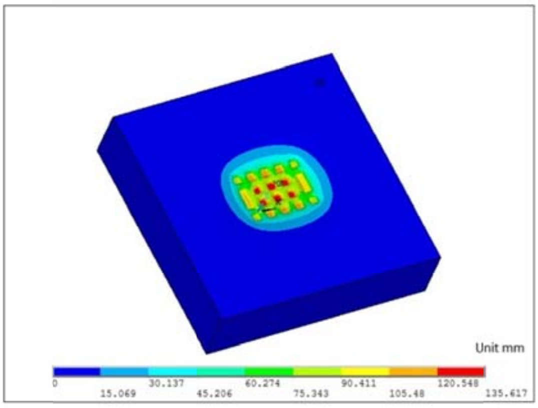
Table 8 shows the comparison of bending moment M_x in the footings for NIA-SW, LIA-SW and NLIA-SW systems.

Under vertical loads (Load case LC1), LIA results in highly significant increase in the values of M_x in all the footings. The maximum increase of nearly 23 times is found in the footing F3 and the minimum increase of nearly 2.5 times is found in the footing F7. NLIA also results in highly significant increase in the values of M_x in all the footings except footing F7 in which marginal decrease is found. The maximum increase of nearly 17 times is found in the footing F3 and the decrease of nearly 13% is found in the footing F7. Change of sign in the value takes place in the footing F6.

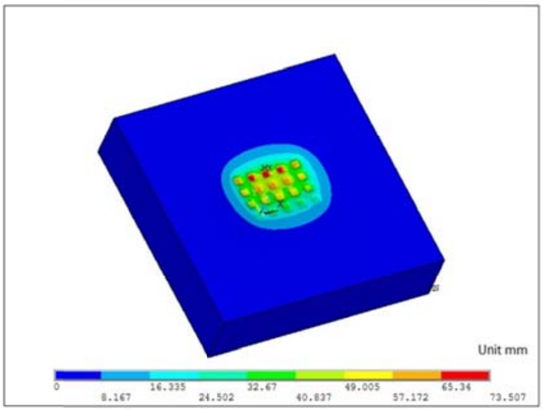
For most of the seismic load combinations, the LIA results in significant increase in the values



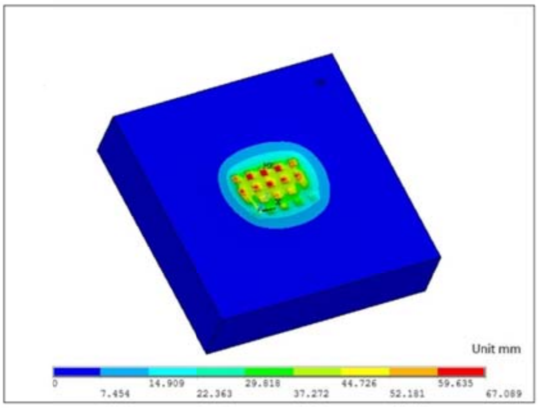
(a) NLIA-SW system : load case LC1



(b) NLIA+SW system : load case LC1



(c) NLIA-SW system: load case LC6



(d) NLIA+SW system: load case LC6

Fig. 5 settlements in the footings under vertical and typical seismic loads

of M_x in most of the footings but decrease is also found in some of the cases. The maximum increase of nearly 92% and the maximum decrease of nearly 14% is found in the footing F3 under different seismic load combinations. The NLIA results in significant increase in the values of M_x in all of the footings under all seismic load combinations. The maximum increase of nearly 84% is found in the footing F3 and the minimum increase of nearly 7% is found in the footing F8.

NLIA causes further significant changes in the values in many cases as compared to LIA.

The comparison of bending moment M_x in the footings for NIA+SW, LIA+SW and NLIA+SW systems is provided Table 9. Highly significant increase is found in the bending moments in all footings due to the interaction effect.

Under vertical loads (load case LC1), the maximum increase of nearly 23 times in LIA and nearly 17 times in NLIA is found in footing F3. The minimum increase of nearly 3 times in LIA and nearly 50% in NLIA is found in footing F8.

Due to all combinations of seismic loads (LC2 to LC7), the maximum increase of nearly 12.5 times is found in the footing SF1 (under column C6) and the minimum increase of nearly 6 times is found in the footing F8 in LIA. Under NLIA, the maximum increase of nearly 15 times is found in the footing SF1 (under column C6) and the minimum increase of nearly 7 times is found in the footing F1.

NLIA causes significant change in the values in many cases as compared to LIA.

Table 10 shows change in bending moments M_x in the footings under various load cases due to presence of shear wall in the space frame.

Table 8 Comparison of bending moment (M_x) in the footings for NIA-SW, LIA-SW and NLIA-SW systems under various load cases

Footing Designation	Coordinates (m)			Analysis type	Bending moment M_x for various load cases (KN-m)						
	X	Y	Z		LC1	LC2	LC3	LC4	LC5	LC6	LC7
F1	0.0	0.0	0.0	NIA-SW	15.74	-325.09	350.00	-407.85	436.02	-413.56	430.32
				LIA-SW	143.31	-383.35	626.64	-507.54	756.46	-522.98	724.81
				% diff.#	*810.25*	*17.92*	*79.04*	*24.44*	*73.49*	*26.46*	*68.44*
				NLIA-SW	100.83	-464.05	623.49	-603.39	758.41	-559.12	719.03
				% diff.#	*540.43*	*42.75*	*78.14*	*47.94*	*73.94*	*35.20*	*67.09*
F2	6.5	0.0	0.0	NIA-SW	10.39	-344.34	360.69	-433.50	447.78	-436.43	444.86
				LIA-SW	225.44	-314.80	680.07	-441.39	800.74	-492.38	742.85
				% diff.#	*2070.20*	*-8.58*	*88.55*	*1.82*	*78.82*	*12.82*	*66.99*
				NLIA-SW	163.29	-395.82	658.31	-532.38	789.53	-529.23	729.56
				% diff.#	*1471.91*	*14.95*	*82.51*	*22.81*	*76.32*	*21.26*	*64.00*
F3	13.0	0.0	0.0	NIA-SW	10.09	-344.94	360.81	-434.20	447.99	-437.03	445.16
				LIA-SW	245.68	-296.59	691.95	-424.93	808.84	-484.71	745.27
				% diff.#	*2334.64*	*-14.02*	*91.78*	*-2.13*	*80.55*	*10.91*	*67.42*
				NLIA-SW	176.69	-379.54	664.33	-516.05	793.61	-521.89	729.82
				% diff.#	*1650.97*	*10.03*	*84.12*	*18.85*	*77.15*	*19.42*	*63.95*
F6	0.0	6.0	0.0	NIA-SW	2.00	-374.77	377.85	-470.13	470.65	-470.26	470.51
				LIA-SW	12.41	-408.39	426.40	-513.33	532.02	-527.79	538.17
				% diff.#	*520.12*	*8.97*	*12.85*	*9.19*	*13.04*	*12.23*	*14.38*
				NLIA-SW	-3.51	-466.67	459.27	-578.39	580.42	-562.67	570.31
				% diff.#	*75.50**	*24.52*	*21.55*	*23.03*	*23.32*	*19.65*	*21.21*
F7	6.5	6.0	0.0	NIA-SW	7.48	-386.27	398.13	-486.12	494.38	-487.80	492.71
				LIA-SW	24.52	-376.94	413.03	-475.42	509.87	-496.79	514.59
				% diff.#	*227.92*	*-2.42*	*3.74*	*-2.20*	*3.13*	*1.84*	*4.44*
				NLIA-SW	6.53	-438.38	446.53	-543.64	563.64	-531.41	548.95
				% diff.#	*-12.70*	*13.49*	*12.16*	*11.83*	*14.01*	*8.94*	*11.41*
F8	13.0	6.0	0.0	NIA-SW	7.41	-386.73	398.49	-486.70	494.83	-488.35	493.18
				LIA-SW	28.18	-368.13	409.86	-465.54	504.53	-488.91	509.16
				% diff.#	*280.36*	*-4.81*	*2.85*	*-4.35*	*1.96*	*0.11*	*3.24*
				NLIA-SW	9.23	-430.77	443.04	-534.58	558.97	-523.36	543.57
				% diff.#	*24.57*	*11.39*	*11.18*	*9.84*	*12.96*	*7.17*	*10.22*

%difference with NIA-SW results; *with reversal of sign

Table 9 Comparison of bending moment (M_x) in the footings for NIA+SW, LIA+SW and NLIA+SW systems under various load cases

Footing Designation	Analysis type	Bending moment M_x for various load cases (KN-m)						
		LC1	LC2	LC3	LC4	LC5	LC6	LC7
F1	NIA+SW	20.85	-39.44	72.26	-51.54	88.08	-58.98	80.64
	LIA+SW	147.22	-336.51	593.37	-447.46	714.76	-439.89	671.42
	% diff.#	*606.12*	*753.28*	*721.16*	*768.16*	*711.48*	*645.79*	*732.62*
	NLIA+SW	111.05	-396.70	572.13	-521.15	685.92	-470.20	647.48
	% diff.#	*432.64*	*905.91*	*691.77*	*911.14*	*678.74*	*697.18*	*702.94*
F2	NIA+SW	10.02	-30.61	46.65	-41.23	55.35	-44.05	52.53
	LIA+SW	226.70	-245.73	605.42	-357.22	706.20	-423.30	647.60
	% diff.#	*2163.15*	*702.67*	*1197.85*	*766.49*	*1175.86*	*860.98*	*1132.87*
	NLIA+SW	168.22	-316.11	591.93	-434.80	700.05	-455.85	641.29
	% diff.#	*1579.35*	*932.57*	*1168.93*	*954.67*	*1164.75*	*934.87*	*1120.85*
F3	NIA+SW	10.10	-30.57	46.74	-41.24	55.40	-44.07	52.57
	LIA+SW	244.68	-227.83	613.50	-340.85	710.20	-416.98	646.35
	% diff.#	*2322.33*	*645.18*	*1212.69*	*726.46*	*1182.04*	*846.13*	*1129.60*
	NLIA+SW	177.01	-303.09	594.19	-420.83	700.88	-450.23	639.74
	% diff.#	*1652.40*	*891.33*	*1171.38*	*920.39*	*1165.22*	*921.58*	*1117.02*
SF1 (below column C6)	NIA+SW	16.72	-97.37	124.05	-127.41	149.37	-131.82	144.96
	LIA+SW	101.57	-1297.40	1458.40	-1646.90	1794.50	-1712.40	1795.80
	% diff.#	*507.44*	*1232.42*	*1075.65*	*1192.60*	*1101.38*	*1199.04*	*1138.82*
	NLIA+SW	104.22	-1524.50	1688.20	-1921.90	2070.80	-1824.60	1906.60
	% diff.#	*523.29*	*1465.65*	*1260.90*	*1408.44*	*1286.36*	*1284.16*	*1215.26*
F7	NIA+SW	7.39	-37.09	48.92	-49.66	57.86	-51.30	56.22
	LIA+SW	32.21	-309.20	358.44	-391.80	440.06	-416.56	445.95
	% diff.#	*335.70*	*733.56*	*632.68*	*688.90*	*660.61*	*711.98*	*693.25*
	NLIA+SW	16.25	-367.04	395.33	-457.50	492.81	-448.72	475.48
	% diff.#	*119.89*	*889.49*	*708.08*	*821.19*	*751.79*	*774.66*	*745.78*
F8	NIA+SW	7.43	-37.12	49.01	-49.69	57.97	-51.34	56.32
	LIA+SW	28.45	-308.68	350.98	-390.98	430.71	-415.97	436.72
	% diff.#	*282.71*	*731.57*	*616.20*	*686.90*	*642.96*	*710.18*	*675.49*
	NLIA+SW	10.91	-363.54	382.40	-451.78	478.73	-445.00	464.86
	% diff.#	*46.76*	*879.36*	*680.31*	*809.27*	*725.80*	*766.72*	*725.46*

%difference with NIA-SW results

Under load case LC1, the presence of shear wall causes highly significant increase of nearly 7.5 times in the bending moments M_x in shear wall footing SF1 (below column C6) for NIA, nearly 7 times in case of LIA and nearly 29 times (with change of sign) in NLIA. Insignificant change is found in footings F2 and F3. In remaining footings lesser significant changes are found.

Due to all combinations of seismic loads, the NIA suggests that the presence of shear wall

Table 10 Change in bending moments (M_x) in the footings under various load cases due to presence of shear wall in the space frame

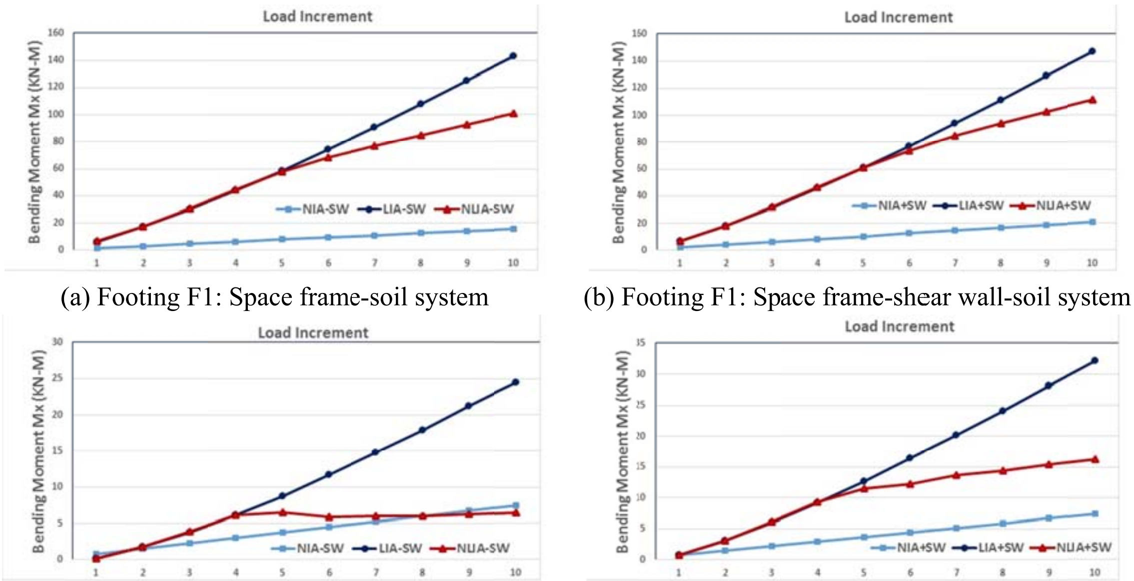
Footing Desig-nation	Cases compared	% change in bending moment M_x for various load cases						
		LC1	LC2	LC3	LC4	LC5	LC6	LC7
F1	NIA+SW and NIA-SW	32.43	-87.87	-79.35	-87.36	-79.80	-85.74	-81.26
	LIA+SW and LIA-SW	2.73	-12.22	-5.31	-11.84	-5.51	-15.89	-7.37
	NLIA+SW and NLIA-SW	10.14	-14.51	-8.24	-13.63	-9.56	-15.90	-9.95
F2	NIA+SW and NIA-SW	-3.57	-91.11	-87.07	-90.49	-87.64	-89.91	-88.19
	LIA+SW and LIA-SW	0.56	-21.94	-10.98	-19.07	-11.81	-14.03	-12.82
	NLIA+SW and NLIA-SW	3.02	-20.14	-10.08	-18.33	-11.33	-13.87	-12.10
F3	NIA+SW and NIA-SW	0.10	-91.14	-87.05	-90.50	-87.63	-89.92	-88.19
	LIA+SW and LIA-SW	-0.41	-23.18	-11.34	-19.79	-12.20	-13.97	-13.27
	NLIA+SW and NLIA-SW	0.18	-20.14	-10.56	-18.45	-11.68	-13.73	-12.34
F6 (SF: below column C6)	NIA+SW and NIA-SW	735.67	-74.02	-67.17	-72.90	-68.26	-71.97	-69.19
	LIA+SW and LIA-SW	718.58	217.69	242.03	220.83	237.30	224.45	233.69
	NLIA+SW and NLIA-SW	2869.23 (*)	226.68	267.58	232.28	256.78	224.28	234.31
F7	NIA+SW and NIA-SW	-1.15	-90.40	-87.71	-89.78	-88.30	-89.48	-88.59
	LIA+SW and LIA-SW	31.34	-17.97	-13.22	-17.59	-13.69	-16.15	-13.34
	NLIA+SW and NLIA-SW	148.99	-16.27	-11.47	-15.85	-12.57	-15.56	-13.38
F8	NIA+SW and NIA-SW	0.34	-90.40	-87.70	-89.79	-88.28	-89.49	-88.58
	LIA+SW and LIA-SW	0.96	-16.15	-14.37	-16.02	-14.63	-14.92	-14.23
	NLIA+SW and NLIA-SW	18.21	-15.61	-13.69	-15.49	-14.35	-14.97	-14.48

*with reversal of sign

causes significant decrease of nearly 65 to 90% in the values of bending moments in the footings. However, the interaction effect causes substantial reversal effect (restoration) in the bending moments except in the shear wall footings in which increase of nearly 2 to 2.5 times is found. The significant decrease of nearly 5 to 23% in LIA and 8 to 20% in NLIA is found in the footings other than shear wall footings.

Thus, it is noticed that the interaction effect causes significant change in the value of M_x in the footings. The results obtained from NIA may be highly misleading in deformable soils as moments in the footings do not actually get reduced to that extent as that in case of NIA. Therefore, the footings of shear wall space frame, designed on the basis of NIA, may be more vulnerable.

Figs. 6(a)-(d) show variation of bending moment M_x in the footings with load increments for the vertical loads (load case LC1). The corner footings F1 and the inner footing F7 are selected for the comparison. Both, the space frame-soil system as and space frame-shear wall-soil system are considered. The results of NIA, LIA and NLIA are plotted for comparison. It can be seen that LIA results in highly significant increase in the values of M_x in the footing F1 and significant increase in the footing F7 as compared to NIA. NLIA also results in highly significant increase in the footing F1 but to somewhat lesser extent as compared to LIA. Up to about 40% load increment, the results of LIA and NLIA are almost the same. Thereafter, with each further load increment, the



(a) Footing F1: Space frame-soil system (b) Footing F1: Space frame-shear wall-soil system

(c) Footing F7: Space frame-soil system (d) Footing F7: Space frame-shear wall-soil system

Fig. 6 Variation of bending moment M_x in the footings with the load increments under load case LC1

difference in results go on increasing in a nonlinear manner.

7.2 Bending moment M_y in footings

Table 11 shows comparison of bending moment (M_y) in the footings of NIA-SW, LIA-SW and NLIA-SW systems.

The footings F3 and F8 are not subjected to moment M_y, as they exist on the line of symmetry. Significant increase in the values of M_y is found in all the footings under all the load cases under LIA except in the footing F2 under load case LC7. The maximum increase of nearly 14.5 times is found in footing F6 and the maximum decrease of nearly 20% is found in the footing F2. Under NLIA also significant increase is found in most of the footings under most of the load cases. However, in a few cases decrease is also found. The maximum increase of nearly 12 times is found in the footing F6 and the maximum decrease of nearly 75% is found in the footing F7. The highly significant increase in otherwise meagre values converts them into significant values.

Reversal in the sign is found in footing F2 under LIA and NLIA for certain load cases, as compared to NIA.

The comparison of bending moment M_y in the footings of NIA+SW, LIA+SW and NLIA+SW systems is shown in Table 12.

The footings F3 and F8 are not subjected to moment M_y, as they exist on the line of symmetry. The interaction effect causes significant increase in values of M_y in all the footings except F7 in which increase/decrease is found under different load cases. The maximum increase of nearly 88 times in footing F2 and the maximum decrease of nearly 94% in footing F7 occur under LIA. Whereas, the maximum increase of nearly 30 times in footing F2 and the maximum decrease of nearly 54% in footing F7 occur under NLIA. The highly significant increase in otherwise meagre values converts them into significant values.

Reversal in the sign is also found in the footings F2 and F7 under some load cases.

Table 13 shows change in bending moments M_y in the footings under various loads due to presence of shear wall in the space frame.

Table 11 Comparison of bending moment (M_y) in the footings for NIA-SW, LIA-SW and NLIA-SW systems under various load cases

Sr. No.	Footing Designation	Analysis type	Bending moment M_y for various load cases (KN-m)						
			LC1	LC2	LC3	LC4	LC5	LC6	LC7
1	F1	NIA-SW	-18.91	-14.07	-16.21	-15.36	-18.03	-8.68	-11.35
		LIA-SW	-128.55	-82.53	-109.43	-72.75	-105.82	-24.03	-58.87
		% diff.#	*579.76*	*486.51*	*575.20*	*373.74*	*487.07*	*176.73*	*418.50*
		NLIA-SW	-84.40	-70.38	-73.32	-64.63	-69.96	-22.85	-47.31
		% diff.#	*346.31*	*400.16*	*352.39*	*320.90*	*288.10*	*163.10*	*316.69*
2	F2	NIA-SW	-2.65	-1.05	-3.22	1.17	-1.54	1.24	-1.48
		LIA-SW	-16.34	-12.39	-9.58	-11.87	-7.98	-3.13	-1.16
		% diff.#	*517.24*	*1081.59*	*197.62*	*914.53 (*)*	*416.92*	*152.42 (*)*	*-21.66*
		NLIA-SW	8.27	-0.35	9.58	-4.65	7.19	-2.79	5.13
		% diff.#	*212.08 (*)*	*-66.56*	*197.52 (*)*	*297.44 (*)*	*366.88 (*)*	*125.00 (*)*	*246.62 (*)*
3	F3	NIA-SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		LIA-SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		% diff.#	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*
		NLIA-SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		% diff.#	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*
4	F6	NIA-SW	-14.25	-11.57	-11.22	-10.01	-9.57	-6.09	-5.66
		LIA-SW	-197.90	-147.70	-155.98	-138.01	-148.25	-69.96	-79.39
		% diff.#	*1288.87*	*1176.58*	*1289.82*	*1279.27*	*1448.67*	*1048.95*	*1303.58*
		NLIA-SW	-140.86	-115.53	-119.20	-111.62	-116.51	-68.12	-74.57
		% diff.#	*888.56*	*898.53*	*962.10*	*1015.53*	*1117.11*	*1018.82*	*1218.46*
5	F7	NIA-SW	-10.89	-8.88	-8.54	-6.30	-5.87	-3.86	-3.44
		LIA-SW	-28.55	-21.65	-20.35	-19.60	-17.97	-9.09	-7.56
		% diff.#	*162.09*	*143.69*	*138.22*	*211.19*	*206.09*	*135.33*	*119.94*
		NLIA-SW	-2.68	-4.73	-2.04	-9.22	-6.16	-8.02	-4.54
		% diff.#	*-75.41*	*-46.76*	*-76.17*	*46.48*	*4.92*	*107.47*	*32.08*
6	F8	NIA-SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		LIA-SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		% diff.#	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*
		NLIA-SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		% diff.#	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*

%difference with NIA-SW results; * with reversal of sign

Table 12 Comparison of bending moment (M_y) in the footings for NIA+SW, LIA+SW and NLIA+SW systems under various load cases

Footing Designation	Analysis type	Bending moment M_y for various load cases (KN-m)						
		LC1	LC2	LC3	LC4	LC5	LC6	LC7
F1	NIA+SW	-18.87	-15.07	-15.17	-16.62	-16.74	-9.96	-10.08
	LIA+SW	-124.04	-78.34	-105.40	-68.61	-101.94	-20.40	-56.82
	% diff#.	*557.48*	*419.76*	*594.79*	*312.83*	*508.92*	*104.77*	*463.48*
	NLIA+SW	-80.23	-64.34	-72.40	-58.48	-69.81	-19.54	-48.41
	% diff#.	*325.25*	*326.84*	*377.26*	*251.88*	*317.01*	*96.17*	*380.12*
F2	NIA+SW	-2.58	-2.08	-2.13	-0.14	-0.21	-0.09	-0.16
	LIA+SW	-13.32	-12.31	-4.86	-12.34	-2.92	-2.85	1.70
	% diff#.	*417.04*	*493.32*	*127.93*	*8805.57*	*1276.94*	*3061.77*	*962.5(*)*
	NLIA+SW	12.89	5.64	10.03	0.98	6.58	-2.44	3.07
	% diff#.	*399.61 (*)*	*171.15 (*)*	*370.89 (*)*	*600 (*)*	*3033.33 (*)*	*2600.35*	*1818.75 (*)*
F3	NIA+SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	LIA+SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	% diff#.	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*
	NLIA+SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	% diff#.	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*
SF1 (below column C6)	NIA+SW	-15.71	-14.36	-10.74	-13.10	-8.58	-8.76	-4.23
	LIA+SW	-212.56	-149.37	-172.51	-135.77	-165.06	-56.61	-94.69
	% diff#.	*1253.02*	*940.18*	*1505.94*	*936.33*	*1824.07*	*546.50*	*2136.20*
	NLIA+SW	-154.57	-129.09	-133.49	-122.96	-130.72	-56.26	-89.19
	% diff#.	*883.90*	*798.96*	*1142.69*	*838.55*	*1423.77*	*542.42*	*2006.47*
F7	NIA+SW	-11.35	-10.70	-7.44	-8.69	-4.60	-6.02	-1.94
	LIA+SW	-9.84	-3.16	-9.58	-0.49	-9.09	7.24	-6.79
	% diff#.	*-13.32*	*-70.48*	*28.86*	*-94.36*	*97.42*	*20.27(*)*	*249.75*
	NLIA+SW	16.32	20.66	3.42	17.22	-3.69	8.61	-7.56
	% diff#.	*43.79(*)*	*93.08(*)*	*-54.03(*)*	*98.16(*)*	*-19.93*	*43.02(*)*	*289.58*
F8	NIA+SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	LIA+SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	% diff#.	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*
	NLIA+SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	% diff#.	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*

%difference with NIA-SW results; * with reversal of sign

The significant change in moments M_y take place due to presence of shear wall. Under NIA the maximum increase of nearly 98% and the maximum decrease of nearly 93% is found in the footing F2 under different load cases. Under LIA the maximum increase of nearly 47% is found in the footing F2 and the maximum decrease of nearly 98% is found in the footing F7. Under NLIA

Table 13 Change in bending moments (M_y) in the footings under various load cases due to presence of shear wall in the space frame

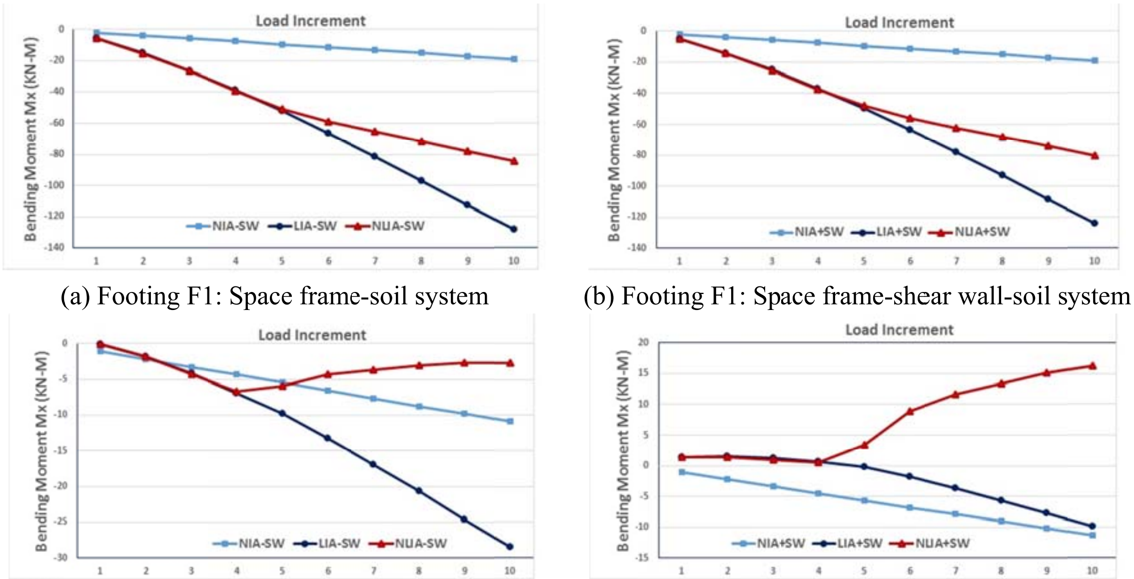
Footing Designation	Cases compared	% change in bending moment M_y for various load cases						
		LC1	LC2	LC3	LC4	LC5	LC6	LC7
F1	NIA+SW and NIA-SW	-0.24	7.11	-6.40	8.22	-7.12	14.70	-11.19
	LIA+SW and LIA-SW	-3.51	-5.08	-3.68	-5.69	-3.67	-15.13	-3.49
	NLIA+SW and NLIA-SW	-4.94	-8.59	-1.25	-9.52	-0.21	-14.48	2.33
F2	NIA+SW and NIA-SW	-2.70	97.91	-33.72	-88.03 (*)	-86.25	-92.74 (*)	-88.90
	LIA+SW and LIA-SW	-18.50	-0.62	-49.24	3.91	-63.38	-8.81	46.55 (*)
	NLIA+SW and NLIA-SW	55.78	1511.43 (*)	4.73	-78.92 (*)	-8.45	-12.66	-40.12
F3	NIA+SW and NIA-SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	LIA+SW and LIA-SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	NLIA+SW and NLIA-SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
F6 (SF1: below column C6)	NIA+SW and NIA-SW	10.25	24.11	-4.29	30.93	-10.38	43.82	-25.14
	LIA+SW and LIA-SW	7.41	1.13	10.60	-1.62	11.34	-19.07	19.27
	NLIA+SW and NLIA-SW	9.73	11.74	11.99	10.16	12.20	-17.42	19.61
F7	NIA+SW and NIA-SW	4.18	20.47	-12.96	37.92	-21.60	55.91	-43.53
	LIA+SW and LIA-SW	-65.55	-85.41	-52.92	-97.50	-49.44	-20.35 (*)	-10.20
	NLIA+SW and NLIA-SW	508.96 (*)	336.79 (*)	67.64 (*)	86.77 (*)	-40.17	7.36 (*)	66.57
F8	NIA+SW and NIA-SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	LIA+SW and LIA-SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	NLIA+SW and NLIA-SW	0.00	0.00	0.00	0.00	0.00	0.00	0.00

*with reversal of sign

the maximum increase of nearly 15 times is found in the footing F2 and the maximum decrease of nearly 79% is also found in the footing F2 under different load combinations.

The reversal in the sign of bending moment takes place in the footings F2 and F7 under some load cases.

Figs. 7(a)-(d) show variation of bending moment M_y in the footings with load increments for the vertical loads (load case LC1). The corner footings F1 and the inner footing F7 are selected for the comparison. Both, the space frame-soil system as and space frame-shear wall-soil system are considered. The results of NIA, LIA and NLIA are plotted for comparison. It can be seen that LIA as well as NLIA results in highly significant increase in the values of M_y in the footing F1 as compared to NIA. The extent of increase is more in LIA than NLIA. In the footing F7, LIA causes significant increase in space frame-soil system and significant decrease in space frame-shear wall-soil system, whereas, NLIA results in significant decrease in both the systems. Up to about 40% load increment, the results of LIA and NLIA are almost the same. Thereafter, with further increase in load increment, the difference in these quantities increases in nonlinear manner.



(a) Footing F1: Space frame-soil system (b) Footing F1: Space frame-shear wall-soil system

(c) Footing F7: Space frame-soil system (d) Footing F7: Space frame-shear wall-soil system

Fig. 7 Variation of bending moment M_y in the footings with the load increments under load case LC1

8. Conclusions

The important research findings are summarized below:

• The interaction effect causes significant redistribution of moments in the footings of space frame-soil and space frame-shear wall-soil systems.

• Significant difference is found in the values of footing moments between linear and nonlinear interaction analyses.

• The interaction effect significantly increases the value of bending moments M_x in all footings of space frame-soil system under most of the load cases for both the interaction analyses. However, it causes manifold increase in the value of bending moments in all footings for all load cases in the space frame-shear wall-soil system. The nonlinear interaction analysis causes further significant changes in the values in many cases as compared to linear interaction analysis.

• Both of the interaction analyses suggests that due to the presence of shear wall, bending moments M_x in the footings do not actually get reduced to that significant extent as found in case of non-interaction analysis. Therefore, the footings of a shear wall-space frame, designed on the basis of non-interaction analysis, may be more vulnerable.

• The interaction effect causes highly significant increase in values of bending moments M_y in most of the footings for both linear and nonlinear interaction analysis. The highly increased values may require revision of design for biaxial bending.

• Up to 40% of the load increment the results obtained for linear and non-linear interaction analysis are almost same. Thereafter, with further load increment the values of bending moments vary in nonlinear manner.

• More realistic and rational design of multi-storey space frame buildings, with and without shear walls, can be effectively achieved by carrying out the interaction analysis using the proposed methodology.

References

Agrawal, R. and Hora, M.S. (2010), "Effect of differential settlements on nonlinear interaction behaviour of plane frame-soil system", *ARPN, J. Eng. Appl. Sci.*, **5**(7), 75-87.

ANSYS Inc. (2009), *ANSYS Structural Analysis Guide*, ANSYS Inc., Southpointe, 275 Technology Drive, Canonsburg, PA 15317.

Arlekar, J.N., Jain, S. and Murthy, C.V.R. (1997), "Seismic response of R.C. frame buildings with soft first storeys", *Proceedings of the CBRI Golden Jubilee Year Conference on Natural Hazards in Urban Habitat,* New Delhi, India.

Bishop, A.W. and Henkel, D.J. (1962), *The Measurement of Soil Properties in the Tri-axial Test*, Edward Arnold (Publishers) Ltd., London, UK.

Code IS 875 (Part 2) (1987), "Indian Standard code of practice for design loads (other than earthquake) for buildings and structures, part 2, imposed loads", Bureau of Indian Standards, New Delhi, India.

Code IS 1893 (Part 1) (2002), "Criteria for Earthquake Resistant Design of Structures (Part 1: General Provisions And Buildings) (Fifth Revision)", Bureau of Indian Standards, New Delhi, India.

Edgers, Lewis, Sanayei, Masoud and Alonge, Joseph L. (2005), "Modeling the effects of soil-structure interaction on a tall building bearing on a mat foundation", *Civ. Eng. Practice: J. Boston Soc. Civ. En. Sec*., ASCE, Fall/Winter, 51-68.

Garg, Vivek and Hora, M.S. (2012), "Interaction effect of space frame-strap footing-soil system on forces in superstructure", *ARPN J. Eng. Appl. Sci*., **7**(11), 1402-1415.

Hora, M. (2006), "Nonlinear interaction analysis of infilled building frame-soil system", *J. Struct. Eng*., **33**(4), 309-318.

Jain, D.K. and Hora, M.S. (2015), "Analysis of settlements of space frame-shear wall-soil system under seismic forces", *Earthq. Struct*., **8**(5), 1255-1276.

Mylonakis, G. and Gazetas, G. (2000), "Seismic soil-structure interaction: Beneficial or detrimental?", *J. Earthq. Eng*., **4**(3), 277-301.

Natarajan, K. and Vidivelli, B. (2009), "Effect of column spacing on the behaviour of frame-raft and soil systems", *J. Appl. Sci*., **9**(20), 3629-3640.

Noorzaei, J., Viladkar, M.N. and Godbole, P.N. (1995), "Elasto-plastic analysis for soil-structure interaction in framed structures", *Comput. Struct*., **55**(5), 797-807.

Renzi, Stefano, Madiai, Claudia and Vannucchi, Giovanni (2013), "A simplified empirical method for assessing seismic soil-structure interaction effects on ordinary shear-type buildings", *Soil Dyn. Earthq. Eng*., **55**(2013), 100-107.

Shakib, H. and Atefatdoost, G.R. (2011), "Effect of soil-structure interaction on torsional response of asymmetrical wall type systems", *Procedia Eng*., Elsevier, **14**(2011), 1729-1736.

Stavridis, L.T. (2002), "Simplified analysis of layered soil-structure interaction", *J. Struct. Eng. Div*., ASCE, **128**(2), 224-230.

Tabatabaiefar, S., Hamid, Reza, Fatahi, Behzad and Samali, Bijan (2013), "Lateral seismic response of building frames considering dynamic soil-structure interaction effects", *Struct. Eng. Mech*., **45**(3), 311-321.

Thangaraj, D.D. and Ilamparuthi, K. (2010), "Parametric study on performance of raft foundation with interaction of frame", *EJGE*, **15**(H), 861-878.

Yahyai, Mahmoud, Mirtaheri, Masoud, Mahoutian, Mehrab and Daryan, Amir (2008), "Soil structure interaction between two adjacent buildings under earthquake load", *Am. J. Eng. Appl. Sci*., **1**(2), 121-125.

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