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Approximate solution for seismic earth pressures on rigid walls retaining inhomogeneous elastic soil

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ABSTRACT

An approximate elasto-dynamic solution is developed for computing seismic earth pressures acting on rigid walls retaining continuously inhomogeneous elastic material and excited by vertically propagating shear waves. The shear modulus of the soil is represented as a nonlinear function of depth, in a manner that is consistent with established analytical and empirical relationships, while mass density and Poisson's ratio are assumed constant. Solutions are presented for a single wall and for a pair of walls spaced at a finite distance. A shape function characterizing the vertical variation of horizontal displacement of the soil column in the free-field is assigned, and simplifying assumptions regarding the dynamic vertical stresses and the vertical-to-horizontal displacement gradient are made to facilitate closed-form expressions for horizontal displacement and stress fields. These solutions are used to compute the distribution of dynamic horizontal earth pressure acting on the wall. A Winkler stiffness intensity relationship is then derived such that the proposed method can be extended beyond the boundary conditions considered herein. These solutions agree well with exact analytical elasto-dynamic solutions for inhomogeneous soil that are considerably more complicated to implement. Causes of differences between the solutions are discussed.

1. Introduction

Seismic earth pressures acting on embedded walls are most commonly analyzed using a limit equilibrium concept originally developed by Okabe [1] and Mononobe and Matsuo [2], commonly known as the Mononobe-Okabe (M-O) method. The M-O method was subsequently modified in various manners (e.g., [3–6]). This family of methods (referred to as limit state methods) assumes that an inertia force acts on an active wedge to produce a dynamic increment of earth pressure.

Limit state methods overlook several important aspects of the problem, such as wave propagation, poroelasticity of saturated soils (e.g., [7–9]), and soil-structure interaction that produces mismatches between wall and free-field soil displacements. Inertial forces in the backfill do not load the wall directly, as assumed in limit state methods. To illustrate this point, consider an embedded wall with the same mass and stiffness as the soil. Vertically propagating shear waves will induce no increment of lateral seismic earth pressure because inertia forces are transmitted entirely by shear, in accordance with the solution for one-dimensional shear wave propagation. Hence, there is no fundamental association between backfill inertia and seismic wall pressures.

In a realistic wall-soil system (with wall elements stiffer than those for soil), dynamic body forces in the backfill induce dynamic deformations, which are incompatible with wall kinematics, causing interaction stresses to develop between the wall and soil. Furthermore, inertial loads arising from differences in mass between the wall structure and soil, or from the dynamic response of an above-ground structure attached to the wall, will produce force and overturning demands on the wall that in turn induce relative deformations and seismic earth pressures at wall-soil interfaces. As these phenomena are overlooked in limit analysis, M-O type methods fail to properly capture the fundamental physics of soil-wall interaction.

It is therefore not surprising that the literature is mixed on the accuracy of the M-O method and its variants. Recent experimental studies have challenged the M-O method as being overly conservative for cantilever U-shaped walls [10] and free-standing retaining walls [11], and as providing a reasonable upper-bound for braced walls [12]. By contrast, analytical elasto-dynamic solutions [13,14] and numerical modeling studies [15] have challenged M-O as being unconservative. This has led to confusion among practicing engineers and researchers regarding appropriate methods of analysis.

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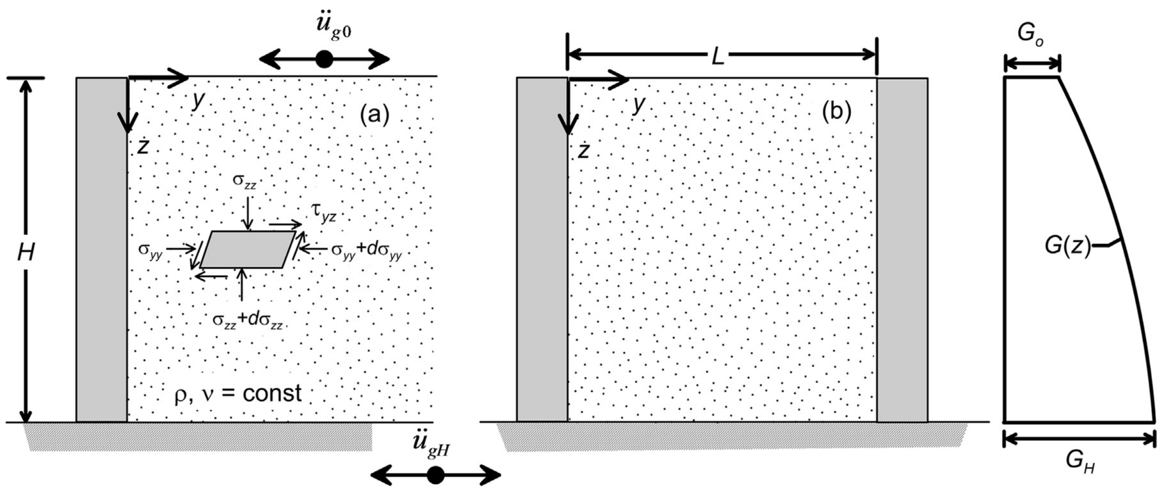

Fig. 1. Vertically heterogeneous soil retained (a) by a single rigid wall, and (b) between a pair of rigid walls.

Brandenberg et al. [16] developed an elasto-dynamic Winkler-based framework for the kinematic wall-soil interaction problem that explains both the lower-than-M-O experimental observations and the higher-than-M-O analytical and numerical simulations. The key parameter controlling relative wall-soil displacements, and hence mobilized earth pressures, is the ratio of wavelength, λ, of the vertically propagating shear wave, to wall height, H, which can be interpreted as a dimensionless frequency. Walls founded on thick soil deposits (like the experimental studies and most retaining structures) tend to have high λ/H ratios, which are associated with modest depth-dependent free-field displacements that are largely followed by wall-foundation systems. Under such conditions, earth pressures are low for a given surface motion amplitude. In contrast, the response of a uniform soil deposit resting on a rigid base is often dominated by the first mode of horizontal vibration, which corresponds to λ/H=4 for retained soil deposits that are long relative to their thickness. Rigid walls resting on a rigid base (as often assumed in elasto-dynamic solutions) can therefore mobilize significant kinematic interaction and high earth pressures.

Although the Brandenberg et al. [16] solutions explain several key features of behavior, assumptions that limit their applicability include (1) uniform shear modulus with depth, (2) rigid walls, (3) a lack of gapping at the soil-wall interface, and (4) elastic soil behavior. The purpose of this paper is to address the assumption of uniform shear modulus with depth. To this end, an approximate elasto-dynamic solution is developed for continuously inhomogeneous soil, defined as a soil layer with a smooth variation of shear modulus with depth (as opposed to layers with abrupt transitions in shear modulus), using simplifications similar to those employed by Kloukinas et al. [17]. An expression for equivalent Winkler stiffness intensity is developed, and the solutions are compared with more rigorous numerical formulations from the literature.

2. Vertical variation of soil shear modulus

A number of empirical and theoretical equations have been suggested to capture the dependence of soil shear modulus on mean effective stress. Hertz [18], in his landmark 1882 paper, derived an expression in which the shear modulus of a particulate medium composed of elastic spheres is proportional to the mean effective stress raised to a power, n, which he found equal to 1/3. The same result was later obtained by more elaborate, yet still idealized particle contact models. Hardin and Richart [19] suggest a form in which the shear modulus is also a function of void ratio. Building upon the earlier work by Mindlin et al. [20], Hardin and Drnevich [21] found experimentally that n=0.5 and also introduced an overconsolidation ratio term for plastic soils. Yamada et al. [22] also recommend n=0.5 for granular soils and suggested that n=1 for plastic clay-sand mixtures (they did not include a void ratio term in their formulation, which may explain why n is higher). All of these forms result in zero shear modulus when the effective stress is zero, which is unrealistic as it does not account for cementation, cohesion, capillary effects, and can be numerically problematic near the surface.

Although shear modulus fundamentally depends on effective stress, it has also conveniently been formulated as a function of depth to facilitate analytical solutions. For example, Wood [13] and Veletsos and Younan [23] formulated solutions for the seismic increment of lateral earth pressure exerted by a soil deposit with a parabolic variation of shear modulus with depth, $G(z)$. Rovithis et al. [24] suggest a form for shear wave velocity as a function of depth, $V_s(z)$, that is equivalent to the equation for $G(z)$ in Eq. (1) for mass density, ρ=const., where z is depth, z_r is a reference depth, G_r is the shear modulus at $z=z_r$, and b is a constant that influences the depth gradient and the value of $G_O=G(0)$ (note that $G_O=G_r b^{2n}$). Rovithis et al. [24] utilize n for the depth-variation of V_s rather than for G, and it is therefore multiplied by 2 here to represent G. Their application was vertical wave propagation through a vertically inhomogeneous layer resting on a rigid base.

$$G(z) = G_r\left[b + (1 - b)\frac{z}{z_r}\right]^{2n} = G_r f(z) \tag{1}$$

Vrettos et al. [25] utilized the form provided in Eq. (2), where G_∞ is the modulus at an infinite depth (approached asymptotically as $z \to \infty$), and η is a constant that controls the rate of change of G with depth. Using this form, Vrettos et al. [25] developed exact analytical solutions for the response of a continuously inhomogeneous soil layer on a rigid base restrained between two rigid walls subjected to horizontal base shaking (illustrated in Fig. 1b).

$$G(z) = G_0 + (G_\infty - G_0)(1 - e^{-\eta z/H}) \tag{2}$$

In this paper, we formulate an approximate analytical solution for seismic earth pressure following the approach developed by Kloukinas et al. [17], but for soil with vertically inhomogeneous shear modulus resting on a rigid base (Fig. 1). The functional form for vertical inhomogeneity of shear modulus follows Rovithis et al. [24]. Solutions are developed for a single rigid wall retaining an infinitely long soil deposit (Fig. 1a) and for two rigid walls retaining a finite length deposit (Fig. 1b) (the common case of basement walls, with soil pressures on the outside, would be analyzed using the geometry in