SOIL-FOUNDATION-STRUCTURE INTERACTION ANALYSIS

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Contents

Summary

Soil-structure interaction (SSI) analysis sheds light on the response of foundations on deformable soil media. Soil deformation due to foundation loading may affect the stresses in the foundation and adjoining structural elements, which may have important

design implications. Most widely used model for static SSI analysis involves using independent Winkler springs to represent soil reaction. A better model is definitely the elastic continuum model involving more rigorous analysis procedures but this method requires correct assessment of soil parameters which may be a challenging task in itself. Some analytical closed form solutions exist for specific problems of beam foundation, plates and pile foundations, some for Winkler foundation and some for elastic continuum. There are also other models, the so-called two parameter elastic model, which are improvements over the Winkler spring offering some continuity (connection) between Winkler springs, however they have not been popular, possibly due to additional mathematical complexities. Dynamic soil-structure interaction (DSSI) consists of three different physical processes. First, soil flexibility results in an increase of the structure period compared to fixed base structure. Secondly, the soil acts as an energy dissipation medium mainly through radiation damping. Third, stiffness of the foundation may impede the incident seismic waves producing a ground motion different from the free-field motion. The first two processes constitute inertial interaction, while the third one is known as kinematic interaction. The first step in the study of a DSSI problem is the evaluation of the complex-valued dynamic impedances (representing stiffness and damping) of the foundation. Simplified methods exist for the determination of dynamic impedance for different types of foundations under different soil conditions using simple equations and charts, but for the general case, rigorous numerical methods may be required. In most cases, only inertial interaction is considered and kinematic interaction is ignored. Finite element method (FEM) and Boundary element method (BEM) have been used extensively for studying problems involving DSSI. More research on DSSI effects is needed so that it can be incorporated in greater detail and clarity in seismic design codes.

1. Introduction

1.1. Soil-Structure Interaction

Civil engineering structures are built on soil (or rock) formations, and the loads acting on the structures are transmitted to the supporting soil through foundation elements such as footings, raft, piles etc. The imposed loads cause deformation in the supporting soil which may in turn modify stresses in the foundation and superstructure. This interaction between soil and foundation affecting the stresses in different structural elements (including foundation) may be defined as soil-foundation-structure interaction, more commonly known as soil-structure interaction (SSI). As an example of soil-structure interaction analysis commonly practiced, mat foundation is considered as a slab resting on springs (representing soil). Conventional design practice for ordinary structures usually neglect soil-structure interaction effects, but for important structures it may be considered in the design. The scope of soil-structure interaction includes retaining walls, buried pipe lines, bridge abutments, tunnels etc. This chapter will, however, primarily treat foundations.

1.2. Static and Dynamic Loading Effects

Based on the type of loadings, soil-structure interaction problems may be broadly classified into two different classes: (i) Static soil-structure interaction, dealing with static loads acting on the structure and (ii) Dynamic soil-structure interaction dealing with the case of dynamic loads. The two classes of problems are fundamentally different with characteristically different analysis procedures.

Static SSI

The interaction between soil and foundation of a structure due to static loads acting on the structure may be defined as "static soil-structure interaction". It is common practice to omit the word "static", hence the term "soil-structure interaction" or SSI may be used.

The primary difficulty in soil-structure interaction problem lies in the determination of the contact pressure between the foundation and the soil. Conventional foundation design may overcome this difficulty by adopting some arbitrary simplification such as assuming contact pressure to be linear. While such assumptions may be considered satisfactory for preliminary studies or unimportant foundation elements, they should not be used for the analysis of important structures.

Description of the condition of the interface between soil and foundation presents a difficult task. Most structural foundations will exhibit some frictional characteristics at the interface. On the other hand, the frictional forces will also have limiting values due to finite strength of the soil. In addition, factors such as pore water pressure, nature of loads on the foundation, foundation flexibility and time-dependent effects may influence the condition at the interface. It may therefore be prudent to consider the two extreme cases of interface behavior ranging from the completely smooth (frictionless) to the completely adhesive (friction) case. The assumption of smooth contact considerably simplifies the analysis of the interaction problem. In general, the effect of adhesion or friction is to reduce the settlement of a foundation (Shield and Anderson, 1966).

Dynamic SSI

The interaction between soil and foundation of a structure due to dynamic loads acting on the structure or due to incident seismic waves (propagating through the supporting soil) may be defined as dynamic soil-structure interaction (DSSI). The importance of considering DSSI effect is briefly discussed. For ease of discussion, hereafter in this chapter, SSI will be used to represent both static and dynamic loading cases. The dynamic characteristics of a structure resting on flexible soil are different when compared to the same structure resting on a rigid support. The flexibility of soil results in longer fundamental period of the structure. Also part of the vibrational energy of the flexibly supported structure is dissipated into the supporting and surrounding soil by radiation of waves as well as by hysteretic action of soil. Considering the case of propagating seismic waves through the soil, the presence of the relatively rigid foundation may scatter the incident seismic wave thereby producing a base motion different from the free-field motion.

2. Static Soil-Structure Interaction Analysis

2.1. Modeling of Soil

The most simple soil model assumes linear elastic behavior of the supporting soil. While it is recognized that this assumption may not be always rigorously satisfied by natural soil, linear elastic behavior considerably reduces the analytical complexity and provides useful information to many practical problems of foundation engineering which would otherwise be intractable.

There are several approaches for modeling soil behavior in soil-structure interaction analysis: (i) Winkler Model where soil is represented by single parameter spring elements attached to the foundation (ii) Elastic Continuum Model where soil is represented as an elastic continuous medium (iii) Two-Parameter Model where soil is represented by two-parameter shear-coupled springs. (iv) Three-Parameter Model where soil is represented by three parameter double layer shear-coupled springs

Winkler Model

Winkler Model represents the simplest model incorporating linear elastic behavior of the soil. In this model, soil is represented by a set of closely spaced linear elastic independent springs (Winkler, 1867) attached to the foundation. It is based on the assumption that load acting at a point on the foundation causes soil displacement at that point only. This assumption is not correct since displacements at one point are influenced by forces at other points due to the continuity of the soil medium. Furthermore, it is also difficult to obtain a realistic value of the spring constant. In spite of such limitations, Winkler springs have been widely used due to its mathematical simplicity and the fact that in more advanced analysis the accuracy may often be lost due to uncertainty in the soil parameters. Figure 1(a) shows Winkler model for rigid foundation (foundation does not deform) while Figures 1(b,c) represent flexible foundation (foundation is free to deform). Another limitation of Winkler foundation is that for uniform loading, the settlement of a flexible foundation on Winkler springs is uniform as shown in Figure1(b), which is incorrect. The single parameter elastic spring constant of Winkler spring is determined from an estimated or measured value of the modulus of subgrade reaction k which is defined as the pressure q required to cause a soil deflection w of unity:

$$q(x, y) = kw(x, y) \quad . \qquad (1)$$

The modulus of subgrade reaction can be evaluated from typical plate load tests where vertical load is applied on $1' \times 1'$ footing (plate) resting on the soil and resulting settlement is measured. The modulus of subgrade reaction (k_1) thus obtained is valid for a foot (0.3m) wide footing. Corrections need to be applied for larger size footings used for structures. A simple correction may be based on the stress bulb concept. Larger size footings produce larger stress bulbs and thus influence larger depths of soil; hence the soil deformation (settlement) is larger. For a B m by B m square footing in a homogeneous cohesive soil with uniform elastic modulus, the modulus of subgrade reaction can be estimated as $k = k_1(0.3/B)$. In cohesionless soils, the elastic deformation modulus increases with depth due to larger confining stresses and k may be estimated (Terzaghi and Peck, 1967) as $k = k_1\left[(B+0.3)/2B\right]^2$ or as $k = k_1(0.3/B)^n$ where $n = 0.4$ to 0.7. Bowles (1997) prefers not to use these relationships and instead recommends using the expression: $k = k_1[0.3E_1' I_{S1} I_{F1} / (BE' I_S I_F)]$. This comes from theory of elasticity based solutions for the settlement of a rectangular footing with flexible base on the surface of an elastic half-space. Here, $E' = (1-\nu^2)/E$ is a function of the stress-strain modulus E and Poisson's ratio ν. The shape influence factor I_S is a function of L/B and H/B ratios and ν, where the foundation size is $L \times B$ and H is the soil stratum depth causing settlement. The depth influence factor I_F is a function of L/B and H/B ratios and ν, where the foundation base is at a depth D . $E_1' I_{S1}$ and I_{F1} correspond to the $1'$x$1'$ plate load test. According to Bowles (1997), the stress-strain modulus E may be estimated as the weighted average of β_i within the stratum depth H causing settlement, where H is equal to $5B$ below the foundation base or the stratum depth to a hard (with 10 times higher E) stratum, whichever is smaller. If q_{ult} is the bearing pressure (kPa) causing a foundation settlement of say 1″ (0.025m), this means k (kN/m^3) can be expressed as $k = 40q_{ult}$. Considering $q_{ult} = q_a(F_S)$, k may be given as $k = 40q_a(F_S)$ where q_a is the allowable bearing pressure and F_S is the factor of safety. Different approaches for obtaining values of modulus of subgrade reaction *k* have been discussed. As a further check, typical values of modulus of subgrade reaction for different types of soil given in Bowles (1997) or Das (2007) may be used as a guide. Typical values for k_1 given by Das (2007) for sand ranges from 8 to 375 MN/m^3 and for clay (undrained shear strength 50 to more than 200 kPa) ranges from 12 to more than 50 MN/m^3. Analytical closed-form solutions exist only for simplified problems of SSI. Numerical methods may be needed to solve practical problems of structural foundations supported by Winkler springs, where the analysis may be done with or without considering the superstructure.

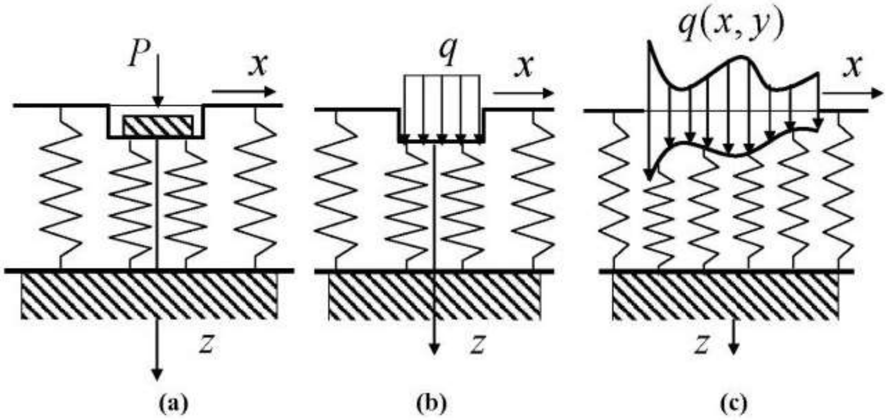

Figure 1. Winkler model: (a) rigid foundation (b) flexible foundation (uniform load) (c) flexible foundation (non-uniform load)

Elastic Continuum Model

In the elastic continuum approach, the soil is considered as a continuous medium with elastic properties. In general, the application of the continuum theory of classical elasticity to solve soil-foundation interaction problems presents considerable mathematical complexity. Theoretical closed-form solutions exist only for few special idealized cases. For practical problems, numerical methods have been used.

Boussinesq (1885) did the pioneering work for the elastic continuum approach by obtaining solutions to the problem of a concentrated force acting normal to the surface of a semi-infinite homogeneous isotropic elastic half-space. Solutions of the axisymmetric problem of a loaded circular plate resting on an elastic half-space can be obtained by using the Boussinesq solution and employing Hankel integral transform techniques or a method of superposition. A similar technique can be employed for the analysis of the two-dimensional problem of an elastic half-plane subjected to a uniform loading of finite width (strip loading) by making use of Flamant's problem (Timoshenko and Goodier, 1970) for line load acting on the surface of a half-plane.

Two-Parameter Elastic Models:

The inherent deficiencies of the Winkler model in representing the continuous behavior of soil and the mathematical complexities of the elastic continuum model have led to the development of other simplified soil models. Such models have two independent elastic constants and attempt to provide some form of continuity between springs. Four such models are introduced below

(a) Filonenko-Borodich Model: Continuity between the individual Winkler springs is introduced by connecting them to a thin elastic membrane (Figure 2) under a constant tension T (Filonenko-Borodich: 1940, 1945). This results in the addition of a tension term in the relationship between the deflection w and pressure q:

$$q(x,y) = kw(x,y) - T\nabla^2 w(x,y), \qquad (2)$$

where $\nabla^2 = \dfrac{\partial^2}{\partial x^2} + \dfrac{\partial^2}{\partial y^2}$.

For two-dimensional (plain strain) problems, Eq. (2) reduces to:

$$q(x) = kw(x) - T\frac{d^2\{w(x)\}}{dx^2} \qquad (2a)$$

(b) Hetenyi Model: In this model proposed by Hetenyi (1946), interaction between the Winkler springs is accomplished by connecting the springs to an elastic plate. The corresponding pressure-deflection relationship is given as:

$$q(x,y) = kw(x,y) - D\nabla^4 w(x,y), \tag{3}$$

where, $D = E_p h^3 / (12(1-n_p{}^2))$ is the flexural rigidity of the plate.

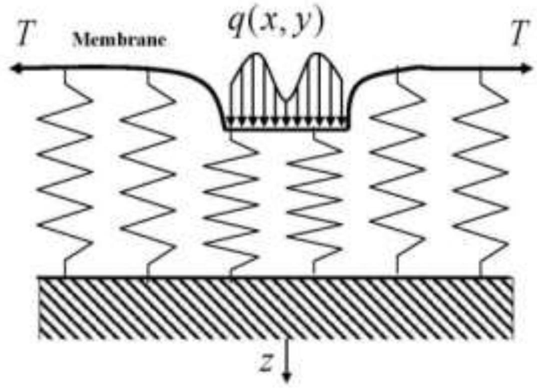

Figure 2. Filonenko-Borodich model

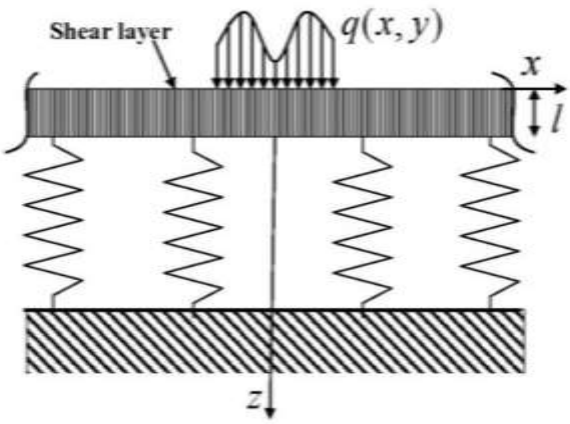

Figure 3. Pasternak model

(c) Pasternak (1954) Model: The spring elements are connected to a layer (Figure 3) of incompressible vertical elements which deform in transverse shear only. If the shear modulus of the layer is G, the relationship is similar to Eq. (2) with G replacing T.

(d) Vlazov and Leontiev Model: This two-parameter model is derived starting from the elastic continuum model. Simplifying assumptions are made with respect to the distribution of displacements such as $w(x,z) = w(x)h(z)$. The variational method is applied (Vlazov and Leontiev, 1966) to obtain soil response function similar in form with Eq. (2) where k and T is obtained as a function of soil elastic constants E, ν and distribution function $h(z)$.

Three-Parameter Elastic Models:

As another improvement over the Winkler spring, three-parameter soil model has been developed by Kerr. Here, soil is represented as two layers of linearly deforming springs interconnected by a thin, incompressible shear layer. The corresponding pressure-deflection relationship is given as:

$$\left(1+\frac{k_2}{k_1}\right)q(x)-\frac{G}{k_1}\left(\frac{d^2\{q(x)\}}{dx^2}\right)=k_2 w(x)-G\frac{d^2\{w(x)\}}{dx^2}, \tag{4}$$

where, k_1, k_2 are the spring constants and G is the shear coupling constant.

2.2. Analytical Solutions

Closed-form solutions obtained for the so-called classical “Beam on Elastic Foundation” problems may be considered as the starting point for soil-structure interaction research. According to the terminology used here, this problem may be renamed as “Beam Foundation on Elastic Soil”.

Beam on Winkler Spring

Let us first consider the simple model of a straight beam of width B supported along its entire length by Winkler springs. Classical treatment of this problem has been given by Hetenyi (1946). If $w(x)$ is the vertical deflection of the beam subjected to a vertical load of intensity $p(x)$, the reaction at any point x is given as $q(x)=kw(x)$, where k is the modulus of subgrade reaction. Considering equilibrium of a beam element and using Bernoulli-Euler beam theory, the following differential equation is obtained:

$$E_b I_b \frac{d^4 w(x)}{dx^4}+Bkw(x)=Bp(x), \tag{5}$$

where, E_b and I_b represent Young’s Modulus and Moment of Inertia respectively of the beam. This equation is also applicable for the plane strain problem, where a beam of width B is subjected to state of cylindrical bending, with $E_\mathrm{b}I_\mathrm{b}$ to be replaced by $E_\mathrm{b}Bh^3/12(1-\upsilon_\mathrm{b})$. This term represents the flexural rigidity of the strip, υ_b is the Poisson’s ratio of the beam material. The homogenous solution of Eq.(5) is:

$$w(x)=e^{\lambda x}\left(A_1\cos\lambda x+A_2\sin\lambda x\right)+e^{-\lambda x}\left(A_3\cos\lambda x+A_4\sin\lambda x\right), \tag{6}$$

where, $\lambda=\left(Bk/4E_\mathrm{b}I_\mathrm{b}\right)^{0.25}$. A_1, A_2, A_3, A_4 can be determined using appropriate boundary conditions at the ends of the beam for any particular external load $p(x)$. The parameter λ^{-1} known as the characteristic length, is indicative of the relative stiffness of foundation (beam) and soil, and appears to have a significant influence on the deflected shape of the beam. For some idealized problems, closed-form solutions for beam deflection have been obtained applying appropriate boundary conditions. Some results for infinite beam on Winkler springs are given in Table 1.

Hetenyi (1946) presents analytical solutions for many cases of practical interest related to finite beams on Winkler medium. It is convenient to consider the finite beam as a part of the infinite beam and to employ the method of superposition using the solutions of the appropriate infinite beam problem. Unknown forces (shear force or moment) are applied at points corresponding to the ends of the finite beam. These forces are evaluated from four simultaneous equations so as to satisfy necessary boundary conditions at ends of the finite beam. Hetenyi (1946) has also shown the use of other methods to solve the finite beam problem namely method of initial parameters and strain energy method. In the method of initial parameters, the general form of deflected shape of the beam is considered and the constants A_1, A_2, A_3, A_4 in Eq. (6) are evaluated in terms of boundary conditions at end of the finite beam. In the strain energy method, energy, not force has the primary role and the balance of energies is accomplished using variational methods. This method can be extended to take into account variations in $E_b I_b$ or k along the beam length. The strain energy method provides a useful alternative technique for the analysis of foundation interaction problems where governing differential equations and their solutions cannot be represented in simple forms.

Beam Response Parameter	**Infinite Beam subjected to concentrated load P at origin**	**Infinite Beam subjected to concentrated moment M_0 at origin**	**Infinite Beam subjected to uniformly distributed load p over finite length $(a+b)$ of beam**
Deflection, $w(x)$	$\frac{P\lambda}{2Bk}A_{\lambda x}$	$\frac{M_0\lambda^2}{Bk}B_{\lambda x}$	---
Rotation, $\theta(x)$	$-\frac{P\lambda^2}{Bk}B_{\lambda x}$	$\frac{M_0\lambda^3}{Bk}C_{\lambda x}$	---
Moment, $M(x)$	$\frac{P}{4\lambda}C_{\lambda x}$	$\frac{M_0}{2}D_{\lambda x}$	
Deflection w below loaded area	---	---	$\frac{p}{2k}\left(2-D_{\lambda a}-D_{\lambda b}\right)$
Rotation θ below loaded area	---	---	$\frac{p\lambda}{2k}\left(A_{\lambda a}-A_{\lambda b}\right)$

$A_{\lambda x}=e^{-\lambda x}\left(\cos\lambda x+\sin\lambda x\right)$, $B_{\lambda x}=e^{-\lambda x}\sin\lambda x$, $C_{\lambda x}=e^{-\lambda x}\left(\cos\lambda x-\sin\lambda x\right)$, $D_{\lambda x}=e^{-\lambda x}\cos\lambda x$

Table 1. Analytical solutions of Hetenyi (1946) for infinite beam on Winkler medium

Various alternative techniques (Selvadurai, 1979) have been used by different researchers for the analysis of this finite beam problem such as relaxation technique, iterative methods, graphical successive approximation procedure, integral transform techniques, influence line methods, finite difference solution, Green's function approach, invariant imbedding techniques, semi-empirical method, soil line method, etc.

Hetenyi classifies beams (length L) on Winkler media into three groups based on the dimensionless parameter λL: (i) Short beams $(\lambda L < \pi / 4)$ (ii) Beams of medium length $(\pi / 4 < \lambda L < \pi)$ and (iii) Long beams $(\lambda L > \pi)$. λ defines the curvature of the deflected shape of the beam and the rate at which the effects of external loads diminish along the length of the beam. The short beam can be considered as a rigid footing. Beams of medium length are flexible beams where appropriate analysis is necessary. For long beams, the length is large enough so that the effect of end conditioning forces can be neglected and computations become simplified. Analytical techniques for infinite beam can thus be applied.

Beam on Two-Parameter Elastic Medium

Filonenko-Borodich (1940), Pasternak (1954), and Vlazov and Leontiev (1966) present details of a general method of analysis for finite beams resting on two-parameter soil medium. The basic solutions for the infinite beam subjected to concentrated force or moment can be solved in closed form in terms of elementary functions. Techniques such as the method of initial parameters and the method of superposition can then be effectively used in the solution of the finite beam problem.

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Biographical Sketch

Tahmeed M. Al-Hussaini is professor of civil engineering department at Bangladesh University of Engineering and Technology (BUET). He obtained his PhD from State University of New York at Buffalo, USA in 1992, Masters Degree at Asian Institute of Technology, Bangkok in 1987 and Bachelor's degree from BUET in 1984. In the US, he was involved in NSF and NCEER funded research projects dealing with wave barriers for reduction of ground vibration and seismic response of base-isolated structures. Working with Earthquake Protection Systems of USA, he gained research and practical experience in seismic isolation. He joined BUET as faculty member in 1994. He was a visiting research scientist at Ecole Centrale Paris in France during 2001 and 2002 for the European project on "Control of Vibrations from Underground Railway Traffic (CONVURT)". Dr. Al-Hussaini received specialized training on Seismology in China and Italy. Dr. Al-Hussaini had a major role in the planning and design of the digital seismic instrumentation project (first of its kind in Bangladesh) for Jamuna Bridge, which has been installed in 2003. He has been appointed a Regular Associate of the Abdus Salam International Centre for Theoretical Physics (ICTP) in Trieste, Italy for 2005-2010. He is currently conducting joint collaborative research with ICTP for the deterministic seismic hazard assessment for Bangladesh. He worked as a technical expert member in the committee for preparing specifications for establishing four modern seismological observatories in Bangladesh. He is the vice-president of Bangladesh Earthquake Society since 2008. He is a Member of TC4 Committee for Earthquake Geotechnical Engineering, International Society for Soil Mechanics and Geotechnical Engineering. He has been in charge of making major revisions to the existing seismic design provisions of the 1993 Bangladesh national building code. His main research interests include BEM/FEM analysis in geomechanics and structural dynamics, wave propagation, vibration isolation, seismic hazard and vulnerability assessment, dynamic response of structures, soil-structure interaction, slope stability, disaster mitigation and seismic code provisions.