Non-Linear Soil-Structure Interaction Analysis Based on Coupling of Fe-Be Method in Time Domain

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Abstract

The present work introduces a direct time domain coupling (BE-FE) method for the most general case of three-dimensional linear and nonlinear soil-structure interaction analysis. The proposed (BEM) is based on an accurate and effective 3-D wave propagation in a layered vescoelastic half-space which is formulated and solved in time domain in terms of highly efficient factorizations for multiple source-receiver geometry in each layer. The present work focuses on the determination of the dynamic stiffness matrix of soil at arbitrary soil-structure interface for any configuration and location of the structure. It has introduced realistic damping for layered soil system by the determination of attenuation factor for each layer analytically.

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Introduction

The time domain BEM has recently captured increasing attention. Cole, (1978); & Niwa, (1980) (1,3), who were the first ones to obtain the dynamic response in the time domain directly. Karabalis and Beskos (1984) (4) presented an approximate three-dimensional time-domain elastodynamic formulation for half-space problems using constant temporal and spatial variation of the functions. Banerjee and Ahmad (1985) (11) presented an advanced algorithm using isoparametric quadratic elements and a better integration scheme. Later (1988), They improved the algorithm using higher order variations of functions in time. Banerjee and Ahmad (1990) developed the first inelastic transient dynamic algorithm for three-dimensional problems. Wolf, J. P.; Obernhuber, P. (1985) (5) formulated the contribution of the unbounded soil using the dynamic flexibility coefficients in the time domain, which are calculated as the inverse Fourier transform of the corresponding value in frequency domain. The dynamic flexibility coefficients in the time domain are calculated for a rigid circular disc resting on the surface of elastic half-space and of a layer built-in at its base. Wolf, J. P.; Motosaka, M. (1989) (14-16) evaluated recursively the interaction forces representing the contribution of the linear unbounded soil to the equations of motion of a non-linear soil-structure interaction analysis in time domain. This evaluation is starting directly from the dynamic stiffness coefficients in the frequency domain, affecting only the right-hand side of the equations of motion. Y. Hayashi & H. Katukura, (1990), ‘ Y. Hayashi & I. Takahashi, (1992) (18) presented an approximate method and a rigorous method for the time-domain soil-structure interaction analysis, which use the stiffness of soil obtained numerically in the limited frequency range. Y.K. Cheung, J. X. Zhu (1992) (19) present the transient response of a circular shell embedded homogeneous, isotropic and vescoelastic half-space subjected to explosion wave. The method employed is a combination of the time domain (BEM) used for the soil and the finite strip method used for circular shell. Y.K. Cheung, Lie, and Tham (1992) (21) have developed a time domain (BEM) in a cylindrical coordinate for the wave propagation in a half-space. The integral formulation is based on dynamic reciprocal theorem and stokes fundamental solutions. And they (1993)(23) have developed a time domain (BEM) in a cylindrical coordinate for the wave propagation in a layered half-space. T. Touhie & T. Ohmachi , (1993) (22) have developed a numerical method for dynamic response analysis of dam-foundation-reservoir systems in the time domain. During formulation, the weighted residual procedure was applied to the coupling of equation of motion for solid and fluid using (FEM) and (BEM), and an algorithm similar to the Newmark beta procedure was obtained. H. Takemiya, G. Fei & Y. Sukeyasu, (1994) (25) have shown an effective implementation of the half-space Green function for surface strip impulses into time domain (BEM) for two- dimensional problem. Wolf, J. P.; Song, C. (1994) (26) have developed the infinitesimal finite element cell method based solely on the finite element formulation and working exclusively in the time domain. Z. Chuhan, J. Feng & O. A. Pekau, (1995) (29) have presented the time domain procedure of three-dimensional coupling of finite elements, boundary elements, and infinite boundary elements for seismic interaction of arch dams and canyons. A. A. Stamos & D. E. Beskos, (1995) (27) have determined the dynamic response of large three-dimensional underground structures to external dynamic forces or to seismic waves using the (BEM). Numerical examples involving lined cavities buried in the full or the half-space subjected to transient external forces are presented to illustrate the method and demonstrate its advantage. The transient case is treated with the aid of numerical Laplace transform with respect to time. J. Qian, L. G. Tham, Y. K. Cheung & D. L. Karablis, (1996) (31) have developed the dynamic cross-interaction between flexible surface footings by combined (BEM) and (FEM). R. Betti (1997)(32) has presented the effects of the dynamic cross-interaction in the seismic analysis of multiple embedded foundations. D. Bernal & A.Youssef, (1998) (33) have developed a hybrid time frequency domain formulation for non-linear soil-structure interaction. The equations of motion are solved in the time domain with due consideration for non-linearities and with the unbounded medium represented by frequency dependency of the impedance coefficients evaluated in frequency domain at the end of each iteration. D.C. Rizos & D.L.Karabalis, (1998) (34) have developed a time domain (BEM) for 3-D elastodynamic analysis using the B-Spline fundamental solutions compute the dynamic response of the soil domain through a superposition of the characteristic B-Spline impulse responses. D.C. Rizos, (1999) (35) has applied a rigid surface foundation for the B-Spline direct time domain (BEM). D.C. Rizos & M.L.Asce & Z.Y. Wang, (2000) (35) have introduced a direct time domain (BEM-FEM) formulation for 3-D soil-structure interaction analysis. The proposed (BEM) that is based on the B-Spline of fundamental solutions and standard direct integration (FEM) procedures are used to compute the dynamic response of the structure. A staggered solution process is proposed for the coupling of the two methods.

Employment Of Time Domain's Fundamental Solution In Layered Media

To analyze the effects of soil-structure interaction on the dynamic response of a structure, it is convenient to partition the soil-structure system at the interface between the soil and the structure. Thereby, the characterization of the interaction between the soil and structure can be combined with a separate analysis to determine the dynamic response of the structure when excited by both external forces and incoming seismic waves. Once the dynamic stiffness matrix of soil-structure system and input motion vector are determined, then the complete soil-structure interaction problem for any configuration of the structure can be formulated and solved, since there are accepted techniques as (FEM) for modeling the structure. The present work focuses on the determination of the dynamic stiffness matrix in time domain for various non-linear analyses of soil-structure systems. It is the objective of this study to remove both limitations by representing the soil as a layered viscoelastic half-space medium and by considering the effect of structure embedment into the soil. In the present work, the problem of determining the non-linear dynamic response of arbitrary shaped three-dimensional structure will be reduced to determine the dynamic stiffness matrix of soil in time domain. The fundamental solution involves the Green's functions for a layered viscoelastic half-space so that the differential equations of motion automatically satisfy the continuity conditions at the layer interfaces, the traction-free conditions, and geometric radiation conditions in the underlying half-space. Such Green's functions, are expressed in time domain, are now available as derived in Paper-I of the author. The boundary conditions at the soil-structure interface will be imposed by numerically solving the integral equation with relaxed as well as welded type contact assumed at soil-structure interface.

- **Formulation Of The Time Domain Boundary Element:**

Using Graffi's dynamic reciprocal theorem, one can show that, if the body force is ignored, Love's integral identity for small displacement vibration of an elastic body, which is initially at rest, is:

$$\varepsilon_{ij}(\vec{x})\left\{D_i(\vec{x},t)\right\} = \int_\tau\int_s \Big[\left[G_{ji}(\vec{y},t;\vec{x},\tau)\right]\left\{T_j(\vec{y},t)\right\} - \left[H_{ji}(\vec{y},t;\vec{x},\tau)\right]\left\{D_j(\vec{y},t)\right\}\Big] ds\ d\tau \tag{1}$$

Where :

$$(i,j=x,y,z), \qquad \varepsilon_{ij}(\vec{x}) = \begin{cases} 0, & \vec{x}\in V' \\ \frac{1}{2}, & \vec{x}\in S \\ \delta_{ij}, & \vec{x}\in V \end{cases}$$

In which; *Hij*($x,t;y,\tau$) denotes Green's functions corresponding *i*-component of the traction vector at point $y \in S$ at proceeding time interval τ due to unit impulse point load in the *j*-component at x point in the volume V' at time interval t, *Gij*($x,t;y,\tau$) denotes Green's functions corresponding *i*-component of the displacement vector at point $y \in S$ at proceeding time interval τ due to unit impulse point load in the *j*-component at x point in the volume V' at time interval t, *Tj* (y,t), *Dj* (y,t) represent respectively the *j*-component of the traction vector and the displacement vector at point $y \in S$ at proceeding time interval τ due to unit impulse point load at x point in the volume V' at time interval t. δij is delta dirak function.
Applying the equation (1) for the source located at point x in the volume V' as shown in figure (1) result in:

$$\int_\tau\int_s \left[G_{ji}(\vec{y},t;\vec{x},\tau)\right]\left\{T_j(\vec{y},t)\right\} ds\ d\tau = \int_\tau\int_s \left[H_{ji}(\vec{y},t;\vec{x},\tau)\right]\left\{D_j(\vec{y},t)\right\} ds\ d\tau \tag{2}$$

The integral equations must be solved numerically by discretizing the formulation. The discrete set of impulse forces *Q*(*x'*) located at discrete points on surface *S*' which offset by small distance from surface *S* within the volume *V'* has to be imposed on the discretization of integral equations. The traction *Tj* (*y,t*), or the displacements *Dj* (*y,t*) can be expressed by means of *Q j*(*x'*) forces.

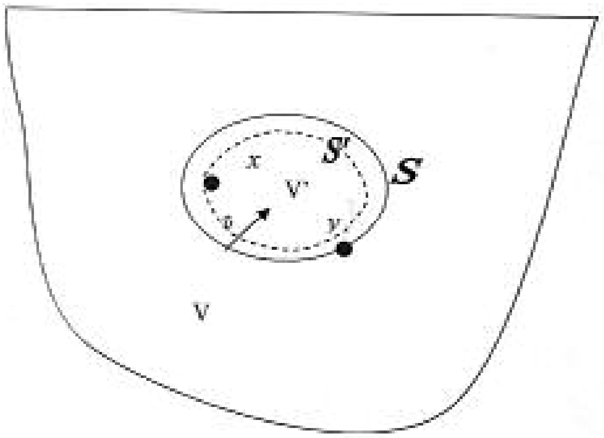


Figure (1): Model geometry represents various radiation and scattering problems in elastodynamic with surface *S* defining surface interface between *V,* and *V'* volumes, and source surface *S'*.

Three different types of boundary conditions are plausible at soil-structure interface; 1) displacements prescribed everywhere on *S* (e.g. , for welded contact between soil and structure); 2) traction prescribed everywhere on *S* (e.g. , for diffraction by a canyon); or 3) mixed boundary conditions where the displacements are prescribed on a portion *S2* of *S* while the tractions are prescribed on the remaining portion *S1* of *S* so that the soil is constrained to move with the structure only on *S2* (e.g. when separation exists along *S1*).
Once the integral equation (2) is solved for distributed forces *Q j*(*x'*) according to the types of boundary conditions at soil-structure interface, the dynamic stiffness matrix in time domain of the soil at arbitrary

surface interface *S* between soil and structure can be obtained, which contains the forces required to produce unit-impulse displacements.
In the analysis, the integration over surfaces *S,* and *S'* are replaced with formulae of quadrature as the first step for the discretization of integral equations. The temporal integration can be implemented according to a numerical integration scheme along the period span *T* of dynamic excitation, which is, discretized in to *N* interval *Δ t.*

- **The Basic Equation Of Motion In The Time Domain:**

The basic equation of motion in time domain to analyze the interaction of non-linear structure and an irregular soil with the linear layered vescoelastic half-space soil domain can be formulated as following:

$$\begin{bmatrix} [M_{ss}] & [M_{sb}] \\ [M_{bs}] & [M_{bb}]-[M^{e}_{bb}] \end{bmatrix} \begin{Bmatrix} \{\ddot{r}^{t}_{s}\} \\ \{\ddot{r}^{t}_{b}\} \end{Bmatrix} + \begin{bmatrix} [C_{ss}] & [C_{sb}] \\ [C_{bs}] & [C_{bb}]-[C^{e}_{bb}] \end{bmatrix} \begin{Bmatrix} \{\dot{r}^{t}_{s}\} \\ \{\dot{r}^{t}_{b}\} \end{Bmatrix} +$$

$$\begin{bmatrix} [K_{ss}] & [K_{sb}] \\ [K_{bs}] & [K_{bb}]-[K^{e}_{bb}] \end{bmatrix} \begin{Bmatrix} \{r^{t}_{s}\} \\ \{r^{t}_{b}\} \end{Bmatrix} + \begin{Bmatrix} \{0\} \\ \left\{\int_0^t [S^{f}_{bb}(t-\tau)]\ \{r^{t}_{b}(\tau)\}\, d\tau\right\} \end{Bmatrix} =$$

$$\begin{Bmatrix} \{0\} \\ \left\{\int_0^t [S^{f}_{bb}(t-\tau)]\ \{r^{f}_{b}(\tau)\}\, d\tau\right\} \end{Bmatrix} \qquad (3)$$

Where:

$$\{r\}, \{[M]\ ,[C]\ ,[K]\}, \{S^{f}\}$$

Denotes respectively; the vector of the total displacements, the mass matrix, the damping matrix, the stiffness matrix, and the dynamic stiffness matrix of unbounded soil domain at surface interface between soil and structure. The superscript *e*, and *f* denote respectively to excavated soil within surface *S,* and free field of layered half-space, and

subscript *s,* and *b* denote respectively to structure nodes and boundary element nodes.
The interaction forces of the unbounded soil domain acting on the node *b* are equal to the convolution integral of the dynamic stiffness matrix of unbounded soil domain and the motion relative to the ground as:

$$\left\{R_b^f(t)\right\} = \int_0^t \left[S_{bb}^f(t-\tau)\right] \left(\left\{r_b^t(\tau)\right\} - \left\{r_b^f(\tau)\right\}\right) d\tau \qquad (4)$$

$$\left\{R_b^f(t)\right\} = \int_0^t \left[S_{bb}^f(t-\tau)\right] \left(\left\{r_b^t(\tau)\right\} - \left\{r_b^f(\tau)\right\}\right) d\tau \qquad (4)$$

If the structure and the irregular soil region exhibit a non-linear behavior, non-linear internal forces replace the second and third terms on the left-hand side of equation (3).

- **Determination Of Internal Damping Of Layred Vescoelastic Half Space System:**

Several previous studies have shown the need for incorporating material damping in the solution, particularly when large strain are involved or when the medium representing the soil is layered. The effects of material or internal damping are automatically incorporated into the fundamental solution of the layered vescoelastic half-space system.
It is well known that dissipation of energy accompanies transmission of stress waves in the soil. The dissipation properties of solid are commonly measured by analyzing the decay rate of the amount of energy dissipated per cycle of free vibration response. From this principle, the free vibration response of the layered half-space system has been determined in the present work. Accordingly, The amount of energy transmission *E* through thickness of each layer has been calculated during a cycle of free vibration response of the layered vescoelastic half-space system. The amount of energy *ΔE* dissipated per cycle of free vibration of the system has also been calculated. The following relation can determine the attenuation factor of each layer in layered half-space system:

$$\frac{2\pi}{\xi} = \frac{\Delta E}{\Sigma E} \qquad (5)$$

The effect of introducing the attenuation factor of soil into the multilayred half-space not only results in a more realistic model, but also allows the numerical integration procedure to be performed without recourse to principal values and shifts the singularities of the fundamental solution of layered half-space system.

Validation Of The Present Work

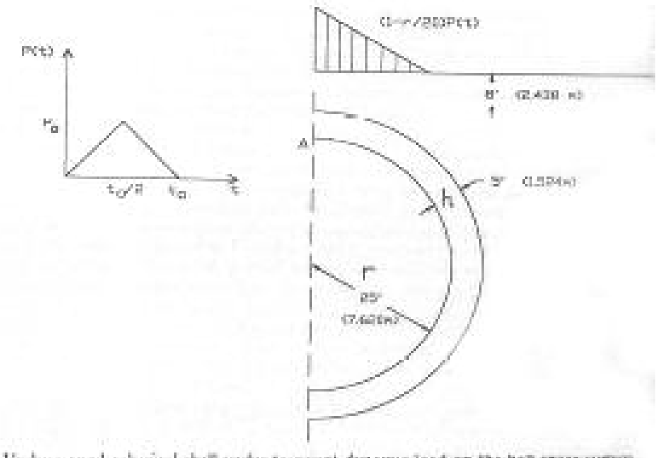
Comparative studies of present work are performed to demonstrate the employment of the fundamental solution of wave propagation in layered half-space media in the time domain, which it is employed to implement the dynamic stiffness matrix in time domain at arbitrary soil-structre interface, as it is considered the second step of soil-structure interaction problem. These studies include comparisons with actual structures presented in various published papaers.

- **Linear Analysis Of Soil-Structure Interaction:**

The curve with discrete points refers to reference study, and continuos curve refers to present work.

- **Comparison With Underground Structure:**

(A. A. Stamos & D. E. Beskos, 1995) (27) have studied a spherical cavity lined with a concrete shell and embedded in half-space as shown in the accompanied figure. The material constants are Eg = 215.547E06 (N/m2), ρg = 1665.452 (kg/m3), vg = 0.33 and the shell material constants are Ec = 215.547E08 (N/m2), $\rho\ c$ = 2402.788 (kg/m3), vc = 0.15. The spherical cavity is subjected to vertical explosion.
They also have studied a spherical cavity in the infinite space lined by a shell of thickness $h = 0.01\alpha$ as shown the related figure. The material constants are Ec= 2.5 Eg, $\rho\ c$ = 1.56 $\rho\ g$, vg = 0.25, vc = 0.20. The spherical cavity is subjected to horizontal plane compressional wave explosion propagates in the soil along the x direction with speed $C1$.



Underground spherical shell under transient dynamic load on the half-space surface

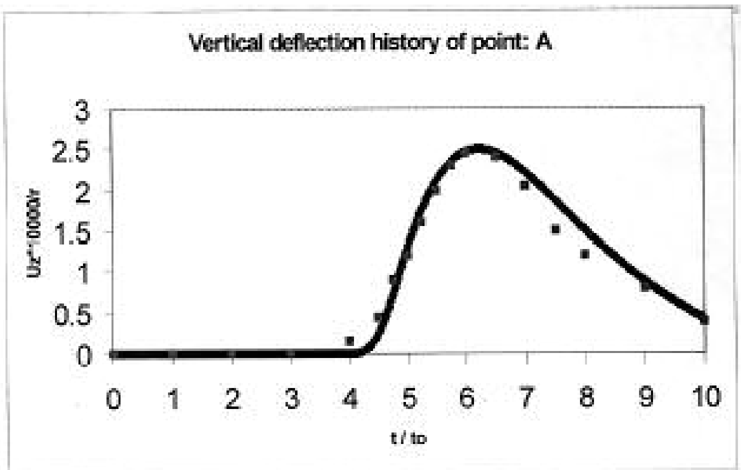
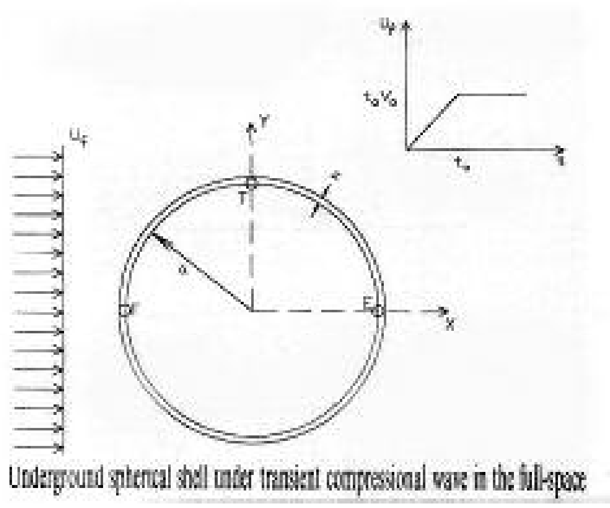


Figure (2): comparison between present work and the reference study for vertical deflection history of point A of the shell.



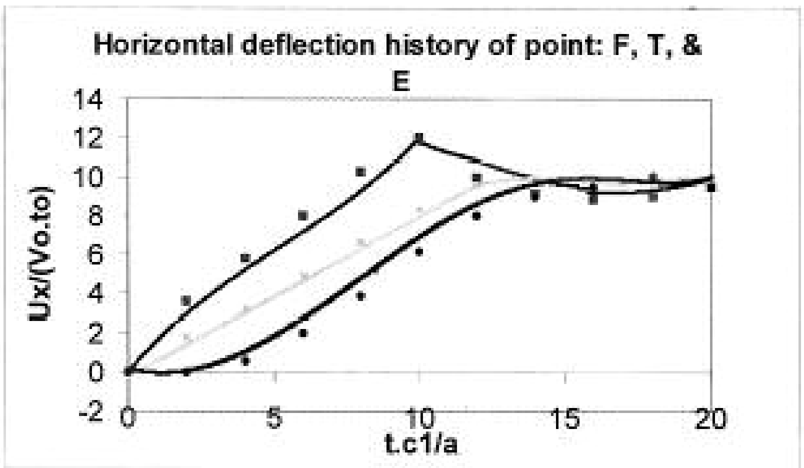
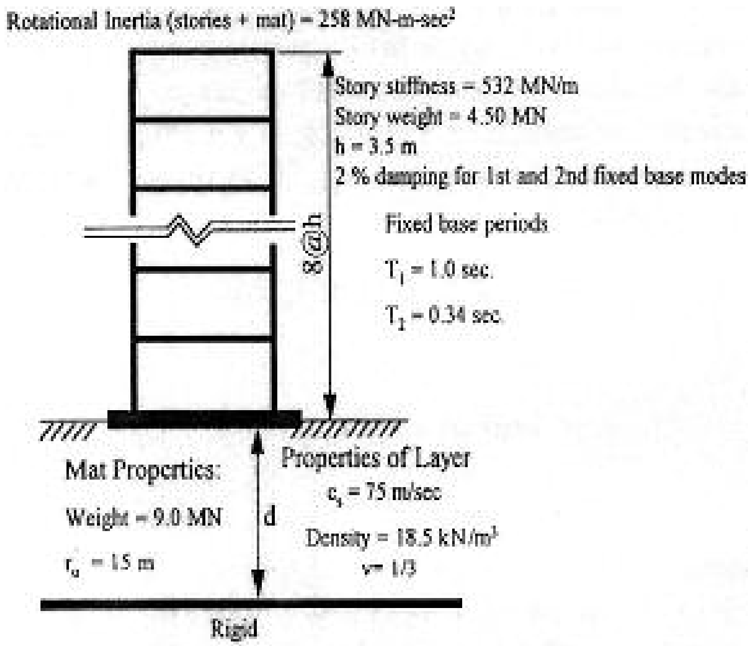


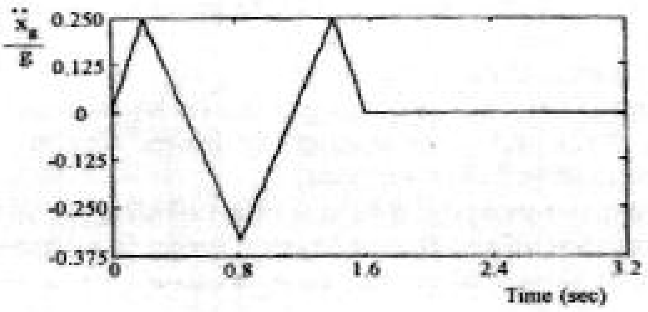
Figure (3): comparison between present work and the reference study for horizontal deflection history of point F, T, & E of the shell.

- **Comparison With Hybrid Time Frequency Domain Method:**

(D. Bernal & A.Youssef, 1998) **(33)** have studied soil-tower building interaction subjected to pulse acceleration at the building base using the hybrid time frequency domain method as shown in the following figures:



Shear building and foundation data



Ground acceleration pulse

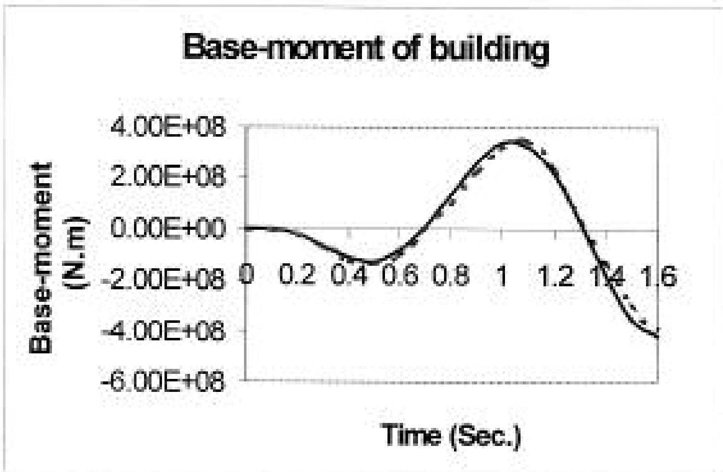


Figure (4): comparison between present work and the hybrid time frequency domain method for the base-moment of the building.

• **Comparison With Staggered (Bem-Fem) Solutioms To Soil-Structure Interaction :**

(D.C. Rizos & M.L.Asce & Z.Y. Wang, 2000) **(36)** have introduced a direct time domain (BEM-FEM) for 3-D soil-structure interaction analysis, which based on the B-Spline family of fundamental solution of half-space. The comparison between the present work and this method is obtained for the problem of the dynamic through-the soil interaction of massive foundations.

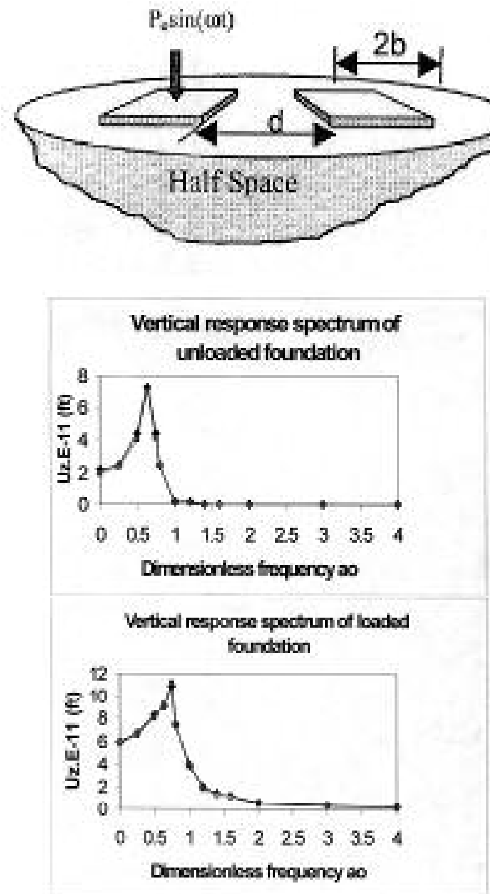


Figure (5): comparison between present work and the staggered (FE-BE) method for the through-soil interaction of massive rigid foundations.

- **Comparison With Dynamic Flexibility Matrix Of Half-Space To Soil-Structure Interaction :**

(J.P.Wolf, 1985) **(6)** has presented the behavior of investigated soil-structure system with a rigid basemat resting on the surface of a half-space as shown in the accompanied figure. The mass moment of inertia at the basemat equals (¼ *a2.m*). The following parameters are applied: *a/Cs* =0.06 sec., *h/a* = 1.5, *m/(ρ a3)* =3. (*ρ* = mass density of the soil), *ν* = 0.33. Horizontal artificial time history normalized to 0.21 *g* is applied at the basemat.

	Convolution Integral	Spring and damper	Present work	Case of the basemat
Horizontal at top	4.1	3.5	4.32	The basemat
Horizontal at top of basemat	2.3	2.3	2.33	on a half-space
Horizontal at top	8.44	5.01	6.32	The basemat
Horizontal at top of basemat	2.35	2.42	2.53	on a layer

Table (1): maximum total acceleration

The results of analysis for present work are shown in the table (1) compared with the reference study.

- **Non-Linear Analysis Of Soil-Structure Interaction:**
  - **Non-Linear Behavior Of Investigated Soil-Structure System Due To Partially Separation Of The Basemat From The Soil:**

The previous investigated structure is subjected to Horizontal artificial time history normalized to 0.4 *g* is applied at the basemat.

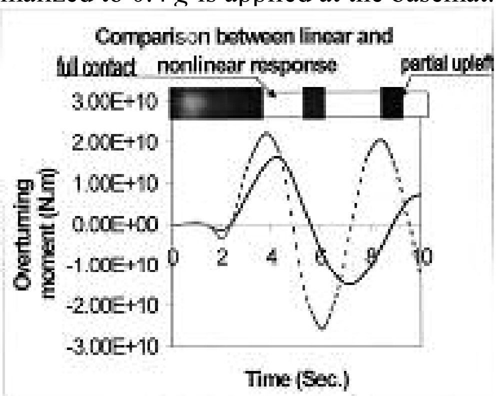


Figure (6): Time history of overturning moment for linear and non-linear behavior of investigated structure. Dash curve refers to linear analysis and continuos curve refers to non-linear analysis.

- **Non-Linear Behavior Of Investigated Soil-Structure System Due To Base Isolation:**

The previous investigated structure is subjected to Horizontal artificial time history normalized to 0.21 *g* is applied at the basemat.
The results of analysis for present work are shown in the table (2) compared with the reference study.

	Convolution integral	Spring and damper	Present work
Horizontal at top	1.85	1.84	1.89
Horizontal at upper basemat	2.46	2.19	2.28
Horizontal at lower basemat	2.66	2.79	2.73

Table (2): maximum total acceleration

- **Non-Linear Behavior Of The Spherical Underground Structure Due To Partially Separation Of The Shell From Surrounding Soil:**

The spherical cavity, which lined with the concrete shell and subjected to vertical explosion as shown in the first example in validation section, is analyzed taking into account the non-linear behavior of the spherical shell due to separation of the shell from the surrounding soil. The non-linear analysis is compared with the linear behavior, which is considered a full contact with the soil for a vertical displacement of the point A at the top of the spherical shell as shown in figure (7). The intensity of the explosion load was ten times of the explosion applied in the first example.

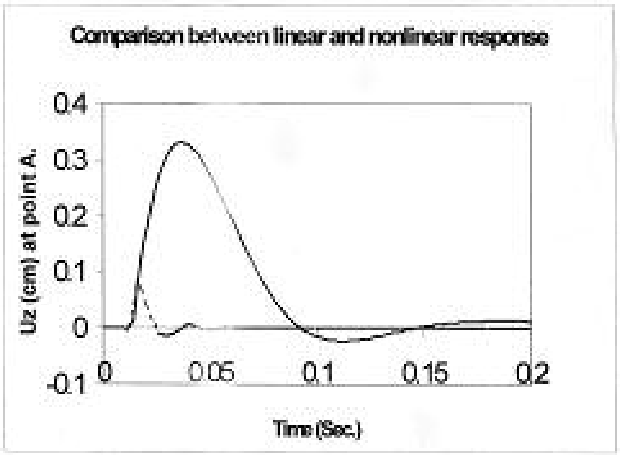


Figure (7): Time history of vertical displacement of point A for linear and non-linear behavior of spherical structure. Dash curve refers to linear analysis and continuos curve refers to non-linear analysis.

Conclusion

A methodology is developed for the efficient coupling of the Finite Element with Boundary Element Method for three-dimensional wave propagation in layered vescoelastic half-space system and soil-structure interaction analysis in the direct time domain. The method uses the developed time domain formulation for 3-D wave propagation in layered soil by applying the boundary-value theory with standard FEM processes. So, the complete soil-structure interaction problem for any configuration of the structure is solved. The present work is removing both limitations by representing the soil as a layered viscoelastic half-space medium and by considering the effect of a structure embedment into the soil in direct time domain to take into account all types of non-linearity behavior of the structure and irregular soil near the structure. The comparison studies found in the validation section include comparisons to particular methods. The results of these studies demonstrate the validation and accuracy of the proposed method. Numerical examples involving investigated soil-structure system and lined cavities buried in the half-space subjected to transient external forces are presented to illustrate the method and demonstrate its advantage for determining the non-linear behavior of the structure due to separation of the structure from the soil or base isolation.

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