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AN ANALYTICAL SOLUTION TO KINEMATIC & INERTIAL INTERACTION OF BUILDINGS WITH DEEP BASEMENTS

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ABSTRACT

Superstructures interact kinematically and inertially with the underlying soil-foundation system under earthquakes. Combination of these two interactions constitutes the overall Dynamic Soil Structure Interaction (DSSI) response. While considerable research has been done to understand inertial interaction, little study has been carried out on kinematic interaction. Though Finite Element (FE) analysis, gives directly the combined effect (direct method), yet, it is not a regular practice because of the large computational efforts that is sometimes prohibitively expensive to execute. The alternative multi-step method suffers from a number of limitations, like selection of appropriate cut off boundaries, availability and correct input of geotechnical data and finally limited knowledge on how to use the output from this analysis as an input to super structure design. Thus in practice, engineers often carry out only inertial interaction and ignore kinematic interaction though its effect could be quite profound, especially when the foundation is substantially massive or deep to influence and modify the free-field ground response.

Present paper proposes an analytical technique whereby the effects of kinematic interaction can be considered for structures with deep basements without resorting to exhaustive FE analysis. The study is carried out for a multi storied service building in a nuclear power plant with multi-basements below ground housing sundry equipment. The study shows that combined effect of kinematic and inertial interaction gives much higher response compared to the results obtained from a conventional “fixed base” analysis.

INTRODUCTION

Dynamic soil structure interaction (DSSI) has recently drawn considerable interest from researchers around the world dealing with earthquakes. With increased knowledge on seismology, it is clear that in order to unravel the seismic behaviour of coupled soil-foundation-structure system, understanding strong motion acceleration is critical. The phenomenon of DSSI comprises of two behaviours (Kramer 1996) namely,

- Inertial Interaction (II)
- Kinematic Interaction (KI).

While inertial interaction explains the structural behaviour during dissipation of the excited energy into the elastic half space, kinematic interaction deals with the interaction of propagating seismic waves within the elastic half space with the foundation and superstructure. Though significant study has been carried out on the inertial effect of DSSI, little research is available as to how kinematic interaction influences the overall response. Since understanding DSSI response holistically requires an understanding of both inertial and kinematic aspects, especially when adopting the sub-structuring technique, the current state of knowledge in this regard can be considered as partial. On the other hand, the direct method, which offers total response using FE techniques, remains expensive and tedious for a comprehensive analysis in two and three dimensions.

The present paper develops a mathematical model to quantify kinematic DSSI behaviour of a building resting on a deep basement and elaborates how this phenomenon affects the overall response of the structure. The mathematical model proposed herein is closed-form, analytical in nature, sufficiently accurate and can be applied in regular design office projects.

PROPOSED METHOD

Shown in Fig. 1, is a site where seismic shear waves are propagating in a vertical direction. For a shallow small foundation the propagating wave pattern remain unaltered as the foundation stiffness and dimensions are far too small compared to the seismic wavelengths. The shallow foundation moves in sync with that of the waves and motion is free field in nature. In such cases, inertial interaction (II) is sufficient to predict the complete response accurately.

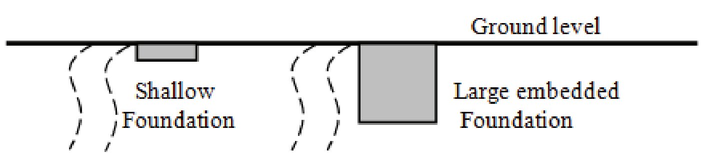

Figure 1. Vertically propagating shear wave in a soil medium

However, when the foundation is massive and embedded substantially, due to the significant geometrical presence and large stiffness, the free field response (i.e. when the foundation was not there there) gets modified due to interaction between the waves and foundation. This is called kinematic interaction (KI). In FE analysis (Fig. 2), this interaction is automatically considered when input at base/bed rock level is provided in the model.

A typical 2D FE model for such analysis is shown in Fig. 2. Notwithstanding the fact that such FE analysis is quite expensive, use of such models are also limited by a number of practical problems (Chowdhury & Dasgupta 2008). Some of these issues, that have persistently troubled the analyst can be summarised as follows:

- In FE analysis bed rock motion data is required. Unfortunately, in most of the cases site specific response spectra are furnished at ground level and not at base rock level. So the spectra need to be transferred to the base rock level and if the soil is too much layered, the analysis results obtained with the transferred data can be dubious.
- At some sites bedrock level is quite deep and can be at 150 m to 250 m below the surface. Reliable geotechnical data at such depths is usually not possible. Some codes recommend 30 m as the cut off level, but even getting accurate soil data to 30 m depth is expensive.
- Some sites may not have any bed rock at all. The boundary of the FEM model that would realistically reflect the free field behaviour in such situation is completely a matter of individual opinion. Considering the boundary at shallow depth result in reflection of waves generating spurious modes. While taking it deeper, makes the analysis expensive and most importantly, geotechnical data may not be available. Generally the boundary is considered where SPT value is more than 50.

- Modelling the soil to a depth to which geotechnical data is known and to ensure that waves are not reflected back, one may use infinite finite element or paraxial boundaries. However if the software in hand do not have such element or boundary options, the analysis is difficult to execute.
- Finally, analysing a structure as per a particular code like say IS 1893 or UBC etc. and also within the framework of modal analysis (most common application for earthquake analysis in industry), incorporating kinematic interaction is difficult as practically no guideline exist.

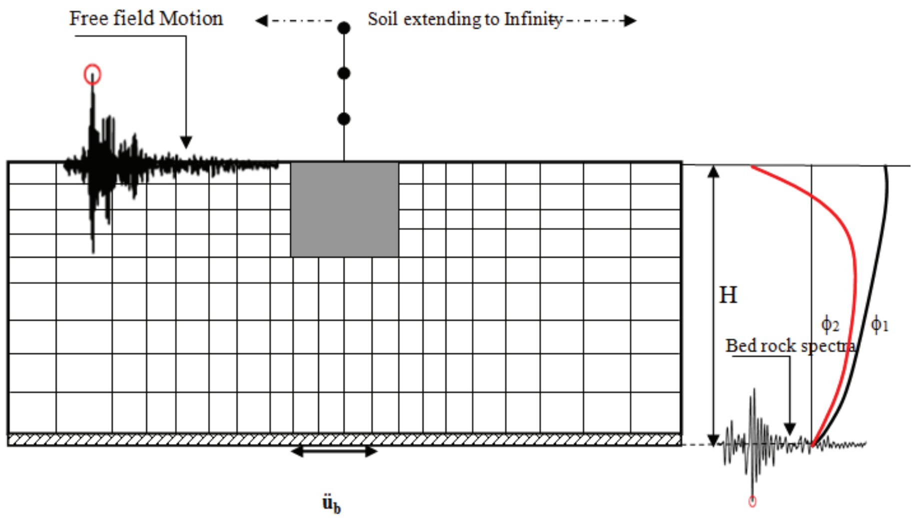

Figure 2. FEM model for superstructure and foundation along with surrounding soil

It is for these reasons engineers commonly perform II analysis but ignore KI effects despite the foundation having significant dimensions below ground. So the question that naturally arises is that, is it possible to device a rational procedure whereby KI effects can be considered within modal analysis frame work for regular engineering?
For this, let us re-evaluate Fig. 2, where a superstructure with multi-degree freedom is resting on foundation embedded to a depth D_f in soil, where D_f is quite large. The site extends infinitely in horizontal direction. H is the depth of soil layer above bedrock or where the SPT value is more than 50. The free field time period of the site considering dynamic shear modulus (G) constant with depth can be expressed as Dowrick (2003),

$$T_n^{si} = 4H/[(2n-1)V_s] \tag{1}$$

Here n= soil mode number 1, 2, 3..n..., V_s= Shear wave velocity of the soil and T^{si}_n= Free field modal time period of the site. It has been shown by Chowdhury and Dasgupta (2013) that free field modal amplitude at ground surface in such case can be expressed as,

$$u_n = \kappa_n \beta \frac{S_{an}}{\omega_n^2} \cos(2n-1)\frac{\pi z}{2H} \tag{2}$$

Here u_n= Modal amplitude of the soil at the site.
κ_n = Modal mass participation factor and is equal to 8/(π+2), -8(3π-2) and 8/(5π+2) for first three modes for shear modulus values constant with depth.
β= Code factor @ ZI/2R
S_{an}= Modal spectral acceleration of the site corresponding to evaluated time period, Eq. (1) above.
Z= Zone factor
I= Importance factor for soil, usually considered same as the structure it supports

R= Response reduction factor and can be considered as 2.0 (Chowdury &Dasgupta 2007)
ω_n= Natural circular frequency of the site corresponding to time period, Eq. (1) above

Based on Eq. (2) as per definition of modal analysis, Clough et al (1983), acceleration is expressed as

$$\ddot{u}_n = \kappa_n \beta S_{an} \cos(2n-1)\frac{\pi z}{2H} \tag{3}$$

At ground surface, @ z=0, acceleration is:

$$\ddot{u}_n = \kappa_n \beta S_{an} \tag{4}$$

At bottom of the basement, @ z=D_f, acceleration is:

$$\ddot{u}_n = \kappa_n \beta S_{an} \cos\frac{(2n-1)\pi D_f}{2H} \tag{5}$$

Thus average acceleration experienced by the basement of depth D_f (Figure-3) can be expressed as

$$\ddot{u}_{nd} = \frac{1}{2}\kappa_n \beta S_{an}\left(1+\cos\frac{(2n-1)\pi D_f}{2H}\right) \tag{6}$$

Thus if we isolate the foundation and draw the free body diagram of the system we can develop a mathematical model as shown in Fig. 3

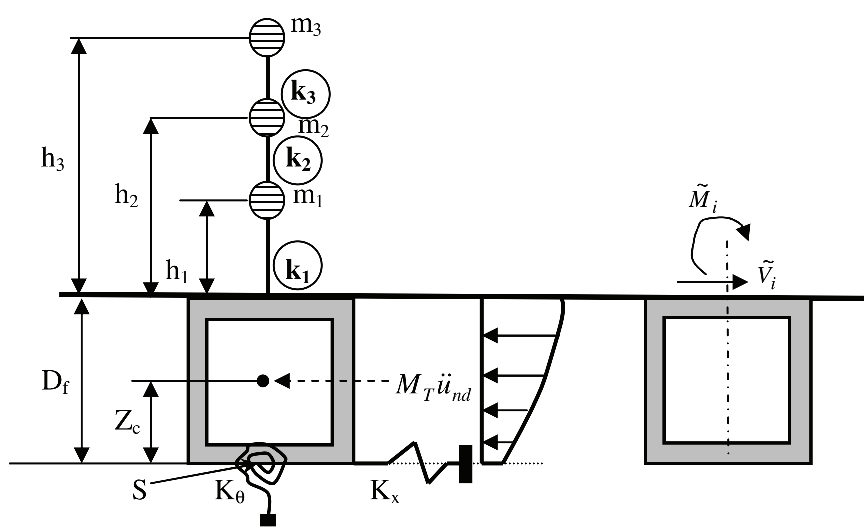

Figure 3. Free body diagram of the structure including basement

It is thus observed that the acceleration induced in the soil will cause the soil to move physically dragging the basement along with it, inducing a lateral thrust,

$$V_f = M_T \times \ddot{u}_{nd} \tag{7}$$

Where, M_T= Total mass of basement plus superstructure and expressed as $M_b + \sum_{i=1}^{3} m_i$. M_b= Mass of the basement including all the appurtenances, m_i= sum of the lumped mass of the structure.
Considering point S as the centre of stiffness the force will also induce an additional moment to the basement that can be expressed as,

$$M_f = M_T \times \ddot{u}_{nd} \times Z_c \tag{8}$$

Here Z_c= Distance of the common centre of gravity of the basement and superstructure measured from the centre of stiffness.
The equation of equilibrium can be expressed as;

$$\begin{bmatrix} M_b+\sum_{i=1}^{3}m_{si} & M_bZ_b+\sum_{i=1}^{3}m_{si}(h_i+D_f) & m_1 & m_2 & m_3 \\ M_bZ_b+\sum_{i=1}^{3}m_{si}(h_i+D_f) & J_\theta+M_bZ_b^2+\sum_{i=1}^{3}m_i(h_i+D_f)^2 & m_1(h_1+D_f) & m_2(h_2+D_f) & m_3(h_3+D_f) \\ m_1 & m_1(h_1+D_f) & m_1 & 0 & 0 \\ m_2 & m_2(h_2+D_f) & 0 & m_2 & 0 \\ m_3 & m_3(h_3+D_f) & 0 & 0 & m_3 \end{bmatrix}\begin{Bmatrix}\ddot{u}_x\\ \ddot{\theta}\\ \ddot{u}_1\\ \ddot{u}_2\\ \ddot{u}_3\end{Bmatrix}+$$

(9)

$$\begin{bmatrix} C_x & 0 & 0 & 0 & 0 \\ 0 & C_\theta & 0 & 0 & 0 \\ 0 & 0 & c_1+c_2 & -c_2 & 0 \\ 0 & 0 & -c_2 & c_2+c_3 & -c_3 \\ 0 & 0 & 0 & -c_3 & c_3 \end{bmatrix}\begin{Bmatrix}\dot{u}_x\\ \dot{\theta}\\ \dot{u}_1\\ \dot{u}_2\\ \dot{u}_3\end{Bmatrix}+\begin{bmatrix} K_x & 0 & 0 & 0 & 0 \\ 0 & K_\theta & 0 & 0 & 0 \\ 0 & 0 & k_1+k_2 & -k_2 & 0 \\ 0 & 0 & -k_2 & k_2+k_3 & -k_3 \\ 0 & 0 & 0 & -k_3 & k_3 \end{bmatrix}\begin{Bmatrix}u_x\\ \theta\\ u_1\\ u_2\\ u_3\end{Bmatrix}=\begin{Bmatrix}V_f\\ M_f\\ 0\\ 0\\ 0\end{Bmatrix}$$

In Eq. (9) the stiffness and damping values of soil-foundation system can be estimated from the expressions furnished by Wolf (1988), Gazetas (1987), Lysmer & Richart's(1966), Whitman(1970) after considering embedment effects. Damping for concrete is considered as 5%.
Z_b= Distance of the centre of gravity of the basement only measured from the centre of stiffness.

Eq. (9) in matrix notation can be expressed as Clough (1983),

$$[\mathbf{M}]\{\ddot{U}\}+[\mathbf{C}]\{\dot{U}\}+[\mathbf{K}]\{U\}=\{\mathbf{P}_t\} \tag{10}$$

Considering $\{U_i\}=\{\phi_i\}\{\xi_i\}$ (11)

Where $\{\xi\}$ is the generalized coordinate and multiplying Eq. (10) by $\{\boldsymbol{\varphi}\}^T$ we have

$$\{\boldsymbol{\varphi}\}^T[\mathbf{M}]\{\boldsymbol{\varphi}\}\{\ddot{\xi}_i\}+\{\boldsymbol{\varphi}\}^T[\mathbf{C}]\{\boldsymbol{\varphi}\}\{\dot{\xi}_i\}+\{\boldsymbol{\varphi}\}^T[\mathbf{K}]\{\boldsymbol{\varphi}\}\{\xi_i\}=\{\boldsymbol{\varphi}\}^T\{P\} \tag{12}$$

$$\rightarrow M_i\{\ddot{\xi}_i\}+C_i\{\dot{\xi}_i\}+K_i\{\xi\}=\{\boldsymbol{\varphi}\}^T\{P\} \tag{14}$$

Dividing each term of Eq. (14) by M_i we have

$$\{\ddot{\xi}_i\}+2\zeta_i\omega_i\{\dot{\xi}_i\}+\omega_i^2\{\xi\}=\frac{\{\boldsymbol{\varphi}\}^T\{P\}}{M_i}=q_i \quad \text{(nodal force, say,)} \tag{15}$$

Eq. (15) thus generates a set of five uncoupled equations,

$$\ddot{\xi}_x+2\zeta_x\omega_x\dot{\xi}_x+\omega_f^{\,2}\xi_x=q_x \tag{16}$$

$$\ddot{\xi}_\theta+2\zeta_\theta\omega_\theta\dot{\xi}_\theta+\omega_\theta^{\,2}\xi_\theta=q_\theta \tag{17}$$

$$\ddot{\xi}_1+2\zeta_1\omega_1\dot{\xi}_1+\omega_1^{\,2}\xi_1=q_1 \tag{18}$$

$$\ddot{\xi}_2+2\zeta_2\omega_2\dot{\xi}_2+\omega_2^{\,2}\xi_2=q_2 \tag{19}$$

$$\ddot{\xi}_3+2\zeta_3\omega_3\dot{\xi}_3+\omega_3^{\,2}\xi_{31}=q_3 \tag{20}$$

General solution to Eqs. (16 to 20) can be expressed as,

$$\xi_i = \frac{q_i}{\omega_i^2}\left[1 + e^{-\zeta_i \omega_i t}\left\{\sqrt{\frac{1-\zeta_i}{1+\zeta_i}}\sin\left(\omega_i\sqrt{1-\zeta_i^2}\,t\right) - \cos\left(\omega_i\sqrt{1-\zeta_i^2}\,t\right)\right\}\right] \tag{21}$$

Where, the boundary conditions are at 1) t=0 u_i=0 and 2) t=0 $\dot{u}_i = S_{vi} = S_{ai}/\omega_i$.

In Eq. (21) the subscript i= f, θ and 1,2,3 respectively as shown in Eq. (9). In Eqs.(16 to 20), the nodal force q_i is actually a function of the free field natural frequency or time period of the site as expressed in Eq. (1). Accounting for this, Eq. (21) can be expressed as,

$$\xi_i = \frac{q_i}{\omega_i^2\sqrt{\left(1-r_i^2\right)^2 + (2\zeta_i r_i)^2}}\left[1 + e^{-\zeta_i \omega_i t}\left\{\sqrt{\frac{1-\zeta_i}{1+\zeta_i}}\sin\left(\omega_i\sqrt{1-\zeta_i^2}\,t\right) - \cos\left(\omega_i\sqrt{1-\zeta_i^2}\,t\right)\right\}\right] \tag{22}$$

Here $r_i = \omega_1^{si}/\omega_i$ where the subscript 1 stands for fundamental natural mode of site. It is obvious that in this case if the natural frequency of the site is in proximity to the natural frequency of the system there will be significant excitation due to development of resonance like condition. On deriving Eq. (22) in uncoupled modes, they are finally converted to the structural coordinate by the relation as expressed by Eq. (11). The structural nodal force of the building for all modes due to kinematic interaction then can finally be expressed as,

$$\{f\}_{i=1,2,3} = \begin{bmatrix} k_1+k_2 & -k_2 & 0 \\ -k_2 & k_2+k_3 & -k_3 \\ 0 & -k_3 & k_3 \end{bmatrix}\begin{Bmatrix} u_1 \\ u_2 \\ u_3 \end{Bmatrix}_{i=1,2,3} \tag{23}$$

Above steps can be repeated for n number of soil modes to determine the force on the superstructure due to kinematic interaction. Based on computation it has been observed that it is the fundamental mode of soil that dominates the response. Higher modes have negligible effect and can be ignored. Above computation has been carried out based on soil having dynamic shear modulus G invariant with depth. But it is often found that dynamic shear modulus varies with depth in different form. Chowdhury and Dasgputa (2013) has computed the free field time period for different types of soil giving a general expression $T_n = C_{Tn}(H/V_s)$, where the values C_{Tn} are as furnished in Table-1 below.

Table-1: Values of coefficient C_T for various soil profiles

Mode ⟶	1	2	3
$G=G_0(z/H)$	5.19	2.133	1.228
$G=G_0(z/H)^{0.5}$	4.486	1.668	1.005
$G=G_0(z/H)^2$	7.826	3.301	1.66
$G=G_0(1+z/H)$	3.094	1.095	0.66
$G=G_0(1+z/H)^2$	2.421	0.906	0.545

In Table-2 the value $G_0(z/H)^n$ depicts the variation of dynamic shear modulus with depth where z=0 depicts the ground surface and H the bedrock level. The modal participation factors κ_n for various types of soil profile for first three modes are furnished in Table-2.

Table-2: Values of κ_n for first three modes

Mode ⟶	κ_1	κ_2	κ_3
$G=G_0(z/H)$	-1.534	1.27	0.529
$G=G_0(z/H)^{0.5}$	-1.586	-1.159	0.531
$G=G_0(z/H)^2$	1.480	0.864	0.465
$G=G_0(1+z/H)$	1.571	1.126	0.513
$G=G_0(1+z/H)^2$	1.581	1.143	0.536

Having computed the time period of the site based on values as furnished in Table-2, the procedure cited above can be used to compute the nodal forces in the super structure due to kinematic interaction for varying shear modulus values or for other modes of soil excitation or a combination thereof.
The KI base shear needs to be added to the II base shear to obtain the final force.
Though inertial interaction has not been the major focus of the present paper, however for completeness of the problem the same is discussed briefly hereafter.
If $\tilde{V}_i$ and $\tilde{M}_i$ are the fixed based shear and moment on the top of the basement for the fixed based structure in the i[th] mode (figure 3) the dynamic equilibrium with respect to the foundation is;

$$\begin{bmatrix} M_b + m_i & M_b.Z_b + m_i(h_i + D_f) \\ M_b.Z_b + m_i(h_i + D_f) & J_\theta + M_b Z_b^2 + m_i(h_i + D_f)^2 \end{bmatrix} \begin{Bmatrix} \ddot{u}_f \\ \ddot{\theta} \end{Bmatrix} + \begin{bmatrix} C_f & 0 \\ 0 & C_\theta \end{bmatrix} \begin{Bmatrix} \dot{u}_f \\ \dot{\theta} \end{Bmatrix} + \begin{bmatrix} K_f & 0 \\ 0 & K_\theta \end{bmatrix} \begin{Bmatrix} u_f \\ \theta \end{Bmatrix} = \begin{Bmatrix} \tilde{V}_i \\ \tilde{M}_i \end{Bmatrix} \quad (24)$$

In Eq.(24) m_i= Mass participating in i[th] mode and is expressed as L_i^2 / M_i where $L_i = \{\phi_i\}^T [\mathbf{M}] \{I\}$
And $M_i = \{\phi_i\}^T [\mathbf{M}] \{\phi_i\}$, $h_i = \left| \tilde{M}_i / \tilde{V}_i \right|$.
Solving Equation (24) we get the value of u_f, θ which are the displacement and rotation of the foundation respectively. The translational displacement of the structure u_{si} is in same form as cited in Equation (22) but for the fixed based frame.
It has been shown by Chowdhury et al (2013, 2015) that amplification factor AF_i due to II is,

$$AF_i = 1 + \left[\frac{u_{fi}}{u_{si}} + \frac{(h_i + D_f)\theta}{u_{si}} \right] \left[1 + e^{-\zeta_i \omega_i t} \left\{ \sqrt{\frac{1-\zeta_i}{1+\zeta_i}} \sin\left(\omega_i \sqrt{1-\zeta_i^2}\, t \right) - \cos\left(\omega_i \sqrt{1-\zeta_i^2}\, t \right) \right\} \right] \quad (25)$$

The fixed base response is multiplied by AF_i to get the final value of II as in Eq (26). Finally SRSS values of kinematic and inertial interactions are added to get the total response. Total base shear at base of building is given by Eq. (27) and can be distributed along the floor height by usual procedure.

$$V_{bi}^{II} = \tilde{V}_i \times AF_i \quad (26)$$

$$V_T = V_b^{KI} + V_b^{II} \quad (27)$$

RESULTS AND DISCUSSION

The results are studied against a 3 storied service building in a nuclear power plant with basements having two floors below ground housing sundry control equipment. The dimensions of the building are as shown in Fig. 4. Roof carries equipment loads like cooling plants and other functional assets of 500 kN. Roof and ground floor slabs are 200 mm thick, while for other floors it is 125 mm thick. External walls are 250 thick masonry and internal partition walls are 100 mm thick. The basement slab is 250 mm thick. Walls of basement are 450 mm thick. Base slab thickness is 600 mm. All beams along Y direction are 300×700 mm. All beams in X direction are 300×600mm. All columns are 400×600 mm up to ground floor. Shear wave velocity of soil is 310 m/sec. Poisson's ratio of soil is 0.4, Density of soil is 19kN/m^3. The bed rock level is at -35.0 meters below ground level (EL 0.0). Earthquake zone is Zone IV as per Indian code. The loads calculated at each floor are as furnished in Table 3. The centre of gravity of building and basement and that of basement alone were computed as EL +8.1 and EL 3.5 meter respectively.

Table 3: Computed weight at each floor of the building and basement

Elevation(M)	12.0	9.0	6.0	0.0	-4.50	-9.0
W(DL+LL+EL) kN	5150	4997	5141	5594	12893	18215

Table 4: Time period of the building system for fixed base, DSSI and site response

Methods Adopted	Time Periods				
	Mode-1	Mode-2	Mode-3	Mode-4	Mode-5
Fixed Base Dynamic Analysis	0.642	0.123	0.072	-	-
Proposed method with DSSI	0.654	0.124	0.096	0.072	0.048
Free field time period of site	0.452	-	-	-	-

Table 5: Base Shear at EL 0.0 due various effects of DSSI

Base Shear Fixed base dynamic analysis (kN)			Max. Base Shear Inertial Interaction (kN)			Max. Base Shear Kinematical Interaction (kN)			Total design Base Shear (kN)		
Mode-1	Mode-2	Mode-3	Mode-1	Mode-2	Mode-3	Mode-1	Mode-2	Mode-3	Mode-1	Mode-2	Mode-3
1292	3.74	0.125	1386	4.38	0.131	386	25	15	1772	29.38	15.13

Table 6: Amplification Factor AF due to DSSI for different mode**

Structural Mode 1		Structural Mode-2		Structural Mode-3	
Translation	Rocking	Translation	Rocking	Translation	Rocking
1.0734	1.0734	1.1712	1.1705	1.052	1.052

** For each structural mode there are two submodes in translation and rocking

Table 7: Shear and Moment at bottom of basement

	Kinematical interaction			Inertial interaction			Total force(KI+II)		
	Mode1	Mode2	Mode3	Mode1	Mode2	Mode3	Mode1	Mode2	Mode3
Shear - basement bottom (kN)	398	20	12	2027	11	0.227	2425	31	12.22
Moment - basement bottom (kN.m)	7489	883	524	25864	56	1.123	33353	884	525

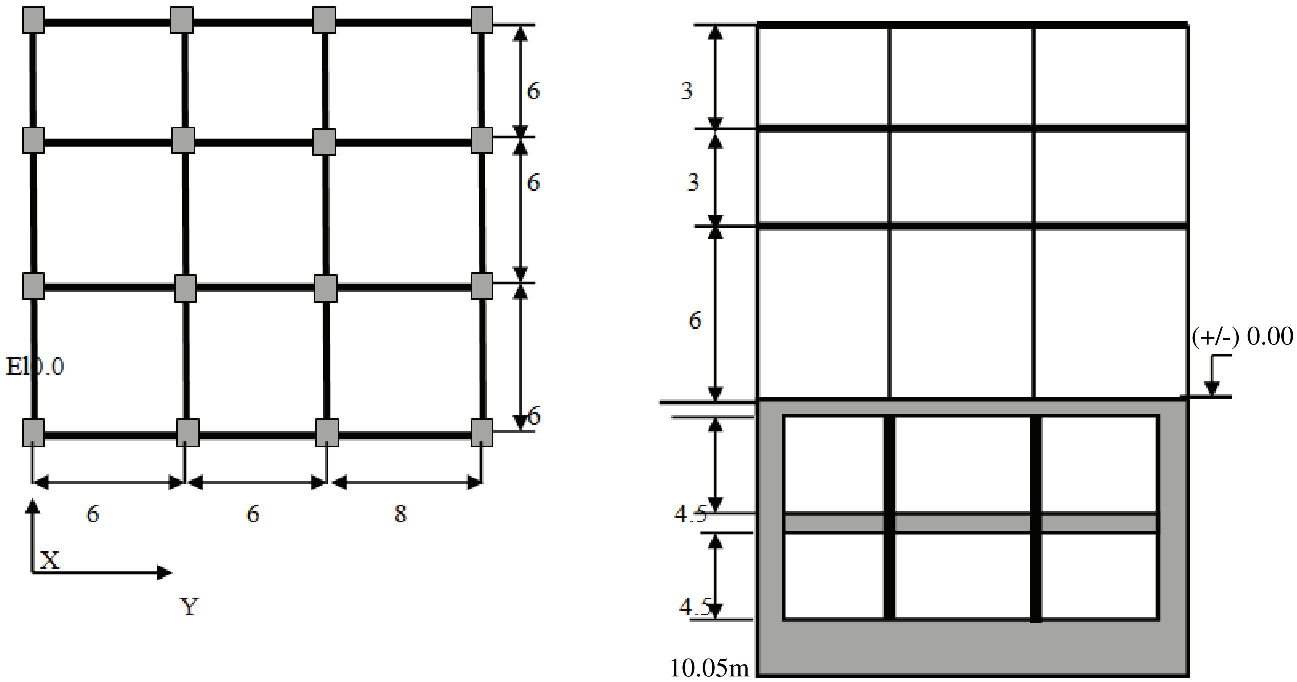

Fig. 4. Plan and elevation of the service building complex analysed for soil structure interaction

The time periods of the building, site and modal damping ratios are summarised in Table 4. Design shear at base of the building due to kinematic and inertial interaction vis-à-vis fixed base analysis are presented in Table 5. Peak Values of amplification due to DSSI are shown in Table-6.Variation of amplification factor for first two modes with respect to time are shown in Fig. 5.

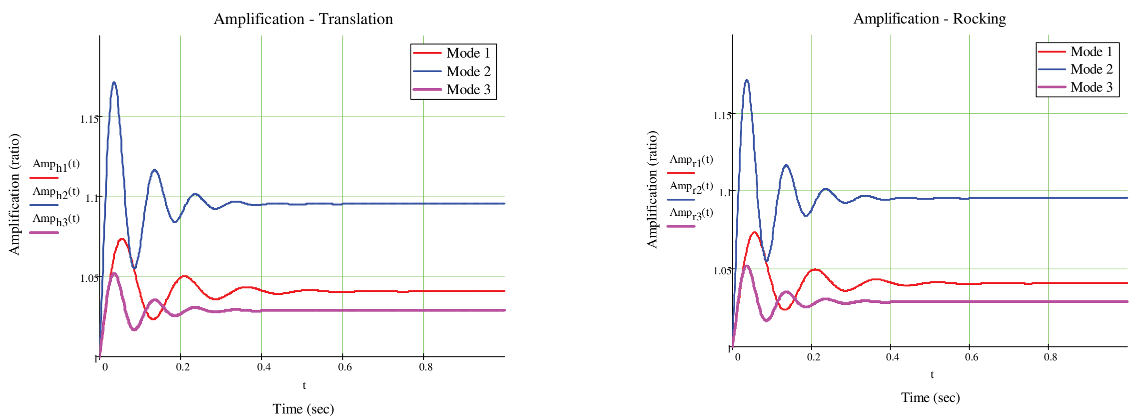

Fig. 5. Variation of Amplification factor(AF) with time due to DSSI first two modes

Results as depicted in Table 6 shows, that there is a significant amplification (about 37%) of fixed base shear when inertial kinematic interactions are considered. The maximum moment and shear at base of foundation due to combined kinematic and inertial interaction are as furnished in Table-7.
Above data shows that while for higher modes inertial contribution is insignificant yet the kinematic interaction has much higher contribution. The SRSS value of moment at the base of basement is 12662kN-m, while that considering DSSI the base moment is 33369 kN-m which is about 2.63 times more than the conventional fixed base response. This shows that while planning layout for buildings having deep basements in areas susceptible to earthquake, architects and engineers should give careful

consideration to both inertial and kinematic interactions. If the surrounding soil is soft and the free field time period approaches the time period of the structure foundation system, the amplification due to kinematic DSSI can be quite severe. The basement will behave as a rigid block and will excite the superstructure significantly under this near resonance condition.

CONCLUSION

An analytical method has been proposed herein that takes care of kinematic as well as inertial interaction effect of structures having deep embedded foundation. Considering the solution is analytical in nature, it is sufficiently accurate for practical applications. The major strength of the proposed method lies in the fact that it does not require any special purpose software for analysis and can be developed in MS excel or MathCad template. The proposed method clearly shows that for structures with deep basement the profound influence of both kinematic and inertial interaction on the overall behaviour cannot be ignored.

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