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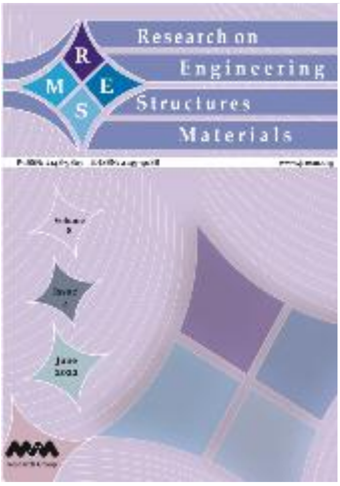

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Review Article

Approaches considering non-linearity in soil-foundation-interaction: A State of the Art Review

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Abstract

Seismic excitation causes the soil to begin acting nonlinearly at higher strain. Hence, the nonlinearity of the soil, foundation, and structure should be appropriately considered. This can be achieved by proper modelling of soil-structure-foundation interaction (SSI). The continuum, Winkler-based, and Macroelement models are the major modelling techniques for considering SSI. The continuum method involves determining absorbing boundaries, the size of the soil domain, soil element size, constitutive soil model, and soil structure interface. In contrast, the Winkler-based model uses nonlinear spring and dashpot to represent inelastic behaviour and energy dissipation properties of soil, respectively. Macroelement replaces the entire soil foundation arrangement with one element at the bottom of the superstructure. The trade-off between the advantageous effects of the SSI model, particularly in terms of energy dissipation, and its unfavourable effects, such as settling or tilting, should also be optimised during the analysis and design phases. The present paper aims to provide a concise review and comparative analysis of the several methodologies proposed by the researchers that consider the nonlinearity in soil-foundation-structure interaction (SSI). The importance of the study lies in the adoption of an approach that reduces computational effort and time. Moreover, the experimental works are also reviewed with regard to the soil structure interaction. It can be inferred from the current study that various approaches have some benefits and drawbacks; thus, these approaches can opt accordingly.

1. Introduction

Soil structure interaction (SSI) is considered a multidisciplinary field that combines soil and structural dynamics, earthquake engineering, geomechanics and geophysics, material science, and various other technical fields. Following the successful result of SSI, many theories, methodologies and experimental settings are employed to continue the study of SSI. Several analytical methods, numerical methods, analytical-numerical techniques, experiments, and prototype observation paved the roadways for SSI analysis after technology advancement.

Nonlinearity in the superstructure, foundation, and soil can be geometric nonlinearity, material nonlinearity, or both. Consideration of the nonlinearity of soil-structure-foundation is crucial for better accuracy of results simulating the actual behaviour of the entire system. Nonlinearity comes into the scenario due to various reasons, including (a) deformation in the seismic force-resisting element of the superstructure, (b) foundation structural element yielding, (c) gapping between foundation base and soil (e.g., base

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uplift), (d) soil yielding, which can get amplified by pore-pressure-induced strength loss. Modelling nonlinearity has become easy with the advancement in computation techniques suggested by researchers.

Typically, structural components are modelled as skeletal, but soil can be either modelled as a skeletal spring or a continuum. Early research into SSI analysis, often known as the "Winkler Model," began in the 1860s. Following this, different soil idealisation types were examined using two-parameter and three-parameter models that solely consider the elasticity of the soil [1]. The majority of Winkler's nonlinear model is also covered in the subsequent sections. With the introduction of FEM in the 1960s, the concept of modelling soil as a continuum emerged. This led to the development of numerous constitutive relations for modelling soil as linear or nonlinear elastic and elastoplastic. Popular constitutive relations include, for instance, Linear Elastic Model, Mohr-Coulomb Model, Hyperbolic Model, Strain Hardening Model, etc. [2]. The concept of incorporating an interface element between two distinct materials was first proposed in the 1970s. The interface makes it possible to simulate how the structure and soil move in relation to one another [3]. Macroelement concept was introduced in foundation engineering, which allows taking into account the coupling phenomena involved in SSI while avoiding the complexity and the numerical cost of nonlinear finite element dynamic analysis. It is equipped with a nonlinear "constitutive law" (defined by the mean of the relationship between forces and displacements) formulated in accord with the theory of plasticity or hypo-plasticity and making it possible to model the dynamic couplings (linear and nonlinear) in several directions between the superstructure, the soil and the foundation [5]. The primary contribution of this novel technique is to consider all such nonlinearities and the coupling between different degrees of freedom.

Summarising concepts discussed in the above paragraph, classification of SSI analysis with nonlinear soil and foundation behaviour can be done majorly in three ways: (1) continuum models, (2) beam-on-nonlinear Winkler foundation models, and (3) Microelement modelling. This study reviewed previous methods adopted to model nonlinearity in SSI analysis. The study also discusses various advantages, disadvantages and applicability of the abovementioned techniques.

Many experiments were performed to verify the result obtained from the analysis methods discussed in the previous section. Shake table and centrifuge tests are the major experiments performed, but a few full-scale models have been conducted in recent days. A concise review of some experimental setups relevant to numerical technique is also presented herein.

2. Modelling Approaches

2.1. Continuum Approach

Continuum modelling of the soil gives its precise and meticulous behavioural response for an SSI problem. The idea of the elastic continuum comes from using Boussinesq's theory for estimating static stresses. This well-known theory assumes the soil domain to be semi-infinite (soil boundary extended infinity in one direction), homogeneous, isotropic, linear elastic solid. Development in the continuum approach enabled the complete and thorough modelling of the semi-infinite soil domain using finite elements. Other assumptions of such theories are realised in the continuum approach, as shown in Fig. 1. Different continuum methods proposed by many researchers are presented hereafter.

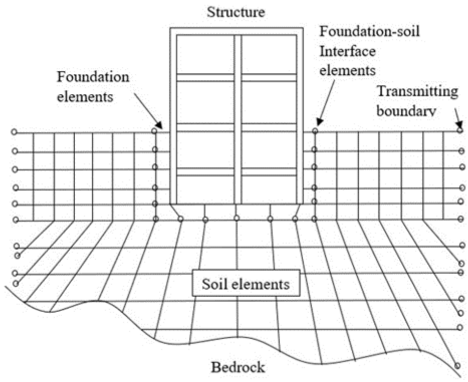

Fig. 1 Schematic illustration of SSI using continuum modelling by FEM (NIST, 2012)

2.1.1 Finite Element Method

FEM is known to be an efficient and multifaceted technique for performing numerical analyses because its applicability is extensive and can be applied to many classes of problems. Also, it can deal with real and complex tasks. To address the SSI problem, FEM considers specific parameters that are discussed below.

a. Absorbing Boundaries

The soil domain should be large enough to deal with the problem of radiation damping generated due to wave propagation within semi-infinite space, which increases a substantial amount of time and internal memory for complete FE analysis. To find an optimal solution (i.e., to reduce computation time and scale of soil domain), wave-absorbing boundaries can be adopted. All boundaries are generally classified into elementary, local and consistent (global) boundaries, which are briefly discussed below.

Elementary boundaries are typically employed in static analyses. Since this boundary is unable to simulate the wave energy radiation toward the infinite soil domain, it is more practical in situations where the wave energy radiation has little impact, such as the interface between soft and hardened soils. A few examples of elementary boundary conditions are Free field, fixed boundaries, and tie boundary conditions. In the soil-pile interaction analysis of a pile embedded in a deep multi-layered soil under seismic excitation, Peiris et al. [6] made successful implementation of elementary boundary conditions.

The viscous boundary produces the absorbing effect by employing a viscous damper or dashpot attached to the boundary element. It gives better results when the boundary is positioned at a suitable distance from the region of interest [7]. Absorbing boundary conditions for the dashpot system can be computed using Eq (1) and represented schematically, as shown in Fig. 2.

$$\begin{aligned} \sigma &= a\rho v_p w \\ \tau &= b\rho v_s u \end{aligned} \qquad (1)$$

where σ and τ are the normal and shear stress, ρ is unit mass, v_p and v_s are the primary wave and secondary wave velocities at the boundary, w and u are the normal and tangential velocities, a and b are the dimensionless parameters.

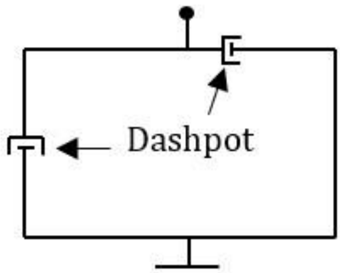

Fig. 2 Viscous boundary element

Another such boundary is the unified boundary proposed [8], which is quite similar to the viscous boundary [7]; the difference arises in the value of the dimensionless parameter, which varies with the value of Poisson's ratio of soil. The viscous boundaries were found most appropriate to apply in time domain analysis among the numerous absorbing boundaries [9].

Another idea to overcome finite boundary difficulties was the infinite element [10]. The formulation of the infinite element is the same as that of finite elements; addition is the domain mapping. This element doesn't require any other boundary condition (BC) to simulate zero displacements at infinity which is an added advantage over other BC. In the case of SSI analysis, frequency-based dynamic infinite elements were used to describe the far-field response of a 2D layered half-space. Nonlinear analysis was not possible because the formulation was frequency-based [11].

Kelvin elements [12] work more effectively than viscous dampers, provided their constants are properly calculated. While using the Kelvin element, the required mesh size of the soil element also gets reduced. The predominant frequency of loading governs the stiffness and damping constant values in the Kelvin model, and the stiffness value can be calculated using Eq (2). Kelvin element can be represented as shown in Fig. 3.

$$k_r = \frac{G}{r_0}[S_{u1}(a_0, v, D) + iS_{u2}(a_0, v, D)] \tag{2}$$

where k_r refers to the complex stiffness, r_0 refers to the distance in a plan between the foundation's centre and the node to which a Kelvin element is coupled, G refers to the modulus of rigidity of the soil, S_{u1} and S_{u2} are the dimensionless quantities obtained from closed-form solutions, a_0 ($= \frac{r_0\omega}{V_s}$, where ω is the excitation frequency and V_s is the shear wave velocity of the soil medium) is the dimensionless frequency, i is the imaginary unit, v is the Poisson's ratio, and D is the damping ratio of the material.

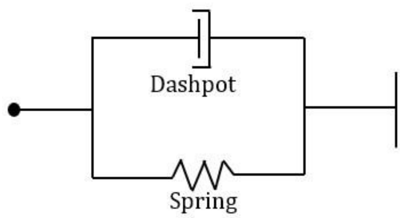

Fig. 3 Diagram representing Kelvin element

An advanced plasticity-based constitutive soil model and hierarchical single surface (HiSS), employed with the Kelvin element to perform the dynamic analysis of soil-pile interaction of single pile and pile groups. It was found that the suggested model performs satisfactorily with the Kelvin element [13].

A viscous spring artificial boundary (VSAB) condition was developed by modifying the spring constant and damping coefficient in the spring-dashpot system and using it to solve the dynamic excitation problem [14]. The values of the constants are calculated from Eq (3).

$$\begin{aligned} k_1 = k_2 &= \lambda_t \frac{G}{R} A \\ c_1 = c_2 &= \rho v_s A \\ k_3 &= \lambda_n \frac{G}{R} A \\ c_3 &= \rho v_p A \end{aligned} \tag{3}$$

where c_1, c_2 and c_3 are the damper and k_1, k_2 and k_3 are spring constants in x, y and z directions, respectively, v_p and v_s are primary wave and secondary wave velocities, G is the modulus of rigidity of soil, ρ denotes mass density, R represents the distance between the load point and the soil boundary, A is the total area contributing from surrounding nodes, λ_n and λ_t represents the constant in normal and tangential directions of the boundary in modified form.

The wave radiation issue in an infinite soil domain can be addressed by coupling the finite and infinite elements at their junction [15]. The dynamic SSI problem of the semi-infinite soil domain was examined using the finite elements in conjunction with 2D and 3D infinite elements in ABAQUS [16]. Domain Reduction Method (DRM) combined with Perfectly-Matched-Layers (PMLs) using ABAQUS to absorb outgoing waves perfectly [17].

b. Domain of Soil Model

To determine the sufficiency of the range in the horizontal direction of the soil domain, two criteria were taken into consideration: (a) a shear soil column formed of the identical material under linear elastic undamped conditions should have an outcome as close as viable to that of a soil column away from the boundary, and (b) the nonlinear vertical soil reaction should be modest in contrast to the horizontal response at any point in the realm of computation [18]. The horizontal distance of soil lateral boundaries must be at least five times more than the width of the structure, usually up to 60 m. Since the greatest amplification of wave generally occurs up to the depth of 30 m of the soil profile, recommended depth of bedrock can be up to 30 m while performing numerical analysis [19].

c. Size of Soil Element

The accuracy and reliability of SSI analysis results may get altered due to the size (Δx) of the element and the time-step (Δt) size used to model soil. Proper wave propagation is not guaranteed when the elements assumed to discretise the soil are of inappropriate size. The size of the elements should not be more than $\left(\frac{1}{8}\right)^{th}$ of the minimum wavelength (λ_{min}) or maximum frequency (f_{max}) of the seismic wave radiation travelling through the soil domain [20] and can be evaluated as per Eq. (4). This condition assures that even the shortest wavelength can easily propagate through the soil medium. To ensure stability and accuracy while performing the numerical analysis for time-step (Δt) size, Eq. (5) can be used [21].

$$\Delta x = \frac{\lambda_{min}}{8} = \frac{v_s}{8 f_{max}} \tag{4}$$

$$\Delta t = \frac{\Delta x}{v_p} \tag{5}$$

where v_p and v_s are longitudinal and shear wave velocities, respectively.

d. Soil Constitutive Model

The constitutive soil model consists of mathematical equations representing the nonlinearity of soil using a single element which can be further used in numerical computations to represent the relationship between stress and strain of a particular type of soil. Some of the constitutive models are discussed in Table 1.

Table 1. Different constitutive soil model

Type of Model	Model	References	Attributes
Elastic	Hooke's Law	[22]	It represents the linear elastic behaviour of soil but doesn't hold well for the elastoplastic behaviour of soil.
	Hyperbolic	[23]	Calculating the tangential modulus at any point of stress during loading can represent nonlinear elasticity, but because hardening behaviour is ignored during unloading, it cannot be applied.
Simple Elastic Plastic	Mohr-Coulomb	[24]	It is used where strength is dominating criterion, hexagonal failure cone is used to represent the real failure pattern. After achieving the maximum strength, it fails to incorporate the softening effect.
	Drucker-Prager	[25]	It is the same as Mohr-Coulomb but uses a simple cone to represent the failure pattern. The strength parameter shows valid representation but stiffness nonlinearity is not considered.

Critical State	Modified Cam Clay	[26]	It describes the strength, dilatancy and critical state of the soil and also represents loading and unloading effectively since the nonlinearity is modelled by hardening plasticity. It may allow for unrealistically high shear stresses.
	Elastic-viscoplastic	[27]	It is developed to describe the rate-sensitive behaviour of normally consolidated clay, and viscoplastic strain is used as a hardening parameter.
Single Yield Surface	Single Hardening	[28]	It is developed for frictional materials based on elasticity and plasticity. Elastic behaviour is represented by Hooke's law, while plastic behaviour is by failure criterion, yield criterion and non-associated flow rule.
Double Hardening	PLAXIS Hardening	[29]	Friction hardening and cap hardening are used in this model to simulate plastic shear under deviatoric loading and volumetric plastic strain under primary compression, respectively. Excludes both creep and anisotropic stiffness strength.

e. Soil Structure Interface

Another critical parameter that needs to be taken into account is interface modelling. Modelling of interfaces helps to understand the phenomenon of slip, bonding & rebonding between soil and structure. The most common interface elements are two-node elements, continuum elements comprising finer meshing, zero thickness, and thin-layer elements (Fig.4). Dashpot elements and node-to-node spring elements are examples of two-node elements. Interface behaviour may be represented for certain scenarios by refining a conventional finite element mesh adjacent to the interface and imparting appropriate characteristics [30]. Viladkar et al. [31] pointed out that one of the primary drawbacks of the methodology proposed by [30] is its inability to properly mimic the failure or slip plane when two types of materials are sandwiched. A widely employed interface element obtained from Goodman's hypothesis relates to the stresses and relative displacement of nodes. It is known as a zero-thickness element since it is a four-node element with no thickness. On the other hand, zero-thickness elements have drawbacks, such as being prone to inaccuracies in normal stress and deformation calculations [32]. In finite element analyses, zero-thickness interfaces are better for modelling solid-on-solid contact [33]. Another option is to think of the soil-structure contact as a thin layer or continuum. It was proposed to overcome the mentioned difficulties of zero-thickness elements. Thin-layer elements are preferable to zero-thickness elements because both field and simple shear tests reveal the presence of a transition zone alongside the interface of two stiff bodies [34]. A simple shear test can be used to measure the thickness of the thin-layer interface [35]. Formulation of an isoparametric interface is applied between soil and footing base to assess the behaviour of shallow foundations when subjected to eccentrically inclined load [36].

FEM necessitates the usage of distinct transmitting boundaries or infinite elements, which might result in inaccuracies. Despite the addition of transmitting boundaries, the whole structure-soil model is still huge. Analysis using FEM needs a significant amount of time and internal memory compared to other continuum approaches, which limits its application in certain problems. Moreover, a detailed review of modelling SSI systems using FEM is available [37].

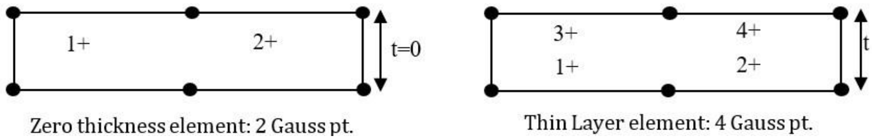

Fig. 4. Interface element

2.1.2 Boundary Element Method

The boundary element method (BEM), another numerical approach in progress to FEM, is more favourable than FEM since it simply needs a surface (or boundary) discretisation, which helps to meet the radiation condition without the requirement of complex non-reflecting boundaries.

The indirect BEM produced fairly accurate findings among the several BEM formulations (the weighted residual formulation, the direct and indirect BEM) [38]. Boundary integral problem approach applicability broadened from isolated foundations to numerous rigid foundations of various shapes and sizes laid on an elastic or viscoelastic half-space, applied with the seismic waves along with other possible external stresses. It is observed that the discretisation of the foundation had a considerable impact on the computed impedance functions in the case of relatively minor separations [39-40]. It also found that numerical outcomes documented by some authors [41-42] in the scenario of diminishing a small gap between the foundations contain an error. The time-domain BEM, combined with the Stokes fundamental solutions, was used to solve a 3D structural system composed of a large rigid square footing lying on isotropic, homogeneous and linear elastic half-space [43].

Furthermore, some investigations have been done on the interaction of nearby rigid foundations on a multi-layer viscoelastic soil media. A 3D frequency-domain-based BEM framework is used in aggregation with infinite space fundamental solutions and the successive stiffness technique to simulate a soil medium of several layers [44-46]. A boundary element technique of the substructure deletion approach is available for seismic evaluation of the dynamic soil-structure interaction among numerous embedded foundations [47-49]. In the frequency domain BEM, an impedance function is developed on and below the foundation surface [50]. The difficulty of applying BEM in the event of heterogeneous media is one of its drawbacks. Similarly, the benefit will be lost if BEM is used to solve a nonlinear issue.

2.1.3 Finite Element - Boundary Element Method

BEM, in conjunction with FEM employed to solve the differential equations of several SSI cases; one such example is the transient analysis of dynamic soil-structure interaction (DSSI) administered to SH motion [51]. Coupled boundary element with finite element method used for three-dimensional soil foundation interaction where boundary elements represent the soil medium. The condition of equilibrium and continuity along the soil and structure interface was used to ensure the continuity of both elements [52]. Finite element–boundary elements coupling models are used to study the dynamic interaction between three-dimensional mass lumped and distributed structures applied to harmonic excitation constructed on square foundations embedded in soil media [53]. The attributes of seismic response of a nuclear power plant composed of a reactor, control and a turbine building examined that was shaken by an artificially induced motion using 3D BEM and 2D FEM [54]. BEM was also used in conjunction with FEM to analyse the DSSI of coupled shear walls [55] and adjacent piled buildings [56]. Finite Element - Boundary Element - Infinite Element - Infinite Boundary Element technique devised to account for various SSI effects. The approach was shown to be capable of earthquake-resistant design and evaluation of structures, mainly of nuclear power plants based on multi-layered soil deposits. The whole structure was dealt with in the frequency domain. Also, the combined model reduces the computational effort by representing the nonlinearity of the near-field soil in an equivalent linear fashion [57].

BEM, on the other hand, isn't well adapted to inhomogeneous or anisotropic media. As a result, researchers intended to develop a technique that included the benefits of both the Finite element and Boundary element methods. In the outcome, the Scaled Boundary Finite Element Method (SB-FEM) is a semi-analytical technique that came into existence [58]. DSSIA-3D software improved by adding a novel approach in which the concept of SBFEM was used to model unbounded soil and the FEM employed in the modelling structure [59]. An alternate method [60] for the computational homogenisation of heterogeneous structures is presented using the idea of the Scaled Boundary Finite Element Method (SBFEM).

Besides the aforementioned methods, the Domain Reduction Method (DRM) came into existence, wherein the whole domain was separated into two sub-domains, (a) one for simulating earthquake origin and propagation path effect, removing localised features and (b) the other for modelling local site effects. The size of the domain considered for analysis was also significantly reduced [61]. Evaluation using DRM resulted in a 50% reduction in computing time compared to traditional absorption boundaries with viscous dashpot systems [62].

2.2 Winkler-based approaches

To describe the general behaviour of the soil-structure interface, Winkler-based techniques may only make use of one-dimensional spring elements or one-dimensional spring elements in addition to two-dimensional or three-dimensional soil components. The Winkler spring technique is desirable in design due to its simplicity and little computational effort. Since it is a spring-based model, its mechanical aspect can be easily calibrated. These characteristics of the Winkler-based approach signify its advantage over the continuum approach.

Initiating with the revolutionary effort of McClelland and Focht [63], Beam-on-Nonlinear Winkler foundation (BNWF) models have been carry forwarded for several decades for analysing the behaviour of foundations mostly for piles subjected to static loads case [64] and then taken forwarded to the application of subjected dynamic loads [65-66]. Multiple executions of the dynamic p-y technique were attempted, and it found that the characteristics of the nonlinear springs and dashpots can affect computations [67]. Dynamic nonlinear response output of offshore pile assessed in a both qualitative and quantitative manner [68]. Issue of complete interaction among the whole soil-pile raft-superstructure arrangement addressed in the study bearing in mind the change in design forces of various components in the structure, which were left out in previous studies [69]. The Nonlinear Winkler model for the composite caisson-piles foundation is by joining the caisson and the pile group. The nonlinear four-spring Winkler model is used for the caisson, and the axial-lateral coupled vibration equations are deduced for the pile group [70].

The methodology adopted for the pile foundation proposed above is taken further to the shallow foundation. Some of the early efforts to use a model based on the Winkler approach for capturing the rocking response of shallow footing are discussed hereafter. With the use of elastic-perfectly-plastic springs and coulomb slider elements, an analytical framework was created to predict the moment-rotation behaviour of rigid foundations. Coulomb slider elements manage to capture the uplift of the foundation, whereas elastic-plastic springs are believed to respond to compression only [71-72]. A model was developed based on two methods: (i) a two-spring model and (ii) a distributed Winkler spring model. The author developed three mechanisms to consider nonlinearity at the foundation interface: (i) viscous dampers, (ii) elastic-perfectly plastic springs, and (iii) an impact mechanism permitting energy dissipation at impact [73]. An analytical framework is presented to evaluate the rocking response of a single-degree-of-freedom system while considering the foundation uplift, along with an expression to estimate the base shear of a flexible structure allowed to uplift. The framework considers individual springs as linear elastic [74]. At the base of a shearwall structure, Nakaki and Hart [75] utilised separately and individually positioned vertical elastic springs along with viscous dampers. The property of the Winkler spring used in this study has zero tension capacity and elastic compression resistance. A nonlinear stiffness degrading hysteretic model was used to model the inelastic shearwall structure. The Winkler-type finite element model reflects the nonlinear behaviour of shallow strip footing subjected to lateral cyclic loading. This model uses the nonlinear spring backbone curve calibrated against the pile. The main limitation of the model was the calibration done mostly with moment-dominated strip footings [76]. The study [77] includes a foundation uplift in performance-based design as in continuation of the prior study. To capture the cyclic response of shallow foundations, Allotey and Naggar used a Winkler-based modelling approach in which linear backbone curves were adopted from the earlier work performed by the author. According to their findings, the model can anticipate the moment–rotation and settlement reaction fairly well. On the other hand, the model cannot accurately reflect the sliding response, which could be due to the absence of coupling amongst the various forms of deformation [78-79].

The BNWF model incorporated in OpenSees comprises elastic beam-column elements simulating structure-foundation behaviour and independent zero-length soil elements simulating soil-foundation behaviour, as shown in Fig. 5. The developed model is only suitable for two-dimensional analysis, and it has also been discovered that the model underpredicts sliding response [80]. An investigation [81] assessed the influence of SSI effects on the seismic performances of 2D moment-resisting reinforced concrete frames by means of FEM and BNWF. The finding suggests adopting the FEM model in the case of a four-story 2D structure can reduce seismic demand by up to 50% for maximum inter-story drift ratio and up to 20% for maximum base shear when compared to a fixed-base model.

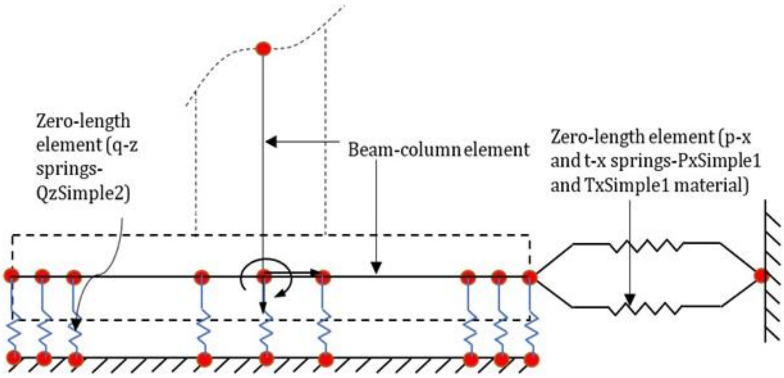

Fig. 5 Schematic diagram of a Beam-on-Nonlinear Winkler Foundation [80]

In contrast, modelling by adopting the BNWF model can change the seismic demand only if the structure has a longer time period (eight-story 2D frame) constructed on very soft soils. Although, the reductions compared to a fixed-base model (up to 20% for maximum base shear and maximum inter-story drift ratio) are less than what a full FEM model would have given as an outcome, as shown in Fig. 6. The fixed-base assumption overestimates the design of the shear wall element while underestimating the design of the coupled moment frame [82]. The influence of nonlinear SSI on the seismic response of acceleration-sensitive non-structural components of a four-story steel moment-resisting frame is investigated. The results suggest that nonlinear SSI positively impacts the performance of the non-structural components of the structure [83].

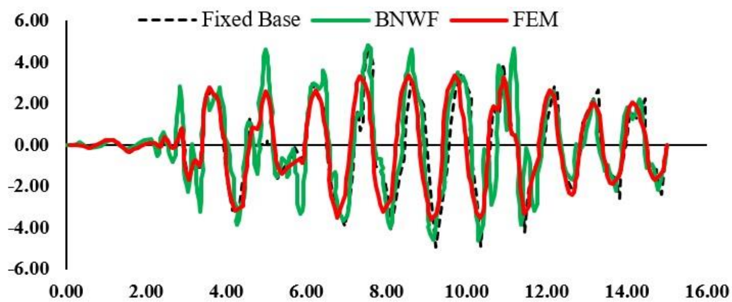

Fig. 6 Acceleration of top storey of 4-storey building

Study [84] aimed to examine the impact of SSI on the seismic outcome and susceptibility of RC structures. Authors [85] reviewed several soil-foundation-interaction models focusing on raft foundations.

2.3 Macro Modelling

Since the finite element approach contains computational complexity and requires a thorough understanding of the concept, specifically to deal with the numerical intricacy pertaining to the soil-foundation-structure behaviour during severe earthquakes, it is barely an alternative for this purpose. Another discussed a simpler technique, the beam-on-nonlinear-Winkler-foundation (BNWF) family is easy to perform. However, it is pointed out that the BNWF model cannot suitably take care of the coupling between its different degrees of freedom [86-87].

The nonlinear macro element (NLME) concept, which involves replacing the complete soil foundation arrangement with one element at the base of the superstructure, has garnered increasing attention in recent years. In this approach, the foundation and the soil arrangement are treated as a macro element, and a 3 DOF (in case of 2D) or 6 DOF (in case of 3D) model is developed to describe the vertical and horizontal force-displacement, moment-rotation behaviour of a point at the centre of footing. The coupling between the different DOFs of macroelement represents the key improvement over the BNWF technique.

The concept of nonlinear macro-element (NLME) was first put forward to estimate settlement and rotation for strip footings placed on the sand under the joint action of eccentric and inclined loading. Here, isotropic-hardening elastoplastic law allows the coupling of displacement and rotations [4]. Furthermore, several NLMEs-based models have been proposed for various types of loading, foundation geometry, and soil type. For example, a bounding surface plasticity concept was added to the previous model and expanded to cyclic loading [88]. Macro-element frameworks [89] were used to investigate nonlinear dynamic soil-structure interaction (DSSI) subjected to seismic excitation, utilising an elastic-perfectly plastic concept of the model described in the earlier study [4]. A coupled plasticity-uplift model was adopted to incorporate footing uplift into the macro-element formulation. The proposed model was limited to strip footings placed on cohesive soils subjected to seismic loads [90-91]. A stiffness degradation model was introduced to consider lowering the soil-footing contact area to capture minor detail of the observed rocking response [92]. A coupling factor in the soil-foundation stiffness matrix helps to include the uplift in the nonlinear microelement plasticity model of the initial phase [93]. Grange S et al. [94] extended the work of Cremer et al. [5] to 3D circular footing and incorporated uplift with the plasticity model framework.

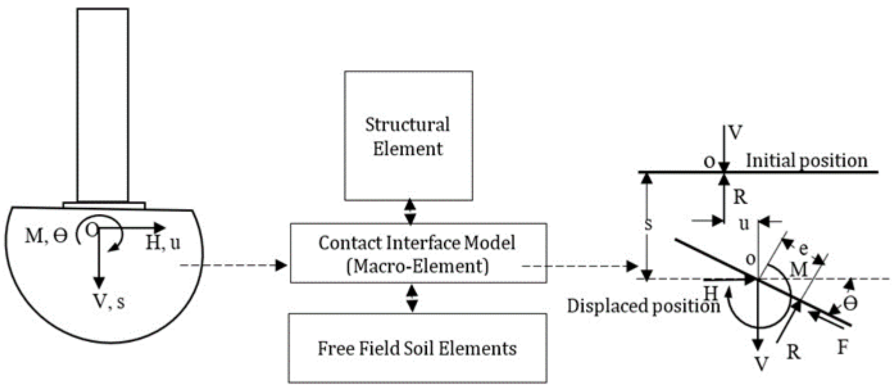

Fig. 7 Macro-element contact interface model [97]

A macro-element model has been developed using the theory of hypoplasticity for modelling shallow foundations on sands. The projected approach uses a simpler mathematical framework, allowing easy application in existing structural analysis FE codes [95]. The numerical implementation of the 6–dof hypoplastic microelement is proposed in the form of the Finite Element code incorporated in Abaqus [96]. The contact interface model (as shown in Fig. 7) proposed [97] to track the progress of the soil-footing contact area. The critical contact area ratio (A/Ac) is defined as the ratio of the footing area (A) to the required footing contact area for vertical and shear loads (Ac). Six model input variables, mainly user-defined and the contact interface model, are required to capture the fundamental properties of shallow foundations under coupled cyclic loading, such as load capacities, energy dissipation, stiffness deterioration, and deformations.

Chatzigogos CT et al. [98] presented bounding surface plasticity in combination with the uplift formulation suggested by [89]. Further, Figini R et al. carry forwarded the previous research on dense sand [99]. A macro-element was proposed [100] for a single pile in cohesive soil exposed to the lateral earthquake force imposed at the head of the pile. The method relies on a nonlinear elastic constitutive model integrated with a boundary plasticity model. It represents the elastic behaviour under small displacement using the elastic linear impedances suggested by Gazetas [101] and incorporated in EC8 [102].

A macro-element presented [103] for a single vertical pile in sand devised within hypo-plasticity theory inspired by macro-element formulation for the shallow foundation of Salciarini et al. [95]. Generalised load and displacement vectors are used to characterise the behaviour of the pile are shown below:

$$t = \{V, H, M\}^T \tag{6}$$

$$u = \{w, u, \theta\}^T \tag{7}$$

Also, the below-recommended matrix can be used to calculate the elastic stiffness matrix regulating the elastic response of the microelement:

$$\kappa^e = \begin{bmatrix} k_v & 0 & 0 \\ 0 & \alpha k_{hh} & \alpha^{2/3} k_{hm} \\ 0 & \alpha^{2/3} k_{hm} & \alpha^{1/3} k_{mm} \end{bmatrix} \tag{8}$$

Where α is the dynamic interaction factor and k_v, k_{hh}, k_{mm} and k_{hm} represents the vertical, horizontal, rotational and combined horizontal-rotational elastic stiffness at the head part of the pile, respectively.

The macro-element formulation suggested above can be adapted for single batter piles in the sand [104] and cassion foundations in the sand [105]. The inability to account for changes in geometry and loading boundary conditions in previous studies was accounted for in the model proposed by F. Pisano et al. [106].

The macroelements for deep foundations developed to date have several limitations that do not allow reproducing (at least in a direct way) the response of a pile group under seismic loading. A newly established frequency-dependent macroelement technique is capable of reproducing the dynamic attributes of the system at multiple levels of increasing ground shaking [107].

Failure surface (point A):

$$\left(\frac{H'}{\xi N\mu H_o}\right)^2 + \left(\frac{M'}{\xi N\mu M_o}\right)^2 - 1.5\frac{H'}{(\xi N\mu H_o)}\frac{M'}{(\xi N\mu M_o)} - \left[1 - \left(\frac{V'}{\xi N\mu V_o}\right)^2\right] = 0$$

A (H',M',V')

B (H^*,M^*,V^*)

Failure suface (point B):

$$\left(\frac{H^*}{N\mu H_o}\right)^2 + \left(\frac{M^*}{N\mu M_o}\right)^2 - 1.5\frac{H^*}{(N\mu H_o)}\frac{M^*}{(N\mu M_o)} - \left[1 - \left(\frac{V^*}{N\mu V_o}\right)^2\right] = 0$$

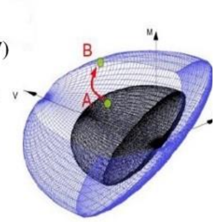

Fig. 8. Loading and failure surfaces of the hypoplastic macroelement model for a single vertical pile [103]

3. Experimental Works

The result and conclusions at which many researchers arrived by adopting different approaches mentioned in the review above for soil foundation modelling need to be validated or complemented by performing experimental work. Experiments performed by researchers are reviewed hereafter, some validating the soil foundation models.

3.1 N-g Centrifuge Experiments

Centrifuge experiments are based on the concept of scaling down a prototype design in terms of geometry by introducing a higher gravitational force (e.g., N = 20-100 times that of gravity). The methodology turns out in the preservation of prototype soil stresses at the model size due to similitude and scaling rules [108]. The centrifuge is a significant instrument for examining the performance and getting better outcomes of the soil-foundation system relating to the rocking shallow foundation because the nonlinearity in the system is primarily due to the soil's unpredictable behaviour. Several centrifuge tests were performed on the strip and square footings accompanying the shearwall and several other structures for different types of loading (monotonic, lateral cyclic and dynamic) [109-111]. Moreover, Ugalde executed tests on a model bridge pier supported by shallow footings [112]. Another study discussed shallow footings supporting a shearwall and a moment frame as part of a combined load-resisting system. The influence of the combined nonlinearity of structure and foundation on the response of the complete structural system is demonstrated in the centrifuge test [113]. Centrifuge experiments were conducted to find out how the degree of liquefaction influences the seismic and post-seismic settlement of shallow foundations lying on saturated sand [114]. The introduction of a new centrifuge tube-actuator was used to discharge spherical projectiles at single-degree-of-freedom (sdof) models resting on sand-filled shallow foundations. This enables the generation of dynamic impulse excitation, which is utilised to assess dynamic rocking stiffness in modest strains [115].

A comparison of dynamic p-y analysis results with the result of dynamic centrifuge model testing shows satisfactory agreement across a wide range of ground motions [116]. Geotechnical centrifuge tests were used to look at the dynamic reactions of soil foundation models with different pile arrangements, and they estimated both kinematic and inertial interaction [117]. The theoretical concept of the BNWF model is complemented by performing a centrifuge test [80].

A set of dynamic centrifuge tests was conducted to evaluate the impacts of soil conditions and structural variables on SSI effects, resulting in a dataset that might be critical for engineering practice [118]. Test performed on single-degree-of-freedom (sdof) system to obtain a period-lengthening ratio (PLR) for the different seismic intensities, which represents the nonlinear properties of soil; the result shows the PLR of sdof system increases with peak ground acceleration at the surface [119].

3.2 1-g Experiments

To study the SSI impacts of shallow foundations, one-g experiments can be conducted employing either input at the base (e.g., using a shake table) or inertial loading (e.g., by structure-mounted hydraulic jacks). These tests have an advantage over centrifuge tests in preserving genuine soil-to-structure size scaling. Tests are, however, restricted by large scale, which significantly raises experimentation expenses. Also, the soil box's size constraint may result in boundary effects on the free-field soil conditions. Barlett tested small plate footing (simulating the real scenario) of size 0.5m×0.25m placed on clay. In contrast, Wiessing performed the cyclic test on identical footing supported on sand [120-121]. These experiments found that a spread footing can yield soil at a moment less than the moment carrying capacity of the column and can prevent the formation of a hinge at the base of the column. By performing the shake table test, Gazetas and Stokes demonstrated the trustworthiness of the impedance functions enlisted in Gazetas [122-123]. To study the true failure mechanism of the foundation, shake table experiments were performed on a strip footing positioned on dry four-layered sand embedded at various depths into the deposit. It is observed when eccentricity is introduced; the seismic bearing capacity is reduced [124]. Particle Image Velocimetry (PIV) paired with the aid of photogrammetry and rapid filming to monitor the seismic effect on soil and investigate the failure mechanism of a shallow foundation on a one-g shaking table. Moment effects are severe for structures with a centre of mass at a considerable height from the foundation level, leading to a significant decline in bearing capacity due to uplift. The failure mechanism was shown to be influenced by the depth of embedment and surcharge [125]. Overturning moments at the soil-foundation interface were studied by Paolucci et al. [92] through the use of a shake table test.

Performing the shake table test, assessment of the effects of pile arrangements, pile caps, and superstructures on time period elongation and damping ratio of a system under sinedwells, white noise and natural EQ motion done [126]. A five-storey scaled-down model structure was put on a shake table to observe the response of two structures, one with the fixed base condition and the other assisted by model piles embedded in soft clay. The result indicated that as the structure height increases, the response amplifies in SSI conditions, which can alter the functioning of the structure [127].

The majority of experiments to assess the effect of SSI on structure uses shake table testing or dynamic centrifuge model studies. But a full-scale dynamic test on a portal frame railway bridge was recently performed [128]. Amendola et al. [129] conducted full-scale field tests on soil-structure interaction to calculate the foundation impedance function.

Gajan et al. [130] correlated rocking foundation performance characteristics with capacity parameters and parameters relating to earthquake demand by taking data from 142 centrifuge and shaking table tests that included a different type of soil, varying foundation geometry, different structural models and ground motion.

4. Conclusions

Since nonlinearity in soil and structure is considered the most critical aspect, it leads to the failure of the entire system. Different methodologies proposed by the researchers are reviewed in this study to find out the best way to model nonlinearity in soil-structure interaction (SSI). The following conclusions are made from this extensive review presented:

FEM helps model the nonlinearity of soil, structure, and foundation to give results similar to the actual scenario but is time-consuming and computationally expansive due to the huge soil domain. The boundary condition, size and soil element type are critical aspects of energy dissipation to reduce the soil domain. The review shows the possibility of evolving a particular technique [17].

Selecting a constitutive model for different soil conditions requires special attention. The Mohr-Coulomb, Modified Cam Clay, and Hyperbolic models are most commonly used. The hyperbolic model is frequently applied in SSI problems pertaining to drained and undrained soil conditions [131].

Adopting FEM in the direct approach can result in reductions in the calculation of the seismic demand of up to 50% in the case of maximum inter-storey drift ratio, whereas up to 20% in the case of maximum base shear when compared to a fixed-base model.

BEM came as an alternative to FEM, reducing the effort to discretise the soil-structure element. But it has its shortcoming in dealing with heterogeneous media and solving the nonlinear issue.

The idea of combining FEM and BEM suggested by researchers reduces computation effort and time. The theoretical development of combined FEM-BEM technique in the application of DSSI has been taken way forward, yet hardly any commercial software uses this concept.

Beam-on-nonlinear Winkler Foundation (BNWF) model evolves as a technique that uses spring to tackle the problem of a huge soil domain and its nonlinearity. Still, the main drawback is the model under-predicts sliding response. In addition, other Winkler's hypothesis also fails to depict the continuum nature of the soil, which is practically unacceptable in seepage, stress analysis, etc.

Only longer-period structures on extremely soft soil modelled using the BNWF technique can affect the assessment of the seismic demand.

In the context of both maximum base shear and maximum inter-storey drift ratio, the reductions in seismic demand estimations compared to a fixed-base model are lower (up to 20%) than what a FEM model predicts.

The primary distinction between the FEM and BNWF approaches might be linked to how the overall damping is characterised. Comparing simplified and rigorous techniques of validating the damping percentage typically provided by different codes using synthetic graphs could be an intriguing future development.

The inclusion of radiation damping into the macroelement formulations is a significant issue that has not yet been fully covered in the existing macroelement models. Considering its significance in the overall response of the system, the issue must be addressed in future.

Several experimental setups have been established to validate numerical results, but few experiments have been performed in the field. Therefore, more experimental studies are necessary to improve understanding of SSI nonlinearity.

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